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Pullout Behaviour of Screw and Suction Anchors in Soft Marine Clays

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Introduction

In the field of ocean engineering, anchors are used for several purposes. They are used to keep in position floating bodies such as ships, navigational buoys, and semi submersible rigs, and to provide stability to bottom-fixed structures such as articulated towers, jacket platforms, seabed pipelines, etc. For oil production in deeper waters, tension leg platforms and guyed towers are being introduced and these require anchoring to the seabed. On the buoyant superstructures, the loads acting are repetitive tensile loads with static pull. The repetitive loading is due to both rocking and bobbing motion of the superstructure caused by wind and waves acting on it. During storm wave conditions, these loads are significant and may lead to failure of anchors.

Some of the anchors which are in use are illustrated in Figures 1 to 4.

Screw Anchors – Literature Review and Advantages involved

Trofimenkov and Mariupolski (1965) and Adams and Klym (1972) were among the first to study the behaviour of screw anchors under uplift conditions and adopt them for tower foundations. Valent, Taylor, Atturio and Beard (1979) suggested that screw anchors might be applied in marine environments to supplement the vertical holding capacity of dead-weight anchors. Mooney, Adamczak and Clemence (1985) were among the first to attempt to develop reliable rational concepts for the design of screw anchors. They proposed a cylindrical failure surface between top and bottom screw plates in order to derive the required capacities. Hoyt and Clemence (1989) and Narasimha Rao et al. (1990) suggested that at higher SR values, capacities could be derived by an individual plate bearing method.

The geometry and behaviour of screw anchors and under-reamed pile anchors are similar in that the screw plates provided in the screw anchors increase the capacities significantly, as do the under-reams in the under-reamed piles.

Design approaches to under-reamed pile anchors have been suggested by Mohan and Jain (1958), Bassett (1977), Chandrasekharan et al. (1978) and Sharma et al. (1978).

In under-reamed pile anchors, a cylindrical failure surface between the under-reams was suggested for the piles with closely spaced under-reams

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((SR) <1.5-2). For larger (SR) values end-bearing failure from each under-ream was suggested.

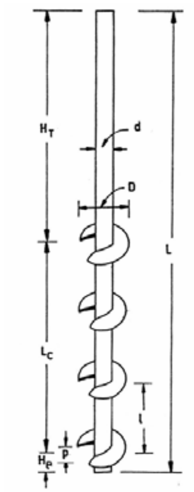

Fig. 1 Helical Anchor Used

Fig. 2 Rig for the Installation of Screw Anchor

A limited number of investigations have been reported on the influence of repetitive cyclic loading on pullout capacity of anchors.

Bemben et al. (1973) studied the behaviour of vertical holding capacity of marine anchors in sands and clays subjected to static and cyclic loading. Andreadis et al. (1981) reported the behaviour of plate and fluke anchors under long-term cyclic loading in sandy soil. They concluded that a realistic failure criterion could be interpreted using deformation rate. Ponniah and Finlay (1988) brought out by the capacities of plate anchors in a normally consolidated cohesive soils subjected to a long-term cyclic load. From these test result, it was found that the anchors did not fail up to a cyclic load ratio of 50 ± 20%. Gulhati (1990) reported the undrained behaviour of plate anchors in cohesive soil.

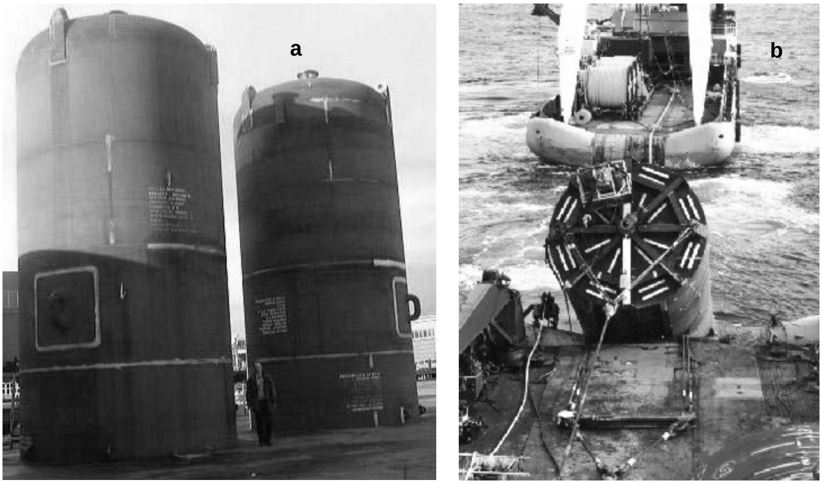

Fig. 3 (a) Typical Suction Caisson and (b) its Deployment into Seabed

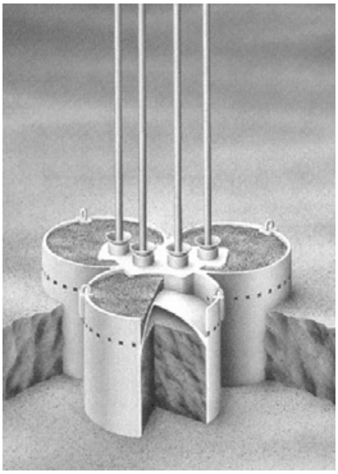

Fig. 4 Assembly of Suction Caissons

Anderson et al. (1978) reported a significant reduction of strength with cyclic loading. Yet a few investigators, such as that of Koutsoftas (1978), reported that the loss of strength due to cyclic loading under undrained conditions was very small. Motherwell and Wright (1978) reported that there was no loss of strength, due to cyclic loading. Brown et al. (1977) reported that there was an increase in strength due to cyclic loading.

Screw anchors are simple steel shafts to which one or more screw plates attached at regular intervals. They are installed into the soil by applying torque to upper end of the shaft. The screw anchors can be easily installed, and they do not require any grouting to keep them in position. They are used underwater,

I. to secure moorings and cables to the seabed

II. to provide stability to seabed pipelines

III. to supplement the vertical holding capacity of dead weight anchors.

Each guy wire has an individual anchor, greatly reducing the risk of catastrophic failure. In anchor replacement, screw anchors do not require removal of the old anchor. In bearing load applications, screw piles allow you to transfer the load to a more stable layer. Screw anchors allow the engineer to utilize stable soil layers well beyond the reach of deadman anchors. Screw anchors are environmentally friendly. The weather is not a factor.

Screw anchors do not depend on the availability of concrete. Guy cables may be attached immediately after installation. Screw anchors can be installed in areas too remote for concrete trucks. Anchor installations do not require excavation. Anchor installation torque measurements provide an immediate indication of anchor strength.

These types of anchors are ideally suited in soft to medium stiff clay, medium dense clayey sands / sandy clay and silty clays. These can be used in soil profile as shown in Figures 5 and 6 [Soil Profile from KG Basin]. A typical screw pile is shown in Figure 7.

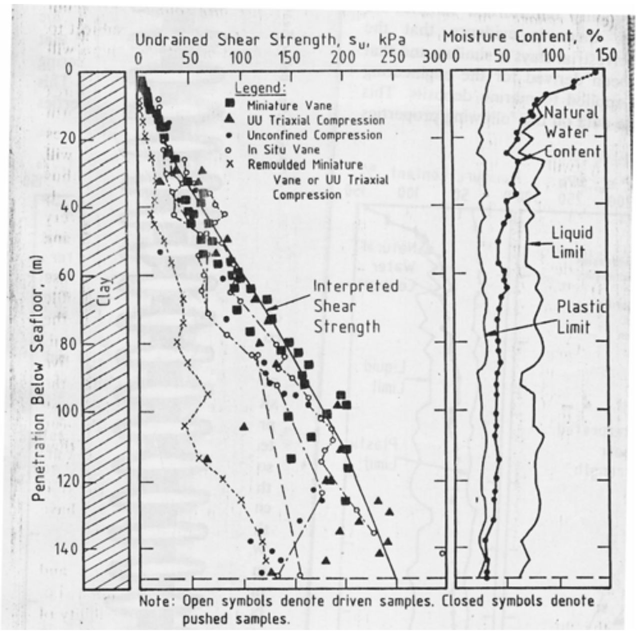

Fig. 5 Typical Soil Conditions in the Gulf of Mexico

(After Poulos, 1988)

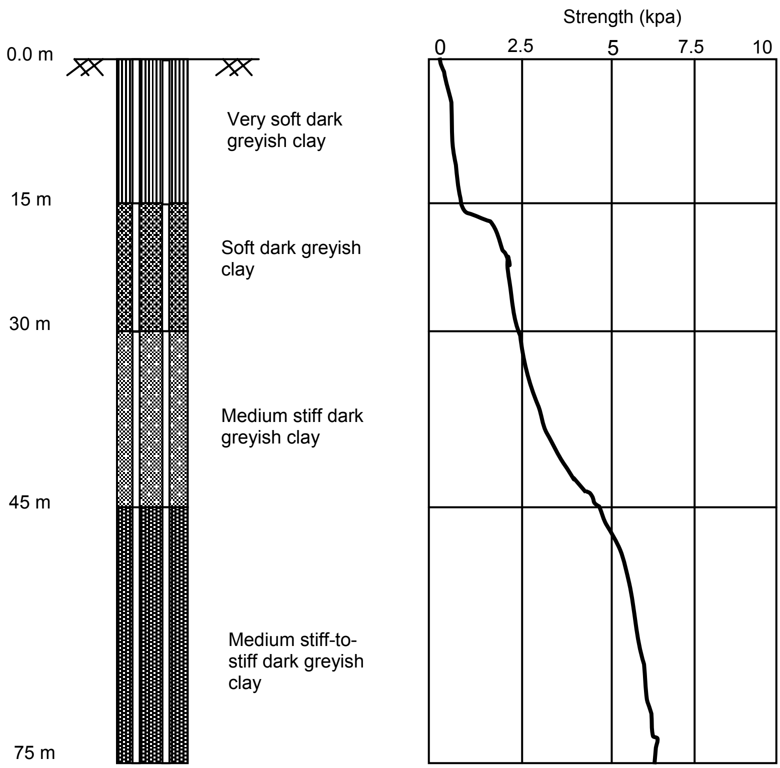

Fig. 6 Typical Soil Profile in Bay of Bengal

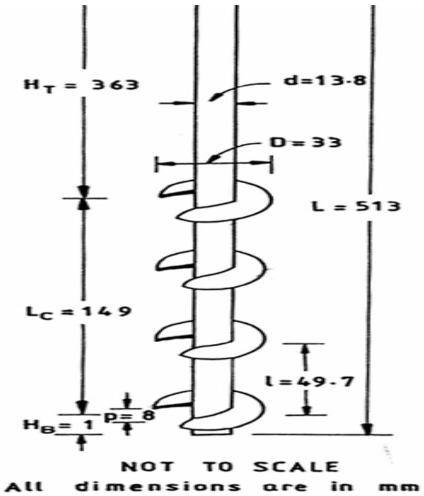

Fig. 7 Helical Anchor Used

Mooney et al. (1985) were among the first to attempt to develop reliable rational concepts for the design of screw anchors. They proposed a cylindrical failure surface between top and bottom screw plates in order to derive the required capacities. They reported that at a spacing ratio SR (SR is the spacing between any two adjacent screw plates divided by their average diameter) of up to 1:5, cylindrical failure surface formulations were valid. Hoyt and Clemence (1989) and Narasimha Rao et al (1990) suggested that at higher SR values, capacities could be derived by an individual plate bearing method.

