THE BEHAVIOUR OF MODEL SCREW PILES IN COHESIVE SOILS

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ABSTRACT

Anchors are being extensively used in foundations for the structures subjected to both compressive and tensile forces. An attempt has been made to develop screw piles as anchors in clayey soils. The screw piles are made of hollow galvanized iron pipes (dia. 25 mm, 44 mm, 60 mm and length 640 mm and 1000 mm) to which helical plates (dia. 75 mm, 100 mm, 150 mm) are welded. Number of plates are varied from 2 to 4 and there are different spacings of these plates adopted. These piles are screwed into soft to medium stiff clay bed prepared in a test tank. Standard type of loading frames have been made for effecting both compressive and tensile forces. In all about 40 Nos. of tests have been conducted in three soils. The behaviour of these piles has been studied through the load deformation curves. Decrease in the moisture contents leads to significant increases in the capacity of the piles. By keeping the ratio of the spacing of the helical plates to diameter of the helical plates between 1.0 and 1.5, optimum capacities of the piles could be obtained. The test results indicate that theoretical predictions for the load carrying capacities can be made by making use of the measured shear strength of the soils.

Key words: anchor, cohesive soils, compression, moisture content, screw pile, tension (IGC : E4)

INTRODUCTION

Many a types of structures founded on land and offshore are subjected to both compressive and tensile forces. Some of the typical structures subjected to tensile forces on land are high rise buildings, chimney towers, and transmission towers. In case of offshore structures, wave forces can produce enormous tensile forces in the foundation system. Besides these,

foundations located below the ground water table are subjected to significant uplift forces due to subsoil water inducing tensile forces. In such situations, ground anchors in one form or the other have been used. In this paper, an attempt has been made to use screw piles as anchors. These screw piles can be of simple steel or concrete shafts to which one or more sets of helical plates are attached. These piles can be screwed into the ground

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by applying torque to the upper end of the shaft and it is a common practice to keep the diameter of the helical plate at about 2 to 3 times the diameter of the shaft. The first application of such type of screw pile was made for the foundations of Maplin Sands light house in the Thames estuary in 1838. Subsequently, these screw piles were used in Nineteen forties for the foundations of wharfs and jetties. In the recent times, these have been conveniently used for mast and tower foundations as anchor foundations to resist large uplift forces. A brief literature review on the use of screw piles is presented in the next few paragraphs.

BRIEF LITERATURE REVIEW

A reference made by Alexander Mitchel in 1838 on the use of screw piles for Maplin Sands light house was referred to by Little (1961). This work has been followed up by several others (Wilson, 1950). A very good amount of work has been done on the bearing capacity of screw piles by Wilson (1950). These investigations included model experiments and full scale loading tests on screw piles and cylinders, observations on the settlement of piles in actual structures, development of theory, a check on the relationship between driving torque and the bearing capacity of the piles and the formulation of design rules. A relationship has been developed between penetrative effort and the bearing capacity of a screw pile and the results are found to be encouraging. Later, several investigators (Morgan, 1950; Meyerhof, 1950; Skempton, 1950) came up with suggestions and modifications to Wilson's theory. Meyerhof (1950) suggested the 'theory of plasticity' for determining the bearing capacity of screw piles. Skempton (1950) also suggested formulation to predict the capacity of this type of piles and reported field test results supported the formulations.

In recent years, screw piles are in use for Mast and Tower Foundations in USSR and Canada. In the USSR during 1961~64, many communication masts upto 254 m high are

erected quite economically using these piles. About two hundred piles are tested to develop reliable methods of determining the bearing capacity of screw piles by Trofimenkov and Mariupoiskii (1965). On the basis of the experiments a method of determining the bearing capacity of deep screw piles has been developed and this method takes into account the bearing capacity of soil and displacement. Robinson and Taylor (1969) reported the use of screw piles (i. e. helical anchors) in two guyed transmission lines. In each of these projects extended a length of about 869 km in British Columbia, helical anchors are investigated for possible use in extensive clayey deposits. Some anchors are tested and the resistance to pull out is related to the installing torque. A field testing program has been carried out in northern Manitoba to evaluate the creep behaviour and load capacity of power installed screw anchors embedded in permafrost by Johnston and Ladanyi (1974). The test results indicated that the anchors behaved under uplift loads, in a manner that is essentially very similar to that exhibited by deep footings of the same size. Clemence and Pepe (1984) reported the results from helical anchors installed in sandy soils. A design method for determination of uplift capacity of helical anchors in both sand and clay has been developed from extensive field and laboratory tests by Bobbitt and Clemence (1987). Results of these tests have been compared with previous research and found to be in good agreement.

