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Experimental Studies of Screw Anchors in Cohesive
and Cohesionless Media.

• Bernard F. Moore

A thesis submitted to the Faculty of
Graduate Studies and Research in Par-
tial fulfillment of the requirements
for the degree of Master of Engineering

Faculty of Engineering
• Carleton University
Ottawa, Ontario
• August 1979

The undersigned recommend to the Faculty
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"Experimental Studies of Screw Anchors
in Cohesive and Cohesionless Media"

submitted by Bernard F. Moore, B. Eng.,
in partial fulfilment of the requirement for
the degree of Master of Engineering

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ABSTRACT

A review of available literature on soil anchors is presented in order to summarize the present state-of-the-art and to highlight areas in which more research is needed. Screw anchor tests, that were performed in Ottawa Sand, are reported and discussed. The results provide valuable insight into the interaction of anchors in a group as well as soil - anchor interaction. Further studies, using a screwplate in compacted kaolin, yield encouraging results, regarding the estimation of the insitu elastic modulus by applying a relationship obtained in the course of the literature review.

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V

INTRODUCTION

There are many instances in civil engineering practice where the foundation of a structure must resist pullout forces. Over the last two decades, the development of techniques which provide this type of support has resulted in what is referred to as the Earth Anchor.

Although well documented, the design of anchorages especially their group behavior is not well understood. Numerous empirical studies have been performed. However, the empirical rules generated usually apply only to specific situations. In addition to this, these rules apply almost exclusively to the ultimate resistance of anchors. Consequently, much remains to be known about the behavior of an anchor when subjected to stresses within its designed working load range and the effect of interaction on the working load behavior of the anchor.

This thesis is a comprehensive exposition on anchors, attempting to generate a better understanding of the behavioral attributes of anchors. Various anchors and their applications are discussed in Chapter 1. A description of the significant factors which influence the performance of an anchor in general is presented in Chapter 2. Chapter 3 summarizes several theoretical studies in order to document the present state-of-the-art and subsequently highlight areas in which

more research is needed.

Chapter 4 describes the experimental program which was developed to study the performance of anchors in the range of working loads. The emphasis was on behavioral trends rather than the evaluation of design parameters. Two types of soils were used in the program; their characteristics and the possible influence of these properties on the test results are also discussed in Chapter 4. The experimental results are presented and discussed in Chapter 5. Wherever possible comparison is made with analytical studies. All the conclusions inferred from the results are reported in Chapter 6 along with the author's recommendations for future study in this area.

CHAPTER I

THE TYPES OF ANCHORS AND THEIR APPLICATIONS

Anchors can be classified as one of three types; (a) those providing uplift resistance only, (b) those providing resistance to uplift as well as downward movement, (c) those designed for specific situations. Anchors in the first group are usually smaller in size, easier to install and of a smaller capacity than those in the second group. The third classification refers to anchors which are utilized in one or two special situations. The resistance realized in these cases is unlikely to be achieved if they are used in situations other than those for which they were designed.

The grouted anchor system is probably the most widely used and the most extensively reported; Robinson (1969), Dupeuble et. al (1969), Littlejohn (1970), Adams (1972), Trow (1974), Adams et. al. (1975) and Zobel et. al. (1975). In its basic form, it consists of a steel rod bonded at one end to a mass of concrete. However, there are many variations, the most common of which is shown in Figure 1.1a. Installation requires the drilling of a hole to the required depth. A steel rod is lowered into place and the cavity is filled with pressurized grout. In cohesionless material, a casing is used to prevent collapse of the hole. It is possible to use a bell-shaped cavity in cohesive soils as the likelihood of collapse

(a) Grouted Rod

(b) Malone (after Zobel, 1965)

(c) Power Installed Multi-Helix

(d) Plate

FIGURE 1.1 IDEALIZED ANCHOR CONFIGURATIONS

is greatly reduced. The **Malone or explosive anchor**, Figure 1.1b, utilizes an explosive charge to create a spherical cavity into which the grout is injected. Some of the civil engineering situations in which grouted anchors have been successfully utilized are described in detail by Littlejohn (1970). With regards to works of a temporary nature, grouted anchors have been used to support base-free retaining walls for excavations and those adjacent to new highways, railways and canals. Pipe-jacking or thrust boring operations require some form of thrust block to provide a reaction to the jacks. Prestressed inclined grouted anchors have been used to provide both horizontal and vertical force components to increase the resistance to sliding.

More permanent resistance to pullout forces is provided in cases such as the anchoring of cofferdams, dry docks and effluent tanks against upward water pressure. Tall buildings, masts and transmission towers are highly susceptible to the overturning forces of wind. Grouted anchors have been used to provide substantial resistive action in situations such as these. In some cases, anchors have been used to apply loads and not resist them. Grouted anchors have been used to preload sections of a new building in order to minimize structural settlement in soils having low permeability. In addition to this, they have been used to apply loads in pile and plate load tests.

Many of the applications mentioned above have been preferred in relation to helical and plate anchors (see Figures

1.1c and 1.1d). Helical anchors, having single or multi-helical plates are mechanically screwed into the soil to various depths to obtain their resistance to load. Plate anchors, however, are installed in a manner somewhat similar to grouted anchors. A hole, slightly larger than the plate, is excavated. The plate is placed in the hole and a cable passing through a seam inclined at a predetermined angle is attached to it. The hole is backfilled with grout or backfill. In soft boulder-free soils, the helical anchor will be a more attractive alternative than the grouted anchor because of its ease of installation. The helical anchor can be likened to the grouted anchor in some respects. The length and diameter of the anchorage zone in both cases is fixed. Their respective methods of installation create a highly densified zone around the anchor and the major portion of the uplift resistance developed is attributed to surface shear.

The three anchors described above are designed to resist uplift forces only. Four basic types of anchors are used as combined footings i.e. to resist uplift as well as downward forces. They are:

- (i) Piles
- (ii) Steel grillages (Figure 1.2)
- (iii) Caisson
- (iv) Formed concrete footing

Fig. 1.2 a) Standard Grillage Anchor; b) Pyramid Grillage Anchor (after Zobel, 1965)

Fig. 1.3 Schematic View of Suction Anchor

Piles of wood or concrete are used as combination foundation anchorages where suitable material is at too great a depth to use anchors of the types previously mentioned. The wooden stake is used in the same manner as a pile but at shallower depths and to supply simple anchoring requirements. In addition to the standard loads that anchors are used to resist, piles have been utilized in foundations which have to provide lateral restraint against earthquake, explosion and thermally induced lateral forces. This broadens considerably the range of applications.

The steel grillage is comprised of a number of steel beams arranged in a variety of patterns. The mat is usually placed in open excavations. In the event that the resistance to downward movement required is minimal, the mat may be quite small. Steel grillages are used mostly to support transmission towers.

A caisson is a hollow shaft or box that is utilized as the outer part of a pier, straight caissons are used in cohesionless soils, with placement facilitated by applying a bentonite slurry to the excavation. In cohesive soils, where cohesion prevents cave-in, the base of the caisson may be belled by under reaming. The enlarged base enhances both the bearing capacity and pullout resistance of this type of foundation.

A similar type of foundation is the formed concrete footing which, like the grillage, is bulky and prohibitive in

cost. As the name implies, the footing is made of concrete poured into a formed excavation. The uplift resistance provided by anchors in this category is derived from their weight and/or frictional resistance mobilized along the surface of the anchor itself.

There have also been advances made in the field of special application anchorages. The most noteworthy of these are anchors used in offshore structures associated with oil exploration operations. The first of these is a suction type anchor, models of which have been tested by Wang et. al. (1977). The suction anchor, (see Fig. 1.3), develops resistance to uplift when the pressure beneath the anchor is reduced to below ambient pressure. The pressure reduction is a direct result of pumping water from inside the anchor.

The Hydropin anchor, designed by Kerr (1972) operates on a different principle. Unlike the suction anchor, it is buried deeply. A nozzle in the head of the anchor, (see Fig. 1.4) fluidizes the underlying seabed and a riser tube effects vertical transportation of the solids. These combine to achieve deep burial of the anchor, and the resulting pullout resistance. One advantage of a large depth of embedment is that the anchor's performance will not be affected by scouring and weathering conditions. Both these anchorages can be used to provide support for (a) research submersibles and buoys (b) drilling platforms (c) salvage and dredging platforms. However, the relative merits of each in a given situation would be highly dependent

Figure 1.4 a) Essential Features of The Hydropin
b) The Hydropin with Jet - Ahead Nozzle

on the cost per unit resistance developed.

No doubt there are numerous other anchors being tested for the various applications mentioned here and even for some not mentioned. However, the ones outlined previously represent the major types of anchors in use today and from the broad spectrum of applications cited, it is reasonable to conclude that as structural foundations, they are invaluable.

CHAPTER II

FACTORS AFFECTING THE BEHAVIOR OF ANCHORS

Despite the fact that a substantial amount of research has been performed on anchors, the soil conditions and anchor types vary. As a result, strict guidelines on the design of anchors are still not available. Nevertheless, valuable information has been provided with regards to the major factors which influence the behavior of an anchor. They are:

- (i) Depth of embedment of the anchor
- (ii) Geometry of the anchor
- (iii) Spacing of the anchor (group behavior)
- (iv) Material properties of the soil
- (v) Type and magnitude of the load

Each of the above factors will be discussed separately so as to emphasize the effect of each.

2.1 Depth of Embedment

The depth of embedment of an anchor determines to a major extent whether it will be classified as shallow or deep. The ratio of embedment depth to the diameter (or width) of the anchor is the non-dimensional indicator used. Meyerhof and Adams (1968) offer the explanation that at shallow depths (at depths less than some critical value), the slip surface caused

by the anchor at failure* extends in a shallow arc from the edge of the anchor to the ground surface. Because of this, the resistance to uplift varies sharply with changes in depth within this range. Carr et. al. (1971) report the results of sand movement measurements near an anchor plate. At shallow depths the sand is displaced vertically, thus giving rise to surface rupture. At greater depths, the failure surface is less distinct, being initially curved at the anchor and becoming straight as it approaches the ground surface. In this case, there is no indication of failure at the surface because at great depths in sand, soil movement is radially outwards. Baker et. al. (1965) and Adams et. al. (1975) suggest that the critical value for the depth to diameter ratio is 6. The value will change depending on the soil properties and the loading conditions (Trofimenkov et. al., 1965). The values reported range from 2 to 13 under the various conditions. However, there doesn't seem to be a discernible trend.

In the analysis of an anchor problem, the effect of depth of embedment is usually accounted for in the overburden term either as weight or lateral pressure. It is conceivable then, that there would be a direct relationship between the depth of embedment and the uplift resistance developed. Figure 2.1

* The condition of failure referred to is signified by a disproportionate increase in uplift movement with applied load.

FIG 2.1 THE EFFECT OF EMBEDMENT DEPTH ON ANCHOR BEHAVIOR (after Zobel, 1976)

summarizes the results of tests carried out by Zobel et. al. (1975). Not only does the uplift capacity vary with the depth of embedment but also with anchor geometry. Of greater significance however, is the initial slope of the load-displacement curves for the explosive anchor and the concrete pile. Its apparent dependence on the depth of embedment suggests that it is an important factor when considering the stiffness of a soil-anchor system.