As the screw anchor is in wide use as a tower foundation, its behaviour under compressive loads is significant. Narasimha Rao et al. (1989), reported on this behaviour, and emphasized the importance of spacing of screw plates. Narasimha Rao and Prasad (1993) proposed simple empirical equations to calculate the uplift capacities of screw anchors with plates at higher SR values in clayey soils.

There are basic differences between the installation methods employed in under–reamed piles and screw piles. Under–reamed piles can be considered as bored cast in situ piles; screw piles are driven. In soft clays, under-reamed piles cannot be cast, because of difficulty in the casting of under-reams. The use of screw piles is preferable in such cases.

The behaviour of multiple anchors in clayey soils at different embedment ratios (H/D is the depth of embedment of the top screw plate divided by its diameter) has not previously been studied extensively, but is examined in this paper using an experimental programme on model screw anchors.

Experimental and Theoretical Work: Screw Anchors

The model anchors

Two sets of model anchors made with mild steel pipes welded with mild steel screw plates were investigated. The set I anchors (anchors A1- A4) were smaller than the set II anchors (A5-A7). Different diameter plates (33 mm and 75 mm), with SR values (1.1 - 4.6) were adopted for the screw plates. Hoyt and Clemence (1989) reported that this is the range of SR values generally adopted in the field.

Soil Used

A marine clay from a coastal deposit in the east coast of India was used and this soil is composed of 90% silt and clay and 10% sand. The liquid limit and plastic limit of the soil were 82% and 32%, respectively.

The Test Tanks

Two types of test tank were used; for the set I anchors, a cylindrical test tank of 300 mm diameter and 600 mm high; for the Set II anchors rectangular test tank 1200 × 1000 × 1200m. These sizes were chosen to eliminate any boundary effects. A schematic arrangement of the test setup is shown in Figure 8.

Testing Procedure

The tests were conducted in a soil bed prepared at a soft consistency I_c ((liquid limit – water content)/(liquid limit – plastic limit)) of 0.28-0.60. These consistencies were considered appropriate for the installation of anchors in the field.

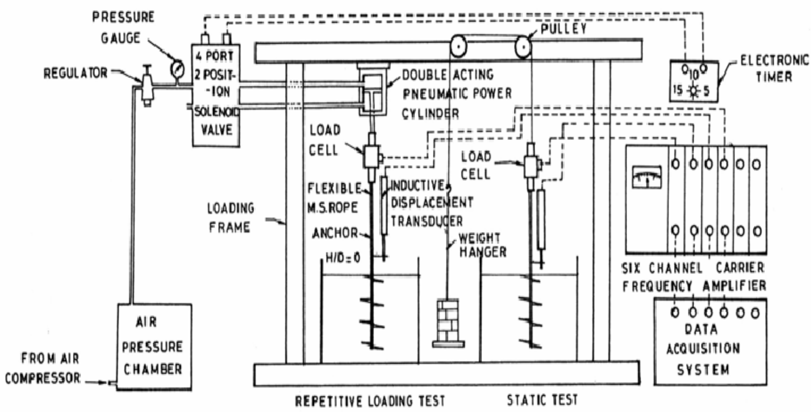

Fig. 8 Experimental Setup

At the end of each test, the undrained shear strength C_u of the soil bed was measured using field in situ vane shear apparatus. The tests were conducted at different depth locations; the average value reported was taken as C_u. Also, a few random samples were taken out and checked for full saturation and homogeneity.

The load at which the anchor came out with a large upward movement was considered the gross ultimate uplift capacity Q_g. At this large movement, the load-movement curve becomes asymptotic to the movement axis. The net ultimate uplift capacity Q_u can be obtained by subtraction of the weight of the anchor w_a and Q_g.

Testing Programme

Anchors A1 – A4 were tested in soil bed prepared at consistency indices of 0.28 and 0.45 and H/D values of 0-10. The influence of the consistency of soil on the model anchors having been elucidated, anchors A5-A7 were tested at I_c = 0.6 and with values of H/D between 0 and 8.

Results and Discussion

Typical pullout load against upward movement curves of anchor A3 (SR =1.5, D = 33.0 mm an I_c of 0.45 are shown in Figure 9 corresponding to H/D values of 0, 1, 2, 3, 4, 6, 8 and 10. From these, it is found that capacities increase with increasing H/D. Yet H/D = 0, the cohesive resistance along with cylinder contributes to the gross ultimate uplift capacity Q_g. As H/D increases from 0, there is an additional contribution to Q_g from bearing resistance on the top screw plate Q_2 and shaft adhesion above the top plate.

A typical value of upward movement of anchor at Q_g corresponding to H/D = 0 is about 15% - 20% of screw plate diameter; at H/D =10 it is about 40%-50%.

All the test results for the Set I anchors are shown in the form of variation of net ultimate uplift capacity Q_u with H/D in Figure 10 which shows this variation at a soil consistency index of 0.28. If H/D increases from 0 to 2, the increase in Q_u is significant, and with further increase in H/D the rate of increase is considerably lower. Also, above an H/D value of 4, Q_u seems to vary linearly with H/D. The capacities in respect of anchors A3 (SR=1.5) and A4 (SR=1.1) are almost identical for all values of H/D ratio, due to the formation of well developed cylindrical failure planes of the same length (149 mm) in anchors with SR ≤ 1.5.

Based on the initial gradient, the transitional area and the final gradient of curves shown in Figure 10 the anchors can be classified as shallow, transition and deep, as shown in Figure 11 and discussed below.

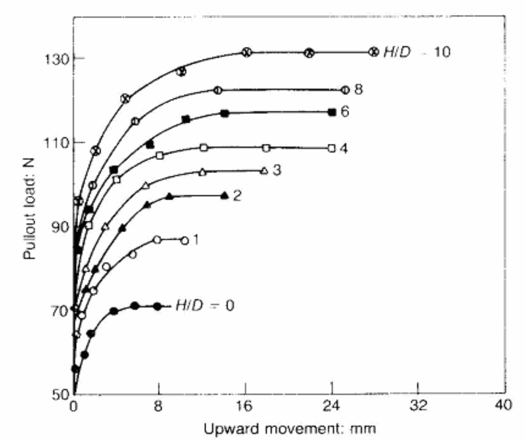

Fig. 9 Typical pullout load-upward movement curves: anchors A3, I_C = 0.45

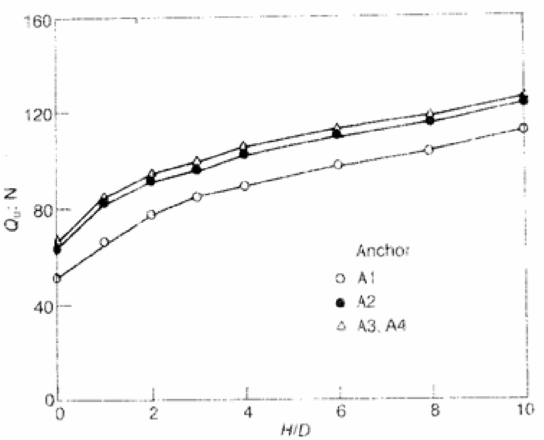

Fig.10 Variation in Pullout Capacity with H/D

Shallow Anchor

Up to H/D=2, Q_u increases significantly with H/D and screw anchor is regarded as shallow. During testing, the failure zones are found to extend up to the surface, with large tensile cracks and surface heave at the top and a clear gap is observed between the anchor shaft and the soil at the top surface (Figure 11a). In this case, the development of shaft adhesion Q_3 can be ruled out, as there cannot be any relative movement between shaft and soil and the load is expected to be transferred to the soil by cohesive resistance between the top and bottom screw plates Q_1, and by the top plate Q_2 only. The top plate can be regarded as a simple plate anchor, and its contribution can be explained. Most of the theories proposed to estimate the uplift capacities of plate anchors in clay overestimate the capacities, perhaps because they have not considered the tensile cracks and large deformations at the failure (Davie and Sutherland, 1977). Vesic's (1971) equation is used here to express Q_2 in terms of break-out factors.

$$Q_2 = A\,(C_u N_{cu} + \gamma D) \tag{1}$$

where A is the area of the plate anchor, N_{cu} is the break-out factor, γ is the unit weight of soil and D is the depth of embedment. Up to H/D=2, Q_u is the sum of the capacity of the anchor at H/D=0 and the bearing resistance on the top screw plate Q_2.

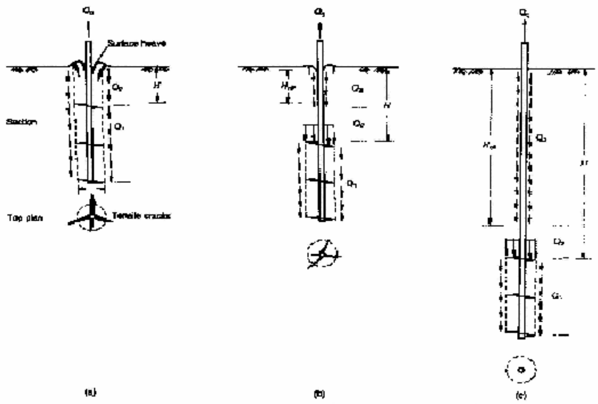

Fig 11 Behaviour of Helical Anchor at Various Embedment Ratios:

(a) Shallow (b) Transition (c) Deep

Transition Anchor

Between H/D values of 2 and 4, the screw anchor can be regarded as a transition anchor. During testing in this embedment range the failure surface extends to the soil surface, with minute tensile cracks and a slight contribution from shaft adhesion Q_3. By taking N_{cu} =9, the Q_3 values are calculated. The effective length of shaft H_{eff} contributing to Q_3 is calculated from

$$Q_3 = \pi d H_{eff} \alpha C_u \tag{2}$$

where d is the diameter of the shaft and α is the adhesion factor (tests on single shafts under similar conditions gave α values of 0.66, 0.58 and 0.53 for I_c values of 0.28, 0.45 and 0.60 respectively). Based on these values of α and the known soil strength C_u, and from the calculated Q_3 values are derived and expressed in terms of screw plate diameter. The values of H_{eff} are in the range 0.7D-0.9D (for H/D=3) and 1.7D-2.5D (for H/D=4).