From the aforementioned review, it is clear that the screw piles are being used in several foundations subjected to large tensile forces. These piles can be considered as light piles and they do not require very heavy drilling rigs. In view of this, in some soil conditions especially in marshy soil areas, where heavy rigs cannot be transported, the light screw piles can be conveniently used. An experimental work using model screw piles has been carried out, the details of which are presented in the next section.

EXPERIMENTAL WORK

Scope of the Investigation: In this experimental study, it has been attempted to develop screw piles suitable for soft and medium stiff clays. The capacity of the piles has been measured for different conditions viz., different placement moisture contents and different sizes with different spacings of helical plates. These have been studied for both compression and pullout conditions.

Model Screw Piles: The investigations are conducted on the screw piles made with galvanized iron pipes welded with helical plates. Two types of screw piles are fabricated and in each of these types, there are two sets of piles. One such typical screw pile is shown in Fig. 1. In the first type, there are 6 Nos. of screw piles; one set of 3 screw piles are made with shaft dia. of 44 mm and in the second set of 3 screw piles are made with shaft dia. of 60 mm. To study the influence of the spacings of helical plates, different number of plates (2, 3 and 4 Nos.) are placed over a length of 457.5 mm. This pattern is same for both the sets in the first type. The plates are made out of 3 mm thick M. S. plates. The diameter of the helical plate is 100 mm in the first set and 150 mm in the second set. The total length of the screw pile is 640 mm. In the second type, totally 5 Nos. of screw piles, one set of three piles and second set of two piles, but with the same shaft diameter of 25 mm are fabricated. To study the influence of the spacing

Fig. 1. Typical screw pile used

of helical plates; in the first set consisting of 3 screw piles are provided with 2 helical plates, 3 helical plates and 4 helical plates respectively and these helical plates are placed over a length of 300 mm. In the second set consisting of 2 screw piles are provided with 2 helical plates and 3 helical plates respectively and these helical plates are placed over a length of 125 mm. The diameter of the helical plate is 75 mm in both the sets. The total length of the screw pile is 1000 mm in the second type. In all these cases, the bottom of the pipe pile is closed. All these details of the screw piles used are given in Table 1.

Table 1. Screw piles fabricated

Sl. No.	Type No.	Set No.	Screw pile designation	Dia. of shaft d (mm)	Dia. of helical plate D (mm)	No. of helical plates	Pitch of helical plate p (mm)	Spacing of helical plates I (mm)	Distance between top and bottom helical plates L_c (mm)	Distance between pile top and top helical plates H_T (mm)	Distance between pile bottom and bottom helical plates H_B (mm)	Total length of screw pile $L = (L_c + H_T + H_B)$ (mm)
1	I	I	P 1	44	100	2	58	457.5	457.5	98.5	84.0	640.0
2	I	I	P 2	44	100	3	58	229.0	457.5	98.5	84.0	640.0
3	I	I	P 3	44	100	4	58	152.5	457.5	98.5	84.0	640.0
4	I	II	P 4	60	150	2	58	457.5	457.5	98.5	84.0	640.0
5	I	II	P 5	60	150	3	58	229.0	457.5	98.5	84.0	640.0
6	I	II	P 6	60	150	4	58	152.5	457.5	98.5	84.0	640.0
7	II	I	P 7	25	75	2	25	300.0	300.0	460.0	240.0	1000.0
8	II	I	P 8	25	75	3	25	150.0	300.0	460.0	240.0	1000.0
9	II	I	P 9	25	75	4	25	100.0	300.0	460.0	240.0	1000.0
10	II	II	P10	25	75	2	25	125.0	125.0	575.0	300.0	1000.0
11	II	II	P11	25	75	3	25	62.5	125.0	575.0	300.0	1000.0

General Experimental Set-up : Figs. 2 and 3 show the general experimental set-ups for compression test and pull out test respectively. In the compression load test set-up, a hollow metallic spacer tube has been placed between pile top and proving ring bottom to adjust the height. The details of the experimental set-up are presented in the following paragraphs.

