GLASGOW UNIVERSITY

DEPARTMENT OF CIVIL ENGINEERING.

The behaviour of plate anchor groups in sand.

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SUMMARY

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This thesis presents the results of an investigation into the uplift resistance of single anchors and various line groups of anchors embedded in dry sand. Fifty-one tests were conducted using 25mm diameter model plate anchors. Anchor spacings of S = 2B, 4B, 6B, (B = diameter of an anchor) and depth of anchor embedment (D) ranging from 3B to 15B were studied.

The author's experimental results are presented in dimensionless factors and ratios so that a possible dimensional similarity between the experimental results and the behaviour of full-scale prototypes can be established. The following general conclusions can be drawn from the author's experimental results.

It can be observed that for a given anchor spacing, the ultimate group efficiency (Ef) decreases to a minimum with increasing D/B ratios, and then increases to as large a value as that found with small values of D/B ratios. There appears to be a distinct critical D/B ratio where (Ef) is a minimum value. For example, from the author's test results for a (2x1) anchor group at S/B = 4, the (Ef) value decreases to a minimum of about 70% with increasing D/B ratios, and then increases to as large a value as about 80%.

It can also be observed that for a given anchor group size, at anchor spacings of 2B, 4B and 6B, the minimum ultimate group efficiency corresponded to critical D/B ratios of about 8, 10 and 12 respectively. Hence, at anchor spacing of 2B, "shallow" anchors in groups are anchor groups with $D/B \leqslant 8$ and "deep" anchor groups have $D/B > 8$. At anchor spacings of 4B and 6B, "shallow" and "deep" anchor groups can be similarly classified.

As anchor spacings increase the (Ef) value also increases. However, as group sizes increase the (Ef) value decreases. For anchor groups at very close spacings (S = 2B), the influence of group configuration on (Ef) is not obvious.

The displacement ratio (Δr) is defined as the displacement of the group at failure to the displacement of a single isolated

anchor at failure. It can be observed that as anchor spacings and group sizes increase, Δr value increases. However, as D/B ratios increase the Δr value decreases.

The distribution of load amongst a group of anchors is non-uniform at failure. Generally the load carried by an anchor is proportional to its distance from the centre of the group where the load is applied. In line groups, the "centre" anchor is shown to carry the smallest load and the "end" anchor the largest load.

The author's experimental results and previous theories are compared and discussed for line groups of anchors. The difference between Meyerhof and Adam's predicted (Ef) results and the author's observed (Ef) results are discussed. In this case, Meyerhof and Adam's predicted (Ef) results ranged from 12.7% greater to 49.8% greater than the author's observed (Ef) results. The range of the differences from Meyerhof and Adam's theory is wide and this suggests that Meyerhof and Adam's theory for predicting the behaviour of anchor groups is not entirely satisfactory.

The difference between Yilmaz's (first analysis for "shallow" anchors) predicted (Ef) results and the author's observed (Ef) results for "shallow" anchor groups (at D/B = 6) are discussed. In this case, Yilmaz's predicted (Ef) results ranged from 6.5% greater to 7.3% less than the author's observed (Ef) results.

In this present investigation a simple analysis has been derived for predicting the ultimate uplift load of "shallow" and "deep" vertical anchors in line groups installed in sand and subjected to static loadings. The difference between the proposed predicted (Ef) results and the author's observed (Ef) results are discussed. In this case, the proposed predicted (Ef) results ranged from 9.3% greater to 6.1% less than author's observed (Ef) results. To further investigate the validity of the proposed analysis, the (Ef) test results from Larnach and McMullan's (1975), and Yilmaz's (1971) investigations are compared separately with the predicted (Ef) results derived from the proposed analysis. The predicted (Ef) results derived from the proposed analysis ranged from 12.1% greater to 3.8% less than Larnach and McMullan's test (Ef) results.

In the second case, the proposed predicted (Ef) results derived from the proposed analysis ranged from 6.5% greater to 7.3% less than Yilmaz's test (Ef) results. The range of the differences from the proposed analysis is narrower than that from Meyerhof and Adam's theory. This suggests that the (Ef) predictions derived from the proposed analysis gives a better estimate than that derived from Meyerhof and Adam's theory (1968). The predicted (Ef) results for "shallow" anchor in groups (with D/B = 6) derived from the proposed analysis are the same as that derived from Yilmaz's first analysis (1971) for "shallow" anchor groups. The proposed analysis, which is basically an improved version of Yilmaz first analysis, can be used to predict the behaviour of "deep" and "shallow" anchors in line groups installed in dry sand.

In Appendix A is an example of how the proposed analysis can be used to design an anchor group system at each stage of the design procedure.

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CONTENTS

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NOMENCLATURE

NOMENCLATURE

S	Centre to centre spacing between each anchor in a group.
Si	When the centre to centre spacing between each anchor in a group is at the spacing where each anchor unit within the group can act as a single anchor.
D	Depth of embedment of an anchor or anchor groups.
B	An anchor footing diameter.
P	Average pressure on an anchor footing at uplift failure load.
N	Number of anchors in a group.
δf	Displacement of a single isolated anchor at failure load.
Δf	Displacement of an anchor group at failure.
Δr	Displacement ratio at failure ($= \Delta f / \delta f$).
Qu	Anchor group uplift failure load.
qu	An isolated anchor uplift failure load.
Ef	Ultimate group efficiency.
ϕ	Angle of friction of sand.
R.D.	Relative density of sand.
ρ	Density of sand.
γ	Unit weight of sand.
Cu	Coefficient of uniformity of sand.

CHAPTER 1

INTRODUCTION

CHAPTER 1

INTRODUCTION

1.1. GENERAL INTRODUCTION:

This thesis is concerned with investigating the behaviour of embedded plate anchors in line groups when subject to a static uplift loading.

An anchor is a structural tension member which usually consist of an embedded anchor body connected to the anchored structure by means of a cable or tie rod. The resistance to a pull-out loading is usually provided by the forces developed at the embedded end of the anchor unit.

One of the earliest traditional methods of dealing with the uplift forces to which some structures are subjected was the gravity anchor (FIG 1.00) which resists the tensile forces, by means of the self-weight of the anchor.

Such methods are now being gradually replaced by the more economic, versatile and attractive direct embedment anchors which utilize the strength of the overlying soil mass to provide uplift resistance. To design these embedded anchors, the soil properties and the failure mechanism of the anchors within the soil must be known and the design procedure is therefore more complex than the relatively simple design of a gravity anchor.

Direct embedment anchors have two essential features: the anchor tendon which transmits the forces from the structure to the anchor body; and the anchor body which is installed in the ground and transfers the load to the surrounding soil thus resisting the applied forces.

1.2. TYPES OF ANCHORS:

Anchors can be classified according to either the techniques of installation in the soil or to their specific applications.

The classification according to specific applications falls into three categories: ground anchors, rock anchors and marine

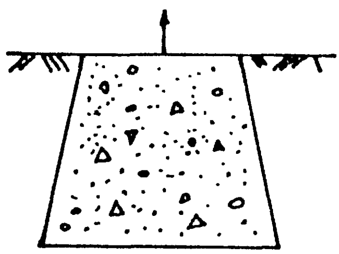


FIG. 1.00. GRAVITY ANCHOR.

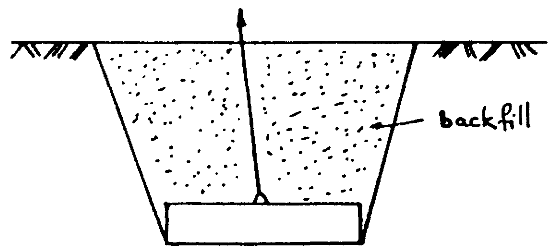


FIG. 1.01. PLATE OR GRILLAGE ANCHOR.

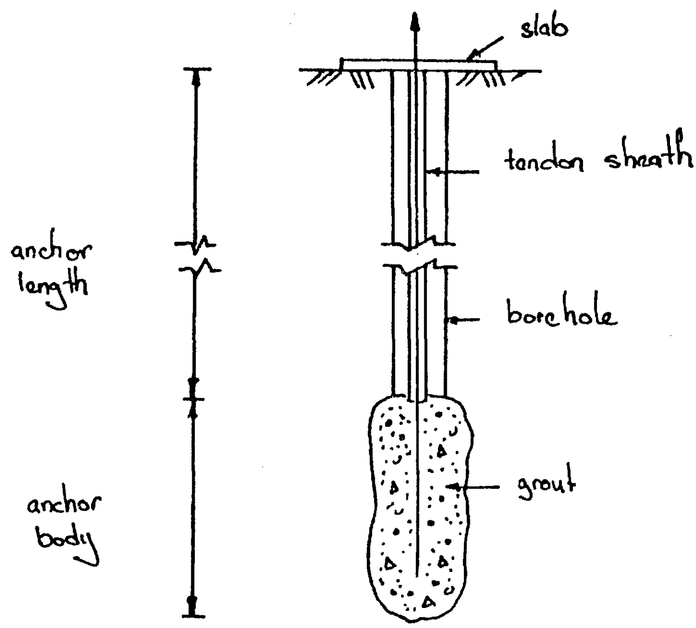


FIG. 1.02. GROUTED ANCHOR.

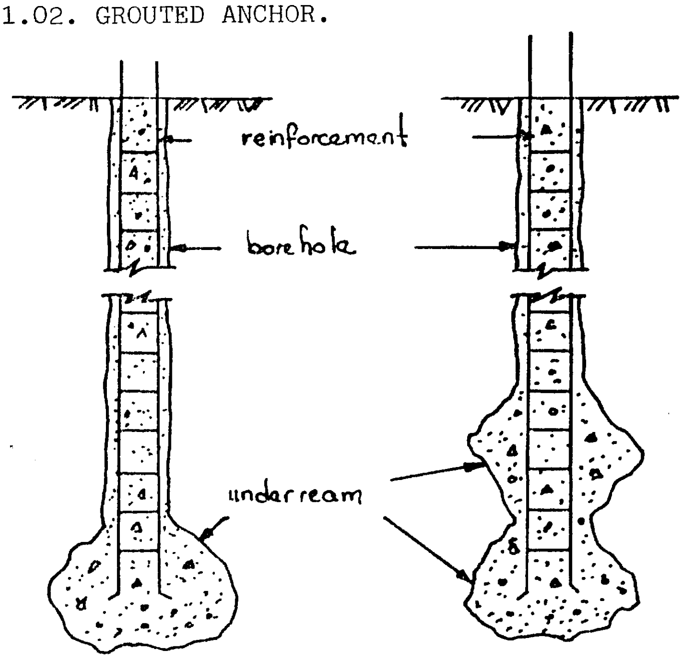


FIG. 1.04.
SINGLE UNDERREAM R.C. ANCHOR

FIG. 1.03.
DOUBLE UNDERREAM R.C. ANCHOR.

anchors. However in this present review anchors are classified under the techniques of installation.

1.3. TECHNIQUES USED IN INSTALLING ANCHORS:

Direct embedment anchors differ from conventional (embedment) anchors in that they do not need to be dragged along the soil to achieve embedment. Instead, direct embedment anchors use the following techniques.

(a) Excavation with backfill:

This installation technique involves placing an anchor body in the base of an excavated shallow trench, attaching an anchor tendon, and then backfilling with gravel above the anchor body to the original ground surface. The applied uplift force on the anchor is assumed to be resisted by the weight of the volume of backfill directly above the anchor body and the shear resistance developed along the failure surface. FIG. 1.01.[54] shows the grillage unit footings used by the Houston Lighting and Power Company for transmission tower footings. However, the major problem in using this technique is that the original soil strength on backfilling such excavations is never achieved. Hence the difficulty arises when determining the shear strength of soil used in the theoretical analysis for predicting the uplift load carrying capacity of these anchors.

(b) Vibration:

Kalajian (1971)[28] investigated the vertical holding capacity of plate anchors in marine applications. The anchor plate is hinged at the bottom end of a rigid steel shaft and is embedded in the sea bed by vibration. To present a minimum frontal area of anchor plate to the soil during vibration, the plate lies along the shaft. Once installed to the required depth the anchor shaft is pulled through a short distance until the plate rotates to present a maximum frontal area to the soil above it. The anchor is then ready to resist loading.