Deep Anchor

Above an H/D of 4, the screw anchor can be regarded as a deep anchor. No tensile cracks or surface heave are observed during testing, perhaps because of local shear failure conditions (Figure 11).It will be established that, at the tip of the pile, the failure zone extends over a depth of almost twice the diameter (Zeevaert, 1983). A height of 1.4D-2.3D is therefore ineffective in mobilizing the adhesion, as it is involved in the bearing zone.

Cylindrical failure surface method

This is suitable only for anchors with screw plates spaced at close intervals (SR ≤1.5). However, suitable spacing ratio factors S_F are suggested by Narasimha Rao and Prasad (1993) for anchors with screw plates at higher SR values. According to this method,

$$Q_g = Q_1 + Q_2 + Q_3 + W_a + \text{suction force} \quad (3)$$

$$Q_g = S_F (\pi DL_c)C_u + A(C_u N_{cu} + \gamma D) + \pi dH_{eff} \alpha C_u + W_a + \text{suction force} \quad (4)$$

where

$$S_F = 1.0 \text{ for } SR \leq 1.5 \quad (5a)$$

$$S_F = 0.863 + 0.069(3.5 - SR) \text{ for } 1.5 \leq SR \leq 3.5 \quad (5b)$$

$$S_F = 0.700 + 0.148(4.6 - SR) \text{ for } 3.5 \leq SR \leq 4.6 \quad (5c)$$

Individual Plate bearing method

The screw anchor is assumed to consist of a series of plate anchors at different embedment ratios (this is valid only for anchors with screw plates at SR>2). The anchors' capacity can be considered as the sum of the uplift capacities of the plate anchors and the shaft resistance between the plates. Above each plate, adhesion over a shaft length of 1.5D - 2.5D should be considered. Shaft adhesion can be calculated by using the α values suggested by Das and Seeley (1982) for pipe piles.

Behaviour under Cyclic Loading

Experimental Setup (For repetitive load tests)

Both the static and repetitive loading tests were conducted in a cylindrical test tank of 350 mm diameter and 500 mm height.

A schematic diagram of the setup used for both static and repetitive loadings is as shown in Figure 8. For repetitive loading tests, the load-controlled pneumatic loading system was fixed to the beam of a loading frame. The pneumatic loading system consisted of (1) an air compressor of adequate capacity with a reservoir, (2) a double-acting pneumatic power cylinder, (3) a solenoid valve, and (4) an electronic timer. The load was measured with a load cell of 250 N capacities, and the upward movement of anchor was measured with an inductive displacement transducer with ± 25 mm travel. The outputs from the load cell and inductive displacement transducer were fed to a multi-channel carrier-frequency amplifier, and the signals were suitably amplified and recorded by means of a data acquisition system.

Testing Procedure

All the tests were conducted with a soil bed prepared to soft consistency. The soil was placed in layers of 50mm thickness. After the clay bed was formed, the anchor was slowly screwed into the soil bed to the desired position by applying torque to the upper end of the shaft using a torque wrench. In addition to torque, enough downward force was applied to the anchor shaft that the rate of advancement of the anchor was equal to the pitch of the screw plate per rotation. To equilibrate the pore water pressures developed during screw anchor insertion, an approximately 2 hour waiting period was employed before the loading was initiated.

The tests using repetitive loading were conducted in two stages: (1) repetitive loading test; (2) post static-pullout test. In the repetitive loading test, in each cycle of loading and unloading, the load on the anchor was varied from zero to the desired cyclic load ratio (ratio of repetitive load applied to static ultimate pullout capacity expressed as percent). This loading pattern was similar to the one adopted by Chan and Hanna (1980) on single piles in sand. The repetitive loading was continued until "elastic" response in upward deformation was attained for the tests with low cyclic load ratios. At higher cyclic load ratios, the test was stopped when anchor movement reached a value equal to the anchor diameter or to a stage at which the required load could not be maintained.

Later, the anchor was subjected to a static pullout test immediately (termed "post static-pullout test") to observe the effect of repetitive loading on anchor behaviour. Further, for the purpose of comparison and to fix the different repetitive cyclic load ratios, static pullout tests were also conducted.

Testing Programme

The repetitive loading tests were conducted at three embedment ratios (H/D) of 0, 2, and 8 to simulate both shallow and deep anchor conditions. At each of these embedment ratios, the tests were conducted at two soft consistency indices.

Tests were conducted at different cyclic load ratios of 30% to 80% and in a time period of 12 s. (For the Indian coast, a 12 s period generally represents storm wave conditions.) However, in one testing series, three different loading periods of 1, 5, and 12 s, were also adopted to observe the behaviour of the anchor under different loading periods.

Results and Discussion

All the test results are discussed in this section for both shallow (H/D = 0 and 2) and deep (H/D = 8) anchors.

Behaviour of Screw Anchor at Limiting Condition H/D = 0

The typical variation of anchor upward movement with number of cycles at $I_c = 0.28$ is presented in Figure 12a. It can be seen from this figure that as the cyclic load ratio is increased from 30% to 80%, the upward movement of the anchor increases rapidly. At very low cyclic load ratios, i.e., at 30%, the anchor movement is practically negligible. It is observed that for cyclic load ratios of 30% to 40%, the displacements become elastic after limited upward movement. In the conventional cyclic shear tests, several investigators observed this type of elastic strain response. In the case of 30% cyclic load ratio, the elastic response in strain has been attained within 50 cycles and the maximum value of strain attained is less than 1 %. In the case of tests with 50 % cyclic load ratio, the elastic response in strain has been attained at around 100 cycles and the maximum value of strain is around 5%. In the case of anchors, strain is the ratio between upward movement of the anchor and screw plate diameter.

In order to have additional information, the behaviour of the anchor at higher cyclic load ratios is presented in Figure.12b. In the case of tests with 55 % cyclic load ratio, the elastic response in strain has been attained at around 400 cycles and the value is around 18%. However, in the case of a test with 70% cyclic load ratio, the displacement increases with number of cycles and no elastic response in strain has been observed. Up to a certain number of cycles (200 cycles), the displacement is limited; after these cycles, there appears to be an enormous increase in upward movement. This test was stopped at a displacement equal to the screw plate diameter (100% strain). In the case of the test with 80% cyclic load ratio also, the movement of anchor increases rapidly after 30% strain (10 mm). In this test, it was found that the required load of 80% could not be maintained.

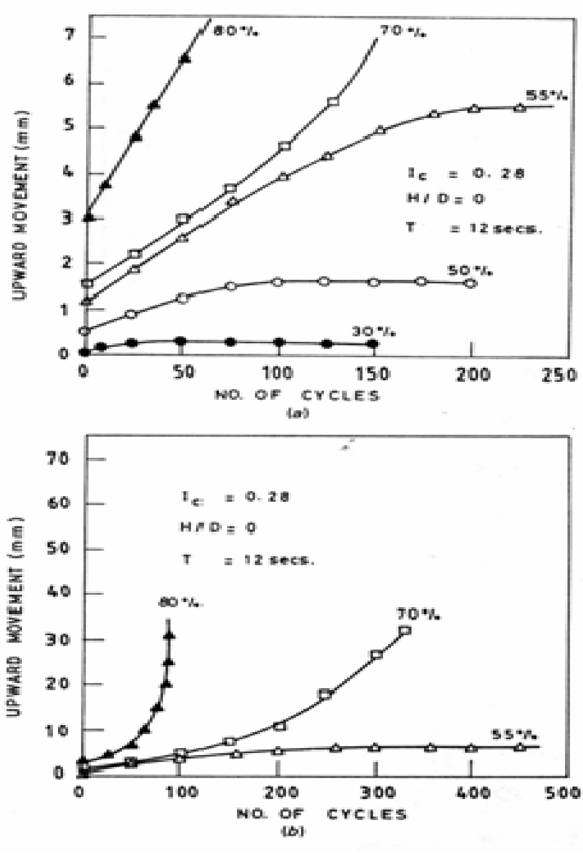

Fig.12 Variation in Anchor movement with no. of load cycles

Figure.13a represents the post static-pullout behaviour of the anchor. The accumulated movement during repetitive loading is also included in these curves. These curves correspond to cyclic load ratios of 50% and 55 %. For the purpose of comparison, the pre-repetitive static-pullout behaviour of the anchor is also presented. In this curve it can be seen that after a strain of 30%, anchor movement is very rapid. Because of the same effect, in the case of the tests conducted at 70% and 80% cyclic load ratios, the displacement increased rapidly with number of cycles after 30 % strain. By comparing pre-repetitive and post repetitive curves, it can be said that in the case of a test with 50% cyclic load ratio, the capacity of the anchor increased by about 8%, and in the test with 55% cyclic load ratio, it increased by about 3 %. It was reported that below a certain critical cyclic load ratio, there would not be a reduction in strength of clay due to repetitive loading (Motherwell and Wright, 1978). A slight increase in capacity of around 5 to 10% due to repetitive loading was reported by Lefebvre et al. (1989). Also, it can be seen that there is reduction of displacements after repetitive loading. The stiffening of soil during repetitive loading has been reported by several authors (Handali, 1986; Hyde and Ward, 1986; Lefebvre et al., 1989; Narasimha Rao, 1988). In these investigations, it was found that the pore water pressures developed during the phase of repetitive loading tests were less as compared to the magnitude of pore water pressures developed during the phase of pre-repetitive static tests and stiffening effect was reported to be caused by this phenomenon.

However, at higher cyclic load ratios of 70% and 80%, there is a reduction in capacity (Figure.13b). In the case of an anchor subjected to 70% cyclic load ratio, the reduction in capacity is around 30%; and in the test with 80% cyclic load ratio, the reduction is around 34%. The reasons for this are twofold: (1) There will be significant reduction in capacity as the depth of embedment reduces for the shallow anchor; and (2) at higher cyclic load ratios, since the soil is subjected to large magnitude of strains, there is likely to be a reduction in undrained shear strength (c) of the clay. To find out their relative contribution, static pullout tests were conducted with the anchor installed at the reduced depth and this depth of embedment was fixed based on the movement of the anchor during the repetitive loading test. This curve is also presented in the Figure. 13b as a dashed line. In the case of a test with 70% cyclic load ratio, it was found that there is reduction in capacity occurred because of the first reason, i.e., anchor movement due to repetitive loading.

It can be seen from Figure.14 (Mooney et al., 1985; Narasimha Rao et al., 1989) that because of upward movement of the anchor, L_c is reduced and hence the cohesive resistance between the top and bottom plates is also reduced. The remaining 20 % reduction in capacity can be attributed to the strength reduction. In the case of a test with 80% cyclic load ratio, these reductions are of the order of 73% and 27%, respectively.