Test Tank : The test tank used is of inner dimensions $800\text{ mm} \times 800\text{ mm} \times 1300\text{ mm}$ and is made out of 5 mm thick M. S. plates. The steel plates used for the test tank are sufficiently thick and are rigid enough. From the literature review it is clear that many investigators (Meyerhof and Sastry, 1978 ; Bowles, 1982) established that at the tip of pile, the failure zone extends to a radial distance of about 1.5 times the diameter or width of pile on either side of the pile and the total width of failure zone is about 3 to 4 times the diameter. In a similar way for skin friction mobilization, the zone of influence can be considered over a radial distance of 0.2 times the diameter from the pile surface (Zeevaert, 1983). At the tip of the pile the failure zone extends over a depth of approximately 2 times the diameter. Based on the above consideration, the size of the tank is considerably large enough and is not likely to interfere with the failure zones or wedges.

Loading Arrangement for Compression Test : The loading has been done with the standard type of hydraulic jack of 15 kN capacity. The maximum travel of ram is about 150 mm and the jack is positioned as shown in the Fig. 2 and enough care has been taken to see that there is no eccentricity in loading. The load reaction has been measured with the help of a standard proving ring of 20 kN capacity with an accuracy of 5 N. The pile settlement has

Fig. 2. Experimental set-up for compression test

been measured with the help of 2 Nos. of dial gauges of least count 0.01 mm placed with the help of datum bars.

Loading Arrangement for Pull-out Test : For tension test, the arrangement made has been shown in the Fig. 3. The pull-out load has been affected with the help of wire rope passing over two pulleys and the wire rope has been subjected to tension with the help of cast iron weights. The upward movement of the pile has been measured with the help of 2 dial gauges.

Soil used for the Test : Totally 3 different types of clayey soils are used for the testing and their properties are given in Table 2.

TESTING PROGRAMME AND PROCEDURE

(a) First type of screw piles : Both compression and tension tests are conducted on all the first

Table 2. Soils-index

Soil designation	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Soil 1	75	25	50
Soil 2	38	16	22
Soil 3	65	23	42

moisture contents of 40.2%, 45.2% and 50.4%. In the case of second set of piles these tests are conducted at only one moisture content i. e. 45.2% (in soil 1).

(b) Second type of piles: Both compression and tension tests are conducted on all the first and second set of piles in the soil 2 at a moisture content of 26%. Pull-out tests are conducted on the first and second set of piles in the soil 3 at a moisture content of 35%.

Method of filling up the Tank: The soil has been placed in layers of 50 mm thickness. Because of high moisture content (conforming either to soft or medium stiff consistency), there is no difficulty in placing the soil as per this procedure and fairly homogeneous soil deposit could be formed around the pile. After the clay bed has been formed, screw pile has been slowly screwed into the desired position by applying torque to the upper end of the shaft. In order to equilibrate the pore water pressure developed, it has been waited for 2 days before the load test has been taken up. The test tank has been covered with polythene sheet to prevent the moisture content variation. At random, a few samples are taken out and checked for full saturation, homogeneity and for pore pressures. All the testing has been carried out with the clayey bed placed at moisture contents of 26% to 50.4% and these moisture contents are sufficiently high and as such there is no difficulty in ensuring a homogeneous and uniformly compacted fill. The number of moisture content and density measurements are taken to confirm the homogeneity of soil layer placed. For this, the degree of saturation works out to be between 97% and 98%. Further, a number of samples are taken out and pore water pressure are measured in triaxial apparatus fitted with pore water pressure transducers. These measurements gave skempton's pore pressure parameter, B of about 0.98 to 0.99 indicating a full saturation as per Bishop and Henkel (1962).

Procedure used in Conducting Compression Test: The test load is applied on a pile head by means of a hydraulic jack at an increment of 100 N and is measured using a proving

Fig. 3. Experimental set-up for pull-out test

ring. Each stage of loading is maintained until the rate of movement of pile is not more than 0.02 mm/hr. The settlement observations are taken from the dial gauge readings. The loadings in stages has been continued until the settlement of pile reached a value approximately equal to 10% of the diameter of the helical plate. As per the relevant code of practice for pile load tests, the safe compressive load on the pile shall be (a) 50% of the final load at which the total settlement is equal to 10% of the pile diameter in the case of uniform piles and (b) 50% of the final load at which the total settlement is 7.5% of the bulb diameter in the case of under-reamed piles. So in this case, the screw pile can be considered as equal to under-reamed piles and accordingly the ultimate loads are calculated from the settlement equal to 7.5% of helical plate diameter.