2.2 Geometry of the Anchor

An analysis of the effect of anchor configuration on the performance of the anchor can be conducted on two levels; (a) variation in size of one type of anchor, (b) variation in overall anchor configuration. In the first case, the anchor capacity increases with increase in anchor size (width or diameter) provided the anchor is buried deeply (see Figure 2.2).

This is not true under all conditions. Neely et. al. (1973) report a decrease in load resistance for vertical anchor plates used as retaining wall tiebacks; (See Figure 2.3). A similar trend is evident in the results of Healy (1971) in that the pullout resistance of spherical anchors in dense sand ($R.D. > 30\%$) decreases with increase in anchor size. The common factor here is that the effect decreases as the depth of embedment and the density decrease. On the other hand, the anchor system in all three cases is different. This may be the factor contributing most to the differences in behavior;

Figure 2.2 Plot of Anchor Pullout Capacity vs Depth of Embedment
(after Baker et. al., 1965)

that is, the horizontal and spherical anchors effect far greater horizontal movement than the vertical anchor.

Tests performed using helical anchors are reported by Adams et. al. (1975). As would be expected, the uplift resistance increases with the size of the helices. The failure load also increases with the number of plates per anchor. A limiting value is not given; however, it is expected that as the number of plates increases, the helical anchor will approximate a cylindrical anchor. Its behavior can be likened to that of a grouted anchor. A comparison of the grouted anchor and the single helix anchor is provided by Johnson and Ladanyi (1974). Essentially, the helical anchor (10ins. diameter) attained only 20% of the grouted anchor capacity for the same displacement, depth of embedment and time of load application. This percentage decreased with increasing time to failure.

Like the plate anchor, the single helix anchor has large prefailure displacements (up to 50% of plate diameter - Bhatnagar, 1969). This is required to compact the overlying soil before sufficient resistance to uplift is obtained. In the case of the grouted anchor, it is already in bonded contact with the surrounding soil. Hence it requires smaller displacements to attain the ultimate shear strength at the soil - anchor interface. Figure 2.1 summarizes the behavior of other anchor configurations. The conclusion that can be drawn is that the anchors which obtain the major part of their uplift resistance from shear stresses developed at the soil - anchor interface will

Fig. 2.3 Experimental Values of M_{yq} for Various B/h and H/h values (Neely et. al. 1971)

have smaller uplift displacements.

2.3 Material Properties of the Soil Medium

When comparing the results of several tests programs, a major obstacle is the variation in soil properties which may occur from one site to another. In order for the results of an experiment to be fully appreciated, it is necessary to monitor the soil properties which influence the results of the tests. Anchor tests are no exception. Baker and Konder (1965) suggest that for tests in sand, it is necessary to monitor the density, angle of internal friction, relative density and void ratio. However it may be necessary to measure only the density and the friction angle as the other two parameters are related to the density, provided the specific gravity and the density limits are known.

On the other hand, Hanna and Carr (1971) present results which show that the pullout behavior of an anchor in sand is dependent on the initial state of stress in the sand. They accept the fact that preconsolidation of the sand may cause an increase in density but conclude that such an increase is negligible. The results of their tests are summarized in Figure 2.4. The peak uplift load increases with the overconsolidation ratio and attains a peak value at a decreasing rate. A very significant increase in the uplift load is achieved for slightly overconsolidated sand. However, at high overconsolidation ratios, there is a considerable decrease in load as compared

to the normally consolidated sand. This is attributed to the inability of the sand to retain the high initial stresses set up during the overconsolidation stage.

Nevertheless, the contribution of sand density should not be discounted. Healy (1971) supports the view that sand density plays an important role in the behavior of an anchor. He concludes that the average pullout stress is independent of the anchor size provided the sand is loose - relative density less than 40%. For sand with a relative density greater than 90%, the pullout stress decreases with increase in anchor size. An explanation for the difference in behavior may lie in the mechanism for developing uplift resistance. The disc anchor used by Hanna and Carr derives most of its strength from the ~~weight of the overlying soil, whereas the anchor used by Healy~~ relies mainly on anchor surface friction. In the latter case, the friction angle as related to the density is of major importance.

Conceivably, the behavior of an anchor will vary with the type of soil medium. Trofimenkov et. al. (1965) report tests on various soils using screw piles. Medium dense sand is found to provide the maximum resistance to uplift while experiencing the greatest deformation at ultimate load. Soft clay develops the least resistance to the uplift force.

With regards to cohesive soils, Davie and Sutherland (1977) recognize the importance of density, cohesive strength, tensile strength, elastic modulus, poisson's ratio and plastic volumetric strain in the analysis of anchor behavior in clay.

Fig. 2.4 Comparison of Uplift Load-Anchor Displacement Curves in Normally and Overconsolidated Sands (Hanna et. al. 1971)

The relevance of some of these factors may be appreciated more if the behavior of deep anchors is examined. When loaded, high tensile stresses exist below the periphery of the anchor plate. The structural stiffness will prevent the propagation of the cracks due to these stresses. Consequently, local failure is induced at the edges of the plate. Shallow anchors fail mainly due to insufficient shearing resistance. Davié et. al. (1977) suggest that for purely cohesive soils, the shear strength and the soil weight are the two principal components of uplift resistance; implying the use of an 'equilibrium of forces' method for determining this resistance. Tests performed in non-layered clay apparently result in smaller anchor displacements than tests in layered clay. The greater structural stiffness of the former is proffered as the cause. This being the primary work in clay reported, it is difficult to discern specific trends with regards to changes in the properties of cohesive soils.

2.4 Type and Magnitude of the Load

In practice, an anchor is usually required to perform adequately under a number of different loading conditions. The anchor will not only be expected to resist tensile and compressive forces but must also accommodate changes in the rate of loading. The load may be applied over a long period of time or as is sometimes the case, it may be inclined at some angle to the horizontal.

In this respect, Trofimenkov and Mariupolskii (1965)

provide valuable insight into the behavior of an anchor (screw pile) under a variety of loading conditions. The four types are:

- (i) Continuous pullout or pressing in
- (ii) Pulling out in steps (load applied after rate of displacement is less than 0.1 mm for 2 hrs.)
- (iii) Pulling out and pressing in by pulsating load
- (iv) Pulling out and pressing in by alternate load increased by steps.

The load carrying capacity of the anchor within its working load range is found to be approximately 1.5 times higher in pressing in tests than in the pulling tests. Three reasons are offered for this type of behavior. In the pressing in tests, the load acts on undisturbed soil and depending on the size of the anchor shaft, the working area in pullout tests may be significantly smaller. ~~Lastly but of equal importance,~~ is that the soil density increases with depth. Figure 2.5a and 2.5b summarizes the results of tests performed in semi-hard clays and in saturated loose sand respectively. Tests using continuous loading produce the greatest ultimate load; the largest deformation at failure and anchor behavior seems to be independent of soil type. The ultimate resistance of anchors subjected to a step load is in all cases 10 to 20 percent less than that produced by continuous loading. The other two loading types produce variations in the ultimate capacity with soil properties.

Fig. 2.5 a) Pullout Tests in Semi-Hard Clays; b) Pullout Tests In Saturated Loose Sands (Trofimenkov et. al. 1965)

An addendum to this is to be found in the work of Andeades et. al. (1978) regarding the effect of repeated loading on the holding capacity of anchorages. The authors conclude that cyclic failure of embedment anchors causes a lower ultimate capacity and displacement stiffness than static pullout. However, if the cyclic displacement is kept below 50% of the displacement at failure in a static pullout test, there is virtually no reduction in strength due to cyclic loading.

With regards to the long term behavior of anchors, Adams et. al (1975) have shown that the uplift capacity of a helical anchor will decrease with time (see Figure 2.6). They attribute this behavior to strain softening associated with the dissipation of pore pressures. In addition to this, prolonged loading will result in the formation of tension cracks, thus further weakening the soil. A further extension to this is provided by Selvadurai (1977) in his theoretical analysis of the time dependent response of a prestressed anchor rod deeply embedded in an isotropic viscoelastic infinite medium. Expressions describing the degree of load relaxation for a specified displacement and the amount of creep deformation due to a constant sustained load are provided.

The results of vertical and inclined anchor tests performed are also compared and reported by Adams et. al. (1975), as shown in Figure 2.7. The difference in performance occurs within the working load range where the curve for the inclined anchor is

Fig. 2.6 Long-term Uplift Test on a Helical Anchor in Clay (after Adams et. al., 1975)

Fig. 2.7 Vertical and Inclined Uplift Test on a Helical Anchor in Clay (after Adams et. al., 1975)

flatter and undergoes a larger deformation at each load level. Das et. al. (1975) approach the problem by applying a load inclined to a horizontal anchor plate in sand as shown in Figure 2.8. The load connection was such that the anchor was free to rotate. Increasing the angle of inclination produces a marginal increase in the ultimate pullout force. In both cases the soil failure is unsymmetrical. The departure from vertical load behavior may then be attributed to this.

2.5 Spacing of Anchors

The behavior of a group of anchors is affected by several variables such as the anchor length to diameter ratio, the shape of the group, the number of anchors in the group, the spacing of the anchors and the stiffness of the pile cap. The stiffness of the anchor cap will affect the distribution of load throughout the group and thus the movement of each anchor in the group. The research on the group action of anchors, reported herein has been performed using rigid anchor caps. Presumably, the load distribution would be different if a flexible cap were used. However experimental results are not available to support this. The only other variables not encountered in the analysis of isolated anchors are the spacing of the anchors and the size and shape of the group. The anchor spacing is seen as the controlling factor where the group action of anchors is concerned.

Evaluation of the group behavior is facilitated by the

Geometric Parameters for Pullout Test

Figure 2.8 Breakout Loads for Inclined Pullout Tests (Das et. al., 1975)

group load efficiency which is

$$\eta = \frac{P}{nP_0}$$

where

P = load on group

P_0 = load on a comparable single anchor

n = number of identical anchors in group

Several researchers have studied the load efficiency variation with spacing of anchors in a group. Das et. al. (1975), Hanna et. al. (1972) and Yilmaz et. al. (1971) report 100% group efficiency of spacings between 4 and 6 anchor diameters.* (See Figure 2.9). This spacing is referred to as the isolation spacing, the spacing at which an anchor in a group behaves as an isolated anchor. A substantially lower value, 1.5 anchor diameter, is reported by Trofimenkov et. al. (1965) as a result of field tests performed using screw piles. This value should not be viewed with skepticism even though the zone of non-interaction may have been reduced due to disturbance caused during installation. A screw pile, like a grouted anchor, depends mainly on skin friction for its uplift resistance. Consequently, the soil wedge, displaced at failure, will be relatively small. Nevertheless, the group efficiency will generally increase with spacing up to the isolation spacing. Under certain conditions it is possible to attain efficiencies in excess of 100%. Yilmaz and Hanna (1971) suggest that at high

*For plate anchors, the diameter of the plate is used and for grouted anchors, the diameter of the grouted column is the value used.

(a)

(b)

Fig. 2.9 a) Plot of Group Efficiency vs Spacing; b) Ultimate Group Efficiency-Spacing Relationship for Lines of Disc Anchors (Yilmaz et. al. 1971)

depth to diameter ratios, the group load efficiency may exceed 100% especially at close spacings. They also report that an increase in the group size will effect an increase in efficiency. It should be noted that because of the interaction of the anchors when closer than the isolation spacing, the movement of the average anchor in a group would increase over that of an isolated anchor at the same load level. There appears to be some dependence on the size and shape of the group as well, however the results are inconclusive.