Figure.15a represents the variation of anchor movement with number of cycles at an I_c of 0.45. Figure.15b represents the variation of anchor movement at higher cyclic load ratios. From the Figures.15a and 15b, it is seen that a similar type of anchor behaviour was observed.

Behaviour of Screw Anchor at H/D=8

For these conditions, the anchors can be considered as deep anchors. Figures.16a and 16b represent the post pullout behaviour of the anchor. The reductions in displacements and increases in capacities can be observed up to 55 % cyclic load ratio. However, at higher cyclic load ratios of 70% and 80%, the reductions in capacities are found to be of the order of 8% and 11%, respectively. In the case of a test with 70% cyclic load ratio, about 30% reduction is due to anchor movement during repetitive loading.

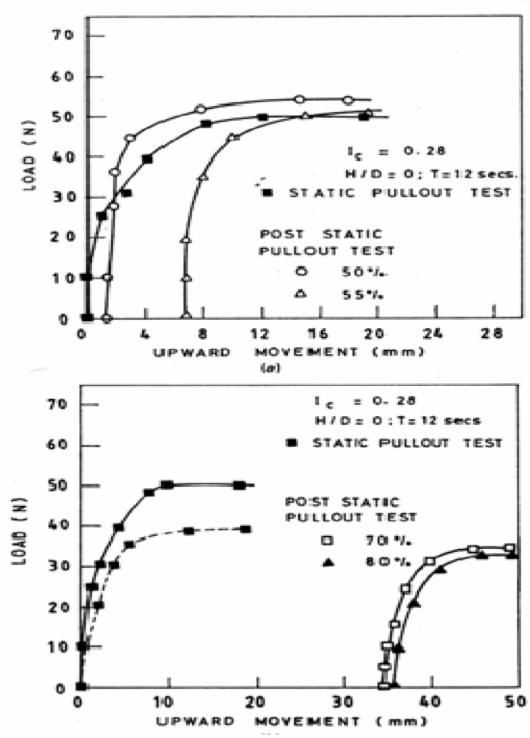

Fig. 13 Post static Pullout behaviour of Anchor (H/D = 0; I_c=0.28)

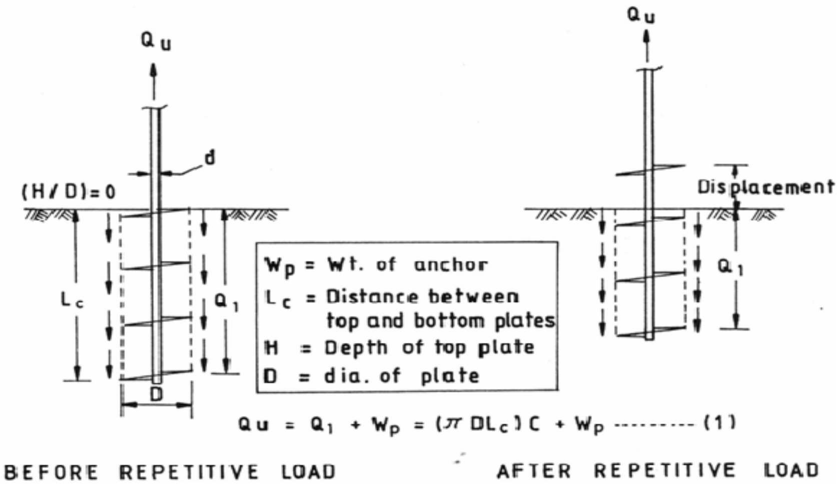

Fig. 14 Behaviour of Shallow Anchor

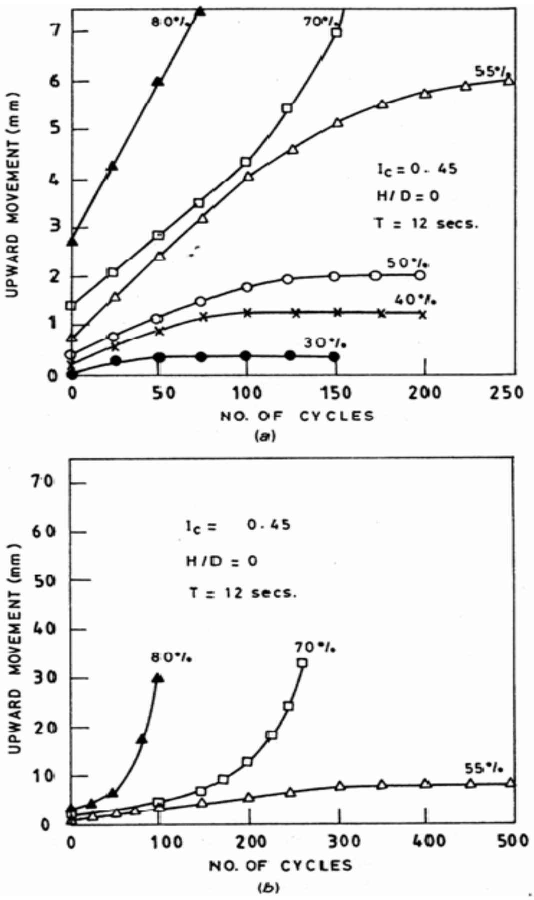

Fig.15 Variation of Anchor Movement with Number of Cycles

From the pull out tests conducted on some of these screw piles under static load conditions, it can be observed that very nearly cylindrical failure surfaces were observed for the anchor systems with closer plate spacings. And these can be seen from the photographs presented in the Figures.17 and 18.

Lateral Capacity of Screw Piles

A four plated screw pile subjected to an ultimate lateral load is shown in Figure.19. The various resisting forces on the pile are (1) lateral resistance offered by the soil on the pile shaft; (2) bearing resistance offered by the soil on the bottom of the screw plate; (3) uplift resistance offered by the soil on top of the screw plate; and (4) frictional resistance offered by the soil on the surface of screw plates.

Lateral Resistance

A number of methods have been established to calculate the lateral resistance of a pile shaft. Poulos and Davis (1980) suggested that for a purely cohesive soil the ultimate lateral resistance pl increases from the surface ($pl = 2C_u$) down to a depth of about three pile diameters and remains constant ($pl = 9C_u$). The same is considered in this investigation. As shown in the Figure.19, the lateral resistance acts in the direction opposite to pile movement down to the point of rotation X, beyond which it acts in the direction of loading.

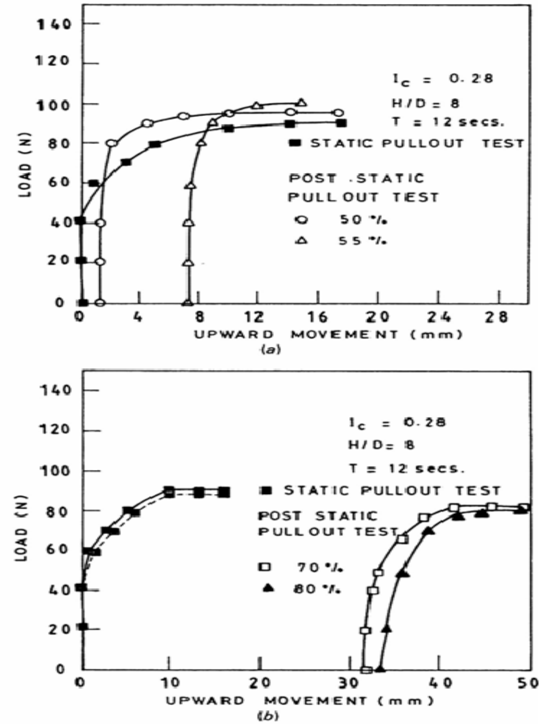

Fig. 16 Post-static Pullout Behaviour of Anchor

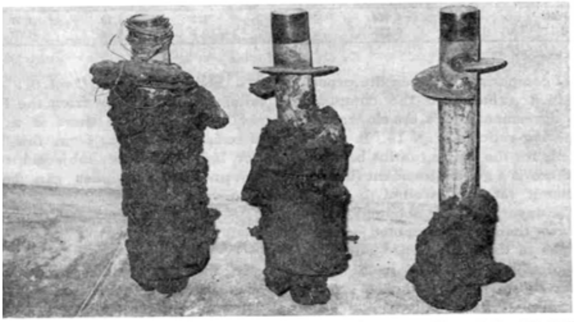

Fig. 17 Photograph of Pulled out Anchors after Pullout Tests (Type I, Set II, Soil I)

Bearing Resistance

As the pile rotates, bearing resistance will be offered by the soil on the bottom of all screw plates. A number of methods are available to estimate the bearing capacity in clays and in this investigation; Skempton's 1951 method is used. According to this method, for a circular footing, the net bearing is equal to $6.2c_u$ at ground level and is equal to 9 c_u at depths of 5 times the plate Diameter or more.

Fig. 18 Photograph of pulled out Anchors – Type I, Set I, Soil I

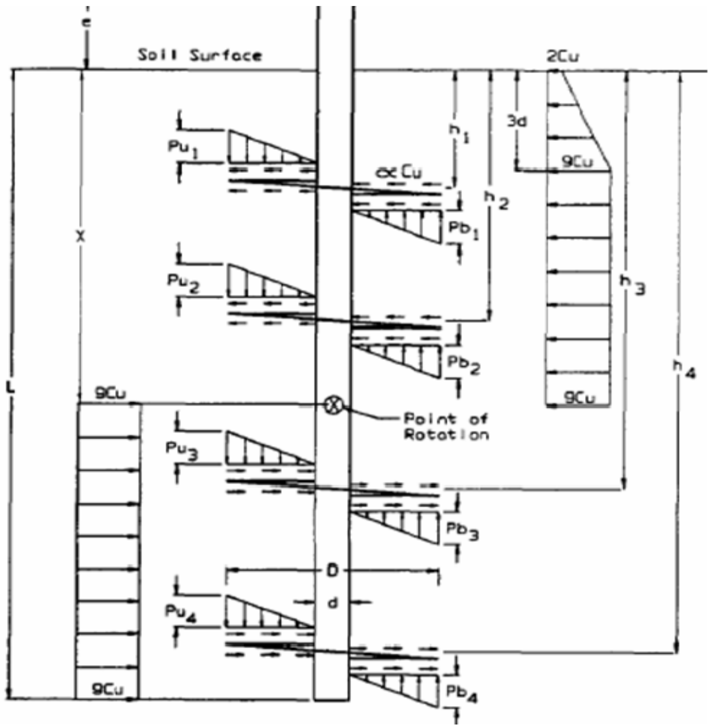

Fig. 19 Helical Pile Anchor at Ultimate Lateral Load

Uplift Resistance

As the pile rotates, uplift resistance will be offered by the soil on the top of the screw plates. Similar to bearing resistance, it is assumed that the net uplift resistance p_u is mobilised along the perimeter of the plate and that it decreases linearly and becomes zero at the junction of the shaft and plate.

$$p_u = C_u N_{cu} \tag{6}$$

where N_{cu} = 0 when the plate is at ground surface and
= 9 C_u when the plate is at a depth equal to twice the diameter.