Procedure Used in Conducting Pull-out Test; In the set-up shown in the Fig. 3, the cast iron weights are used as the loads. Loads are continuously placed on the pan and the upward movement of piles are noted down with the help of dial gauges. Loads are increased until the pile comes out from the soil bed. The load at which pile came out suddenly is considered as the pull out load or uplift resistance of screw pile.

Measurement of in-situ Shear Strength: All the tests are conducted on soft to medium stiff clayey soil ($I_c = 0.5$ to 0.7). The in situ strength has been measured with the help of standard in situ vane shear apparatus. This

apparatus consists of a torque measuring system and is attached to a rod which can be rotated by gear and worm wheel arrangement. Vane blades of 37.5 mm diameter and 75 mm high are welded to the bottom end of the torque rod of 1.6 m length. The rotation of the blades shears the soil along a cylindrical surface and the corresponding torque required to shear the soil can be measured from the calibrated torque measuring system. From this, the shear strength is computed. This is basically as per the specifications conforming to I. S. 4434-1967. Similar type of apparatus but with remote control has been suggested by Ehlers and Babb (1980). These tests have been conducted periodically and at different depth locations. The strengths are measured at four different depths from top of soil surface viz., 250 mm, 500 mm, 750 mm and 1000 mm. At each of these depths, strengths are measured at four locations around the anchor at a distance of 200 mm from the periphery of anchor helical plates. The average of these sixteen strengths is reported as the shear strength of soil bed. It may be mentioned that the in situ vane shear strength measured for the soil 1 indicated very low values of strength varying from 0.0048 to 0.0093 N/mm² and this is the strength in the remoulded condition. This soil is obtained from in situ coastal deposit. The in situ strength tests made in the field and the strength obtained from a thoroughly disturbed or remoulded soil samples indicated the apparent sensitivity values between 5 and 6. For the soil 2, the in situ strength measured is 0.0062 N/mm² and for soil 3, it is 0.0135 N/mm². For these soils also, the measured values of sensitivity vary between 4 and 6. These soils can be considered as of medium sensitivity. Because of very low strengths, satisfactory unconfined compression tests could not be carried out.

RESULTS AND DISCUSSIONS

The behaviour of these piles can better be assessed with the help of results obtained from load tests. The results of both compression and tension tests are presented and analysed

Fig. 4. Compressive load Vs Settlement curves

Fig. 5. Compressive load Vs Settlement curves

critically. The load-settlement curves obtained from compression tests conducted on the first set of piles in the type I are presented in the Figs. 4 to 6. These referred to the tests conducted in soil 1 at different moisture contents, viz. 40.2%, 45.2%, 50.4% and these moisture contents cover a range of consistency index of 0.49 to 0.70. This range of moisture contents is considered quite appropriate for the installation of screw type of piles. Fig. 4 refers to the test results obtained from the screw piles with 2 Nos. of helical plates

Fig. 6. Compressive load Vs Settlement curves

Fig. 8. Pull-out load Vs upward movement curves

Fig. 7. Pull-out load Vs upward movement curves

Fig. 9. Pull-out load Vs upward movement curves

and in Figs. 5 and 6 the results obtained from screw piles with 3 and 4 helical plates respectively are presented. From the curves, it is clear that there is a similar load-settlement behaviour observed in all these tests. As mentioned previously, the ultimate load capacities are taken as loads corresponding to a settlement equal to 7.5% of diameter of helical plates. In this case this limiting value of settlement is 7.5 mm. From the Figs. 4 to 6 it could be seen that the capacities are

considerably decreased with the increase in moisture contents and it can also be observed that increase in the number of plates results in the increase in load capacities. This aspect has been further examined from the pull-out tests.