The load distribution throughout the group is a function of the load level. At low load levels (less than 50% of the ultimate load), Hanna et. al. (1972) report a uniform load distribution. At higher load levels, the load in any anchor is a function of its position relative to the center of the group - peripheral anchors carry a higher percentage of the applied load. This is not an absolute effect however. Yilmaz et. al. (1971) report that these trends will be affected by the group size and the depth of embedment. In addition to this, the percentage of load carried by the central anchors in a group decreases as failure is approached. The tests reported were performed in sand and the explanation offered by Hanna et. al. (1972) is that a decrease in depth of embedment and general upward movement causes a deterioration of the arching system in the sand contained within the central core of anchors. The spacing of the anchors is reported to have no effect on the distribution of load even though the load taken by each anchor will vary with changes in the spacing.

CHAPTER III

Analytical Study Of Anchors

3.0 Conventional Anchor analysis

Many researchers have tried to analyze the uplift behavior of soil anchors. Despite the sizeable number of theories, with their varying degrees of sophistication, there are three basic approaches to predicting the ultimate uplift capacity of shallow anchors. They are The Cone Method, The Earth Pressure Method and semi-empirical methods (Flucker & Teng, 1965). Semi-empirical methods have been developed to predict the uplift capacity of deep anchors as well.

In the cone method, a failure surface is assumed and the cohesive and frictional forces along this surface are added to the dead weight of the soil-anchor system to give the uplift capacity of the anchor. The earth pressure method utilizes a vertical failure surface. The estimate of uplift capacity obtained is the dead weight of the system added to the frictional resistance developed along the sides of the vertical failure plane. The variations proposed by the major exponents of these methods are described by Kovacs et al. (1975), hence they will not be recounted here.

Meyerhof and Adams (1968) proposed a semi-empirical method for estimating the uplift resistance of anchors in

soil by adapting Meyerhof's general bearing capacity theory. The failure surface is assumed to have a truncated parabolic shape which extends to the ground surface in the case of shallow anchors, and is limited to some depth below the ground surface when the anchor is deeply embedded. The height of the failure surface is estimated from experimental observations made in the course of the research. This method is somewhat similar to the cone method mentioned above; the fact that the assumed failure surface is applied to both deep and shallow anchors, suggests that the same may be possible for the cone and earth pressure methods.

Neely et al. (1973), reported a further refinement to the work of Meyerhof and Adams, making it applicable to vertical anchor plates. The two methods reported are The Surcharge Method and The Equivalent Free Surface Method, see Fig. 3.1; both had good correlation with experimentally observed values. In the surcharge method, a failure surface is developed using a combination of logarithmic spirals and straight lines. The soil above the top of the anchor plate is assumed to act as a surcharge. This surcharge, the weight of the anchor and the soil within the failure surface determines the maximum resistance of the system. The shearing resistance of the upper layers of soil is included by making use of the 'equivalent free surface' concept proposed by Meyerhof, (1951). The normal and shear forces along the free surface are calculated using Rankine's active stresses.

(a)

(b)

Fig. 3.1 a) Surcharge Method of Analysis; b) Equivalent Free Surface Method (after Neely et. al., 1973)

Several other semi-empirical works are summarized in Table 3.1, as well as by Kovacs et al. (1975). The design approach will differ depending on the type of anchor being considered. Nevertheless, the principle remains the same, that is, the summation of forces resisting uplift. Anchors with relatively small active parts, for example grouted anchors, rely mainly on cohesion, adhesion and skin friction for their holding power. Larger anchors, such as the grillage, depend more on the weight of the soil-anchor system. The factor common to all these theories is that they deal solely with the ultimate performance of anchors. However several researchers have carried out theoretical studies in an attempt to describe the behavior of an anchor in a soil deforming elastically.

3.1 Selvadurai

Selvadurai (1976) studied the load-deflection characteristics of an anchor embedded in an isotropic infinite elastic medium. For simplicity, it was assumed that a.) the soil was a linearly deformable elastic solid; b.) the anchor was rigid; c.) the anchor was spherical in shape; d.) the anchor was in bonded contact with the surrounding soil medium; e.) frictional effects along the rod and in the loading mechanism were negligible; f.) the depth of embedment was large enough to isolate the anchor from the interference of external boundaries. The problem was solved using an adaptation of the work of Bousinesq (1885) as reported by Sternberg et al. (1951). It was necessary to transform the axisymmetric curvilinear coordinate

system into a spheroidal coordinate system. From the relationships generated for both the prolate and oblate spheroidal shapes, it was possible to obtain solutions for anchors with curvilinear and elongated shapes. A summary of the results obtained and the limiting cases is presented below.

A. For a prolate spheroidal anchor (Figure 3.2)

$$P = 16\pi G\delta A_p(1-\nu)(1-\lambda)^{1/2} \left\{ \frac{1}{2}[(3-4\nu) + 1/(1-\lambda^2)] \cdot \log[(1+(1-\lambda^2))/(1-(1-\lambda^2) - 1/(1-\lambda^2))] \right\}^{-1} \quad (2)$$

where $\lambda = B_p/A_p < 1$

A spherical shape is generated if $A_p = B_p$. Hence

$$P = 24\pi G\delta A_p(1-\nu)/(5-6\nu) \quad (3)$$

The behavior of a rod anchor is obtained where $A_p \gg B_p$:

$$P = 8\pi G\delta A_p(1-\nu) \{ 2(1-\nu) [\log(A_p/B_p) + \log(2)] - \frac{1}{2} \}^{-1} \quad (4)$$

B. For an oblate spheroidal anchor (Figure 3.3)

$$P = 16\pi G\delta C_o(1-\nu) \{ \sinh(\alpha_o) - [\sinh(\alpha_o) - (3-4\nu)] \cdot \cot^{-1}(\sinh(\alpha_o)) \}^{-1} \quad (5)$$

where $C_o^2 = B_o^2 - A_o^2$; $\sinh(\alpha_o) = A_o/(B_o^2 - A_o^2)^{1/2}$

In the special case where $A_o = 0$, and $\sinh(\alpha_o) = 0$,

$$P = 32G\delta B_o(1-\nu)/(3-4\nu) \quad (6)$$

which is the relationship for a circular plate anchor having

a diameter B_0 . The general solutions presented here can be used to generate the equations governing the behavior of an anchor embedded in an incompressible elastic medium by substituting $\nu = 0.5$. However, Selvadurai (1978) derived the equations in a similar form by drawing an analogy between this situation and that of an impermeable object placed in a viscous fluid having a slow uniform velocity. The principle of equivalence was applicable due to the similarity between the behavior of a saturated elastic soil medium and an incompressible Newtonian viscous fluid, flowing at a slow velocity. The relationships so derived are:

(i) Prolate spheroid:

$$P = 8\pi G\delta C_p [(\tau_p^2 + 1)\coth^{-1}\tau_p - \tau_p]^{-1} \quad (7)$$

$$\text{where } C_p^2 = A_p^2 - B_p^2 \quad ; \quad \tau_p^2 = 1 - (B_p/A_p)^2$$

(ii) Oblate spheroid:

$$P = 8\pi G\delta C_o [\lambda_o^2 - (\lambda_o^2 - 1)\cot^{-1}\lambda_o]^{-1} \quad (8)$$

$$\text{where } C_o^2 = A_o^2 - B_o^2 \quad ; \quad \lambda_o^2 = (A_o/B_o)^2 - 1$$

(iii) Sphere:

$$P = 6\pi G\delta A_p \quad (9)$$

(iv) Disc:

$$P = 16G\delta B \quad (10)$$

Fig. 3.2 Prolate Spheroidal Anchor; (after Selvadurai, 1976)

Fig. 3.3 Oblate Spheroidal Anchor (after Selvadurai, 1976)

3.2 Rowe and Booker (1978)

Rowe and Booker (1978) developed a technique for determining the stresses and displacements anywhere within an elastic soil mass due to a given load, P , applied to an anchor of general shape. The technique involves assuming a relationship of the form

$$q_j = \sum_{k=1} (I_{jk} \Delta P_k) \quad (11)$$

between the applied load, P_k , and the stresses and displacements in the soil represented by q_j . The influence coefficients, I_{jk} , are evaluated by using the double Fourier transform (Sneddon, 1951) to convert the equations of elasticity. Provision is made for the effect of anchor shape, layer depth, anchor-soil interface condition, separation of the anchor from the underlying soil and the interaction between anchors in a group.

Solutions for the stiffnesses of strip ($P/\delta E$), circular ($P/B\delta E$), and rectangular ($P/L\delta E$) anchors are expressed as

$$K = I_d \cdot I_h / (1 - \nu^2) \quad (12)$$

where I_d = influence factor for the effect of layer depth, D/B , upon the apparent stiffness evaluated for an anchor with $h/B = \infty$

I_h = correction for the effect of embedment ratio, h/B , on the apparent stiffness.

B = diameter (or width)

L = length

The authors also present a set of influence curves illustrating the effect of anchor spacing and aspect ratio on the anchor stiffness.

3.3 Kay and Qamar (1978); See Figure 3.4

The authors contend that the response characteristics of a grouted anchor consists of five components which may contribute to total movement at the anchor head.

(i) Elastic deformation of the free portion of the anchor shaft:

$$\delta_r = PL_r/E_r A_r \quad (13)$$

where

P = applied load

L_r = free tendon length

E_r = Modulus of elasticity of the steel

A_r = effective cross-sectional area of tendon

This gives the unrestrained deformation, presumably the result would be different should the soil exert a significant frictional force on the shaft.

(ii) Extension of soil-grout composite either as a cracked or uncracked section:

$$\delta_a = PL/3E_c A_a \quad (14)$$

where

E_c = composite anchor modulus

A_a = cross-sectional area of anchor

L = grouted length

This is a simplified relationship as opposed to the more exact solution to be obtained by integrating the strain caused by the attenuating surface shear stress.

(iii) Deformation associated with strains in the surrounding soil:

The cylindrical portion of the anchor is divided into n elements. The shear stress is idealized as a uniformly distributed line load acting along the axis of the element.

From Mindlin's equations, the displacement ρ_i , of the soil immediately adjacent to a typical element, i , due to a distributed line load, P_j , is

$$\rho_{ij} = I_{ij} \cdot P_j \quad (15)$$

where I_{ij} = constant

The displacement due to all n elements is

$$\rho_i = \sum_{j=1}^n (I_{ij} \cdot P_j) \quad (16)$$

with the equilibrium of the system dictating that

$$P = (L/n) \sum_{j=1}^n P_j \quad (17)$$

where P = applied force

Regarding the deformation due to bond slip at the steel-grout interface and creep, the authors make reference to the

Fig. 3.4 Idealization of Anchor Arrangement (after Kay and Qamar, 1978)

Fig. 3.5 The Calculation Drawing and Symbols (after Khan'zhov et. al., 1975)

work of several other researchers. However, they conclude that these two factors are unlikely to be significant factors.

3.4 Khan'zhov and Khan'zhov (1975); See Figure 3.5

Khan'zhov et al. reduce the problem of a circular plate anchor, embedded in an infinite depth to the solution of the integral equation

$$\delta = \int_0^R q(\rho) \cdot k(x, \rho) \cdot d\rho \quad (18)$$

with the boundary condition

$$P = 2\pi \int_0^R q(\rho) \cdot \rho d\rho \quad (19)$$

The displacement of the plate is represented by δ , and $k(x, \rho)$ is a function expressing the displacement of the upper boundary of the gap along a circumference of radius, ρ . The intensity of the contact pressures caused by the applied force P , is represented by $q(\rho)$.