Frictional Resistance

By knowing the various forces acting on the screw pile, the point of rotation X and the ultimate lateral load H_u can be obtained by static equilibrium of forces. Theoretically, it is derived that

$$X = -e \{[324e^2 + 36(10.5de + 10.5d^2 + 9Le + 4.5L^2)]^{0.5} / 18\} \tag{7}$$

$$H_u = C_u d(18X - 10.5d - 9L) \tag{8}$$

where e is the height of the load above the ground/seabed level.

These formulations are verified with experimental results and are found to be in good agreement.

Suction Anchors

Introduction

Some of the offshore structures like tension leg platforms (TLP), guyed towers and articulated towers are held in position by using some special types of anchor foundations installed into the seabed. In case of TLPs, the foundation loads can be divided into sustained (static), low frequency cyclic (transient) loads and cyclic loads. The loading due to cyclic action of waves can be considered as a fast type of loading and based on the soil response, it can be considered as undrained loading. These loads are quite significant to the foundation system and hence it must be designed to withstand the extreme storm conditions during which a large number of high amplitude load cycles are applied. Piles in groups are used as anchors in the conventional foundation systems for TLP, provided the seabed soil possesses adequate strength. In the weak sub marine soils, very long tension piles are required to provide enough of anchorage. An alternative foundation system called suction anchor or suction caisson is being considered as a potential application for TLP, mooring and other floating structures in soft marine clays. A suction anchor is an inverted top capped hollow cylinder of a fairly large diameter with length to diameter ratio (L/D) of 1.0 to 2.0 embedded into the seabed. Self-weight and differential water pressure created can facilitate the easy installation of this type of anchor into seabed with soft soil formation. This differential water pressure can be created by pumping out the water from the interior of the anchor during installation. Since high ambient water pressure is available, this can be considered as simple concept and ideally suited for deep-water locations.

In offshore complaint structures, load transmitted to the foundation is mainly static during the normal sea state conditions and this is predominantly static pullout applied through tethers or tendons/ cables. However, during rough sea conditions, the superstructure is likely to experience dynamic excitation and consequently, the fluctuating superstructure loads are transmitted to the foundations through the anchor cables. It is suggested that these loading conditions can be simplified in the form of cyclic loading in a laboratory testing. With suitable correction factors like sensitivity and area ratios between model and prototype anchors, the results can be extended to prototype conditions. The combination of static and cyclic axial loads may lead to excessive anchor head displacement leading to pullout failure at cyclic load levels, which are significantly less than the static capacity of anchor. The behaviour of anchors under cyclic loading can be analysed in terms of the degradation in the stiffness and post cyclic capacities. The relevant literature shows that a few investigators have studied the behaviour of suction anchors in marine clay. Finn and Byrne (1972) conducted experimental studies to find out the factors governing the breakout capacity of suction anchors and concluded that the breakout capacity mainly depends on the suction developed beneath the anchor. Dyvik et al. (1993) reported field model cyclic tests on suction anchors simulating the loading conditions typical of TLP in soft clays. In these tests, the influence of cyclic loading histories, loading direction and load eccentricities on the anchor behaviour were brought out. Narasimha Rao and Prasad (1993) reported that even in the case of simple plate anchors embedded in soft clays, the main component of pullout capacity was due to the suction developed underneath the plate. Clukey and Morrison (1995) reported a series of cyclic loading tests conducted in a centrifuge to evaluate the behaviour of suction caissons under cyclic pullout loadings with normally consolidated clay soil conditions, which were typical of Gulf of Mexico. Singh et al. (1996) reported the results obtained from tests conducted on laboratory model super pile anchors in clayey soil and in this investigation, the influence of cyclic load level and number of cycles on the anchor response was brought out. Based on their investigation, it was reported that the anchor displacement was stabilized with in 1000 load cycles when the maximum cyclic stress level was equal to the sum of the weight of the anchor and the soil plug. Based on the results of static pullout load tests on model cylindrical suction anchors, Narasimha Rao et al. (1997) suggested load transfer mechanisms and reported significant breakout resistance in the form of suction induced reverse end bearing. Yamazaki et al. (2003) suggested that the required penetration of this type of anchors can be easily achieved through the application of active suction and this further improves overall pullout capacity of this type of suction anchors. Based on the test results obtained from centrifuge model testing, Allersma (2004) found out that even with cyclic loading at a level corresponding to 80% of static pullout capacity, there was not significant decrease in ultimate bearing capacity. Houlsby and Byrne (2005) brought out the installation procedures for the suction caissons. From the above mentioned literature review, it is clear that there were a few investigations carried out on suction anchor in clay soils under the loading conditions typical of tension leg platform. In order to find out wider field applications, it is preferable to conduct the tests on anchor system closer to the field loading conditions. Even though the main components contributing to the capacity of the anchor can be identified, there are still some uncertainties involved in the prediction of the capacities.

If the water depths are quite large and poor sub soil conditions like soft clay of large depths, some innovative types of offshore structures have to be conceived and planned. One such popular offshore area with deeper water is

Mississippi delta area and Mexican Gulf. One typical soil profile from the Gulf of Mexico is indicated in Figure.5. From these profiles, it can be seen that this can reveal a typical normally consolidated or under consolidated deposits. The in-situ undrained shear strength is about 10-15kpa in the depth range of 10 to 15 m below the seabed. Even at a depth of 40m this shear strength is between 30-40 kpa. From the nature of sediments deposited, it is quite clear that the most of the sediments are originated from the river born activities and transported into the sea. According to rough estimate (Poulos, 1988), these river born deposits contribute about 20 billion tonnes each year and most of it coming from Asia.

In the Asian regions, the vast distribution of soft marine clays in the seabed is quite a common feature. Along the east coast of India in Bay of Bengal, soft marine clays, stretching over a distance of 100-150 km from the coast are found with strength between 10 to 15 kpa even upto a depth of 60 to 70m below the seabed. And in these areas the water depths are between 200 to 300 m. In view of this the presence of the weak soil extending to large depths and because of water depths, it beyond the scope of several construction agencies to build even jacket type of structures, which were vogue in Bombay High. Under these circumstances one has to look for alternative type of platform if there is a large scope for exploitation of resources like hydrocarbons. During recent exploration carried out in Bay of Bengal by our Indian Premier organization like ONGC, which had met with big success in locating large reserves of hydrocarbons. ONGC has been striving hard to develop deep-water technology and it is contemplating to deploy many offshore platforms in one area namely Krishna-Godavari basin (K-G Basin). Very recently, for one of the project carried out for ONGC by IIT Madras in the K-G Basin a typical soil profile is indicated in Figure.6. The water depth at this location is over 200 m. The measured strength in the top 10m is 4 kpa. Between 10 and 20m depth the variation in strength is between 4 and 15 kpa. Only from the depth of 35m onwards the strength varies between 25 and 60 kpa. All these clearly indicate the problems involved in the construction of conventional type of platform are many.

Installation and uses of Suction Anchors

The most advanced preset mooring system for ultra-deepwater is the suction anchor system. Typical suction anchor foundations used for Tension Leg Platforms are shown in Figures.3 and 4. This system uses a suction caisson as the anchor. The caisson is cylindrical in shape with the bottom end open. Suction anchors, a relatively new type of mooring system for drilling rigs and semi-submersible production platforms. The anchors are large, hollow cylinders (in this case 12m high and 5m diameter), which are closed at the top. They are installed on the sea floor by pumping out the water inside the cylinders, which sucks them to the sea bottom. The anchor chains of large vessels or rigs are secured to them by the pad eyes, which must be strong enough to bear the forces put upon them.

Mechanisms Involved In Load Transfer of Suction Anchors

If caissons with closed top are used as suction anchors and when these are subjected to pullout moment, there can be significant breakout resistance through the development of negative pore water pressure. The suction developed at the top helps in the formation of soil plug and this adds to the self-weight of the anchor and thus enhances the capacity of the anchor. In the recent times, Albert et al. (1989) proposed suction anchors for anchoring Tension Leg Platform and other compliant structures in deeper waters.

Following this, Steensen Bach (1992) has proposed that the failure mechanisms are similar to reverse end-bearing capacity and also supported the plug formation. Following this approach, a Programme of model testing was initiated at IIT Madras, Chennai (Narasimha Rao et al., 1996, 1997). In fact, the work of Yamazaki et al. (2003) brought out some encouraging features in the installation and in this paper it was brought out that the application of the active suction helps in the required penetration of suction anchor.

These components are the reverse end bearing resistance (R_b), external skin friction (F_{ext}), weight of the soil plug (W_s), weight of the anchor (W_a), weight of ballast (if any) (Singh et al, 1996). Of these, the weight of the soil plug (W_s) and the reverse end bearing resistance (R_b) are important components to be considered in the design. The short term break out capacity of these embedded anchors can be determined by the use of plasticity theory assuming general shear failure as suggested by Steensen Bach (1992). In the present investigation carried out on anchors installed under soft clay conditions, it is quite likely that the pattern of plastic deformation corresponds to a reverse end bearing capacity failure and is similar to a bearing capacity failure beneath an embedded shallow footing, but the direction is reversed. Only in limited cases with anchors embedded in clay bed formed at medium stiff consistency, anchors failed under tension at anchor bottom much before the full mobilization of reverse end bearing takes place. However, in all the types of breakout failure of suction anchor, the plug formation is confirmed.

Adhesion Factor (α)

The soil structure interaction factor called adhesion factor, α has to be evaluated properly. This factor can be conveniently evaluated considering the conditions corresponding to open top anchor. Figure.20 shows the vertical force equilibrium of open top anchor.

In the estimation of the total pullout capacity, there are several contributing components suggested as shown in Figure.20. Out of these, skin friction is quite important and this component is a

$$P_u = F_{int} + F_{ext} + W_a \tag{9}$$

where, F_{int} = Internal Skin Friction
F_{ext} = External Skin Friction
W_a = weight of open top anchor

Internal and External skin friction values can be obtained by multiplying the unit adhesion with the area of the circumference of the anchor inside and out side respectively as given below.

$$F_{int} = \alpha C_u A_{si} \tag{9a}$$

$$F_{ext} = \alpha C_u A_{se} \tag{9b}$$

where, A_{si} = Area for the internal skin friction
A_{se} = Area for the external skin friction

Hence,

$$P_u = \alpha C_u (A_{si} + A_{se}) + W_a \tag{9c}$$

From the equilibrium equation of the pullout tests of anchors with open-top (suction eliminated) with local shear failure along the anchor wall, α values can be obtained from the results to be obtained from a suitable testing.