The same screw piles are installed in the clay deposits at the same moisture contents under almost identical conditions, but subjected to pull-out tests. The load-displacement curves

Table 3. Measured ultimate load carrying capacities
(Type I; Set I; Soil 1)

S. No.	Screw pile designation	No. of helical plates	Placement moisture content (%)	Consistency index	Ultimate Capacity	
					Compression P_{cu} (kN)	Tension P_{tu} (kN)
1	P 1	2	40.2	0.696	1.44	0.84
2	P 2	3	40.2	0.696	1.74	0.97
3	P 3	4	40.2	0.696	1.94	1.34
4	P 1	2	45.2	0.596	1.04	0.67
5	P 2	3	45.2	0.596	1.36	0.91
6	P 3	4	45.2	0.596	1.45	0.97
7	P 1	2	50.4	0.492	0.84	0.55
8	P 2	3	50.4	0.492	0.93	0.63
9	P 3	4	50.4	0.492	1.05	0.73

Fig. 10. Compressive load Vs settlement curves

Fig. 11. Pull-out load Vs upward movement curves

Table 4. Measured ultimate load carrying capacities
(Type I; Set II; Soil 1)

S. No.	Screw Pile designation	No. of helical plates	Placement moisture content (%)	Consistency index	Ultimate Capacity	
					Compression P_{cu} (kN)	Tension P_{tu} (kN)
1	P 4	2	45.2	0.596	2.38	1.48
2	P 5	3	45.2	0.596	2.60	1.67
3	P 6	4	45.2	0.596	2.71	1.72

obtained from pull-out tests are presented in the Figs. 7 to 9. While arriving at the tension capacities of screw piles, there is not much difficulty in arriving at pull-out loads. At the maximum pull-out load which can be considered as ultimate pull-out capacity, the screw piles are almost came out of the soil. However, the displacements recorded at the time of pull

out are within the range of 50 mm to 70 mm and these displacements are 5 to 7 times the maximum settlements recorded in compression tests. From the same figures, it could be observed that the pull out tests also indicated higher capacities for tests with low moisture contents. Increase in the number of plates also resulted in substantial increase in tension

capacities. All the results of the compression and pull-out tests are summarized in Table 3. The summarised values clearly support that the reduction in moisture content increases the capacities both in compression and tension. The number of plates also increase the capacities.

Having confirmed the influence of moisture on capacities, in the second set (I type), screw piles have been fabricated at different diameters (shaft dia 60 mm, helical palte dia 150 mm) but keeping the same length. The tests have been conducted at one moisture content of 45.2%. The compression load test results are shown in Fig. 10 whereas the pull-out test results are shown in Fig. 11. The capacities obtained are summarized in Table 4. A comparison of these results with the results obtained for the previous set, but at the same moisture content (i. e. 45.2%) indicate that in the case of compression tests, for pile with 2 helical plates, the capacity increased from 1.04 to 2.38 kN. In the piles with 3 helical plates and 4 helical plates the carrying capacities increased from 1.36 kN to 2.60 kN and 1.45 kN to 2.71 kN respectively. These results clearly suggest that the increase in sizes proportionately increases the capacities. In the case of pull-out tests for screw pile with 2 helical plates, the increase is from 0.67 kN to 1.48 kN and in piles with 3 helical plates and 4 helical plates the increases are from 0.91 kN to 1.68 kN and from 0.97 kN to 1.72 kN respectively.

In the second type of screw piles, the overall length of piles has been increased to 1000 mm and the distance between top and bottom helical plates also been varied. But all these piles are of smaller diameters. For the pile with shaft diameter 25 mm and helical

Fig. 12. Compressive load Vs Settlement curves

Fig. 13. Pull-out load Vs upward movement curves

Table 5. Measured ultimate load carrying capacities
(Type II ; Set I ; Soil 2)

S. No.	Screw Pile designation	No. of helical plates	Placement moisture content (%)	Consistency index	Ultimate Capacity	
					Compression P_{cu} (kN)	Tension P_{tu} (kN)
1	P 7	2	26	0.545	0.81	0.69
2	P 8	3	26	0.545	0.93	0.83
3	P 9	4	26	0.545	0.99	0.90

**Table 6. Measured ultimate load carrying capacities
(Type II; Set II; Soil 2)**

S. No.	Screw Pile designation	No. of helical plates	Placement moisture content (%)	Consistency index	Ultimate Capacity	
					Compression P_{cu} (kN)	Tension P_{tu} (kN)
1	P 10	2	26	0.545	0.68	0.65
2	P 11	3	26	0.545	0.73	0.71