(c) Borehole with cement grout:

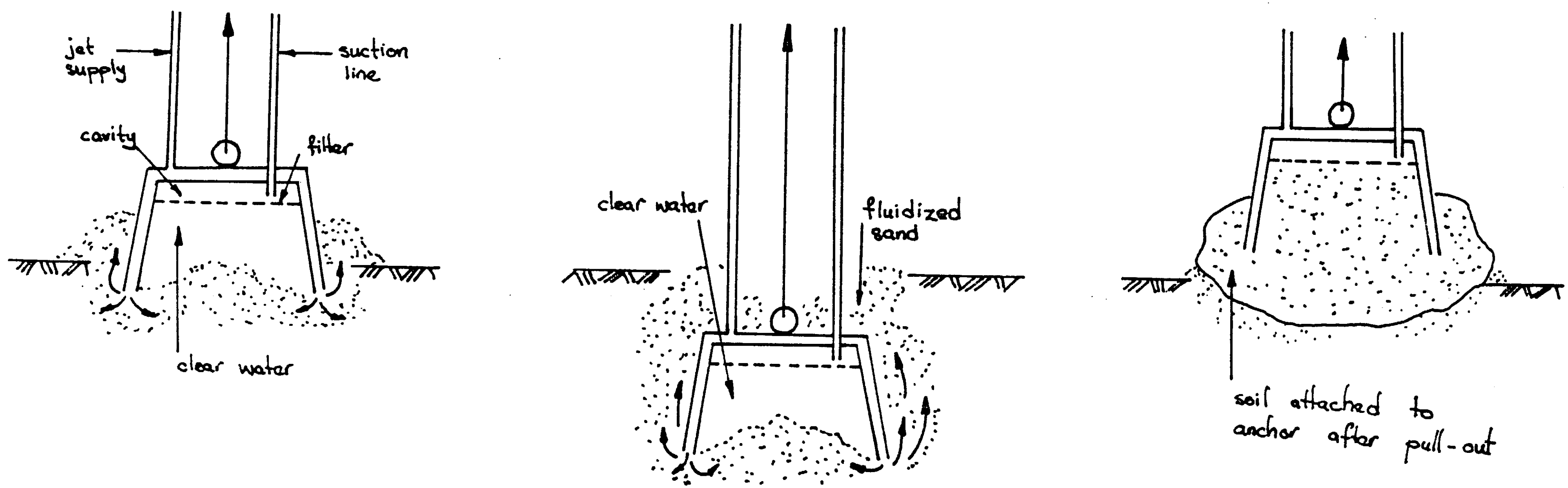
Commercial grouted anchors (see FIG. 1.02.) consist of a steel tendon inserted into a small diameter borehole and anchored to the soil at the lower end of the borehole by pressure grouting with cement to form the anchor body. Littlejohn (1970)[36] reports that in gravel and coarse sands, which are permeable, the grout will permeate the soil surrounding the borehole and an anchor body will be formed with a diameter up to 4 times the diameter of the original borehole. Fine to medium sands, which are less permeable, will not permit the passage of a cement grout. Nevertheless, the remote end of the borehole is subjected to the grouting pressure which may cause some compaction of the surrounding sand and give rise to an anchor body whose diameter is larger than that of the original borehole. To ensure no disruption and heaving of the surrounding ground, all pressure grouting work must be carried out under strict controls to confine the pressure to the region of the anchor under construction.

Ostermayer (1975)[57] described the installation technique for anchors in cohesionless soil, which gave superior results to ordinary boring methods with pressure grouting. The free length of the anchor is drilled in the normal manner, with casing used to support the hole in the cohesionless soil. The remainder of the hole (i.e. the anchor body) is formed by the drill head being surged forwards and backwards without drilling fluid but with the hole being kept full of cement grout. This results in the grout being mixed into the soil, and the soil being compacted. From his laboratory tests, Ostermayer observed that the spread of grout was 3-5 times the diameter of the drill head for sand and gravel and 2-3 times for fine sand.

Another technique involves enlarging the borehole over the length of the anchor shaft with a series of multiple underreams (see FIG. 1.03.)[26] or enlarging just the base of the borehole (see FIG. 1.04.) before installing the tendon and then grouting. This underreaming technique is generally suitable for "stiff clay" ground conditions. However, if the underreaming technique is used in sand, a casing is provided to control the sides of the formed borehole from caving in.

(d) Screw - in action:

Trofimenkov and Mariupolskii (1965)[53] described the use of steel anchor screw piles. The "anchor" is installed by rotating it with a



SHALLOW BURIAL.

A. JETS ONLY

B. JETS ONLY.

C. SUCTION ONLY.

FIG. 1.05. BURYING AND PULLING OUT ACTION OF A BOX ANCHOR.

powered torque so that the blade of the "anchor" forms a cutting edge mounted on a rigid steel shaft and screws itself into the ground to the required depth. The "anchor" is then ready to take compressive and uplift loads.

(e) Jetting action:

Sahota (1978)[48] reported on the development of suction anchors which are capable of resisting large uplift forces in comparison to their own weight. Suction type anchors may have a box shape with an open or closed end. The burying of this anohor can be achieved by supplying pressurised water through the periphery and / or centre using a system of water jets. These fluidize the soil underneath and within the skirt of the anchor, since the jets produce a region of high turbulence which excavates the surrounding soil. Once the required burial depth is achieved the water supply to the jets is cut - off and suction is applied. The suction action dewaters the disturbed soil resulting in consolidation of the soil around the anchor, and provides an extra breakout resistance. When the need arises to retrieve the anchor, the water jet system is reactivated to fluidize the soil around the anchor, hence reducing the pull - out capacity. FIG. 1.05. shows the burying and pulling action of a box anchor.

(f) Driving action with hammer:

FIG. 1.06.[5] shows a fluke system which is installed (initially with closed flukes) by driving with a hammer. At a pre - determined depth the flukes are unlocked by activating an explosive bolt. The final phase of driving causes the soil bearing pressure to force the flukes open until the contained angle is 90° ; the locked flukes then offer a substantial withdrawal resistance when loaded. These anchors are called umbrella anchors.

A propellant actuated anchor, however is driven by an explosive charge. The gun assembly discharges an explosive which drives the anchor projectile into the ground. An example of this type of anchor is as shown in FIG. 1.07.[34]

(g) Explosives to create a cavity:

In rock, grouted anchors are sometimes formed by using

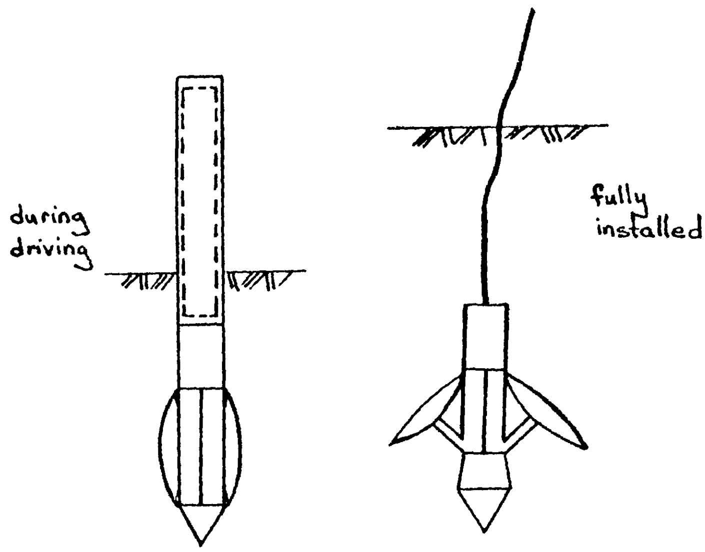


FIG. 1.06. UMBRELLA ANCHOR.

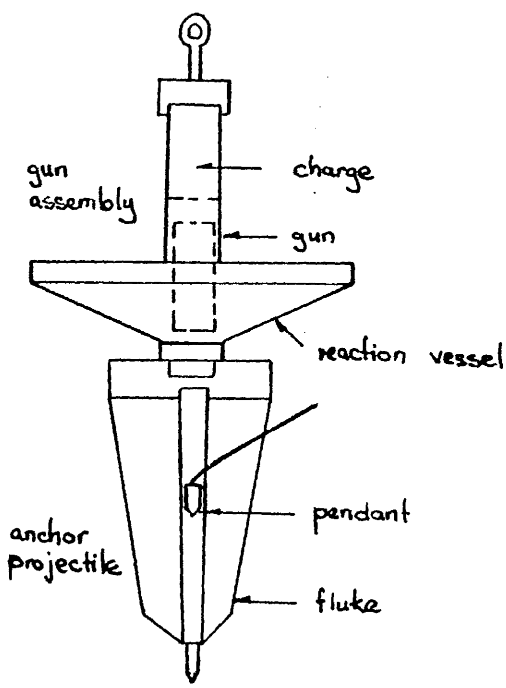


FIG. 1.07. PROPELLANT ACTUATED ANCHOR.

explosives to create a large cavity which is subsequently filled with a quick setting grout. Littlejohn (1968)[37] reported that working loads of 498 kN have been achieved by this technique. However, the practical applications of this technique of anchor construction is limited due to the disturbance and damage that may be caused by blasting operations. Special care must be also be taken when applying grout under pressure for the reasons stated in section 1.3. (c).

1.4. APPLICATIONS OF ANCHORS IN ENGINEERING PRACTICE:

Embedded anchors have been fairly extensively used in constructional work over the last few decades. The main classes of applications will be considered in this section.

(a) Foundations of structures:

The earliest uses of embedded anchors in construction work was to stabilize the foundations of transmission towers, radio and television masts.

Considerable overturning moments and forces which are induced on the structures from exposure to strong winds, snow and ice need to be resisted. The resistance of these structures to uplift forces may be provided by the use of rock or ground anchors as shown in FIG.1.08. For many years the standard foundation to support line suspension towers has been a steel grillage footing. However, augered concrete footings are now widely used instead, for example in transmission construction for southern Ontario.[1]

(b) Foundations for buoyant structures:

One of the applications of anchors to overcome bouyancy effects was reported in Ground Engineering, March 1971. The project involved providing a road and rail link between Sicily and Italy using anchor cables to secure three submerged pipes 40m below sea level. FIG.1.09. shows a sketch of the engineering project.

In the North Sea, where offshore oil exploration activities are intensive, various applications of anchors have been pioneered to anchor buoys, submerged pipe lines, submersibles and vessels for site investigation. The GASUB design of a cable - stayed semi - submerged buoyant oil platform is claimed to be cheaper

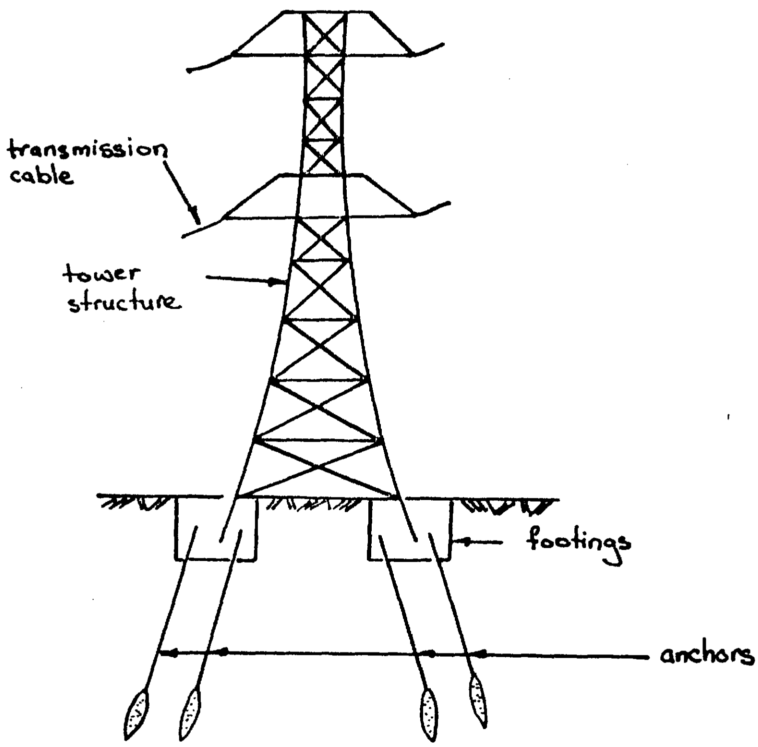


FIG. 1.08. SKETCH OF TOWER SUPPORTED BY ANCHORS.

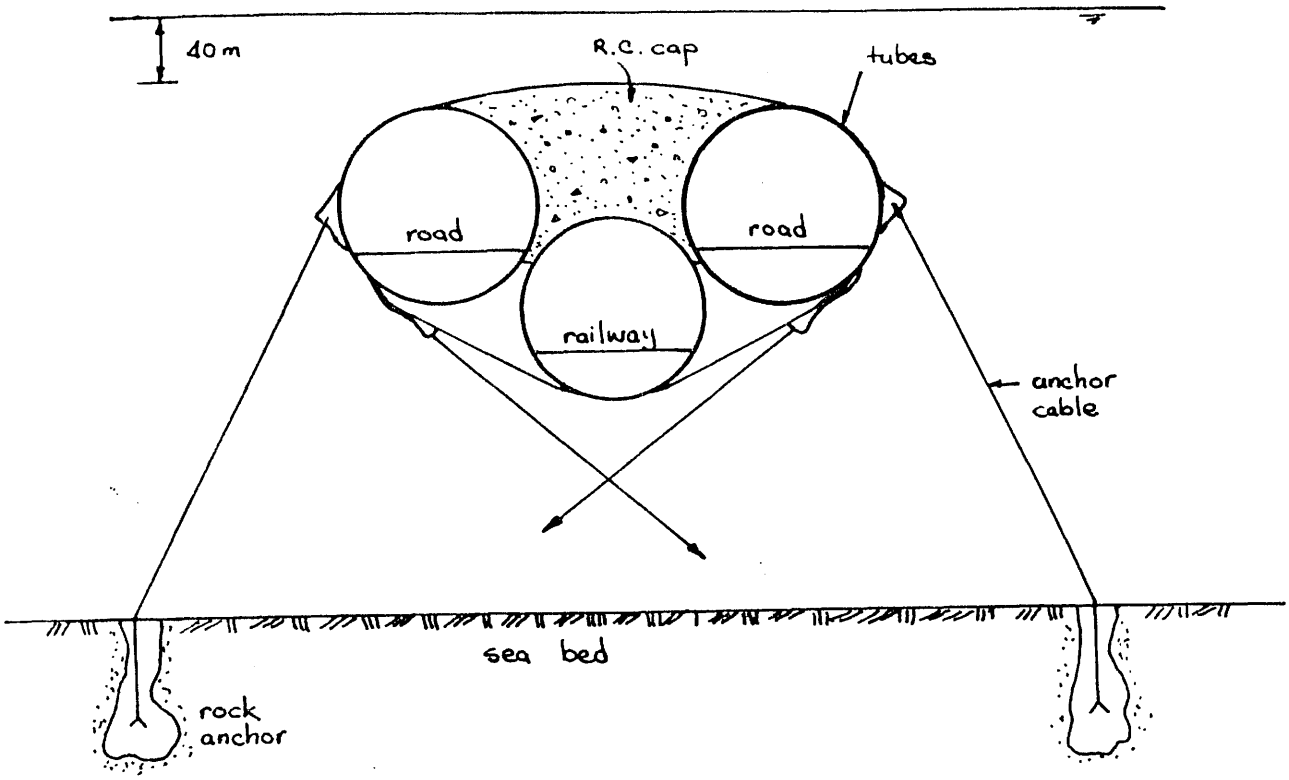


FIG. 1.09. PRINCIPLE OF ANCHORED FLOATING TUNNEL.