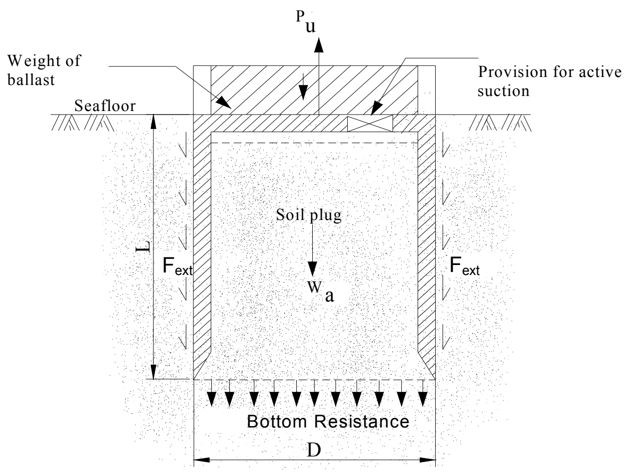

Fig. 20 Forces on Suction Anchors

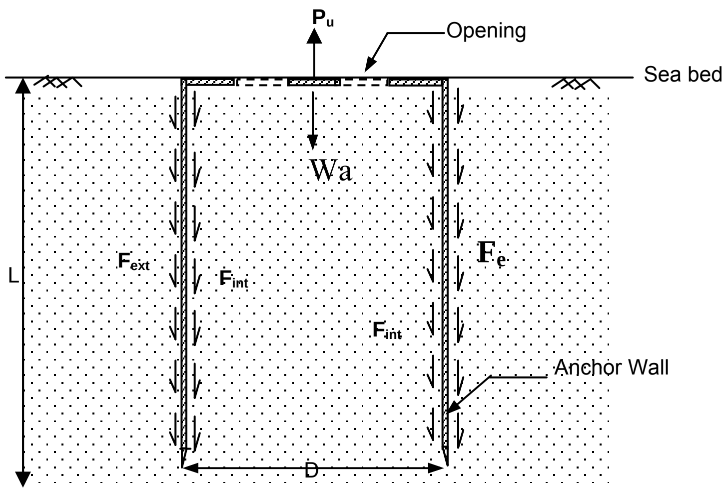

Fig. 21 Forces on Soil Plug

Suction break out factors

From the vertical force equilibrium considerations of suction anchors (referring Figure.21), the ultimate pullout capacity of the suction anchor is given by

$$P_u = W_a + F_{ext} + W_s + R_b \qquad (10)$$

The suction induced end bearing resistance (R_b) can be expressed in a non dimensional form as a bottom break out factor called suction break out factor (N_b). From the consideration of rupture in clay under tensile loading (Vesic, 1971), the bottom break out resistance is expressed in a non dimensional form as given below.

$$R_b = N_b C_u A_b \tag{11}$$

Then, the break out capacity of the suction anchor can be written as

$$P_u = W_a + \alpha C_u A_{se} + W_s + N_b C_u A_b \tag{12}$$

where, A_b is base area of the anchor at bottom.

The bearing capacity factor N_b can then be computed from the measured breakout force at the time of failure in each test.

Break out Factor from Overall and Plug Equilibrium Considerations

The overall equilibrium of the suction anchor and the equilibrium of the soil plug are shown in Figure.22. Referring to the Figure.22a, the overall equilibrium equation of the suction anchor can be written as

$$P_u = W_a + W_s + \alpha C_u A_{se} + R_{b1} \tag{13}$$

From Eq.13 the end bearing resistance can be written as

$$R_{b1} = P_u - \left(W_a + W_s + \alpha C_u A_{se}\right) \tag{14}$$

The non dimensional break out factor can be written as

$$N_{b1} = \frac{R_{b1}}{C_u A_b} \tag{15}$$

The breakout factor value obtained from the overall equilibrium of the anchor is named as N_{b1}. Further, from the considerations of the plug equilibrium (refer to Figure.22b), the equilibrium equation can be written as

$$R_{b2} + W_s = p_{st} A_b + F_{\text{int}} \tag{16}$$

Where, p_{st} is suction pressure measured at the top of soil plug,

$$R_{b2} = p_{st} A_b + F_{\text{int}} - W_s \tag{16a}$$

$$R_{b2} = N_{b2} C_u A_b$$

$$N_{b2} = \frac{R_{b2}}{C_u A_b} \tag{16b}$$

where, N_{b1} and N_{b2} are bottom breakout factors from overall equilibrium and plug equilibrium considerations and F_{int} is internal skin friction. From the equations (15) and (16b) suction breakout factors N_{b1}, N_{b2} can be evaluated.

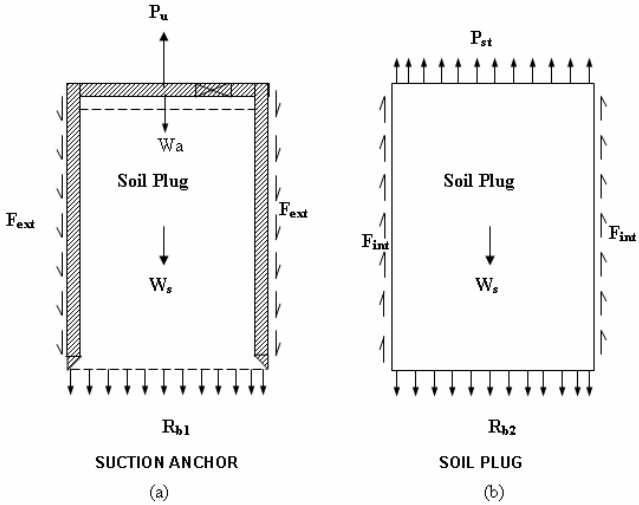

Fig. 22 Overall Equilibrium and Plug Equilibrium of Suction Anchor

Breakout Factors from Suction at Anchor bottom

The suction pressure variation at the bottom is a measure of the end bearing resistance mobilized. Based on the results obtained from centrifuge testing, Clukey et al. (1993) confirmed that the dominant portion, 60 to 70% of the uplift capacity is from the reverse end bearing or suction at the bottom of the anchor. From the mobilized peak suction pressure (p_{sb}) at anchor bottom, the end bearing resistance can be expressed as

$$R_{b3} = p_{sb} A_b \tag{17}$$

$$N_{b3} = \frac{R_{b3}}{C_u A_b} = \frac{p_{sb}}{C_u} \tag{18}$$

Breakout Capacity of Suction Anchor

Finn and Byrne (1972) conducted model studies to understand the factors governing the breakout capacity of suction anchors and reported that the breakout capacity was mainly dependent on the suction developed beneath the anchor and suggested that an upper limit could be fixed by assuming a failure mechanism similar to a classical bearing capacity approach based on limit equilibrium considerations. However, considering that the loading direction is just the opposite, this can be treated as reverse end bearing. From compressive end bearing consideration, slip lines and failure surfaces for the reverse-end bearing of a footing under breakout loading as suggested by Finn and Byrne, (1972) is given in Figure.23. Accordingly, the breakout capacity can be calculated based on the compressive end bearing considerations

$$P_u = \left(C_u N_c - \gamma' d\right) A + W' \tag{19}$$

where, Nc= Breakout factor, α' = Unit weight of soil, d= Depth of the footing and W'= Weight of anchor footing.

In the case of suction anchor, if the plug formation is confirmed, the breakout equation is modified to

$$P_u = \left(C_u N_c - \gamma' L N_q\right) A_b + F_{ext} + W_a + W_s \tag{20}$$

Based on this analysis for undrained condition (Φ=0-analysis), the breakout factor reduces to N_c = 6.2 for cylindrical shape and N_q = 1.0. Substituting these values, the breakout equation is reduced to

$$P_u = \left[C_u N_c A_b\right] + F_{ext} + W_a \tag{21}$$

In the above equation, the first component is the reverse end bearing resistance (R_{bc}). These factors are originally deduced from the conventional end bearing considerations under compressive loading. For soft clays in compressive loading, the soil is normally subjected to positive pore pressure changes. In the case of pullout, the pore pressure in the soil is reduced below the atmospheric pressure leading to a negative pressure or a suction state. In view of this, a correction factor has to be applied to the part of the total uplift capacity resisted by the reverse end bearing as obtained from the compressive end bearing considerations. For the purpose of obtaining a correction factor, the results of the test data can be compared with the calculated values. The correction factor to be applied is termed as "suction efficiency factor" which is obtained using the test results as the ratio of the measured end bearing to the calculated end bearing resistance.

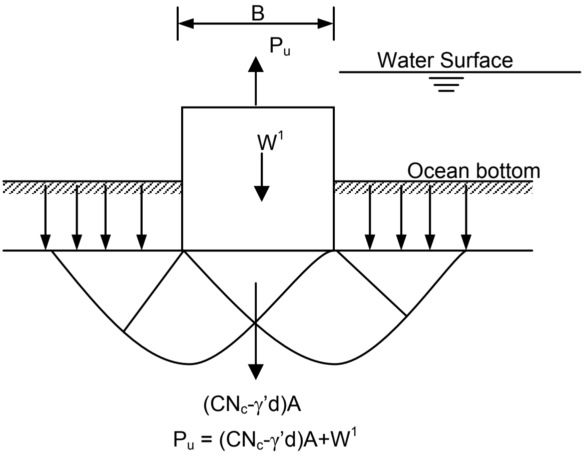

Fig. 23 Concept of End Bearing in Suction Anchor

Suction Efficiency Factor (S_f)

The suction efficiency factor (S_f) = Measured reverse-end bearing/ calculated reverse end bearing. Calculated end bearing (R_{bc}) = Calculated total capacity- F_{ext}+W_a+W_s (22)

The reverse end bearing has been measured from suction pressure measured (R_{bc}) at the anchor bottom and from the overall equilibrium considerations of the suction anchor (R_{b1}).

From the measured quantity of R_{b1} from the overall anchor equilibrium, suction efficiency factor (S_f) is obtained and is referred here as S_{f1}.

$$S_{f1} = \frac{R_{b1}}{R_{bc}} \tag{23}$$

From the reverse end bearing as obtained from measured suction at anchor bottom, S_f is obtained and termed as S_{f2}.

$$S_{f2} = \frac{R_{b3}}{R_{bc}} \tag{24}$$

Suction efficiency factors obtained from both the cases are in the same range and hence the average value is taken as

$$S_f = \frac{S_{f1} + S_{f2}}{2} \tag{25}$$

The breakout capacity equation of suction anchor is suggested as

$$P_u = S_f \left(C_u N_c A_b\right) + F_{ext} + W_a + \mathrm{Ws} \tag{26}$$

Hence, in this investigation, an experimental programme has been drawn up to study the behaviour of model suction anchors under both static monotonic and stress- controlled cyclic loading conditions in a typical marine clay. The objectives are

I. to establish the static pullout capacity
II. to bring out the influence of cyclic loading on the pullout behaviour
III. to relate the variation in passive suction at the top of the soil plug and at the suction anchor bottom with breakout capacity under cyclic loading
IV. to analyse the various components contributing to the pullout capacity of the suction anchor and to formulate a breakout capacity equation accounting for the suction mechanism.