**Table 7. Measured ultimate load carrying capacities
(Type II; Sets I & II; Soil 3)**

S. No.	Screw Pile designation	No. of helical plates	Placement moisture content (%)	Consistency index	Ultimate Capacity Tension P_{tu} (kN)
1	P 7	2	35	0.714	1.52
2	P 8	3	35	0.714	1.86
3	P 9	4	35	0.714	2.13
4	P10	2	35	0.714	1.19
5	P11	3	35	0.714	1.48

plate dia. 75 mm, the tests also have been conducted on the piles with 2, 3 and 4 helical plates (set I, type II). All the compression and pull-out tests have been conducted on soil 2 at a moisture content of 26%. The load settlement curves obtained in compression tests are given in Fig. 12 whereas the load-displacement curves from pull-out tests are shown in Fig. 13. It should also be mentioned herein that these tests on piles with smaller diameters and also the distance between top and bottom plates has been reduced to 300 mm. All the capacities in both compression and tension are summarised in Table 5. In the second set of type II screw piles (by keeping the same shaft dia. 25 mm and helical plate dia. 75 mm as first set of type II) the distance between top and bottom plates is reduced to 125 mm. The results of both compression and tension tests are given in Table 6. The load displacement curves obtained in the second type of piles also show similar trends as observed in the first set of piles. For all the piles covered under the first and second sets of type II, only pull-out tests have been conducted in another type of soil at a water content of 35% (soil 3). They have been conducted for confirmation. All these test results are summarized in Table 7.

LOAD TRANSFER PHENOMENA

The actual load from a pile is normally transmitted to the soil through skin friction along the shaft and end bearing at its tip. In case of piles with enlarged shafts or piles with larger dia helical plates welded to it (as used in this investigation) the formulae proposed for under-reamed type of piles can reasonably be applied. One such formulae proposed to give the ultimate load carrying capacity is (explained in Fig. 14(a)) given below.

$$P_{cu} = A_p N_c C_p + A_a N_c C_a' + C_a' A_s' + \alpha C_a A_s \quad (1)$$

This formulae can be used for an approximate estimate of the load carrying capacity. Using the experimental parameters incorporated in the equation 1, it is possible to arrive at theoretical predictions of load carrying capacity. The important soil parameters required in the computation of this load are the undrained cohesion and the adhesion factor for the portion where the shaft is in contact with the soil. As the soil condition mainly conform to remoulded conditions in soft to medium stiff clays, the adhesion factor equal to unity has been used. So in this type of computation, the insitu vane shear strength measured has been used without any reduction. Also since the same soil is compacted in uniform layers at same water content, it can be approximately assumed that $C_p = C_a' =$

Fig. 14(a). Screw pile in compression-forces acting

$C_a = C$.

Based on this, calculations have been made for the load carrying capacity of these screw piles under compression and these values are presented in the Table 8. The experimentally obtained values are also given in that table for comparative purposes. These results clearly indicate that in general there is a good agreement in these values, especially for the piles with 3 and 4 helical plates. In general, for the pile with helical plates at close intervals, the agreement seems to be better. Irrespective of the number of plates the spacing ratio (i. e. spacing of helical plates/dia. of helical plate) approaching a value of 1.0 to 1.5, the computed values almost agree with the experimental values. This aspect can also be examined based on test results obtained from pull-out tests.

In the case of the piles subjected to uplift, the capacities can be experimentally verified by

Fig. 14(b). Screw pile in tension-forces acting

conducting pull-out tests. The uplift capacity of this type of screw pile is as given (explained in Fig. 14(b)) below.

$$P_{tu} = A_a N_{cu} C_t + C_a' A_s' + \alpha C_a A_s \quad (2)$$

As a first approximation, the cohesion parameters can be equated as $C_a' = C_t = C_a = C$. This cohesion is determined in a similar way by conducting the standard insitu vane shear test. However, the factor N_{cu} for the estimation of uplift capacity has to be estimated using some appropriate methods. As per earlier investigators (Adams and Hayes, 1967; Meyerhof and Adams, 1968; Adams and Klym, 1972), it is assumed that the soil above the top helical plate helps in mobilizing the uplift resistance on the top helical plate similar to the end bearing below the tip of pile. In view of this, the standard bearing capacity factor N_c is replaced by the uplift capacity factor N_{cu} . Bobbitt and Clemence (1987) have reported a relationship between uplift capacity factor (N_{cu}) and relative depth (H/D) and this relationship is used while arriving at the capacities. In the first type of piles, the top plate coincides with the top surface of the