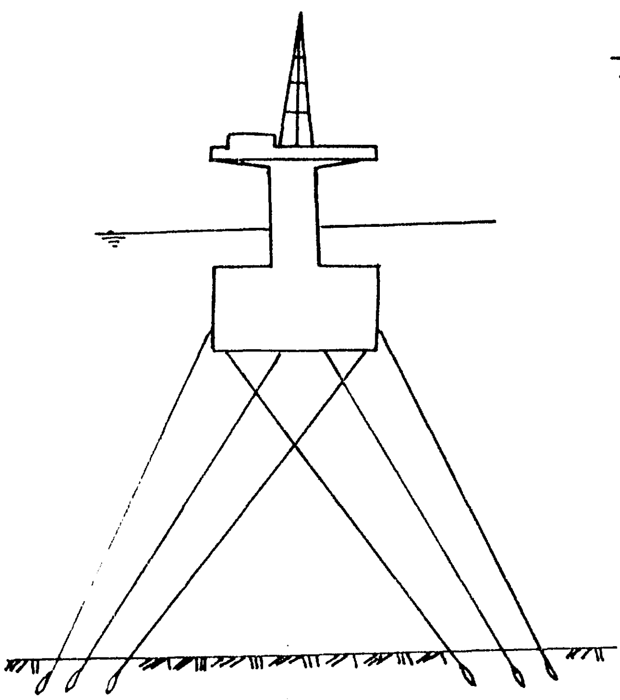


FIG. 1.10. MODEL OF A CABLE-STAYED SUBMERGED BUOYANT RIG.

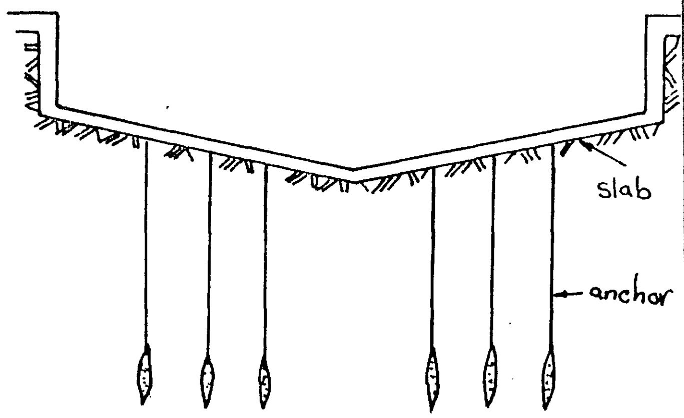


FIG. 1.11. SKETCH OF DRY-DOCK SLAB SUPPORTED BY ANCHORS.

to construct compared with a gravity oil platform and is reputed to be very stable under all weather conditions (New Civil Engineer, 21 February 1984)[42]. FIG. 1.10. shows the oil platform consisting of a 50m diameter multi - cell buoyant chamber positioned at 30m below sea level. On top of the chamber is mounted a working deck.

Buoyancy problems also occur in dry docks or basin shaped structures, FIG. 1.11. When the water in the dock is pumped out, the structure may tend to float due to the presence of hydrostatic uplift pressures. This problem can be solved by controlling the level of the ground water table during the early construction stage and by providing rock or ground type anchors at the base to resist the uplift forces. For example, Greenock Dry Dock in Scotland constructed by the Cementation Company involved installing a single line of 35 anchors each of 2400 kN capacity along the centre line of the dry dock.[58]

(c) Retaining Structures:

Hanna (1968)[20], described the technique that use ground or rock anchors to tie a retaining wall into the retained soil. FIG. 1.12. shows this technique being used to anchor a retaining wall for vertical cuttings adjacent to highways, railways and canals by horizontally positioned ground anchors.

Still to be developed further is the idea of doing away with the the conventional battering to the outside walls of water tanks to control excessive deformations of the walls. Instead, by using anchors with cables as shown in FIG. 1.13. the deformation problem caused by water pressure can be controlled much more economically. The problems related to this technique are the effects of water level fluctuations on the anchor cables and the amount of initial tensioning required on the anchors.

Anchors have also been used for controlling the heave of the toe of dams as reported by Jaspar and Shtenko (1969)[27]. Anchors can also be used to increase the height of existing dams, as shown in FIG. 1.14.[58]

(d) Stabilization of rock face and underground excavations:

Rock anchors are used as shown in FIG. 1.15. to stabilize the

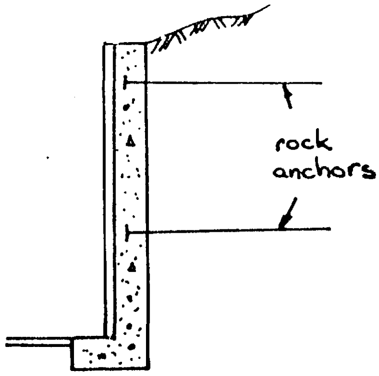


FIG. 1.12. REVETMENT OF ROCK
WITH ANCHORED SOIL
RETAINING WALLS.

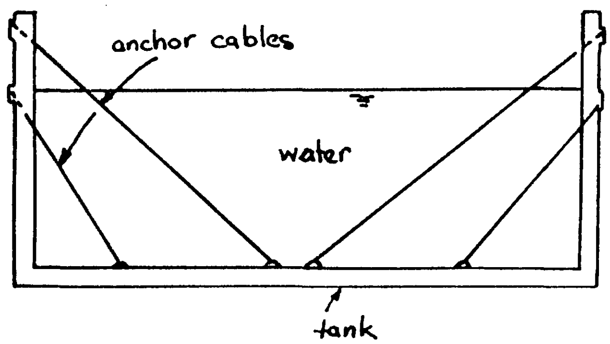
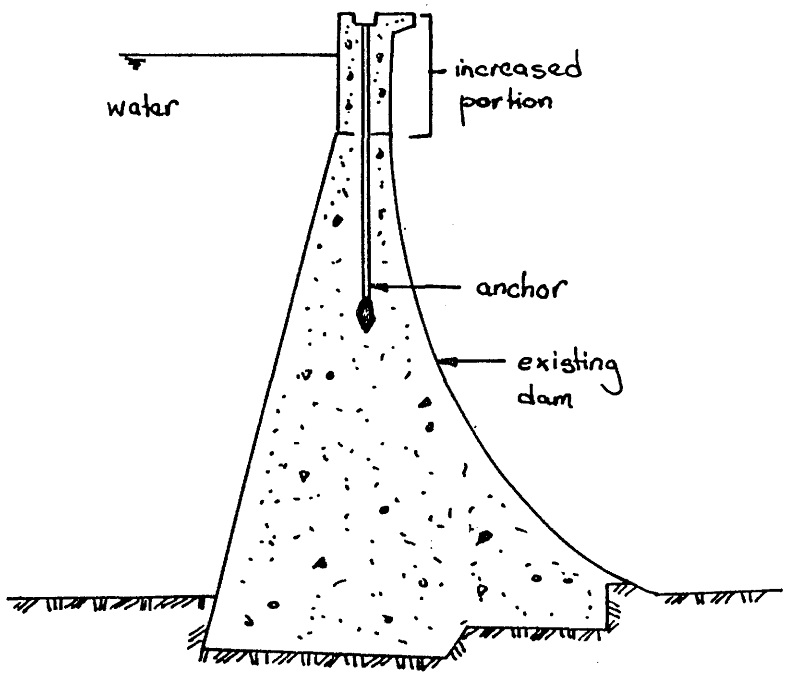
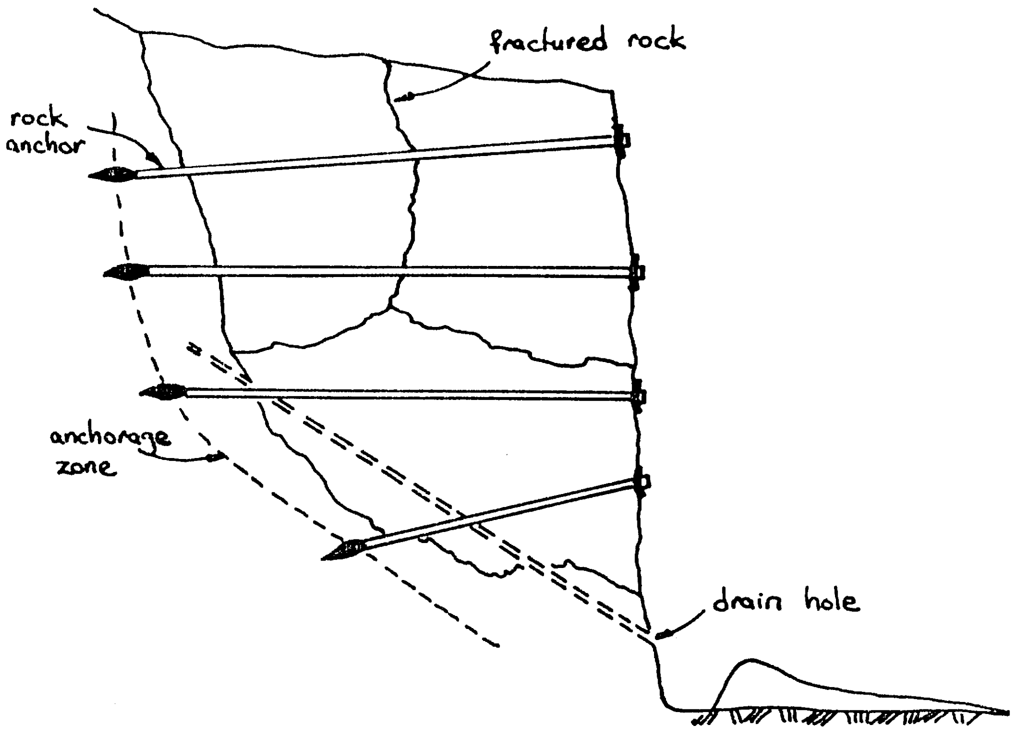


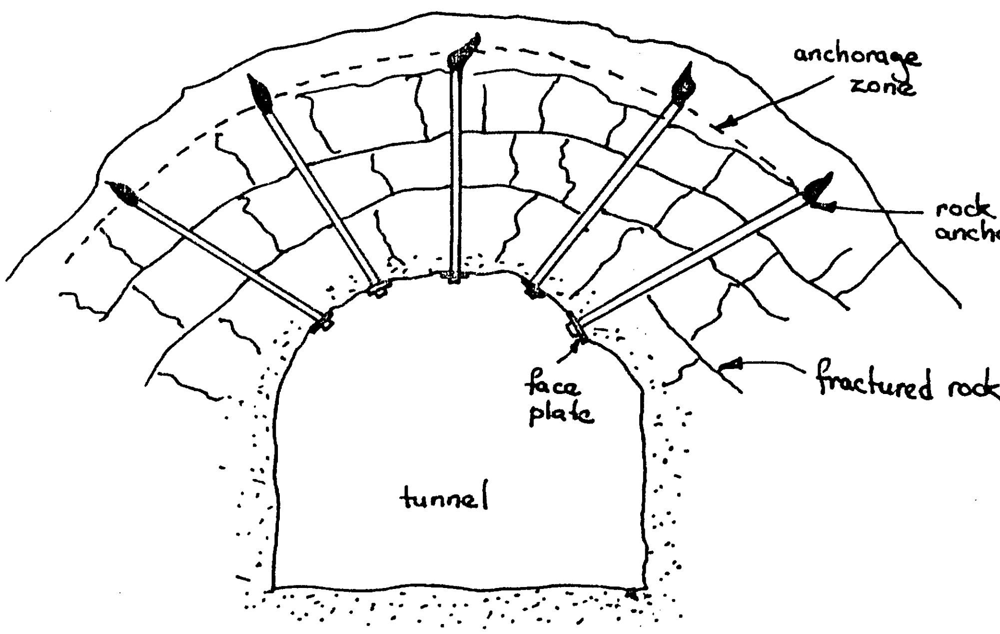
FIG. 1.13. ANCHOR CABLES FOR WATER
RETAINING TANK.