The method of finite differences is used to transform these expressions into their algebraic form. The authors express the applied load in two parts, axisymmetric and antisymmetric with respect to the gap created, in order to simplify the displacement function. The displacement of the anchor in a compressible medium when loaded by a concentrated vertical force P , is then given as

$$\delta = 1.748P \cdot (1 - \nu) / \pi^2 r G \quad (20)$$

BALLA, (1961) Scale: model & field Material: c, ϕ Anchor: circular plate	$F = D^3 \gamma [F_1 + c F_2 / \gamma D + F_3] + G_2$ where G_2 = wt. of anchor less displaced soil F_1, F_2, F_3 - see figure 10, (Ref. 6)
MEYERHOF et al. (1968) Scale: model & field Material: c, ϕ Anchor: circular plate	Shallow: $F = W + \pi c B D + 0.5 \pi s_y B D^2 K_p \tan \delta$ Deep: $F = W + \pi c B H + 0.5 \pi s_y B H (2D - H) K_p \tan \delta$ where $\delta = \frac{2}{3} \phi$ (sand); s - see Table 2 (Ref. 35) H - see Table 1 (Ref. 35)
LITTLEJOHN, (1970) Scale: field Material: c, ϕ Anchor: grouted	Cohesionless: $F = K \gamma (D + L/2) \pi B L \tan \phi + 0.25 \pi N_q \gamma D (B^2 - B_o^2)$ Cohesive: $F = \pi B L c + 0.25 \pi (B^2 - B_o^2) N_c c + \pi B_o L_o C_a$ where N_q, N_c are bearing capacity factors
VESIC, (1971) Theoretical analysis Matn.: isotropic elastic-plastic solid Anchor: various shapes	$P_u = W + A(c F_c + D F_q)$ where F_c and F_q are breakout factors dependent on anchor shape, friction angle and depth ratio, D/B , (see Table 1, 2 & 3 in Ref. 47) P_u = pressure needed to expand cavity
WERNER, (1975) Scale: model Material: ϕ Anchor: horizontal plates and cylinders	$F = 1.5 \pi B L \gamma D A u \psi$ where λ = lock-in factor, u = shape factor ψ = depth factor (see fig. 4, Ref. 49)
KWASNIEWSKI et al., (1975) Scale: model & field Material: ϕ Anchor: circular plate	$F = (\pi B^2 \gamma D / 4) [1 - (2D/B) \tan \phi + (2D \tan \phi / B)^2 / 3]$ residual: $F_r = (\pi B^2 \gamma D / 4) [1 + 2\eta D/B + \rho (2D/B)^2]$ where $\eta = -\cos(2\theta + \phi) \sin \phi / \sin^2 \theta$; $\rho = \eta / 3 \tan \theta$; $\theta = 90 - \phi$
ISMAEL et. al., (1979) Scale: field Material: ϕ Anchor: concrete piers	$F = W + 0.5 \pi D^2 B \gamma K \tan \phi$
KULHAWY et al., (1979) Scale: field Material: ϕ Anchor: concrete piers	$F = W + 0.5 \pi B D^2 K \gamma \tan \phi$ residual: $F_r = \begin{matrix} 0.7F & \text{(loose)} \\ 0.4F & \text{(dense)} \end{matrix}$

KEY

W = weight of anchor
 $\bar{W}$ = weight of anchor & soil in slip surface
 B = diameter of anchor
 B_o = diam. of anchor shaft
 L = length of anchor
 L_o = length of anchor shaft
 D = depth to top of anchor
 γ = unit weight of soil
 c = unit cohesion
 C_a = adhesion
 ϕ = angle of internal friction
 K = lateral earth pressure coefficient, in uplift
 K_o = 'at rest' earth pressure coefficient
 K_p = passive earth pressure coefficient
 F = maximum uplift force

Table 3.1 Summary of Various Analytical Studies of Anchors.

CHAPTER IV

EXPERIMENTAL PROGRAM

4.0 Program Development

The dearth of theoretical solutions to the anchor group problem is testimony to the difficulties involved in analyzing this indeterminate system. The analysis is made more difficult by the fact that the mechanism of load transferral varies with anchor geometry - a grouted anchor derives its resistance to uplift from the shear force mobilized on its surface and a plate anchor relies more on the overburden and passive resistance of the soil. This in turn will influence anchor performance when group interaction is studied.

In planning the program, it was necessary to ensure that the results obtained would be as universally applicable and as practical as possible. The scale of the tests was limited by the facilities available and it was possible to simulate only plane strain conditions. Consequently, only line group configurations were used. Nevertheless, a line of anchors under plane strain conditions can be idealized as a section of a larger, square configuration of anchors; see Appendix III.

Despite the fact that the use of contacting and non-contacting anchor caps are equally widespread, only a rigid non-contacting anchor cap was used. Justification for this lies not only in the ease with which the system could be assembled

but in the research of Butterfield and Banerfee, (1971). They conclude that with an increase in stiffness of 5 - 15% for the contacting cap over its non-contacting counterpart, there was little difference in their load-displacement behavior. In anticipation of a non-uniform load distribution, each anchor was instrumented for load and displacement measurements.

The experimental program was developed for the primary purpose of studying the working load range behavior of anchors in a group. Hence, the anchors were loaded and unloaded in a series of cycles instead of a single continuous loading. The main part of the program involved the testing of single and group anchor systems in a cohesionless medium. The single anchor tests were needed to establish the basis on which the anchor group behavior would be judged. The only variable for the group test was the spacing of the anchors, every effort being expended to keep the soil conditions constant throughout the program.

The facilities needed to perform similar tests in clay efficiently were not available. However, by performing small scale single anchor tests in a cohesive medium, it was possible to explore another possibility that exists regarding the application of anchors, i.e. insitu soil testing. This was proposed by Selvadurai and Nicholas (1979), and several experiments were performed to substantiate the claim.

4.1 Experimental Apparatus

4.1.1 Model Tank

All the tests performed in the cohesionless medium, utilized a rigid steel tank with internal dimensions of 1.52m by 0.38m by 1.07m. Details of its structural components are provided by Kempthorne (1978). Additional bracing, in the form of two 6" x 4" x 0.375" rectangular hollow steel sections was used to prevent bulging of the tank walls. Friction on the interior walls was minimized by using sheets of polished No.16 gauge stainless steel. It was not necessary to anchor the tank to the floor as no vibratory loads were being applied and the uplift forces would be insignificant compared to the 1400kg weight of the tank when full.

The same was true of the box used for the single anchor tests in the cohesive medium; see Farbar, 1977. It had internal dimensions of 0.31m x 0.31m x 0.56m, and had a weight of 100kg when full. A metal plate, 0.5" thick, was used to support the base when the box was placed in the loading device. Sheets of plexiglass were utilized to reduce friction on the interior of the box. During compaction, modified column clamps were placed around the box because they provided greater lateral restraint than the clamps used while the box was in the loading device.

4.1.2 Loading Devices

Two loading devices were used in the experimental program.

DETAIL A

DETAIL B

FRONT VIEW

SIDE VIEW

Fig. 4.1 Support System and Load Mechanism

The loading rig, that was used for the major study, was designed and fabricated, at Carleton University, in such a way that it could be adapted easily for future research.

A manually operated pump and a hydraulic jack were used to apply the pulling forces. The system was capable of applying loads in 4N increments. However, the same was not true during the unload cycle. It was not possible to reverse the flow of hydraulic fluid without releasing the incipient pressure and the corresponding load. Nevertheless, it was possible to return to and maintain the initial load, provided it was less than 100N.

The support system framed a 2.5m square area; see Figure 4.1. Thus the load could be applied at any point within a 2.1m square area. Despite the absence of lateral and vibratory loads, the support columns were braced against the walls of the testing enclosure.

The Instron Machine* was used to apply the loads to the screwplate in the cohesive medium. The anchor shaft was clamped at the top and the supporting table lowered at a constant rate, thus loading the anchor.

4.1.3 The Test Anchors

The device used in the major program was a helical anchor. But, with the helices closely spaced, a more suitable designation would be a screw anchor. Basically, it offered

*a constant strain rate universal testing device, manufactured by Instron Co. Ltd.

the ease of handling and the flexibility needed for laboratory studies of this type. Once the test medium was prepared, the anchor could be screwed into the soil to whatever depth was required. It was expected that installation of the anchor would change the density of the deposited material; this quality being decreased at shallow depths and increased at greater depths where dilation is restricted.

In this way, the screw anchor is not unlike the pressure grouted anchor, which, when installed, has a zone of highly compacted material around it. In addition to this, both the screw anchor and the grouted anchor have anchorage zones of fixed length and diameter.

In its basic form, the screw anchor consisted of a helical section (see Plate 4.1), and a connecting rod. The helical section had a diameter of 0.06m and a pitch of 0.03m. The single anchor tests were performed using sections that were 0.1m and 0.24m in length. Only the 0.1m long sections were used for the group tests. Rods of different lengths were also used. Consequently, numerous configurations could be assembled by varying the anchor - shaft combination, and the effect of changes in the active anchor length and the depth of embedment could be studied..

For tests in the cohesive medium, one cycle of a helix was used; (see Plate 4.2). It was essentially the same as a plate anchor but could be installed by a mechanical screwing action.

Plate 4.1 Screw Anchor

Plate 4.2 Screwplate

4.1.4 Data Acquisition

All the devices used in the acquisition of data were electrical in nature. For the tests in sand, the loads were monitored using model SM-100 Interface load cells. These were 'four active arm' strain gauges which require an excitation voltage of 10 vdc to give a full scale output (450N) of 25 mV. The important feature of these devices was the semi-rigid mounting of the active end to the body of the cell. Consequently, the cells were insensitive to lateral loads and registered only the vertical load applied to the anchor.

Direct current displacement transducers (DCDT's), as supplied by Hewlett Packard Ltd., were used to monitor the displacement of the anchors. Each had a stroke of 2.5 cm. One was placed at diametrically opposite points on each anchor shaft to give an indication of the occurrence of tilting. The DCDT's were linear variable differential transformers with a built-in 6 vdc excited carrier oscillator and phase sensitive demodulator system. The dc input is converted to ac, which then excites the primary windings of the transformer. Displacement of the transducer's moveable core causes a proportionate voltage change in the secondary windings. The voltage is then reconverted to dc. The specifications and performance data for both the load cells and the DCDT's are presented in Appendix I.

A multi-channel digital strain indicator was used to record the output from the load cells and the displacement

transducers. The system, model SY-161 minisystem, was manufactured by B & F Instruments Ltd. and supplied by Intertech-ology Ltd. It was capable of displaying and recording all nine channels used - sequentially in 7 secs. It was necessary to convert the output from this system into the required units by the application of the appropriate conversion constants. Details of the operational attributes the B & F system are provided by Kempthorne, 1978.

The Instron machine had built-in data acquisition systems. However, on several occasions, an x-y plotter was connected to it in order to observe the load - displacement curve as the test progressed.