Table 8. Comparison between experimental and computed values
(compression Tests)

Sl. No.	Designation of screw pile	No. of helical plates	Consistency Index (I_c)	Cohesive strength C N/mm ²	Spacing ratio (I/D)	Experimental value (kN)	Computed value (kN)	Experimental value / Computed value
Type I, Set I, Soil 1								
1	P 1	2	0.696	0.0093	4.6	1.44	2.10	0.69
2	P 2	3	0.696	0.0093	2.3	1.74	2.10	0.83
3	P 3	4	0.696	0.0093	1.5	1.94	2.10	0.92
4	P 1	2	0.596	0.0070	4.6	1.04	1.58	0.66
5	P 2	3	0.596	0.0070	2.3	1.36	1.58	0.86
6	P 3	4	0.596	0.0070	1.5	1.45	1.58	0.92
7	P 1	2	0.492	0.0048	4.6	0.84	1.09	0.77
8	P 2	3	0.492	0.0048	2.3	0.93	1.09	0.85
9	P 3	4	0.492	0.0048	1.5	1.05	1.09	0.96
Type I, Set II, Soil 1								
10	P 4	2	0.596	0.0070	3.1	2.38	2.73	0.87
11	P 5	3	0.596	0.0070	1.5	2.60	2.73	0.95
12	P 6	4	0.596	0.0070	1.0	2.71	2.73	0.99
Type II, Set I, Soil 2								
13	P 7	2	0.550	0.0062	4.0	0.81	1.02	0.79
14	P 8	3	0.550	0.0062	2.0	0.93	1.02	0.91
15	P 9	4	0.550	0.0062	1.3	0.99	1.02	0.97
Type II, Set II Soil 2								
16	P 10	2	0.550	0.0062	1.7	0.68	0.79	0.86
17	P 11	3	0.550	0.0062	0.8	0.73	0.79	0.92

soil and hence there is no additional contribution due to the uplift resistance on the top helical plate (i. e. $N_{cu} = 0$). However, in the second type piles, the top helical plate is embedded to a depth of 450 mm (i. e. $H/D = 6$). So an approximate value of 9 has been used as N_{cu} . Based on these values of uplift capacity factors and soil parameters, the uplift capacities are computed and compared with the experimental values from pullout tests as given in Table 9. These values also support that for screw piles with 3 and 4 helical plates (i. e. spacing ratio = spacing of helical plates/diameter of helical plate = 1 to 1.5), the computed values are in good agreement with experimental values. In fact the pull out tests have revealed that very nearly cylindrical failure surface could be obtained between top and bottom helical plates for the screw piles with the spacing ratio (I/d) of 1 to 1.5. For

Fig. 15. Agreement ratios between experimental values and computed values with spacing ratio

Table 9. Comparison between experimental and computed values
(Pullout tests)

S. No.	Designation of screw pile	No. of helical plates	Consistency Index (I_c)	Cohesive strength C^* N/mm ²	Spacing ratio (I/D)	Experimental value (kN)	Computed value (kN)	Experimental value / Computed value
Type I, Set I, Soil 1, $H/D=0$								
1	P 1	2	0.696	0.0093	4.6	0.84	1.45	0.58
2	P 2	3	0.696	0.0093	2.3	0.97	1.45	0.67
3	P 3	4	0.696	0.0093	1.5	1.34	1.45	0.92
4	P 1	2	0.596	0.007	4.6	0.67	1.09	0.61
5	P 2	3	0.596	0.007	2.3	0.91	1.09	0.83
6	P 3	4	0.596	0.007	1.5	0.97	1.09	0.89
7	P 1	2	0.492	0.0048	4.6	0.55	0.75	0.73
8	P 2	3	0.492	0.0048	2.3	0.63	0.75	0.84
9	P 3	4	0.492	0.0048	1.5	0.73	0.75	0.97
Type I, Set II, Soil 1, $H/D=0$								
10	P 4	2	0.596	0.007	3.1	1.48	1.62	0.91
11	P 5	3	0.596	0.007	1.5	1.67	1.62	1.03
12	P 6	4	0.596	0.007	1.0	1.72	1.62	1.06
Type II, Set I, Soil 2, $H/D=6$								
13	P 7	2	0.550	0.0062	4.0	0.69	0.99	0.70
14	P 8	3	0.550	0.0062	2.0	0.83	0.99	0.84
15	P 9	4	0.550	0.0062	1.3	0.90	0.99	0.91
Type II, Set II, Soil 2, $H/D=6$								
16	P 10	2	0.550	0.0062	1.7	0.65	0.77	0.84
17	P 11	3	0.550	0.0062	0.8	0.71	0.77	0.92
Type II, Set I, Soil 3, $H/D=6$								
18	P 7	2	0.714	0.0135	4.0	1.52	2.16	0.70
19	P 8	3	0.714	0.0135	2.0	1.86	2.16	0.86
20	P 9	4	0.714	0.0135	1.3	2.13	2.16	0.99
Type II, Set II, Soil 3, $H/D=6$								
21	P 10	2	0.714	0.0135	1.7	1.19	1.67	0.71
22	P 11	3	0.714	0.0135	0.8	1.48	1.67	0.89