F IG. 1.14. ANCHOR ARRANGEMENT FOR INCREASING
HEIGHT OF DAM.



F IG. 1.15. TYPICAL SLOPE ANCHOR INSTALLATION.



F IG. 1.16. TYPICAL ROOF ANCHOR INSTALLATION.

natural fissures and joints in rock surfaces caused by the weather, vegetation wedging and flow of water. Drain holes are also provided to let the ground water escape without causing the build up of excess hydrostatic pressure in the joints.

Excavations of tunnels in badly fissured rock requires the installation of rock grouted anchors to stabilize the walls of the tunnel, especially the roof of the tunnel. FIG. 1.16. shows a typical arrangement of rock anchors used to stabilize the roof of a tunnel.

(e) Other applications:

Hanna (1970)[21] describes an interesting application of anchors in the technique of horizontally jacking pipes into slots cut into an embankment. FIG. 1.17. shows the arrangement for this operation.

Anchors are also used to resist the reactions created during load tests on compression piles. Hanna (1970)[21] reported these reactions to be between 500 - 10000 kN.

In some cases, the conventional methods of stabilizing slopes by constructing large gravity walls or cutting a gentler slope are less economical than installing anchors as shown in FIG.1.18.

The above applications of anchors are by no means the only ones. Future and present research on the behaviour of anchors is enabling the discoveries to be applied to many new fields.

1.5. DESIGN AND CONSTRUCTION CONSIDERATIONS OF ANCHORS IN COHESIONLESS SOILS:

A comprehensive ground investigation is essential in any location where anchors are to be installed. The soil profile must be determined accurately, any variations in the level and thickness of strata being particularly important. The basic characteristics and shear strength of the soil should be determined, in order that failure load of the anchors embedded in the soil can be estimated.

Design for a single anchor system can be carried out according to one of the existing semi - empirical methods to obtain a first approximation solution. It is normal practice today to check on the soundness of the adopted design and construction procedures by conducting a set number of full - scale pilot uplift

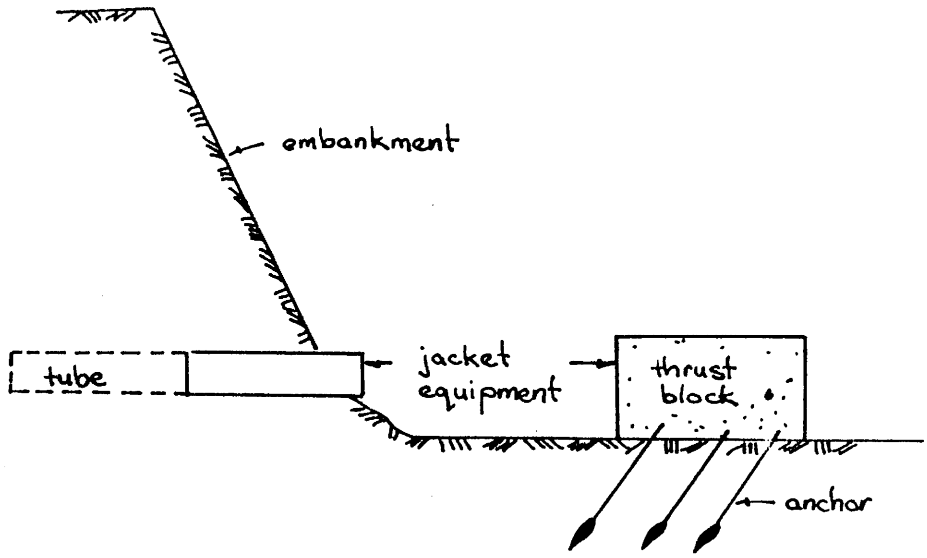


FIG. 1.17. ANCHOR ARRANGEMENT TO PROVIDE A REACTION FOR PIPE JACKING.

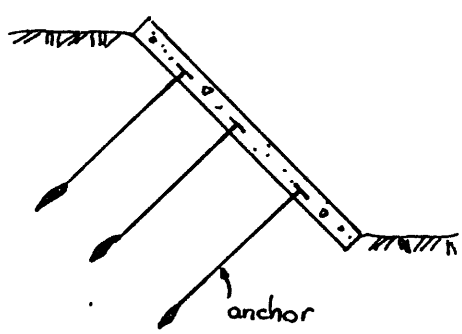


FIG. 1.18. RESTRAINING SIDE OF SLOPE IN A ROAD CUTTING OR OTHER EXCAVATIONS.

tests on prototype anchors installed at the site.

When designing a grouped anchor system, the problems of group action due to the interaction of individual failure zones can occur. This group action, which is measured by the group efficiency (given in Chapter 5) can reduce the ultimate capacity of an average anchor in the group. Furthermore, the grouping effects can influence the displacement of an average anchor in the group and the factor of safety required. However, the degree of influence of group action depends on the anchor spacing within a group, configuration of the anchor group, size of the anchor group, relative embedment depth D/B ratio, and the stiffness of the anchor cap.

Only a few theoretical analyses are available to design a grouped anchor system. Meyerhof and Adam (1968)[40] proposed a theory of group action which only reflects the trend of test results and cannot be heavily relied on. Yilmaz (1971)[58] suggested a purely empirical analysis for grouped anchor design. It is only applicable to shallow anchors, but is a more reliable first approximation to grouped anchor design.

Various practical construction considerations need to be taken into account to complement the theoretical design analysis of grouped anchors in order to achieve an overall predicted stability of the anchored structure. It has been recommended that anchors are installed deeply, so that inaccuracies in the embedded depth will not substantially affect the designed ultimate static capacity. This is significant in cases of considerable change in the topographical features of the sea bed which could otherwise have a catastrophic effect on anchor capacity. Other options like drilling, placing the anchors, grouting and pre-stressing of anchors should be performed with precision and care.

The uncertainty in the theoretical analysis of uplift capacity of anchors and grouped anchors, the allowable displacement of installed in-situ anchors, disturbance caused by installation of anchors and the unpredictable variations in soil conditions necessitate the introduction of safety factors. In general for permanent anchorage works a factor of safety of about 2 to 3 is usual and for temporary works a lower value of 1.5 is allowed.[58]

1.6. CONVENTIONAL PHYSICAL MODEL TESTS:

Full scale or field tests give the most reliable results because the actual field conditions are investigated. However, full scale tests are costly and time consuming, so laboratory model tests are used in this experimental work on anchors in groups.

Scale models used in soil mechanics can be classified into three types according to the purpose of the model tests (Roscoe, 1968 .)[45]

(a) TYPE 1.

To investigate the assumptions and factors used in developing a proposed theoretical analysis applied to a prototype.

(b) TYPE 2.

To establish a simple basis for the possible dimensional similarity between the results of small-scale model tests and the behaviour of full scale prototypes. Hence the behaviour of a full-scale prototype structure can be predicted from the model tests.

(c) TYPE 3.

Model tests conducted to serve theoretical purposes only. Hence they do not necessarily relate to any possible prototype problem. However, this research could lead to possible improvements in new methods of analysis which in turn could lead to better design rules, especially for complicated stress or strain boundary problems and soil-structure interaction investigation.

The present model tests fall into TYPE 2 classification of model tests. The laboratory model tests conducted are reported in detail in Chapter 4.

1.7. PURPOSE AND SCOPE OF PRESENT INVESTIGATION:

Relatively little previous work has been done on the subject of anchor group interaction (see Chapter 2). In this present investigation an attempt is made to derive a simple analysis, for predicting the ultimate uplift load of "deep" and "shallow" anchors in sand, based on the author's experimental results.

The author's experimental investigation used laboratory scale models to study the behaviour of anchor groups of 25mm diameter plate anchors embedded in dry cohesionless soil when subjected to static load. The anchor groups were rigidly capped and free-standing. The model test parameters varied were line configuration of groups of anchors and anchor spacing (distance between centre to centre of adjacent anchors in a group), relative depth of embedment of anchor, and number of anchors in a group. The influence of these parameters on the ultimate group efficiencies and the displacement ratios of a given group of anchors are considered.

Various British Standard laboratory tests were conducted to determine the properties of the cohesionless soil used. The technique used for laying beds of sand of uniform density is reported. Other test apparatus used in this investigation is also mentioned.

The present results and previous theories are compared and discussed for anchors in line groups. The previous theories considered are Meyerhof and Adam's theory (1968) and Yilmaz's first analysis (1971). Yilmaz's second analysis (1971) for groups of circular anchors was omitted because in this investigation the many prerequisite physical test parameters were not measured and calibrated. The results derived from the proposed analysis are compared with the present experimental results. To further investigate the validity of the proposed analysis, the observed results from previous experiments are compared with the predicted results derived from the proposed analysis. The previous experimental results considered are from Larnach and McMullan's (1975)[31] and Yilmaz's (1971) investigations. However, Meyerhof and Adam's (1968) experimental results are not used to validate the proposed analysis. This is because Meyerhof and Adam's investigations considered line groups of only up to two anchors.

In Appendix A is an example of how the proposed analysis can be used to design an anchor group system at each stage of the design procedure.

In Appendix B is a tabulated summary of the author's experimental results on uplift capacity tests.

CHAPTER 2

LITERATURE REVIEW OF PREVIOUS RESEARCH WORK

CHAPTER 2

LITERATURE REVIEW OF PREVIOUS RESEARCH WORK

2.1. INTRODUCTION:

This chapter presents a survey of the previous theoretical and experimental work on uplift capacity of anchor groups installed in sand.

Relatively little theoretical and experimental work has been done on the uplift capacity of anchors in groups. Most work on uplift capacity of anchors has been confined to predicting the ultimate uplift capacity of single anchors. Meyerhof and Adam (1968) derived a semi-empirical theory, for a single strip anchor and later modified the theory for anchors in groups, based on their model test results. The theory gives approximate solutions for "shallow" and "deep" anchors in groups. Assumptions in the theory included the occurrence of "block failure" and the hypothesis that the total shearing resistance of the uplift capacity component was a function of the passive earth pressures acting on a vertical plane, through the footing edge. In another attempt to predict the ultimate uplift capacity of anchor groups, Yilmaz (1971) proposed the following two simple empirical analyses for "shallow" anchors in groups installed in sand.

(a) The load distribution and the ultimate uplift capacity of a line group of 3 strip anchors can be predicted from experimental results of single and pairs of strip anchors. The analysis can be extended for larger line groups.

(b) The load distribution amongst individual anchors in the group can be computed by assuming the uplift failure load of an isolated anchor to be directly proportional to the area of its failure circle created on the surface of the soil, and the isolation spacing is the radius of the failure circle. The prerequisites for the application of this analysis are the availability of the test results of ultimate uplift capacity of an isolated anchor and the "isolation" spacing.

Any small inaccuracy in the observed results is reflected in the predicted results from both Yilmaz's analyses. (see equations E.2.05. and E.2.08.).

2.2. UPLIFT CAPACITY THEORIES OF ANCHOR GROUPS IN SAND:

There are only a few theories for predicting the uplift capacity of anchor in groups.

Meyerhof and Adam (1968) derived from theoretical concepts and experimental results, a semi - empirical analysis to forecast the uplift resistance of a strip or continuous footing. Modifications were then introduced to cater for circular and rectangular footings arranged in groups buried in sand. It was stipulated that the ultimate uplift load of a footing group was the smaller value of either the sum of the uplift loads of the individual footings or the uplift load of an equivalent pier foundation consisting of the footings, and enclosed soil mass. The uplift load of the equivalent pier foundation could be estimated by the suggested "rectangular" footing method.

Hence a rectangular group of circular footings at shallow depth ($D \leqslant H$) has an ultimate uplift capacity of:

$$Qu = 2cD \left[a+b+ (\pi/2) B \right] + \gamma D^2 \left[a+b+s (\pi/2) B \right] Ku \tan \phi + W_1 \qquad (E.2.01.)$$

Similarly, for deep depths ($D > H$) equation E.2.01. becomes:

$$Qu = 2cH \left[a+b+ (\pi/2) B \right] + \gamma (2D-H)H \left[a+b+s (\pi/2) B \right]. Ku \tan \phi + W_1 \qquad (E.2.01.)$$

and the following equations give the upper limits of the equations E.2.01. and E.2.02. The equations E.2.03. and E.2.04. are respectively the sum of the ultimate uplift capacity of "shallow" and "deep" individual footings in groups.

$$Qu = N \left[\pi cBD + s (\pi/2) \gamma BD^2 Ku \tan \phi + W \right] \qquad (E.2.03.)$$

$$Qu = N \left[\pi cBH + s (\pi/2) \gamma B (2D - H) H Ku \tan \phi + W \right] \qquad (E.2.04.)$$

Where:

a and b = distance between centres of "corner" footings on length and width respectively, of group.

N = number of footings in group,

W_1 = weight of soil mass bounded within the "block failure" zone of grouped footings.

W = weight of soil mass bound within an assumed truncated conical failure surface for a circular footing. (FIG. 2.00)

sKu is the uplift coefficient. (FIG. 2.01.)

c = cohesion along the failure plane.

D = depth as shown in FIG. 2.00.

H = depth as shown in FIG. 2.00.