4.2 Cohesionless Material

4.2.1 General Characteristics

The cohesionless material which was used was Flint grade sand supplied by the Ottawa Silica Company, Ottawa, Illinois. Its commercial designation was No. 24 grade, medium, white sand with rounded to subrounded particles. Using the methods outlined by Bowles, 1970, the values obtained for the specific gravity, and the maximum and minimum densities were 2.65, 1880 kg/m³ and 1500 kg/m³ respectively. The material was poorly graded, as indicated by a uniformity coefficient of 1.7, and the grain size distribution shown in Figure 4.2

4.2.2 Placement Procedure

As mentioned previously, the criterion which dictated

the placement method was the necessity to perform all the tests in similar soil conditions. The technique that was employed has been used successfully by several researchers; Atefi, 1975; Drupsteen, 1977; Kempthorne, 1978.

The technique entailed raining the sand into the test box, at a predetermined rate, while the sand hopper traversed the box at a constant velocity. Although it was possible to control the rate of deposition, it was not possible to move the manually operated hopper at a constant velocity. Using gradation markings on the side of the tank, the increase in layer thickness per sweep of the tank was maintained at 1 cm.

The rate of deposition was easier to control. There was a metal plate with a regular pattern of holes at the base of the hopper. By varying the spacing of the holes and the number of holes per unit area, the rate of deposition could be controlled. The holes also served to separate the sand particles; thus maximizing the energy imparted to the particles. The height also influenced the density. However, tests performed by the author as well as Drupsteen (1977), indicated that the density was not increased significantly by depositing the sand at a height greater than 0.2 m. Hence, the height of deposition was maintained at this level or greater.

Using a 1.6 mm. hole diameter, with a spacing of 2.8 mm., a density of 1800 kg/m^3 was obtained. This converted to a relative density of 91% and a 'dense' classification, based on the criteria outlined by Holtz, 1973.

4.2.3 Shear Properties

Although the placement density of the sand was known, the density after installation of the anchor could be entirely different. A series of direct shear tests were performed in order to determine the effect of change in density on the shear properties of the sand.

The angle of internal friction at the maximum density was 48° and 29° at the minimum density; 41° being the value obtained at the placement density of 1800 kg/m^3 ; see Figure 4.3. Of equal importance is the shear modulus of the sand. Figure 4.4 indicates that the shear modulus varies linearly over the range of normal stresses applied, for sand at the placement density. Unfortunately, it was not possible to apply normal stresses of less than 10 kPa in order to ascertain whether or not there would be a change in the slope of the curve in this range

4.3 Cohesive Material

4.3.1 General Characteristics

A synthetic cohesive material, E.P.K. Florida clay, was used. It is referred to as kaolin because it exhibits the basic properties of the clay mineral kaolinite. It was whitish grey in colour and had a specific gravity of 2.85. The liquid limit and plastic limit were 49% and 31% respectively. Proctor compaction tests revealed that the maximum dry density, 1.8 gm/cc , is obtained at a moisture content of 19%. However,

Fig. 4.3 Plot of Shear Stress Versus Normal Stress, for Ottawa Sand

Fig. 4.4 Plot of Shear Modulus Versus Normal Stress for Ottawa Sand

these tests were performed for purposes of classification only because the placement method used was completely different.

4.3.2 Placement Procedure

The kaolin was compacted using a Tinius Olsen "constant stress rate" testing machine. First the material was hand-mixed to a moisture content of 30%, and then stored for a week prior to compaction. The soil was placed in the box and compressed until a stress of 600 kPa was attained. This stress level was maintained for 15 mins. and resulted in a bulk density of 1.8 gm/cc. Because the volume of the material decreased substantially during compaction, it was necessary to increase the initial volume of the box. A detachable segment, which increased the depth of the box by 50%, was used in the initial stages of compaction. This removed the need to refill the box during the compaction process and the accompanying layered effect.

4.3.3 Strength Properties

The Proctor compaction tests also revealed that, with a moisture content of 19%, the soil would be much too stiff to facilitate the installation of the screwplate. Triaxial samples, with moisture content varying from 14% to 35%, were tested in order to select a moisture content which would be suitable in terms of strength and stiffness. These were important factors to consider because rigid body displacement was an assumption made in the theoretical formulation of the

Plate 4.3 Anchor Installation Template

problem. Elongation of the shaft would also be minimized.

The results are summarized in Figure 4.5. On the basis of these results, a moisture content of 30% was chosen. The elastic modulus of the anchor region was 55 times that of the clay under these conditions. Saturated triaxial samples, at 30% moisture, possessed an unconsolidated undrained compressive strength of 210 kPa.

4.4 Testing Procedure

All the tests in sand were carried out in a similar manner. The sand was placed in the tank as previously described. Then, using a placement template (see Plate 4.3), the anchors were installed at preselected locations in the tank. A spirit level was used to ensure that the anchor shaft was vertical. Once the anchor was installed, the platform to be used for displacement measurements and the load cell were attached to the shaft. The anchor cap was then clamped to the load cell and the hydraulic jack. For the group tests, it was necessary to ensure that the anchor cap was horizontal before it was clamped to the load cell and the jack. Lastly, the displacement transducers were placed in position and the zero readings taken; see Plate 4.4a and 4.4b.

All the anchors were loaded in increments of 20N., with each increment being maintained until the displacement was constant. The theoretical elastic extension of the anchor rod was calculated and subtracted from the measured displacement. The load was cycled four times between 0 and

Figure 4.5 The Effect of Moisture Content on the Properties of Compacted Kaolin

Plate 4.4a Single Anchor Test Setup

Plate 4.4b . Group Anchor Test' Setup

40% of the ultimate load, and then it was increased until failure occurred. After each test was completed, the data was converted so that the success of the test could be assessed immediately. Each test was repeated at least three times, and in some cases - single anchors - tests were repeated eight times.

The tests performed in kaolin, were less tedious because the loading device contained all the data acquisition systems. Consequently, once the soil was compacted, and the anchor installed, the box could be placed in the Instron machine and the test commenced; see Plate 4.5. The loading in this case was continuous; a consequence of the constant rate of displacement of 0.02 in/min. The load was cycled between 0 and 50% of the ultimate load and the load - displacement curve was plotted as the test proceeded.

Three series of tests were performed, with each series consisting of two tests at each of three different depths. The screwplate was first installed to a depth of 0.15m. After testing, it was lowered to 0.25m., and loaded again. This was repeated at a depth of 0.35m.

Plate 4.5 Orientation of Test Box in Instron Machine

CHAPTER V

RESULTS and DISCUSSION

5.0 Tests in a Cohesionless Medium

In this section, the experimental data obtained from the tests in sand will be examined. The single anchor test data are presented first in order to emphasize that an appreciation of the behavior of an isolated anchor is required for a better understanding of anchor group behavior. These results will be used as the basis for discussion of the anchor group results. Two aspects of single anchor performance are examined; namely, the load - displacement behavior and the stiffness. The factors involved in the prediction of uplift capacity are also discussed. The anchor group results are presented in a similar manner, along with an analysis of the variation in ultimate group load.

5.1 Single Anchor Performance

5.1.1 Load - Displacement Behavior

The uplift load - displacement curves, presented in Figure 5.1, depict the behavior of a single anchor after being subjected to a series of load - unload cycles. In order to initialize the displacement, it was necessary to subtract any permanent deformation that had occurred as a result of the load cycles. The cyclic displacement was al-

ways less than 5% of the displacement at failure and the reduction in ultimate strength, where apparent, was minimal.

The uplift load, for a given level of displacement, increases with depth of embedment, for both anchor aspect ratios (L/B); see Figure 5.1. This experimental observation substantiates the findings of Hanna et. al., 1972. An increase in the aspect ratio, also causes an increase in the uplift resistance at each displacement level. However, the increase is not as substantial as that caused by an increase in embedment depth.

There was visual evidence of failure at the surface of the sand for tests at depth ratios 1.5 and 4 only. It may be noted that, for deep anchors, failure will not be visible on the surface. Hence the transition from shallow to deep anchor behavior occurs between depth ratios 4 and 7.

5.1.2 Stiffness

It is possible to express the stiffness of the soil-anchor system in several ways. As previously mentioned, shear forces on the slip surface constitute the major portion of the anchor uplift resistance. Therefore, the ideal expression for the stiffness should encompass the shear deformation properties of the medium. However, the shear modulus of the sand varies with confining stress. For this reason, it is convenient to use the initial unit weight of the medium as a normalizing factor. The non-dimensional form chosen

Figure 5.1: Load - Displacement Behavior of the Single Anchor

for the deformational stiffness of the anchor is

$$K = \frac{P}{\delta} \frac{1}{B^2 \gamma} \quad (21)$$

where

P = applied load

δ = displacement of anchor at load level P

B = diameter of the anchor

γ = unit weight of the sand before anchor installation.

The importance of the load cycles is readily apparent because to calculate the stiffness, it is necessary to establish and estimate the load per unit displacement within the cyclic load range. A linear regression technique was used in order to eliminate any inconsistencies in the evaluation of the stiffness.

Although the stiffness of the soil-anchor system increases significantly with embedment depth, only a marginal increase is effected by increasing the aspect ratio of the anchor. This pattern is summarized in Figure 5.2, where the effect of depth ratio and aspect ratio on the stiffness of an isolated anchor is shown.

It would be reasonable to expect an increase in the stiffness with increasing depth since the deformability characteristics of a granular medium is influenced by the depth of embedment and the confining stress; which is directly proportional to the depth of embedment. Anchor installation will also influence the stiffness through its effect on the density of the medium. At large depths of embedment, anchor install-

Figure 5.2 The Effect of Depth Ratio on Anchor Stiffness

tion produces a zone of highly compacted material around the anchor, mainly because the dilatancy of the soil is suppressed. Near the surface, the density will, at best, remain unchanged. Realistically, the density will decrease as the overburden is too small to offer resistance to surface heave.

The increase in stiffness for a cohesionless medium is in contrast to the behavior of an anchor located in a cohesive medium under the same conditions. In this instance, the stiffness of the cohesive soil will approach a limiting value as the depth or confining pressure is increased.

5.1.3, Single Anchor Ultimate Capacity

The preceding discussion suggests that inaccurate predictions of anchor uplift capacity can be the result of the use of sand properties which are unrepresentative of the medium after anchor installation; and although they are assumed to be equal, the diameter of the failure surface is often greater than the diameter of the anchor. However, Shields et. al. (1978), suggest that the lateral earth pressure coefficient in uplift* is the factor most responsible for inaccurate predictions when dealing with grouted anchors. They report values as high as 20 for the ratio of radial stress to overburden pressure. It is reasonable to assume that shearing resistance is the major component of the uplift resistance developed by the anchors used in this study. Consequently, the problems inherent

*hereafter referred to as the lateral coefficient

in the estimation of the uplift capacity of grouted anchors will be encountered in experimental investigations of this nature. Table 5.1 lists the estimates of the uplift resistance based on the works of several researchers, for grouted and circular plate anchors. The experimentally observed uplift capacity is also listed. It is assumed that frictional resistance constitutes 85% (ratio of the lateral surface area to the total active surface area) of the maximum load. A value of 2.3 for the lateral coefficient is inferred from this.

In each case, the uplift capacity is comprised of a dead weight component and a frictional component. Nevertheless, the predicted uplift capacities differ in magnitude.

Balla's (1961) prediction is quite close to the experimental capacity reported in this thesis; however his equation emphasizes the effect of the third power of the anchor depth. The effects of dead weight and shearing resistance are accounted for in the non-dimensional parameters used. Kwasniewski et. al. (1975) combine the weight of the cylinder of soil directly above the anchor with the resultant shearing force on an assumed shear surface. Although the lateral coefficient is relatively small, overestimation may be attributed to the size of the assumed failure surface. Meyerhof and Adams (1968) also assume the shape of the failure surface, but the lateral coefficient is twice the inferred value.