A brief description of the experimental work carried out is presented in the next section.

Experimental Work

Model Anchors fabricated

Laboratory tests were conducted on model anchors of 75mm diameter and 3 mm skirt wall thickness and this can be considered as a 1:100 scale model in comparison with the prototype size anchor used for the snore TLP (Andersen et al., 1992). A steel hook rod welded to the centre of the anchor top cap was used to connect it to the loading system. On either side of the central hook, threaded holes were made to fix two pressure transducers for measuring the suction pressure developed at the top of soil plug.

Soil Used

In the present investigation, a typical marine clay obtained from a coastal deposit on the east coast of India was used. The liquid limit of the soil is 82% and the plastic limit is 32%. This soil is made up of 48% clay, 41% of silt and 11% fine sand fractions.

Test Set Up

The schematic diagram of the test set up is shown in Figure.24 and in this, a provision has been made for a stress - controlled loading. The cyclic loading to the anchor was applied with the help of pneumatic loading system controlled by a double acting pneumatic power cylinder. Similar setup was used for cyclic load tests on screw anchors. The desired maximum cyclic loading was varied from 25% to 90% of the static capacity of the anchor.

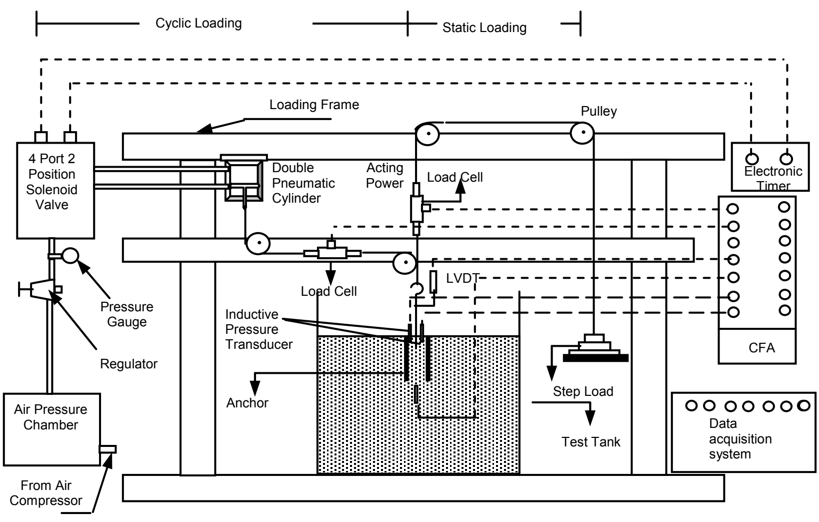

Fig. 24 The Schematic Diagram of the Test Set up

Testing Procedure

After the clay bed had been formed, the pore pressure transducer to measure the pore water pressure was placed with in the soil mass at the anchor bottom. Before placing the transducer inside the soil mass, a rubber membrane was used to cover it safely exposing the diaphragm zone. Then the anchor was installed slowly by pushing the skirt wall vertically into soil ensuring a proper contact of soil mass at both the exterior and interior faces of the anchor walls.

The pore pressure transducer, to measure the suction at the top of the soil plug, was screwed on to the top cap of the anchor in an airtight condition with Teflon lining sealing fitting into the threads.

Test Programme

In cyclic load tests, all the tests were conducted in three stages: a) static monotonic pull out tests b) cyclic tests c) post cyclic- static test. In cyclic load test, the load on the anchor was varied from zero to the desired load level expressed as cyclic load ratio CLR, (cyclic load amplitude/ultimate pullout capacity under static load conditions). In order to bring out the induced effects of cyclic loading on the ultimate static capacity of the anchors, these anchors were initially subjected to certain number of cycles at constant load amplitude conforming to different CLR's in Phase I and at the end of these cycles of loading, the load on the anchor was reduced to zero and then, static monotonic pullout tests were conducted till the anchor came out of soil bed. Generally, in phase 1 under cyclic loading, the loading was continued until the stabilization in displacement was reached and then only static monotonic pullout test was conducted. The number of cycles necessary to reach this stabilization in displacement varies depending on the load level. However, at a very high CLR, there was no stabilization in the displacements and in such cases, the initial cyclic loading was stopped when once an abrupt increase in the displacement was noticed and thereafter the anchors were unloaded to zero load and subjected to post cyclic static pullout tests. Parameters varied in the cyclic loading test programme include. Cyclic Load ratios (CLR): 25%, 50%, 65%, 80% and 90%. The details of the load tests conducted on suction anchors for the various conditions are given in Table 1.

TABLE 1: Test Programme

Anchor Type	Embedment Ratio (L/D)	Liquidity Index (LI)	Cyclic Load Ratio (CLR %)
Suction Anchor	2	0.68	25
			50
			65
			78
			89
	2	0.42	25
			50
			65
			80
			90

Results and Discussion

The performance of suction anchors under cyclic loading is assessed either through the pullout capacities measured or through the anchor displacements observed. The suction pressure measured both at the top and bottom of the soil plug can also be used in the analysis.

Behaviour of suction Anchor under static pullout

A typical set of pullout load-displacement curves obtained from tests on model suction anchors of different L/D ratios tested as surface anchors is shown in Figure. 25. These are the values obtained from tests on anchors embedded in clay bed formed at a consistency index value of 0.4. From these curves, three distinct phases in pullout behaviour could be observed. In phase I, at the initial stages of deformation up to 2.0mm (2.75% of the diameter), there is a steep rise in the pullout resistance. This is followed by phase II, in which there is a gradual and slow increase in the pullout resistance with deformation. However, in this phase the deformation is quite large. In the last (Phase III), there is a reverse curve exhibiting a sudden increase in pullout resistance, which is followed by sudden pullout of the anchor. Further, it is to be noted that in all these tests the soil remained intact inside the anchor, and this confirmed the formation of the soil plug which increased the pullout capacity.

For any load test conducted on this type of cylindrical anchor or pile, the first component to become to important is skin friction, which needs a very small deformation. From the load deformation presented in Figure.25, it is obvious that there is s a steep rise in capacity even at a deformation of less than 1.5mm (2% of the diameter). This indicates that skin friction predominates in this phase As the plug forms, the skin friction mobilized is only external skin friction. Phase I is followed by a large deformation, and at a stage, a failure wedge is formed as shown in Figure.26. This type of failure has been observed by Steensen-Bach (1992) and is called reverse end bearing. This phase lasts to deformations of as much as 12mm (16% of the anchor diameter). Beyond this stage, the tensile strength of the soil at the anchor bottom mobilizes. When the pullout force exceeds the load capacity of these three components, there can be rupture in the soil mass at the base of the anchor and the anchor is pulled out clear out of the base. A shallow crater formation can be seen at the location of the anchor after the anchor is pulled out. Extension of this crater to deeper levels depends on the type of anchor and is discussed in a later section.

Behaviour of Suction Anchor under cyclic Loading

The influence of cyclic loading on any foundation system can be realized in terms of the changes in the displacements. Any increase in the displacements can lead to significant amount of settlements / heave in the supporting structure. There is a necessity to establish the safe limits of cyclic load levels and estimate the number of cycles required to cause failure of the structure.

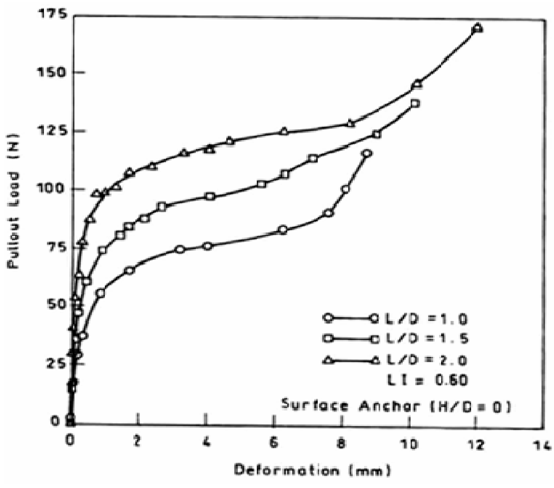

Fig. 25 Typical Pullout Load Displacement curves of suction Anchor, I_c =0.45

Pullout Behavior of Suction Anchors

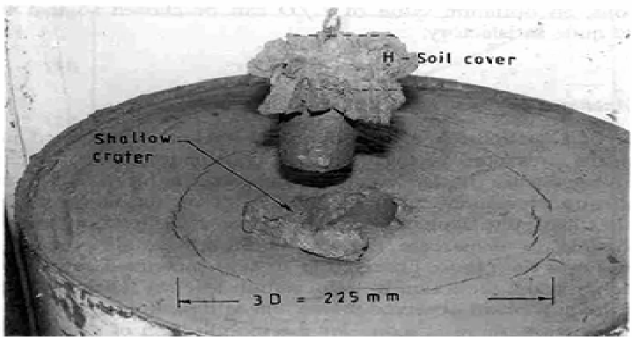

Fig. 26 Photograph of failure Wedge

Influence of Cyclic Load Ratios (CLR) and the number of Cycles (N) on Anchor Displacement

The variations in the anchor displacements with number of cycles for different cyclic load ratios (CLR) are shown in Figure.27. These are the results obtained from tests on anchors embedded in soil formation at LI=0.68 and L/D=2. At low CLRs (i.e. upto 50%) the displacements are not significant and the stabilization in displacement can be noticed at very low number of load cycles, i.e. at 50 cycles. Even at a moderate load level corresponding to CLR=65 %, the anchor displacements tend to stabilize at load cycles of less than 100. Further, the maximum displacement is less than 6 mm (8% anchor diameter) and this is considered to be within the safe limits. With high CLRs (80% and 90%), there is a progressive increase in the anchor displacement leading to a complete pullout failure of the anchor and at a CLR of 80%, there are no signs of stabilization in deformation.

The results in Figure.27 indicate that the permanent displacements increase with the increase in the cyclic load level, as expected. It is better to express these cyclic load levels as fractions of the ultimate capacity or pullout capacity of anchors under static loading. The factors contributing to the pullout capacity of anchors are conceptually shown in Figure.28. From this, the pullout capacity can be expressed as

$$P_u = W_a + F_{ext} + W_s + W_b + R_b \qquad (27)$$

where

W_a =Weight of the anchor
W_s= weight of the soil plug
R_b= the suction induced reverse end- bearing
W_b= Weight of the ballast, if any.
F_{ext}= External skin friction

It has already been stated that cyclic load levels are taken as fractions of static pullout capacities obtained at these two consistency values and the reference pullout load- displacement curves are presented in Figure.27. Corresponding to these consistency conditions, the weights, $W = (W_a+W_s)$ are 17.15 and 18.5 N. From the undrained strength, cu values, skin friction values are evaluated and using the measured pull out capacities, it is possible to estimate these components.