B = diameter of anchor footings.

and $s = 1 + m\ D/B$ or a maximum of $s = 1 + m\ H/B$.

s = the shape factor governing the passive earth pressure on a convex cylindrical wall.

m = a coefficient depending on $\emptyset$ and obtained graphically from FIG. 2.02.

Included in the analysis were observations of experimental results showing that the average angle (α) between the failure surface and the vertical was $\emptyset$ / 3 for circular footing, where $\emptyset$ = angle of friction.

As indicated earlier, Yilmaz (1971) proposed two simple analyses to predict the uplift capacity of grouped anchors. The first analysis was based on the experimental results obtained for an isolated anchor and for a group of two "shallow" strip anchors. A single "shallow" anchor is assumed to develop a failure zone as shown in FIG. 2.00. similar to that suggested by Meyerhof and Adam (1968). The concept of simple interaction of failure zones was applied in the analysis to predict the ultimate uplift capacity and the load distribution for a group of (3x1) "shallow" strip anchors. FIG. 2.03. and FIG. 2.04. show the analysis for groups of strip anchors installed at very close, and close spacings respectively. The assumption was that the area within the rupture zone could be divided into areas x, y & z, each of which represented a proportion of failure load.

The equations, valid for "shallow" strip anchors of spacings from B to 8B were:

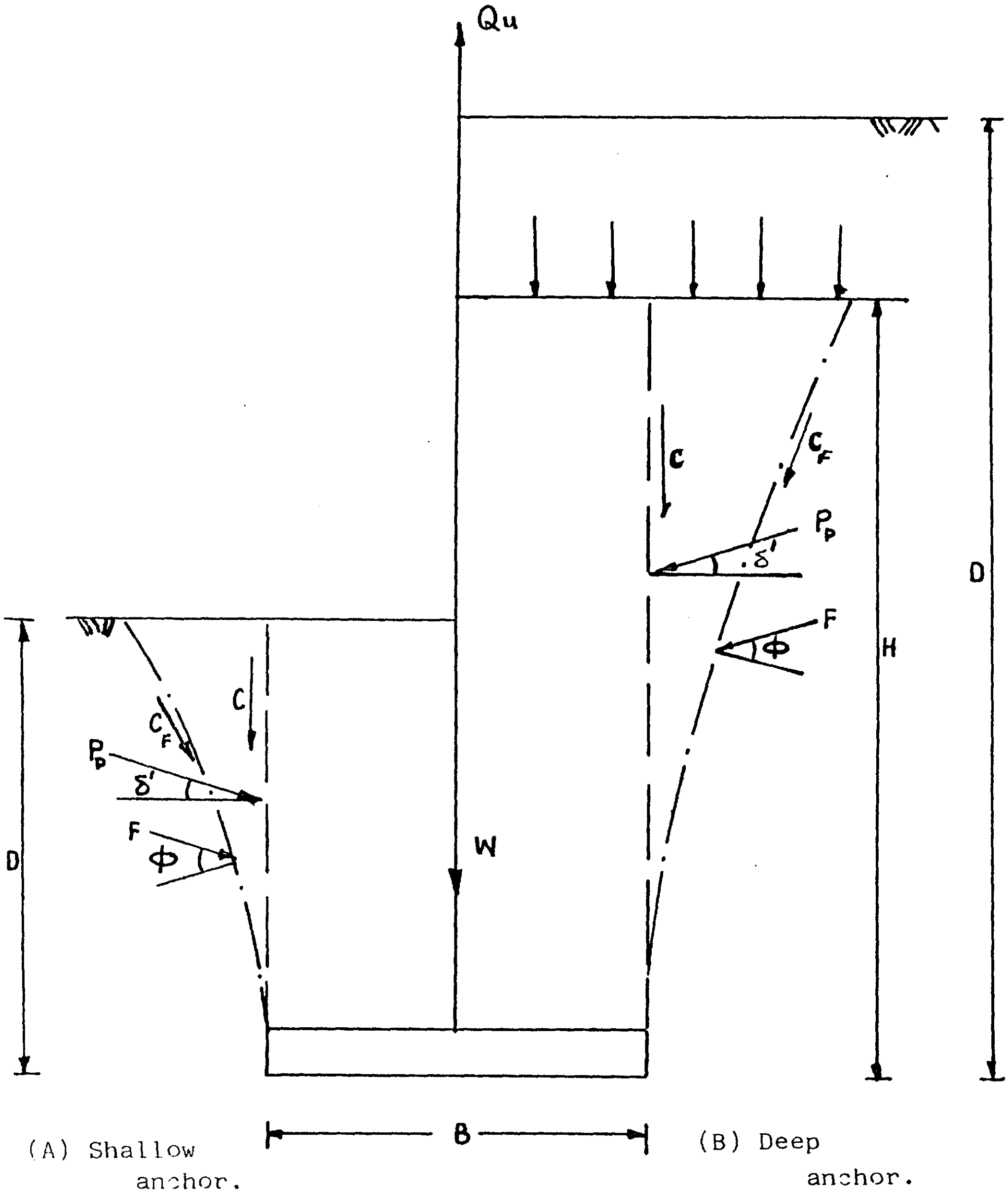
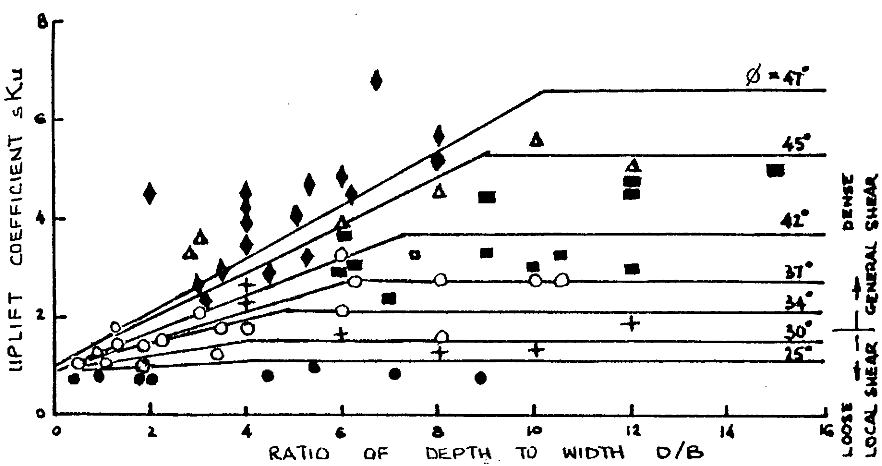


FIG. 2.00. MEYERHOF AND ADAMS THEORY.
- FAILURE ZONES OF "SHALLOW" AND "DEEP" ANCHORS.



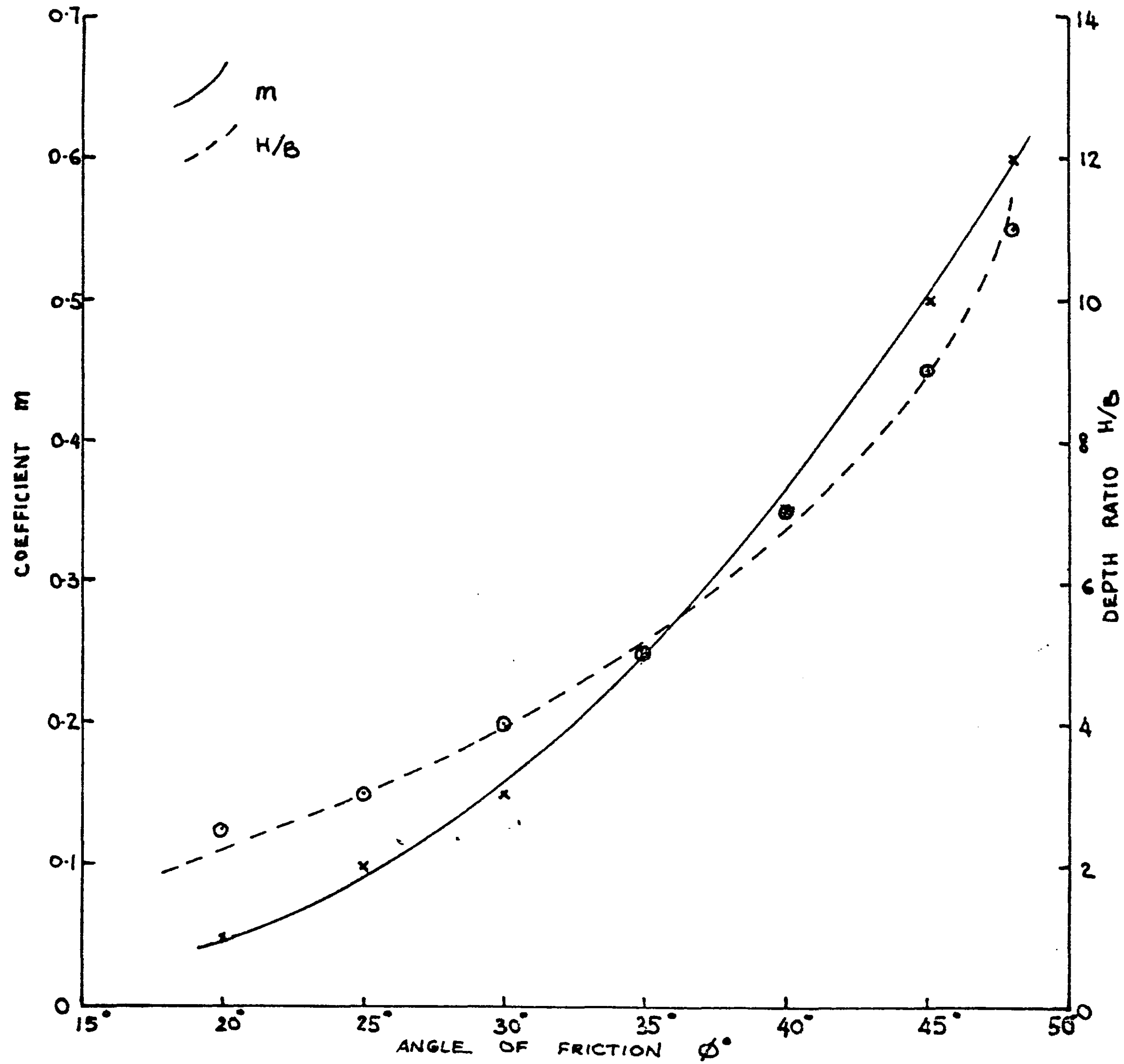
PUBLISHED DATA - CIRCULAR FOOTINGS - SLENDER SHAFTS:

1" - 4" DIAMETER:

- o Dense silica sand Ø = 34° - Adams and Hayes 1957.
- ♦ Dense concrete sand Ø = 47° - Adams and Hayes 1967.
- ■ Dense silica sand Ø = 42° - Baker and Konder 1968.
- ▵ Dense sand Ø - 45° - Macdonald 1963.
- ● Loose silica sand Ø = 28° - Adams and Hayes 1967.
- + Loose sand Ø = 34° - Macdonald 1963.

FIG. 2.01. COMPARISON OF THEORY AND MODEL TESTS FOR FOOTINGS IN SAND.

FIG. 2.02. MEYERHOF AND ADAMS THEORY.
-RELATIONSHIPS OF m vs ϕ AND H/B vs ϕ.