The predictions for a grouted cylinder, highlight more clearly the importance of the accurate estimation of the rel-

Author	Ultimate Capacity (N)	Frictional Component (N)	Lateral Coefficient
<u>Circular Plate</u>			
Balla, (1961)	220	110	τ
Meyerhof et.al., (1968)	1020	1000	$K_p = 4.81$
Kwasniewski et. al., (1975)	320	308	$\sin^{-1}(90-\phi) = 1.33$
<u>Grouted Cylinder</u>			
Littlejohn, (1970)	547	130	1.5
Ismael et. al., (1979)	156	136	1.5
Kulhawy et. al., (1979)	228	208	$\sqrt{K_p} = 2.19$
<u>Observed Value</u>	$L/B = .2$ $D/B = 4$	240	2.3

Table 5.1 Ultimate Capacity of Screw Anchor in Sand, as Predicted by Various Researchers

evant soil parameters. Based on the results of field tests, Littlejohn (1970) uses a single bearing capacity factor to calculate the direct contribution of the overburden to uplift capacity. A value of $N_q = 35$ is suggested and skin friction constitutes only 25% of the uplift capacity. Ishmael et. al. (1979) and Kulhawy et. al. (1979) use identical methods of analysis. However, their predictions are radically different because of the value of the lateral coefficient used. Kulhawy et. al. (1979) suggest that $\sqrt{K_p}$ (square root of the passive earth pressure coefficient) provides the best estimate of the insitu lateral coefficient in dense sand. Their prediction of the ultimate capacity comes closest to the values observed in the tests reported here.

The effect of changes in other parameters, such as the diameter of the failure surface and the friction angle are of less importance, although they are by no means insignificant. The friction angle will increase with density, however, the net effect on the lateral coefficient will be negligible.

5.2 Anchor Group Behavior

5.2.1 Ultimate Load Capacity

The effect of anchor spacing on the ultimate capacity of an anchor group is summarized in Figures 5.3 and 5.4. A total of 24 tests were performed on a 2 - anchor group and 16 tests on a 3 - anchor group. The load - displacement curves are shown in Figure 5.3. The group load was put in a non-dimensional form by division with the ultimate capacity of an

Figure 5.3 Load - Displacement Behavior of the Anchor Groups

isolated single anchor. The curves depict the behavior of the group after it has been subjected to a series of three load - unload cycles. The total load on the group at each level of displacement was determined by adding the individual anchor loads. The load distribution in the 3 - anchor group was non-uniform, with the central anchor carrying approximately 30% of the ultimate group load (as compared to 35% for the outer anchors). At low load levels, as applied during the loading cycles, the distribution was uniform. These observations are in agreement with the findings of Yilmaz et. al. (1971). Increasing the spacing of the anchors by 100%, resulted in a 25% increase in the uplift resistance at each level of displacement.

The anchor spacing has an even smaller effect on the uplift capacity of the group; see Figure 5.4. Although both anchor groups obtain 100% efficiency at a spacing factor of 7, there is only a marginal decrease in efficiency at significantly lower spacing factors. Of the two groups, the load efficiency of the 3 - anchor group appears to be affected more by changes in anchor spacing. Das et. al. (1976) reported sharp decreases in the group efficiency at spacing factors less than 3. Unfortunately, smaller spacing factors were unattainable with the anchor cap used in the present series of experiments. As a results, it was not possible to verify the trend documented by Das et. al. (1976). Nevertheless, a hypothetical estimation of the ultimate group load was attem-

Figure 5.4 Variation of Ultimate Load With Anchor Spacing

pted. For the minimum spacing factor of 1, the cross-sections of the assumed failure surfaces are shown in Figure 5.3. The friction force was calculated using a value of 2.3 for the lateral coefficient. The dead weight component in each case was estimated by utilizing the ratio of the lateral surface area to the total active surface area; as outlined in a previous section.

As expected, the efficiency is greatly reduced. It is possible that the efficiency would decrease sharply due to anchor interaction. It is also conceivable that the lateral coefficient would increase due to the increased stiffness of the system and the stress locked-in by anchor installation. If the uplift capacity of the anchor group is assumed to increase in proportion to the increase in active surface area, this would necessitate values of 4.79 and 4.67 for the lateral coefficient for 2 and 3 - anchor groups respectively. These values are much higher than the value of 2.3 suggested in the previous section and approach the upper limit, the passive earth pressure coefficient, suggested by Kulhawy et. al. (1979).

5.2.2 Group Stiffness

In order to illustrate the effect of anchor spacing on group stiffness, the ratio of group stiffness to individual anchor stiffness (K_{grp}/K_{single}), was plotted against the spacing factor; see Figure 5.5. The equation used to evaluate the stiffness of the anchor group was essentially the same as

Figure 5.5 The Effect of Spacing on Group Stiffness

that used for the single anchor. The group load was substituted for the single anchor load.

Unlike the ultimate load, group stiffness varies significantly with anchor spacing. At spacings greater than six diameters, anchors in both groups each attain stiffnesses consistent with the stiffness of a single isolated anchor. The results are that the stiffness of the 2 - anchor group is twice that of a single isolated anchor, and the stiffness of the 3 - anchor group is three times that of an isolated anchor. Beyond the apparent isolation spacing of six anchor diameters, there is a similarity between the group anchor behavior, depicted by these results, and the behavior of a group of springs loaded in parallel. In this situation, the group stiffness is the sum of the individual anchor stiffnesses.

The stiffness of both anchor groups decreases with anchor spacing. At spacings of less than four anchor diameters, the group stiffness decreases to its lower limit. The lower limit of the stiffness of the 2 - anchor group appears to be 40% less than the single anchor stiffness; while the minimum stiffness of the 3 - anchor group is approximately equal to twice the stiffness of the single anchor.

The mechanism by which the screw anchor develops resistance to uplift suggests that the failure surface is cylindrical in shape, having a diameter approximating the diameter of the anchor. This implies that the zone disturbed by anchor movement does not extend beyond the cylindrical surface of the anchor itself. Consequently, the range of spacing

factors between total anchor interaction and no interaction at all, would be quite small. As shown in Figure 5.5, the stiffness of the anchor group is highly sensitive to changes in the anchor spacing only between spacing factors 4 and 6. Unexpectedly high prefailure movement would occur should the spacing be less than six anchor diameters.

5.3 Tests in a Cohesive Medium

The results obtained from tests performed in compacted kaolin will be examined in this section. These results will enable an assessment to be made of the accuracy to which the theory of Selvadurai and Nicholas, (1979), can be used to evaluate the insitu undrained modulus of a cohesive medium. The proposed theoretical formulation will be presented first and then used to determine the undrained modulus from the load - displacement characteristics of the anchor; namely a single cycle of a helical anchor. A comparison will be made with the results obtained from the triaxial tests, performed on undisturbed samples of the test medium.

5.3.1 Evaluation of the Deformation Modulus

The model used to analyze the anchor problem is similar to that used in the theoretical assessment of the screwplate, attempted by Selvadurai and Nicholas, (1979). In the development of the problem, simplifying assumptions are made regarding the soil - anchor system.

The anchor is assumed to possess finite flexural rigidity and the interface between the anchor and the soil is

considered to be frictionless. In view of this, oil was applied to surface of the anchor before it was installed. The soil medium is idealized as an isotropic and elastic medium; and although the anchor embedment depth never exceeds six times its diameter, the effect of the free boundary on the anchor deflection is neglected. Soil disturbance will have a strong influence on the behavior of the anchor because the stiffness of the column of soil directly above the anchor will be greatly reduced due to the cutting action of the anchor as it is installed. This will be accounted for in the results obtained and should not significantly affect the applicability of the technique as a means of determining the insitu undrained modulus of the soil.

The load - deflection behavior of the anchor is given by the expression

$$\pi a E \cdot \frac{\delta}{p} = \begin{cases} 0.77 & \text{for } \log R > 1 \\ 0.88 & \text{for } \log R < 1 \end{cases} \quad (22)$$

The displacement per unit load of the anchor is represented by δ/p , while a is the diameter of the anchor helix and E is the insitu undrained modulus of the soil. The relative rigidity R , of the soil - anchor system is defined as

$$R = \frac{\pi}{12} \cdot \frac{(3-4\nu)(1+\nu)}{(1-\nu_p)(1-\nu)} \cdot \frac{E_p}{E_s} (h/a)^3 \quad (23)$$

2 2
OF/DE

where E_p & ν_p = elastic constants of helix

h = thickness of helix plate

E_r = modulus obtained from triaxial tests

$\nu = 0.5$

Using these two equations, and the load per unit displacement obtained from the anchor tests, the undrained modulus of the soil can be calculated.

The results are presented in Table 5.2. It was not possible to present clearly the load displacement cycles of the anchor without excessive distortion of the horizontal displacement axis. Separate cycles were indiscernable because the inelastic soil deformation due to the load cycles was practically zero.

Although the stiffness of the soil - anchor system increases with depth, the percentage increase in stiffness per unit increase in depth varies with each test. The probable cause of this is stress relaxation as a result of swelling that may have occurred as the clamps on the box were being changed.

5.3.2 Comparison with Triaxial Results

The values for the undrained modulus, derived from the stress - strain data, refer to the initial modulus; with the samples being stressed over a range of zero to 50% of the ultimate shear stress. Processing of the data involved fitting the data with a hyperbola, having parameters, $c = 1/E_r$

Depth Ratio (D/a)		P/ δ (kN/m)	E (kPa $\times 10^4$)
Series II	2.5	2200	2.1
Box 1	4	5500	5.1
	6	7300	6.8
Series III	2.5	1700	1.6
Box 1	4	3200	3.0
	6	4200	3.9
Series III	2.5	1700	1.6
Box 2	4	2000	1.9
	6	2700	2.5

Table 5.2 Summary of Results Obtained from Anchor Uplift Tests in Compacted Kaolin

Depth Ratio (D/a)		σ_3 (kPa)	E_1 (kPa $\times 10^3$)	E_6 (kPa $\times 10^3$)	E (kPa $\times 10^4$)	E_6/E
Series II	2.5	13	2.3	5.1	2.1	2.4
Box 1	4	37	4.4	9.5	5.1	1.9
	6	55	5.2	14.7	6.8	2.2
Series III	2.5	13	1.5	4.0	1.6	2.5
Box 1	4	37	2.3	6.8	3.0	2.3
	6	55	3.1	8.7	3.9	2.2
Series III	2.5	13	1.6	3.3	1.6	2.1
Box 2	4	37	1.6	3.6	1.9	1.9
	6	55	2.8	4.9	2.5	2.0

Table 5.3 Comparison of Moduli Obtained from Undrained Triaxial Tests and Anchor Tests

and $m = 1/(\sigma_1 - \sigma_3)_u$. These parameters are obtained by applying a linear regression technique to variables $x = \epsilon$ and $y = \epsilon/(\sigma_1 - \sigma_3)$; where ϵ is the percentage axial strain and $(\sigma_1 - \sigma_3)$ is the deviator stress. This eliminates the inconsistencies associated with fitting a tangent to the initial portion of the stress - strain curve.