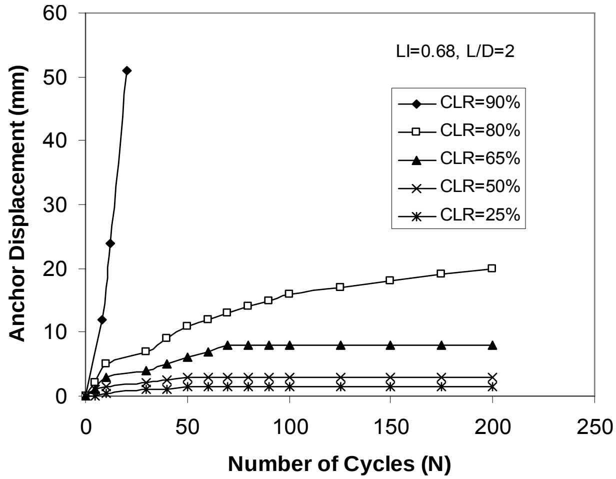

Fig. 27 Variation in Anchor displacement with Number of cycles for different CLRs

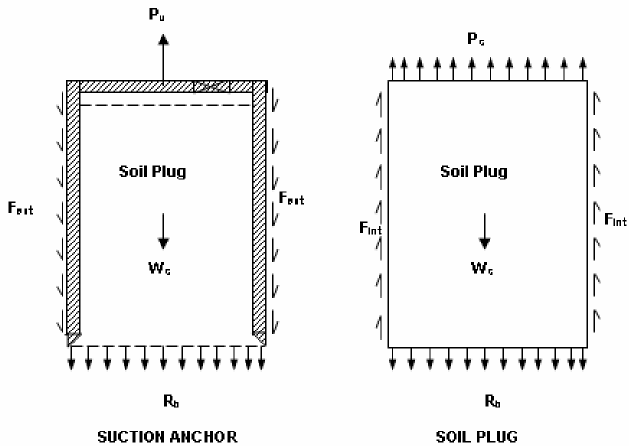

Fig. 28 Overall Equilibrium and Plug Equilibrium of the Suction Anchor

In view of this, CLRs applied are expressed quantitatively in terms of the weight of the anchor and soil plug (W_a+ W_s), the external skin friction mobilized (F_{ext}) and the reverse end bearing resistance (R_b). These values are given in Table 2. Cyclic load levels is found to be a fraction of the pullout capacity under static monotonic loading and at this level, a few components are assumed to be fully mobilized and in certain other components, a fraction is mobilized as indicated. At low CLRs, since the cyclic load is equilibrated mostly with W_a+ W_s and F_{ext} the suction induced components of the end bearing resistance (R_b) is yet to be mobilized. Hence, it is observed that for all the cyclic tests when the CLR $\leq$ W_a+ W_s and for CLR = W_a+ W_s + a part of F_{ext}, the anchor displacement tends to stabilize with in 100 cycles and a near zero-rate of increase of displacement per cycle has been observed. At CLR=25%, which is less that W_a+ W_s, it is mostly the weight balance and hence, there is very little scope for the anchor to be pushed up. At moderate CLR, say CLR=50%, the external skin friction is required to be mobilized and this joins (W_a+ W_s) in resisting the pullout. From the results reported (Narasimha Rao et al., 1997), under static loading, at a load equal to ¼ to ½ of the pullout capacity, the skin friction mobilized is approximately 50% of the ultimate skin friction. As the cyclic load level changes from 25% to 50% in all probability, skin friction gets exhausted. As the load level changes to 75% and above, the final component, i.e. the reverse end bearing also gets mobilized and at this stage, the anchor movements are on the increase and there can be significant permanent movements in the anchor. At higher CLR i.e. at cyclic load ratio of exceeding 80%, (corresponding to static component > W_a+ W_s+ F_s+50% of end bearing), the anchor experienced a progressive upward movement to failure with in the first few cycle. From these results, the critical cyclic load level can be expressed in terms of the breakout components. It is the maximum CLR beyond which the anchor can fail under cyclic loading with progressive upward movement of the anchor. This critical cyclic load ratio is observed to lie between W_a + W_s + F_{ext} + 55% of R_b and W_a+ W_s +F_{ext}+60% of R_{b1} and corresponding to this, the critical equivalent CLR can be taken as 80%. This load level appears to be slightly on the higher side, but such values can be expected in this type of soft clays. It is to be noted that this critical load level obtained is for this soil tested under soft clay conditions. For similar type of soft clays, it is possible to establish such critical load levels.

TABLE 2: Cyclic Load Ratio Expressed as Breakout Components of Suction Anchor
(P_u = Ultimate Pullout Load; F_{ext} = External Skin Friction;
R_{b1} = External Bearing Resistance)

L/D	LI	P_u (N)	F_{ext} (N)	R_{b1} (N)	Cyclic Load Ratio (CLR) Max
2.0	0.42	210	69	122	25%=W+50%F_{ext}
					50%=W+F_{ext}+10%R_{b1}
					65%=W+F_{ext}+30%R_{b1}
					80%=W+F_{ext}+60%R_{b1}
					90%=W+F_{ext}+80%R_{b1}
2.0	0.68	109	45.86	45.44	25%=W+25%F_{ext}
					50%=W+F_{ext}
					65%=W+F_{ext}+20%R_{b1}
					80%=W+F_{ext}+55%R_{b1}
					90%=W+F_{ext}+80%R_{b1}

Variation in Suction Pressure at Anchor Bottom in Suction Anchor under Cyclic Loading

The variation in suction pressure at anchor bottom with number of cycles is shown in Figure.29. At all the cyclic load ratios, the suction pressure increases with number of cycles and peak pressure is attained within 25 cycles. At cyclic load ratios even upto 80%, the suction pressure almost remains constant throughout the test after reaching the peak value. However, at high CLRs, say 90%, the suction pressure shoots up to a very high value at a very few cycles (of about 10-15) and drops down rapidly. At this point of peak suction, it is expected that there is a high pore pressure gradient developed at the bottom and consequently, the system can attract the additional moisture from the surrounding one. This sudden rush of moisture can destroy the suction developed and this naturally leads to the yielding in the soil mass. For the tests conducted at higher CLR, there is a good amount of water collected in the crater formed at the time of failure and no such accumulation of water was seen for the tests conducted at lower and moderate CLRs. From the results presented in earlier Figure.27, at these cycles, deformations are quite large and hence, it can also be stated that the suction pressure variation is a function of anchor displacement. It could be observed that there is a good correlation between anchor displacement and development of suction pressure.

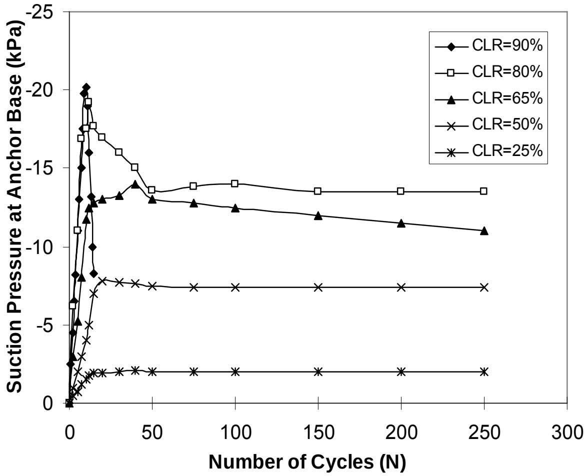

Fig. 29 Suction Pressure Variation at the Anchor Base

Suction pressure variation at the top of soil plug

The variation in suction pressure at the top of soil plug with number of cycles is shown in Figure.30 From this Figure.30, it could be seen that at all the CLRs, the suction pressure increases with number of cycles and attains its peak value in the first few cycles and then it remains constant throughout the test at all the CLR except at CLR of 90% where anchor is pulled out within 15 cycles. In case of highest CLR of 90%, the pressure amplitude is shooting up till failure and the anchor is pulled out with the soil plug in contact. If the suction pressure can be sustained without dissipation, a good pullout capacity can be assured. With large water depths, there should not be any difficulty in assuming that this suction depend component contributes to the pullout capacity in low permeable clays.

Based on the above results, it is possible to develop suction anchors to withstand the environmental pullout loads. For an assumed prototype conditions in one of the typical offshore situation, the pullout capacity of suction caisson/anchor can be worked out.

Conclusions

In some of the offshore areas located along the Indian coast there are thick soft deposits with deeper water columns. Because of poor subsoil conditions and deeper water columns, there may be a necessity of constructing compliant structures with special types of anchor foundations.

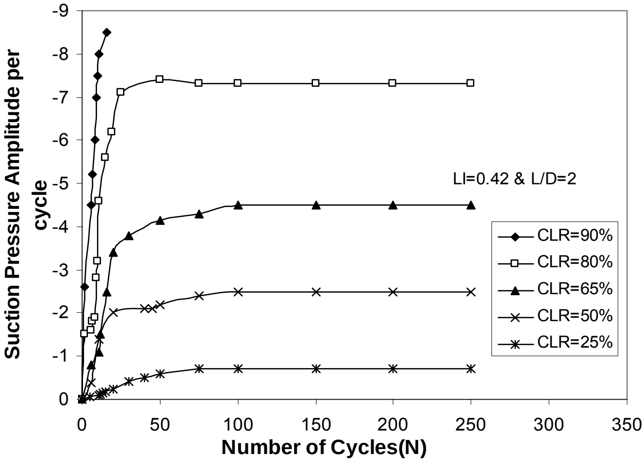

Fig.30 Suction Pressure Variation at top of the soil plug

Screw piles and suction anchors can be proposed to support many marine structures like compliant structures, pipe lines, mooring systems, data buoys etc. Screw piles can be conveniently installed in underwater conditions and there are formulation suggested to estimate the pullout capacities in terms of anchor dimensions, spacing ratios of anchor plates and soil properties. An attempt has been made to estimate the lateral load carrying capacities. For the screw anchors installed in soft clays and tested under cyclic loading conditions, there can be deterioration in the capacities if the cyclic load levels exceed 70-80% of static capacities. The Anchors can be classified as shallow and deep anchors. The soft consistency of Indian Marine soils encourages the development of screw piles.

The studies conducted on suction anchors show a good promise for the use of this type of suction anchors for Indian Offshore areas with large water columns. The active suction applied at the top of the closed caisson encourages the easy penetration of anchor during installation. The pullout capacity is found to be a function of various factors like skin friction, weight of soil plug, suction dependent components and reverse end bearing. Procedures are suggested to estimate the various components.

Tests conducted under cyclic conditions bring out some interesting conclusions. Suction developed at the top and bottom of soil plug increase the uplift capacity of this type of anchors.

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