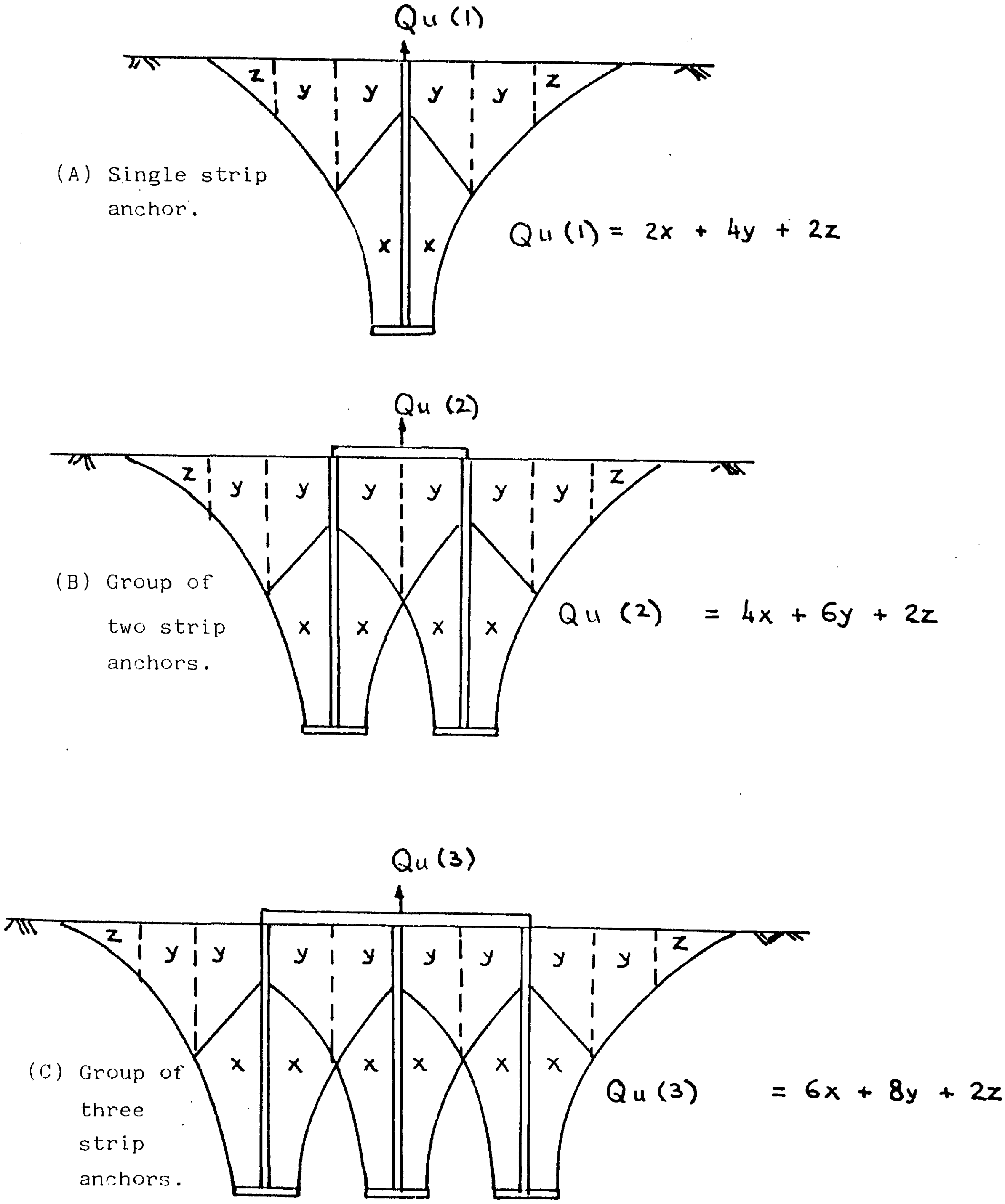
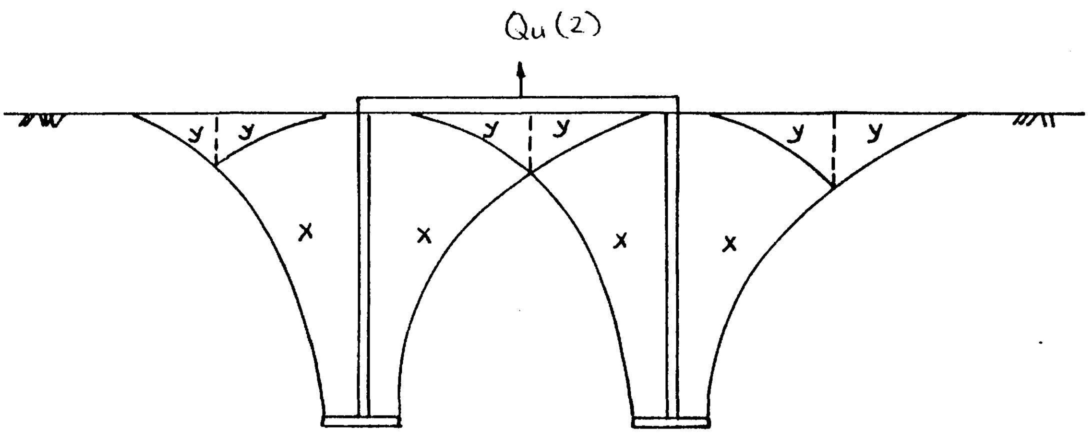


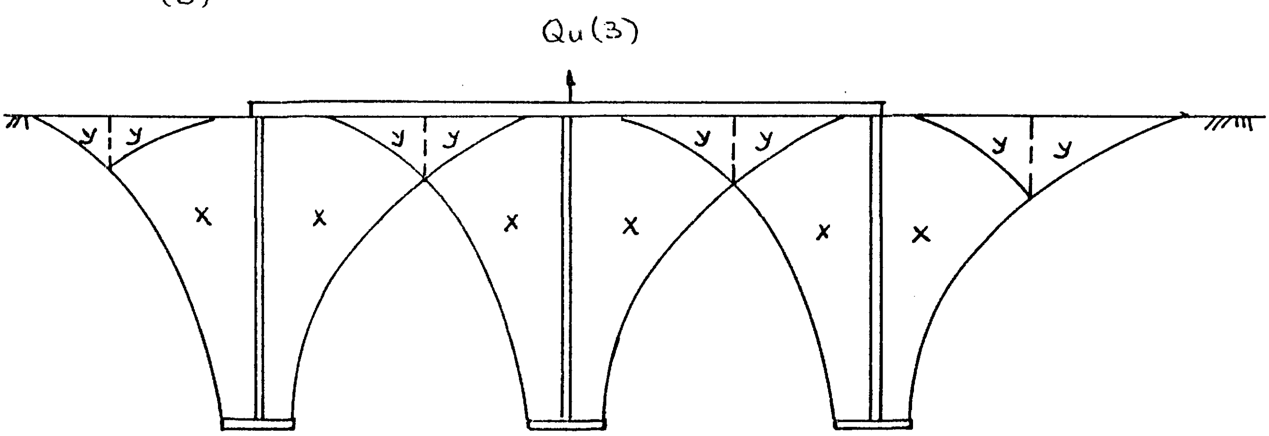
FIG. 2.03. YILMAZ FIRST ANALYSIS.
-INTERACTION OF FAILURE ZONES OF EACH "SHALLOW" ANCHORS IN LINEAR GROUPS AT VERY CLOSE SPACINGS.

(A)



$$Qu(2) = 4x + 6y$$

(B)



$$Qu(3) = 6x + 8y$$

FIG. 2.04. YILMAZ FIRST ANALYSIS.
-INTERACTION OF FAILURE ZONES OF EACH "SHALLOW" ANCHORS IN LINEAR GROUPS AT CLOSE SPACINGS.

(a) ultimate uplift capacity of 3 strip anchors in a line group is given from FIG. 2.03. as:

$$Qu\ (3) = 2\ Qu\ (2) - Qu\ (1) \qquad (\text{E. } 2.05.)$$

(b) the magnitude of uplift load resisted by the middle strip anchor in the (3x1) group is given from FIG. 2.03. as:

$$2x + 2y = Qu\ (2) - Qu\ (1) \qquad (\text{E. } 2.06.)$$

(c) the magnitude of the uplift load resisted by the end strip anchor in the (3x1) group is given from FIG. 2.03. as:

$$2x + 2y = \frac{Qu\ (2)}{2} \qquad (\text{E. } 2.07.)$$

Where:

Qu (3) = ultimate uplift capacity of a line group of three strip anchors.

Qu (2) = ultimate uplift capacity of a line group of two strip anchors.

Qu (1) = ultimate uplift capacity of a single strip anchor.

x and y = each of the areas within the rupture zone of a single anchor which represents a proportion of the single anchor failure load Qu (1). (see FIG's 2.03. and 2.04.)

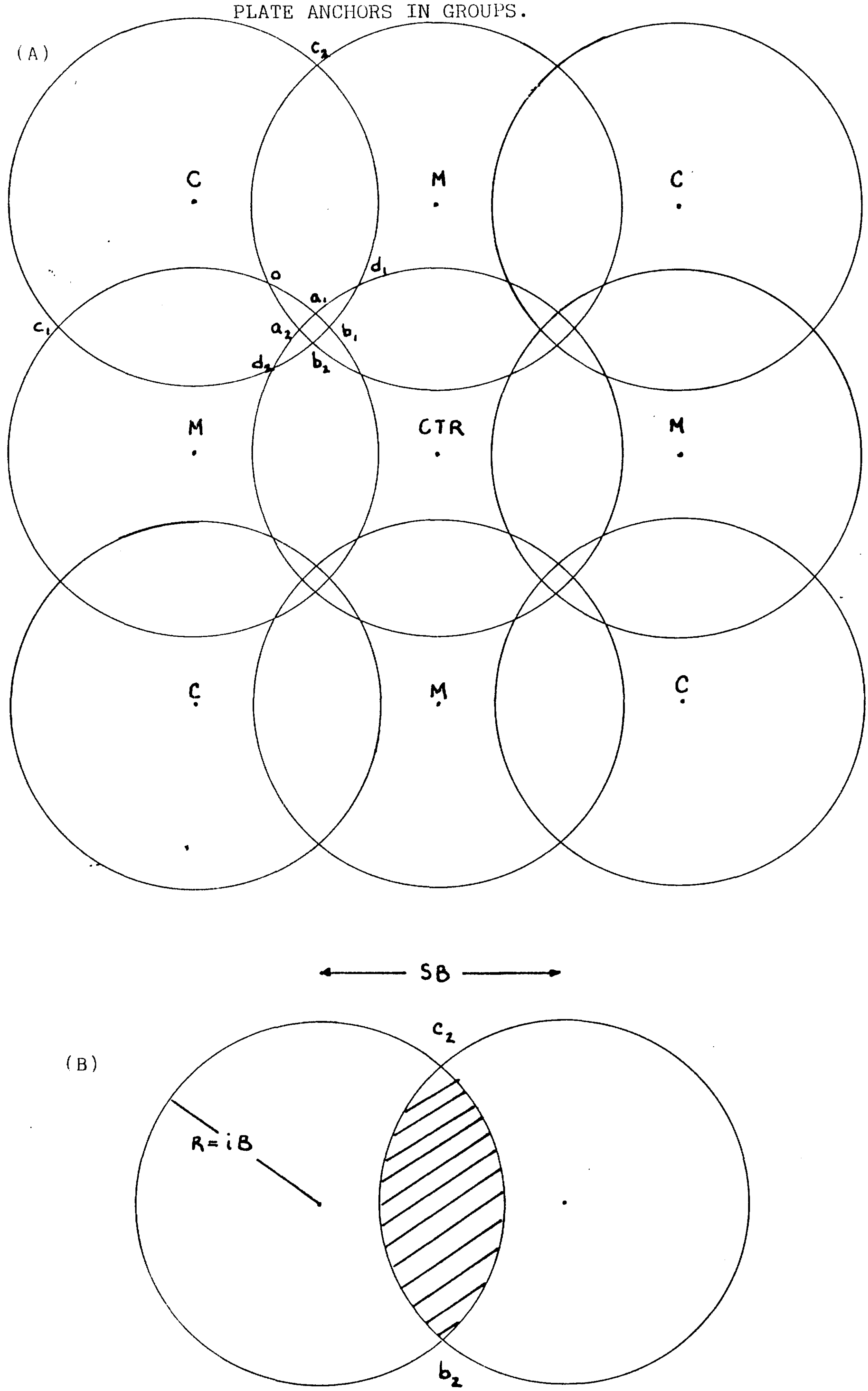
Yilmaz then suggested that the above analysis could be extended to obtain ultimate uplift capacities for larger line groups of strips anchors.

Yilmaz's second analysis was derived for predicting the load distributed to an individual shallow, circular plate anchor in a group. The assumptions of the analysis are:

(a) the ultimate uplift capacity of an isolated anchor was directly proportional to the area of its failure circle created on the surface of the soil.

(b) the observed experimental "isolation" spacing of an anchor was the radius of the failure circle. The "absolute area" of the failure circle of an anchor in a group (i.e. reduced area as a result of the interaction with the adjacent anchors) was found by trigonometry (shown in FIG. 2.05.).

FIG. 2.05. YILMAZ SECOND ANALYSIS.
-INTERACTION OF "FAILURE CIRCLES" OF CIRCULAR PLATE ANCHORS IN GROUPS.



Hence the expression:

$$\frac{qcf}{Qu\,(1)} = \frac{\pi\,(\,iB\,)^2 - \left[A\,(\,C_2 d_1 a_1 OC_2\,) + A\,(\,a_1 d_1 b_1 a_1\,) + \frac{A\,(\,a_1 b_1 \;\; b_2 a_2 a_1\,)}{4} \right]}{\pi\,(\,iB\,)^2}$$

(E. 2.08.)

Where:

Qu (1) = ultimate uplift load of a single isolated anchor.
qcf = failure load on "corner" anchor in a group.
R(=iB) = radius of failure circle.
A = area.
Si(=iB) = "isolation" spacing of anchor.

The above procedure could similarly be carried out for predicting the load resisted by any other anchor in the group. The summation of the predicted uplift capacities of each anchor within a group will give the predicted uplift capacity of the group.

2.3. COMMENTS ON THE PREVIOUS THEORIES PRESENTED:

Meyerhof and Adam (1968) stated that the ultimate uplift resistance of a footing group was the smaller value of either the sum of the uplift loads of the individual footings or the uplift loads of an equivalent pier foundation consisting of the footings and enclosed soil mass. Both values are estimates, the former method tends to overestimate the ultimate uplift capacity while the latter method tends to underestimate the failure load. Hence the assumed "resultant" failure zone of an anchor group in Meyerhof and Adam's theory do not model satisfactory the effects of the interaction between the failure zones of each anchor within the group. Meyerhof and Adam's theory required the D/B ratio - ϕ values relationships, and assumed that the failure surface of a single anchor or an anchor group makes an angle of $\alpha = \phi / 3$ with the vertical, for all types of sand. These assumptions are not true in practice, as sand grain roughness, size and uniformity which are ignored do affect the ϕ values of different types of sand. This is probably the reason why the theory is insensitive to changes in ϕ values.

Yilmaz's empirical solutions derived for the uplift capacity problem have the following limitations:

(a) both the suggested methods are for "shallow" anchor groups only and further modifications are needed for "deep" anchor groups. The classification of "shallow" and "deep" anchors in Yilmaz's investigation was based on the previous work done by Meyerhof and Adam (1968). (see section 2.2.)