the purposes of comparison between the experimentally obtained values and the computed capacities, the agreement ratios are shown as a function of spacing ratio in Fig. 15. It could be seen that only for the spacing ratios between 1.0 and 1.5, there is a good agreement (indicating an agreement ratio of around 1). For other ratios, the experimentally obtained values are much lower than the computed values. The photographs of the same pulled out piles with columns of soils are shown in Figs. 16 to 19. Even for the pile with 2 helical plates as shown in Fig. 19, the failure is nearly cylindrical. However, for the pile with same

two helical plates ($I/D=4.0$), no cylindrical failure could be seen from the Fig. 18. These results confirm that there is a good amount of resistance coming from this type of piles. For light structure subjected to overturning movements these piles can be conveniently used.

CONCLUSIONS

In the recent times, light screw types of piles are being used as anchors to support the foundations of masts and transmission towers subject to overturning moments. Fur-

Fig. 16. Photograph of pulled out piles after pull-out tests (Type I, Set I, Soil 1)

ther, the use of such light weight foundation systems are found to be very convenient in marshy soil conditions and in soft marine clays.

The results of this investigation bring out that these piles are very useful to resist uplift forces. For the optimum use of piles, the ratio of the spacing of helical plates to the diameter of the plates (I/D) should be kept between 1.0 and 1.5 in soft to medium stiff clays. The pull out test results indicate that the failure surface is very nearly cylindrical for these spacing ratios. If the helical plates are spaced at larger distance, the resisting area gets decreased.

This study shows a good promise for screw pile as one type of anchor and these are the piles requiring very simple installation techniques. If enough precautions are taken to counteract corrosion from the aggressive soil conditions, it is possible that these screw piles can be conveniently installed.

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Fig. 17. Photograph of pulled out piles after pull-out tests (Type I, Set II, Soil 1)

Fig. 18. Photograph of pulled out piles after pull-out test (Type II, Set I, Soil 3)

Fig. 19. Photograph of pulled out piles after pull-out tests (Type II, Set II, Soil 3)

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NOTATIONS

- A_a : Area of cross section of the helical plate = $\frac{\pi}{4}(D^2 - d^2)$
- A_p : Cross sectional area of pile stem at the toe level = $\frac{\pi}{4}d^2$
- A_s : Surface area of shaft
- A_s^1 : Surface area of cylinder between top and bottom plates $(\pi D)L_c$
- C : Cohesive strength of soil measured with the standard insitu vane shear apparatus
- C_a : Average cohesion of soil along the pile shaft
- C_a^1 : Average cohesion of soil around cylinder of

- soil between top and bottom helical plates
- C_p : Cohesion of soil around toe
- C_t : Cohesion of soil around the top helical plate
- d : Dia. of the pile shaft
- D : Dia. of the helical plate
- H : Depth of embedment of top helical plate
- H_B : Distance between bottom helical plate bottom
- H_T : Distance between pile top and top helical plate
- I : Spacing of helical plates
- L : Total length of screw pile = $L_c + H_T + H_B$
- L_c : Distance between top and bottom helical plates
- N_c : Bearing capacity factor
- N_{cu} : Uplift capacity factor
- p : Pitch of helical plate
- P_{cu} : Ultimate bearing capacity of screw pile
- P_{tu} : Ultimate uplift capacity of screw pile
- ω : Placement moisture content
- α : adhesion factor

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