Figures 5.6 and 5.7 show typical results for the triaxial tests. The complete set of results is presented in Table 5.3. This behavior is a consequence of the expected occurrence of some disturbance effects as well as bedding error. The maximum value of the modulus, obtained from the triaxial tests, is compared with the modulus, obtained from the anchor tests. The ratio of the two, varies between 1.9 and 2.5. A distinct factor could not be identified as being responsible for the variation. Marsland, (1975), reported moduli obtained from large diameter plate compression tests in clay as being 2 to 4 times higher than those evaluated from laboratory and pressuremeter tests. However, with a standard deviation of only 0.2 from the mean value of the ratio, the results presented here attest the significance of the analysis proposed by Selvadurai and Nicholas, (1979).

5.4 Scale Effects

Model studies have, for a long time, been an integral part of the process of developing analytical solutions to field problems in soil mechanics. By comparing observed and predicted behavior, researchers have been able to estimate

Figure 5.6 Triaxial Test Results

Figure 5.7 Effect of Load Cycles on Triaxial Moduli

the accuracy of a proposed theory. More importantly, however, model studies provide a means of investigating the influence of systematic changes in various parameters. Throughout this thesis, the emphasis has been placed on the latter; that is, the identification of behavioral trends. Hence, the value of the results presented in this chapter will not be lessened by the scale effects inherent in model studies. The parameters being studied, have been presented in dimensionless form, so that the the results may be applied to situations involving different physical dimensions and properties..

Nevertheless, care should be exercised when applying the results of small scale laboratory tests to large scale field situations. Several researchers (Rocha, 1957; DeBeer, 1963; Graham et. al., 1971; Neely et. al., 1973) have studied the effects of the scale of an experimental study on the applicability of the results obtained. Neely et. al., (1973), report the results of tests which suggest a dependence of pullout loads on the scale of the experiments. These loads are presented as dimensionless coefficients and decrease as the scale of the experiment is increased. Similar conclusions are made by Graham and Stuart, (1971), based on the observed effects of scale on bearing capacity factors. It is also conceivable that a change from two - dimensional to three - dimensional behavior would affect the value of these coefficients. Rocha, (1957), and Davie et. al., (1978), suggest that the discrepancy could be eliminated by maintaining dimensional similarity between the model and the prototype.

This, however, would reduce the range of applications of the results. In order to apply the results of a model trend study to a field situation, in absolute terms, it would be necessary to obtain data from a full scale test and then use these in conjunction with the observed trend to predict the behavior of similar full scale anchors.

CHAPTER VI

CONCLUSION

6.0 Summary

This thesis has presented a general summary of the observed behavioral trends of soil anchors as well as the experimental results of tests performed in both cohesive and noncohesive media.

In Chapters 1 and 2, the review of available literature on soil anchors revealed some of the anchors currently in use, and it also highlighted several factors which affect the behavior of a soil anchor. These factors were:

- (i) Depth of embedment of the anchor;
- (ii) Geometry of the anchor;
- (iii) Material properties of the soil medium;
- (iv) Type and magnitude of the load;
- (v) Spacing of the anchors.

Chapter 3 outlined several analytical and semi-empirical studies, which proposed relationships from which the load - carrying ability of an anchor could be determined.

In the course of the literature review, the need for a better understanding of soil - anchor interaction at working load levels, as well as the interaction of anchors in a group, became evident. The experimental program developed for the purpose of studying these aspects of anchor behavior was described in Chapter 4. Details of the tests that were performed

in the cohesive medium, was also given.

Despite some shortcomings, which have been mentioned in previous chapters, the experimental results indicated that:

A. For tests performed in sand,

- (i) the uplift capacity of the anchor, and the stiffness of the soil - anchor system, increases with depth of embedment and anchor aspect ratio (L/B);
- (ii) the load efficiency of an anchor group is only marginally sensitive to changes in spacing, for spacings greater than four anchor diameters. This implies an isolation spacing of four anchor diameters when dealing with anchor uplift capacity;
- (iii) the stiffness of an anchor group is highly sensitive to changes in anchor spacing, between spacings of four and six anchor diameters. Isolated anchor behavior, with regards to stiffness, is obtained at spacings greater than six anchor diameters. At spacings of less than four anchor diameters, the stiffness of the anchor group decreases to a lower limit, which is dependent only on the number of anchors in the group.

B. For tests performed in compacted kaolin,

- (i) the initial slope of the triaxial stress - strain curves reaches a constant value after two load cycles;
- (ii) the ratio of the initial modulus, obtained from the sixth load cycle of the triaxial test, and the calculated modulus, is not dependent on the anchor depth

of embedment and has a value between 1.9 and 2.5.

6.1 Recommendations for future research

Many parameters have not been considered in this thesis but are important enough to warrant further research. The following are some possibilities:

- (i) The series of tests performed using an isolated anchor can be extended by using a screwplate for future tests.
- (ii) The effect of anchor inclination, to the ground surface, on anchor performance, has been noted by several researchers and should be investigated further.
- (iii) The investigation of anchor group behavior should be continued by testing additional anchor groups, along with line configurations, and by using spacings ~~of~~ less than three anchor diameters
- (iv) Large scale or field tests should be performed to study the effect of scale on the observed trends reported herein.
- (v) With regards to loading, the effect of prestressing, inclined loading as well as load relaxation and creep are pertinent subjects, and warrant further study.
- (vi) The scope of the tests should be extended to include tests in a cohesive medium, at both laboratory and field levels.
- (vii) The results obtained from the screwplate tests in compacted kaolin are encouraging. However, further verification should be obtained by performing additional laboratory and field tests.

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APPENDIX I

Specifications and Performance Data for the Load
Cells and Displacement Transducers

A. Super - Mini Load Cell; Model SM-100

Specifications

output resistance		350 $\pm$ 3.5 ohms*
input resistance		350 $\pm$ 3.5 ohms
excitation		10 vdc
non-linearity		<0.03% F.S.
hysteresis		<0.02% F.S.
comp. temp. range		0°F to 150°F
thermal zero shift		<0.0003% F.S./°F
zero balance		<1% F.S.

Performance Data

Load Cell		Calibration Constant
No. 1		0.4079 N/ $\mu\epsilon$ *
2		0.4073 "
3		0.4012 "
4		0.3910 "

* the output from the B & F Indicator was registered in
microstrain, $\mu\epsilon$.

B. Displacement Transducer; Model 7DCDT-100

Specifications

max. non-linearity		$\pm 0.5\%$ F.S.
excitation		6 vdc
output impedance		3000 ohms
comp. temp. range		-65°F to 140°F
weight of core assembly		2.1 gms

Performance Data

Transducer	Calibration Constant
No. 1	6.3×10^{-6} m/ $\mu\epsilon^*$
2	3.7×10^{-6} "
3	3.2×10^{-6} "
4	4.0×10^{-6} "
5	4.2×10^{-6} "
6	4.6×10^{-6} "

* the output from the B & F Indicator was registered in microstrain, $\mu\epsilon$.

APPENDIX II

Fortran IV Computer Programs

Two programs were used to evaluate the results. Program A calculated the force applied to, and, the displacement of each anchor; along with the theoretical extension of each anchor rod. These values were tabulated for purposes of inspection.

Program B calculated the total force acting on the group and the average displacement of the group. A linear regression analysis was then applied to these data to obtain the anchor load per unit displacement.

The data, obtained from the triaxial tests on the compacted kaolin, were processed using a Texas Instruments SR52 programmable calculator.

Program Outlines.

Program A:

- (i) input the number of anchors and diverge to appropriate program section.
- (ii) input data from B & F Indicator
- (iii) calculate individual anchor force
- (iv) calculate rod extension
- (v) calculate anchor displacement
- (vi) tabulate individual anchor force, displacement and rod extension.

Program B:

- (i) input the number of anchors in group and diverge to appropriate program section.
- (ii) input data from B & F Indicator.
- (iii) calculate individual anchor forces and sum, to obtain total group load.

- (iv) calculate extension of each anchor rod.
- (v) calculate individual anchor displacement and evaluate average displacement of anchors in the group.
- (vi) calculate the group load per unit displacement.

Note: 'group' applies to the single anchors as well as the anchor groups.

PROGRAM A

```

;FLAG ( , MOORE ),DB
DIMENSION A(9)
REAL M
REAL DEL1,DEL2,DEL3,DEL4
C1 = 3.6896
C2 = 3.1606
C3 = 6.3414
C4 = 1.9625
C5 = 2.3942
C6 = 2.1355
C7 = 0.4079
C8 = 0.4012
C9 = 0.4073
AE = 26.2124
AL = 0.3

```

* G305

```

1 READ(105,2) NUM
2 FORMAT (12)
N = 1

```

```

3 IF (NUM) 4,10,16

```

```

4 DO 6 I = 1,3
  READ(105,5) A(I)
5 FORMAT (F7.1)
6 CONTINUE

```

```

70 M = 0
  DF1 = 0
  DF2 = 0
  DU1 = 0
  DU2 = 0
  Y1 = 0
  Y2 = 0
  D1 = 0
  D2 = 0

```

```

7 M = M + 1.

```

```

  READ(105,8) DEL1,DEL2,F1
8 FORMAT (3F7.1)

```

```

IF (DEL1 .LT. -3000.) GO TO 22
IF (DEL1 .EQ. 0) GO TO 24
FORCE1 = (F1 - A(3))*C7
EXTEN1 = FORCE1*AL/AE
DELTA1 = (DEL1 - A(1))*C1 - EXTEN1
DELTA2 = (DEL2 - A(2))*C2 - EXTEN1
DELTA1 = (DELTA1 + DELTA2)/2
DF1 = DF1 + DELTA1*FORCE1
DF2 = DF2 + DELTA2*FORCE1
DU1 = DU1 + DELTA1**2
DU2 = DU2 + DELTA2**2
Y1 = Y1 + FORCE1
Y2 = Y2 + FORCE1
D1 = D1 + DELTA1
D2 = D2 + DELTA2
IF (M .LE. 1.) GO TO 7
IF (M .EQ. 1.) GO TO 7
SLOPE1 = (DF1 - (D1*Y1/M))/(DU1 - (D1*D1/M))
SLOPE2 = (DF2 - (D2*Y2/M))/(DU2 - (D2*D2/M))
WRITE(10B,9) SLOPE1,SLOPE2
9 FORMAT(2(4X,F7.3))
GO TO 7

```

C
C
C
C
C

```

10 DO 12 I = 1,6
   READ(105,11) A(I)
11 FORMAT (F7.1)
12 CONTINUE

```

C

```

130 M = 0
   Y1 = 0
   Y2 = 0
   Y3 = 0
   Y4 = 0
   D1 = 0
   D2 = 0
   D3 = 0
   D4 = 0
   DF1 = 0
   DF2 = 0
   DF3 = 0
   DF4 = 0
   DU1 = 0
   DU2 = 0
   DU3 = 0
   DU4 = 0

```

```

13 M = M + 1.