(b) In Yilmaz's first analysis, for predicting the ultimate uplift loads of anchors in line groups, it is a prerequisite to obtain field or test results of the ultimate uplift loads for a single anchor and a pair of anchors. Hence, any small inaccuracy in the observed results is reflected in the predicted results.

(c) the solution obtained from the methods are approximate solutions at the failure condition and not at pre-failure condition.

(d) Yilmaz's second analysis for predicting the load distributed to each anchor in a group will be difficult to compute by hand calculations because of the multiple interactions between adjacent and neighbouring anchors. However, computers can be used successfully to carry out the calculations.

The anchor group analyses discussed, revealed only the general trend in predicting field uplift capacity. Hence the methods should be regarded as a first stage proposal only.

2.4. EXPERIMENTAL WORK ON UPLIFT CAPACITY OF ANCHOR GROUPS IN SAND:

There has been very little study of the behaviour of anchors in groups. Field testing of anchors in groups has been neglected because of practical difficulties in installing field size anchors, the great expense involved, and forseeable instrumentation problems. Hence, the bulk of tests reported here are on laboratory scale anchors in groups. A tabulated summary of the previous experimental work on the uplift capacity of anchor groups is shown in TABLE. 2.06.

Hueckel (1957)[24] experimental with a line group of three square anchor plates, each plate of thickness 100mm, at a depth of embedment to plate thickness ratio of two, buried in medium

TABLE. 2.06. A SUMMARY OF THE PREVIOUS EXPERIMENTAL WORK ON UPLIFT CAPACITY OF ANCHOR GROUPS IN SAND.

INVESTIGATIONS.	TYPE OF TESTS	GROUP CONFIGURATIONS	ANCHOR SPACINGS	PROPERTIES OF SAND				D/B RATIO
				DENSITY	R.D.	Ø	M.C.%	
HUECKEL (1957).	VERTICAL MODEL TESTS WITH SQUARE ANCHOR PLATES.	3x1	2.25B	MEDIUM			DRY	(H/h = 2)*
TROFIMENKOV & MARIUPOLSKII (1965)	VERTICAL FIELD TESTS WITH CIRCULAR ANCHOR PLATES.	3x1	1.5B - 5B	AVERAGE		30°-35°	MOIST	8
MEYERHOF & ADAMS (1968)	VERTICAL MODEL TESTS WITH CIRCULAR FOOTINGS.	2x1 ; 2x2	1B - 8B 1B - 7B	DENSITY LOOSE		45° 25°	DRY DRY	3 ; 8 3 ; 8

NOTE: H= THE DISTANCE FROM THE GROUND SURFACE TO THE BOTTOM EDGE OF THE ANCHOR PLATE.
h= THE HEIGHT OF THE ANCHOR PLATE.

CONTINUED:

INVESTIGATIONS	TYPE OF TESTS	GROUP CONFIGURATIONS.	ANCHOR SPACINGS	PROPERTIES OF SAND				D/B RATIO
				DENSITY	R.D.	Ø	M.C.%	
YILMAZ (1971)	VERTICAL MODEL TESTS WITH CIRCULAR PLATE & STRIP FOOTINGS	LINEAR GROUPS OF UP TO 7x1. SQUARE GROUPS OF UP TO 5x5.	2B - 6B	1541 kg/m^3	0.4	37^{o}	DRY	6;12
NEELY (1972)	VERTICAL MODEL TESTS WITH SQUARE ANCHOR PLATES.	2x1 ; 2x2.	1.75B - 4.5B	1615 kg/m^3			DRY	(H/h = 1.5to4)*
LARNACH (1972)	INCLINED MODEL TESTS WITH STEEL SPHERE FOOTINGS.	2x1 ; 3x1; 5x1.	2B - 10B	MEDIUM	0.59	33^{o}	DRY	16
LARNACH & McMULLAN (1975)	INCLINED MODEL TESTS WITH CIRCULAR ANCHOR PLATES	GROUPS OF UP TO 16 ANCHORS.	2B - 12B	LOOSE		33^{o}	DRY	25

NOTE: H= THE DISTANCE FROM THE GROUND SURFACE TO THE BOTTOM EDGE OF THE ANCHOR PLATE.
h= THE HEIGHT OF THE ANCHOR PLATE.

dense sand. The test results showed that the ultimate load of the grouped anchors decreased as the anchor spacings decreased.

Trofimenkov and Mariupolskii (1965)[53] conducted field tests on anchors arranged in (3 x 1) groups embedded in sand. The anchor spacing considered ranged from 1.5B to 5B and the depth of embedment was 8B (where B = diameter of a plate anchor). The experimental results failed to show the effects of group action when the grouped anchors were subjected to static uplift loads. This was primarily because no information was recorded about the case when the group of 3 anchors was pulled out as one unit. Instead only the ultimate uplift capacity of an individual anchor in a group was measured for each test. The test results showed that the ultimate uplift capacity of an individual anchor in a group, installed at anchor spacing not less than 1.5B, was equal to that of a single anchor embedded in the same soil when tested at the same anchor depth.

Wiseman (1966)[56] conducted some experiments on groups of model footings in sand at Nova Scotia Technical College to investigate the effects of number, spacings, and dimensions of footings in groups, and the properties of the soil on the uplift resistance of the groups. The experimental findings were reported by Meyerhof and Adam (1968)[40] who later derived a theoretical analysis to predict the ultimate uplift capacity of circular footings in groups embedded in sand. A comparison of the experimental results and the theoretical results are shown in FIG. 2.07. The experimental results show that for a given density of the sand the uplift efficiencies of the groups increased linearly with the spacing of the footings, and the efficiencies increased as the depth of embedment decreased. The uplift efficiencies decreased as the number of footings in the group increased and as the density of the sand increased. Although the trend of these observations is reflected in Meyerhof and Adam's theory, comparison between theoretical and experimental results revealed that agreement is much better at great depths than at shallow depths where the predicted results are rather conservative.

Yilmaz (1971)[58] conducted a series of laboratory tests to study the behaviour of groups of plate - shaped anchors buried in

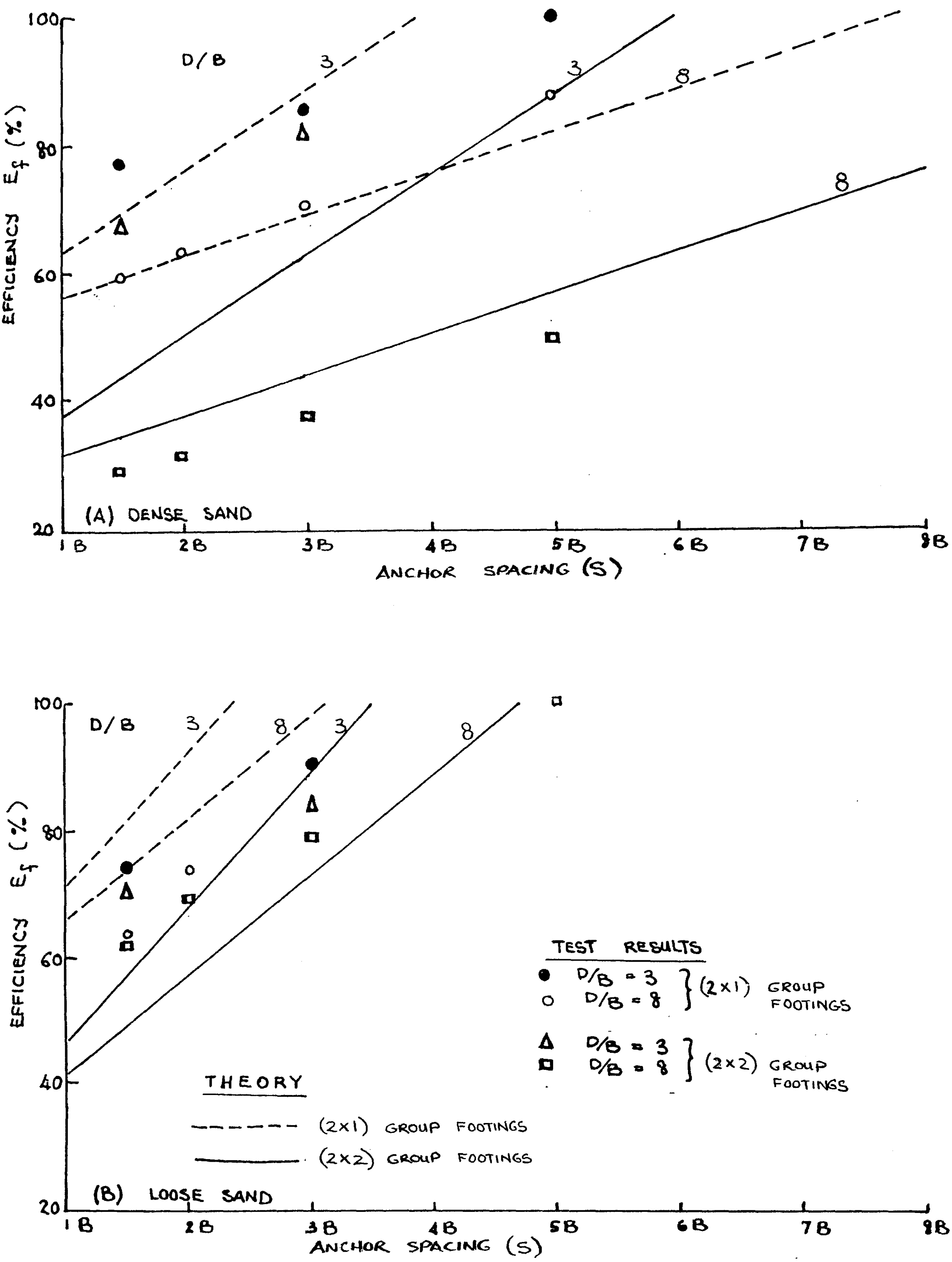
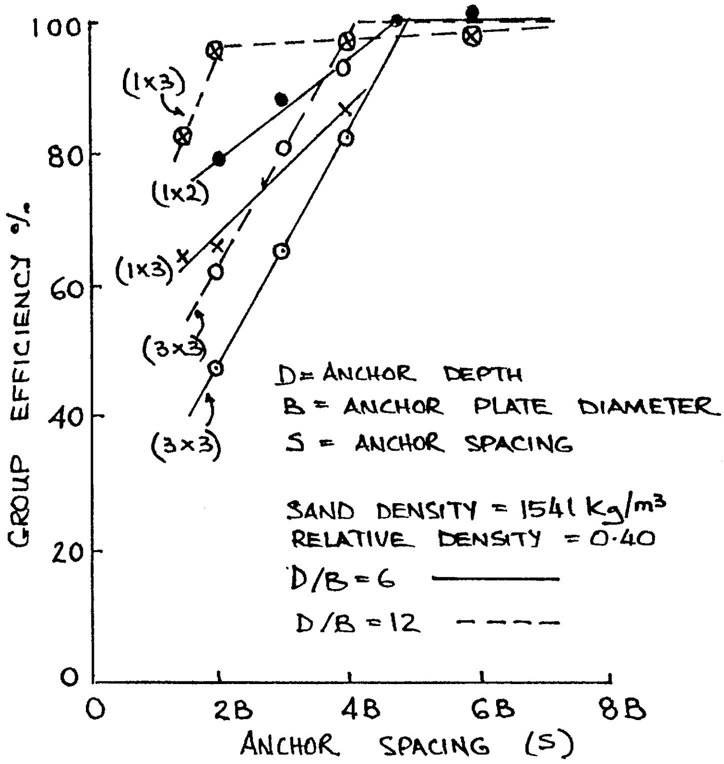


FIG. 2.07. COMPARISON OF MEYERHOF AND ADAMS THEORY WITH MEYERHOF AND ADAMS UPLIFT TESTS RESULTS OF GROUPS IN SAND.

dry sand of bulk density 1541 kg/m^3. Line groups of up to 7 anchors and square groups of up to (5 x 5) were tested. The circular anchor footing used had a diameter of 38mm. The depth/diameter ratios considered were 6 and 12, and the spacing factor ranged from 2B to 6B. The experimental results revealed that the ultimate uplift capacity of a group of n anchors is always less than n times the ultimate uplift capacity of an isolated anchor. The group efficiency is dependent on the anchor centre to centre spacing, the group size and shape, and the depth of anchor embedment. The load distribution within a group of anchors is non - uniform at the failure condition. Generally, the uplift capacity of an anchor depends on its distance from the point of uplift load application, the "centre" anchor carrying the smallest load and the "corner" anchor the largest load. FIG. 2.08. and FIG. 2.09. show the relationships of group efficiency - anchor spacing and group efficiency - group size, respectively.

Another series of experiments on anchors in groups were carried out by Neely (1972)[41]. Square anchor plates of 50mm dimension, in groups of two and four were embedded in Lough Neagh sand (ρ = 1615 kg/m^3) over a range of depth/height ratios (H / h = 1.5 to 4) where H is the distance from the ground surface to the bottom edge of the anchor plate and h is the height of the anchor plate. All the test results obtained indicated that the efficiency of the group decreased as the anchor spacing is reduced (as observed by Wiseman (1966) and Yilmaz (1971)). The displacement ratio at failure increased as the anchor spacing increased. In addition, the displacement ratio also depends on the size of the anchor group. (displacement ratio is defined as the displacement of a group of anchors at failure compared with that of a single isolated square plate at the same depth/height ratio.) Finally, it was found that the most significant variable influencing group behaviour was anchor spacing.