```

C

```

   READ(105,14) DEL1,DEL2,DEL3,DEL4,F1,F2
14 FORMAT (6F7.1)

```



```

IF (DEL1 .LT. -3000.) GO TO 22
IF (DEL1 .EQ. 0) GO TO 24
FORCE1 = (F1 - A(5))*C7
FORCE2 = (F2 - A(6))*C8
EXTEN1 = FORCE1*AL/AE
EXTEN2 = FORCE2*AL/AE
DELTA1 = (DEL1 - A(1))*C1 - EXTEN1
DELTA2 = (DEL2 - A(2))*C2 - EXTEN1
DELTA3 = (DEL3 - A(3))*C3 - EXTEN2
DELTA4 = (DEL4 - A(4))*C4 - EXTEN2
FORCE1 = FORCE1 + FORCE2
DELTA1 = (DELTA1 + DELTA2 + DELTA3 + DELTA4)/4
DF1 = DF1 + DELTA1*FORCE1
DF2 = DF2 + DELTA2*FORCE1
DU1 = DU1 + DELTA1**2
DU2 = DU2 + DELTA2**2
DF3 = DF3 + DELTA3*FORCE2
DF4 = DF4 + DELTA4*FORCE2
DU3 = DU3 + DELTA3**2
DU4 = DU4 + DELTA4**2
Y1 = Y1 + FORCE1
Y2 = Y2 + FORCE1
Y3 = Y3 + FORCE2
Y4 = Y4 + FORCE2
D1 = D1 + DELTA1
D2 = D2 + DELTA2
D3 = D3 + DELTA3
D4 = D4 + DELTA4
IF (M .EQ. 1.) GO TO 13
SLOPE1 = ((DF1-(D1*Y1/M))/(DU1-(D1**2/M)))
SLOPE2 = ((DF2-(D2*Y2/M))/(DU2-(D2**2/M)))
SLOPE3 = ((DF3-(D3*Y3/M))/(DU3-(D3**2/M)))
SLOPE4 = ((DF4-(D4*Y4/M))/(DU4-(D4**2/M)))
WRITE(108,15) SLOPE1,SLOPE2,SLOPE3,SLOPE4
15 FORMAT (4(4X,F7.3))
GO TO 13

```

C
C
C
C

```

16 DO 18 I = 1,9
  READ(105,17) A(I)
17 FORMAT (F7.1)
18 CONTINUE

```

C

```

190 M = 0
  Y1 = 0
  Y2 = 0
  Y3 = 0
  Y4 = 0
  Y5 = 0
  Y6 = 0

```

D1 = 0
 D2 = 0
 D3 = 0
 D4 = 0
 D5 = 0
 D6 = 0
 DF1 = 0
 DF2 = 0
 DF3 = 0
 DF4 = 0
 DF5 = 0
 DF6 = 0
 DU1 = 0
 DU2 = 0
 DU3 = 0
 DU4 = 0
 DU5 = 0
 DU6 = 0

19 M = M + 1.

C

READ(105,20) DEL1,DEL2,DEL3,DEL4,DEL5,DEL6,F1,F2,F3
 20 FORMAT (9F7.1)
 IF (DEL1 .LT. -3000.) GO TO 22
 IF1 (DEL1 .EQ. 0) GO TO 24
 FORCE1 = (F1 - A(7))*C7
 FORCE2 = (F2 - A(8))*C8
 FORCE3 = (F3 - A(9))*C9
 EXTEN1 = FORCE1*AL/AE
 EXTEN2 = FORCE2*AL/AE
 EXTEN3 = FORCE3*AL/AE
 DELTA1 = (DEL1 - A(1))*C1 - EXTEN1
 DELTA2 = (DEL2 - A(2))*C2 - EXTEN1
 DELTA3 = (DEL3 - A(3))*C3 - EXTEN2
 DELTA4 = (DEL4 - A(4))*C4 - EXTEN2
 DELTA5 = (DEL5 - A(5))*C5 - EXTEN3
 DELTA6 = (DEL6 - A(6))*C6 - EXTEN3
 FORCE1 = FORCE1 + FORCE2 + FORCE3
 DELTA1 = (DELTA1 + DELTA2 + DELTA3 + DELTA4 + DELTA5 + DELTA6)
 DF1 = DF1 + DELTA1*FORCE1
 DF2 = DF2 + DELTA2*FORCE1
 DF3 = DF3 + DELTA3*FORCE2
 DF4 = DF4 + DELTA4*FORCE2
 DF5 = DF5 + DELTA5*FORCE3
 DF6 = DF6 + DELTA6*FORCE3
 DU1 = DU1 + DELTA1**2
 DU2 = DU2 + DELTA2**2
 DU3 = DU3 + DELTA3**2
 DU4 = DU4 + DELTA4**2
 DU5 = DU5 + DELTA5**2
 DU6 = DU6 + DELTA6**2
 Y1 = Y1 + FORCE1
 Y2 = Y2 + FORCE1

```

Y3 = Y3 + FORCE2
Y4 = Y4 + FORCE2
Y5 = Y5 + FORCE3
Y6 = Y6 + FORCE3
D1 = D1 + DELTA1
D2 = D2 + DELTA2
D3 = D3 + DELTA3
D4 = D4 + DELTA4
D5 = D5 + DELTA5
D6 = D6 + DELTA6
IF (M.EQ. 16) GO TO 19
SLOPE1 = ((DF1-(D1*Y1/M))/(DD1-((D1**2)/M)))
SLOPE2 = ((DF2-(D2*Y2/M))/(DD2-((D2**2)/M)))
SLOPE3 = ((DF3-(D3*Y3/M))/(DD3-((D3**2)/M)))
SLOPE4 = ((DF4-(D4*Y4/M))/(DD4-((D4**2)/M)))
SLOPE5 = ((DF5-(D5*Y5/M))/(DD5-((D5**2)/M)))
SLOPE6 = ((DF6-(D6*Y6/M))/(DD6-((D6**2)/M)))
WRITE(108,21) SLOPE1,SLOPE2,SLOPE3,SLOPE4,SLOPE5,SLOPE6
21 FORMAT (6(4X,F7.3))
GO TO 19

```

```

22 N = N + 1
WRITE(108,23) N
23 FORMAT ('0',6MCYCLE ,12)
WRITE(108,30)
30 FORMAT ('0')
IF ( NUM ) 70,130,190

```

```

24 READ(105,25) TEST
25 FORMAT (F3.1)
IF (TEST) 28,28,26
26 WRITE(108,27)
27 FORMAT ('1','##### NEW TEST #####')
WRITE(108,31)
31 FORMAT ('0')
GO TO 1

```

```

28 STOP
END

```

```

$EOD

```

PROGRAM B

```

      FLAG, (      ,MUORE      ),DB,LS      B G47J
      DIMENSION A(9)
      REAL DEL1,DEL2,DEL3,DEL4
      C1 = 3.6896
      C2 = 3.1606
      C3 = 6.3414
      C4 = 3.9625
      C5 = 2.3942
      C6 = 2.1355
      C7 = 0.4079
      C8 = 0.4012
      C9 = 0.4073
      AE = 26.2124
      AL = 0.3

      C
      C
      C
      C
      1 READ(105,2) NUM
      2 FORMAT (I2)
      N = 1

      C
      C
      C
      C
      3 IF (NUM) 4,10,16

      C
      C
      C
      C
      4 DO 6 I = 1,3
      READ(105,5) A(I)
      5 FORMAT (F7.1)
      6 CONTINUE

      C
      C
      C
      C
      7 READ(105,8) DEL1,DEL2,F1
      8 FORMAT (3F7.1)
      IF (DEL1 .LT. -3000.) GO TO 22
      IF (DEL1 .EQ. 0) GO TO 24
      FORCE1 = (F1 - A(3))*C7
      EXTEN1 = FORCE1*AL/AE
      DELTA1 = (DEL1 - A(1))*C1 - EXTEN1
      DELTA2 = (DEL2 - A(2))*C2 - EXTEN1
      WRITE(106,9) DELTA1,DELTA2,FORCE1,EXTEN1
      9 FORMAT (3(2X,F7.1),2X,F4.2)
      GO TO 7

```

C
C
C

```
10 DO 12 I = 1,6
    READ(105,11) A(I)
11 FORMAT (F7.1)
12 CONTINUE
```

C
C

```
13 READ(105,14) DEL1,DEL2,DEL3,DEL4,F1,F2
14 FORMAT (6F7.1)
    IF (DEL1 .LT. -3000.) GO TO 22
    IF (DEL1 .EQ. 0) GO TO 24
    FORCE1 = (F1 - A(5))*C7
    FORCE2 = (F2 - A(6))*C8
    EXTEN1 = FORCE1*AL/AE
    EXTEN2 = FORCE2*AL/AE
    DELTA1 = (DEL1 - A(1))*C1 = EXTEN1
    DELTA2 = (DEL2 - A(2))*C2 = EXTEN1
    DELTA3 = (DEL3 - A(3))*C3 = EXTEN2
    DELTA4 = (DEL4 - A(4))*C4 = EXTEN2
    WRITE(108,15) DELTA1,DELTA2,FORCE1,EXTEN1,DELTA3,DELTA4,
    1,FORCE2,EXTEN2
15 FORMAT (2(3(2X,F7.1),2X,F4.2))
    GO TO 13
```

C
C
C
C
C

```
16 DO 18 I = 1,9
    READ(105,17) A(I)
17 FORMAT (F7.1)
18 CONTINUE
```

C
C

```
19 READ(105,20) DEL1,DEL2,DEL3,DEL4,DEL5,DEL6,F1,F2,F3
20 FORMAT (9F7.1)
    IF (DEL1 .LT. -3000.) GO TO 22
    IF (DEL1 .EQ. 0) GO TO 24
    FORCE1 = (F1 - A(7))*C7
    FORCE2 = (F2 - A(8))*C8
    FORCE3 = (F3 - A(9))*C9
    EXTEN1 = FORCE1*AL/AE
    EXTEN2 = FORCE2*AL/AE
    EXTEN3 = FORCE3*AL/AE
    DELTA1 = (DEL1 - A(1))*C1 = EXTEN1
    DELTA2 = (DEL2 - A(2))*C2 = EXTEN1
    DELTA3 = (DEL3 - A(3))*C3 = EXTEN2
    DELTA4 = (DEL4 - A(4))*C4 = EXTEN2
    DELTA5 = (DEL5 - A(5))*C5 = EXTEN3
    DELTA6 = (DEL6 - A(6))*C6 = EXTEN3
```

WRITE(108,21) DELTA1,DELTA2,FORCE1,EXTEN1,DELTA3,DELTA4,
1FORCE2,EXTEN2,DELTA5,DELTA6,FORCE3,EXTEN3
21 FORMAT (3(3(2X,F7.1),2X,F4.2))
GO TO 19

22 N = N + 1
WRITE(108,23) N
23 FORMAT ('0',6MCYCLE ,I2)
WRITE(108,30)
30 FORMAT ('0')
IF (NUM) 7,13,19

24 READ(105,25) TEST
25 FORMAT (F3.1)
IF (TEST) 28,28,26
26 WRITE(108,27)
27 FORMAT ('1', '***** NEW TEST *****')
WRITE(108,31)
31 FORMAT ('0')
GO TO 1

28 STOP
END

1EOD

APPENDIX III

Description of Experimental Plane Strain Condition

The size of the bin, in which tests using the cohesionless medium were performed, restricted the shape of the anchor groups to line configurations. However, the requirement of zero lateral displacement of the soil medium, needed for the plane strain condition, was not fulfilled. Hence, the plane strain condition referred to in Section 4.0 was incorrect, based on the strict definition of the phrase.

Nevertheless, use of the phrase suggests that the line configurations, reported herein, can be thought of as sections of larger square and rectangular arrays of anchors; as shown below. By ensuring that the frictional forces on the walls of the bin were negligible and that there was zero lateral displacement of the walls, the condition required for the aforementioned simplification would be satisfied.

END

15 · 12 · 80

FIN