Limited research work has been reported on inclined anchors. Larnach (1972)[33] studied the pull - out resistance of inclined anchors installed singly and in groups in sand of relative density 0.59. In the experiments conducted only the anchor spacings (up to 10D) and the angle of pull - out (θ = 35° to 90°) were



F IG. 2.08. YILMAZ EXPERIMENTAL RESULTS.
-RELATIONSHIP OF GROUP EFFICIENCY vs ANCHOR SPACINGS.

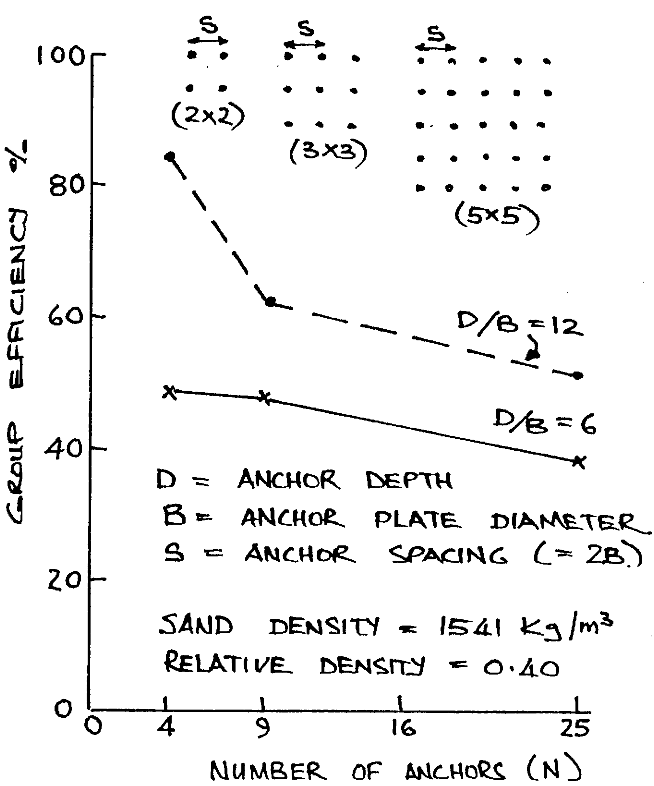


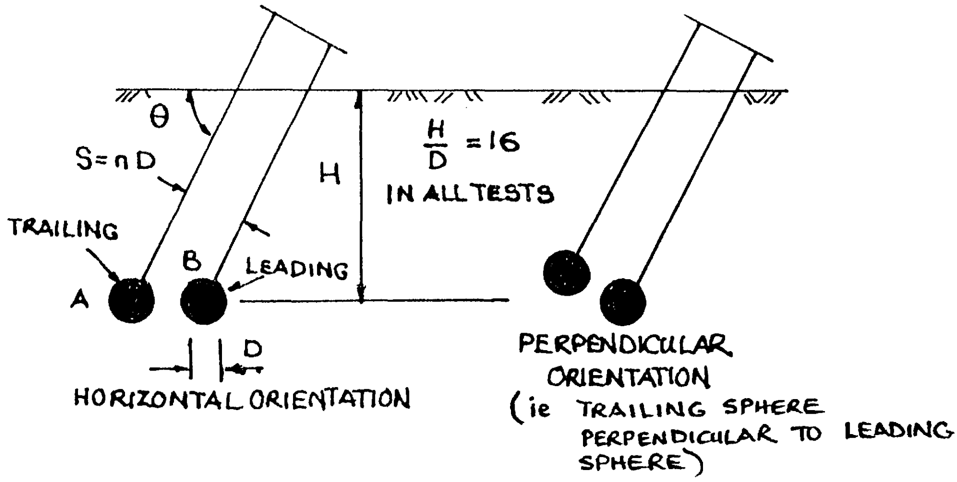
FIG. 2.09. YILMAZ EXPERIMENTAL RESULTS.
-RELATIONSHIP OF GROUP EFFICIENCY vs GROUP SIZES.

varied. Anchors of steel spheres having a diameter D = 22mm were tested in groups of two. The depth of embedment H was kept constant at a ratio H/D = 16. FIG. 2.10. shows typical load/displacement curves for a "perpendicularly orientated" group of inclined anchors at $\theta = 60^\circ$. The test results show that the depth of embedment predominate since the load on "A" is smaller than that on "B". Summarised in FIG. 2.11. is the ultimate uplift capacity of an individual anchor in a "horizontal orientation" group of inclined anchors at $\theta = 90^\circ$, 60° and 35°, plotted against anchor spacing. It is observed that there is no interaction at spacing of about 8D, 5D and 10D respectively. In FIG. 2.12. the relationship between group efficiency and anchor spacing for different angles of inclination is shown. The $\theta = 60^\circ$ case is the optimum, if this is defined in terms of the least value of the spacing at which the efficiency reaches 100 % . From the experimental results, it can be clearly established that interaction can occur to the considerable disadvantage of the trailing anchor in inclined grouped anchors.

McMullan and Larnach (1975)[31] described tests to determine the failure zones around "deep" single anchors and "deep" line grouped anchors subjected to loading. Some qualitative explanations of group efficiency were suggested after examining the experimental results. For a group of "deep" anchors at close anchor spacing (e.g. S=2B), the interference of the failure zones is so intense that adjacent anchors in the group effectively act as a single, larger anchor and efficiencies are increased. For spacings between 4B and 8B a group of "deep" vertical anchors may be taken to act as a group of "shallow" anchors as interference of failure zones is mild, and result in loss in efficiencies. Then for spacings greater than 8B, each anchor in the group acts as a single isolated anchor resulting in a group efficiency of 100%.

2.5. COMMENTS ON THE PREVIOUS EXPERIMENTAL WORK PRESENTED:

Meyerhof and Adam (1968) were pioneers in the investigation of the behaviour of anchors in groups and concluded that there are interactions between anchors in a group. The interaction factor was measured by studying the group efficiency trends of



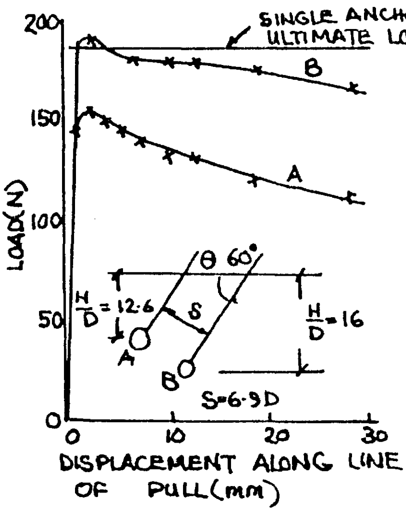


FIG. 2.10. LARNACH TESTS RESULTS.
-GRAPH OF LOAD vs DISPL.

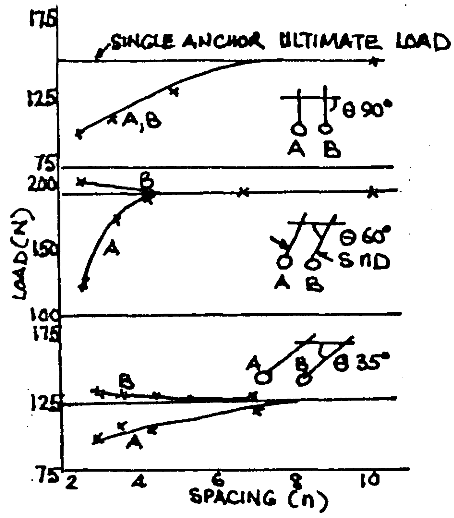


FIG. 2.11. LARNACH TESTS RESULTS.
-GRAPH OF LOAD vs SPAC.

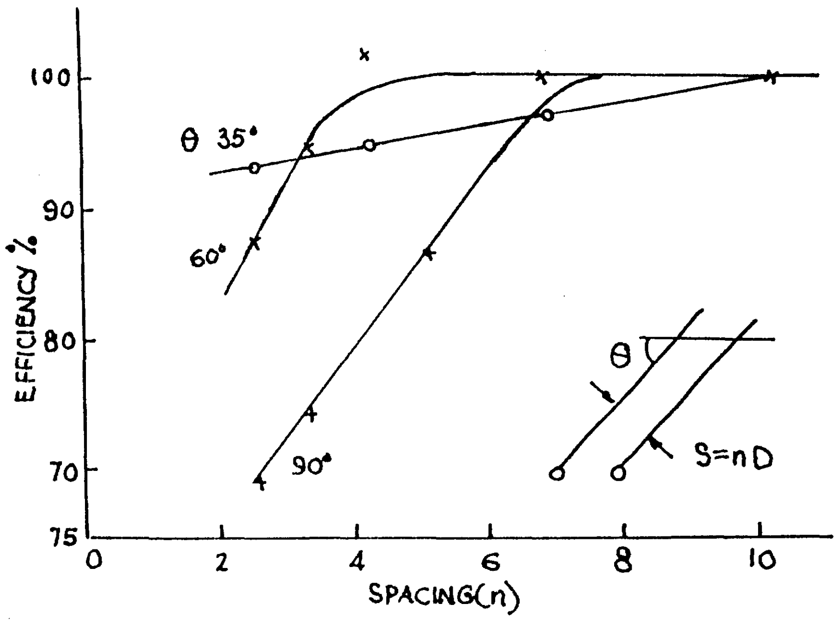


FIG. 2.12. LARNACH TESTS RESULTS.
-GRAPH OF GROUP EFFICIENCY vs SPACINGS.

groups of two and four anchors. From these experimental results, a semi-empirical analysis for predicting ultimate uplift capacity of anchors in groups buried in sand was proposed. However, this analysis lacks a full appreciation of the interaction problem in grouped anchors.

Yilmaz (1971) experimented with larger anchor groups and investigated the variables affecting group efficiency, load distribution amongst anchors in groups and the group anchors displacements. Based on test results an empirical analysis was derived for estimating the ultimate uplift capacity of anchors in a group. Further work on anchor group action was conducted by Larnach (1972). Qualitative explanations of group efficiency were suggested (see section 2.4.). The anchor spacing was found to be the most significant variable affecting group behaviour.

The experimental results from each reported case are difficult to compare because of differences in testing conditions. However, some general similarities between previous test results are:

(a) anchor spacing of an anchor group is directly proportional to its ultimate efficiency.

(b) other variables such as density of soil used, angle of friction of soil, anchor group configuration, anchor group sizes and anchor loading conditions can affect group behaviour. However, the most significant variable affecting group behaviour is anchor spacing.

The review has shown that there is a lack of theoretical analysis for predicting the group action behaviour. Both the Meyerhof and Adam's (1968) and Yilmaz's (1971) analyses for anchors in groups were found to be not entirely satisfactory (see section 2.3. and 5.5.). In this investigation an attempt is made to derive a simple analysis for predicting the ultimate uplift capacity of anchors in line groups installed in sand, based on the author's experimental results. The proposed analysis will be validated using Larnach and McMullan's (1975), and Yilmaz's(1971) experimental results.

CHAPTER 3

PROPOSED ANALYSIS FOR DETERMINING THE ULTIMATE UPLIFT CAPACITY OF ANCHORS IN LINE GROUPS AND DIMENSIONAL ANALYSIS

CHAPTER 3

PROPOSED ANALYSIS FOR DETERMINING THE ULTIMATE UPLIFT CAPACITY OF ANCHORS IN LINE GROUPS AND DIMENSIONAL ANALYSIS.

3.1. INTRODUCTION:

An extensive series of experiments were conducted by the author and are reported in Chapters 4 and 5. Based on these test results, a general empirical analysis to predict the ultimate uplift capacity of "shallow" and "deep" vertically embedded anchors in line groups installed in sand is proposed. Meyerhof and Adam (1968) proposed an analysis in which a single "deep" anchor and a "shallow" anchor were assumed to develop failure surfaces as shown in FIG. 2.00. The proposed analysis is based on a series of straight line failure surfaces as shown in FIGs. 3.00. (a) and 3.01. (a) where the volume within the failure zone of a single isolated anchor can be divided into volumes x, y, z, each of which contributes a proportion of the failure load. The previous treatment of the group action of "shallow" and "deep" anchor groups by Meyerhof and Adam is considered to be not entirely satisfactory. The proposed analysis incorporated the use of a simple interaction concept (see FIGs. 3.00. (b), (c) and 3.01. (b), (c).) to predict the failure loads of anchors in line groups.

Also, in this Chapter is a discussion on the application of dimensional analysis in the investigation of anchor groups. This analysis is often helpful in establishing similitude between small-scale model test results and full-scale prototypes.

3.2. THE PROPOSED EMPIRICAL ANALYSIS TO PREDICT THE ULTIMATE STATIC UPLIFT CAPACITY OF ANCHORS IN LINE GROUPS:

In this analysis it is a prerequisite to obtain field or experimental results of failure load for a single anchor and a pair of two anchors for predicting the ultimate uplift capacity of anchors in line groups.

FIG. 3.00. (a), (b) and (c) indicate the proposed

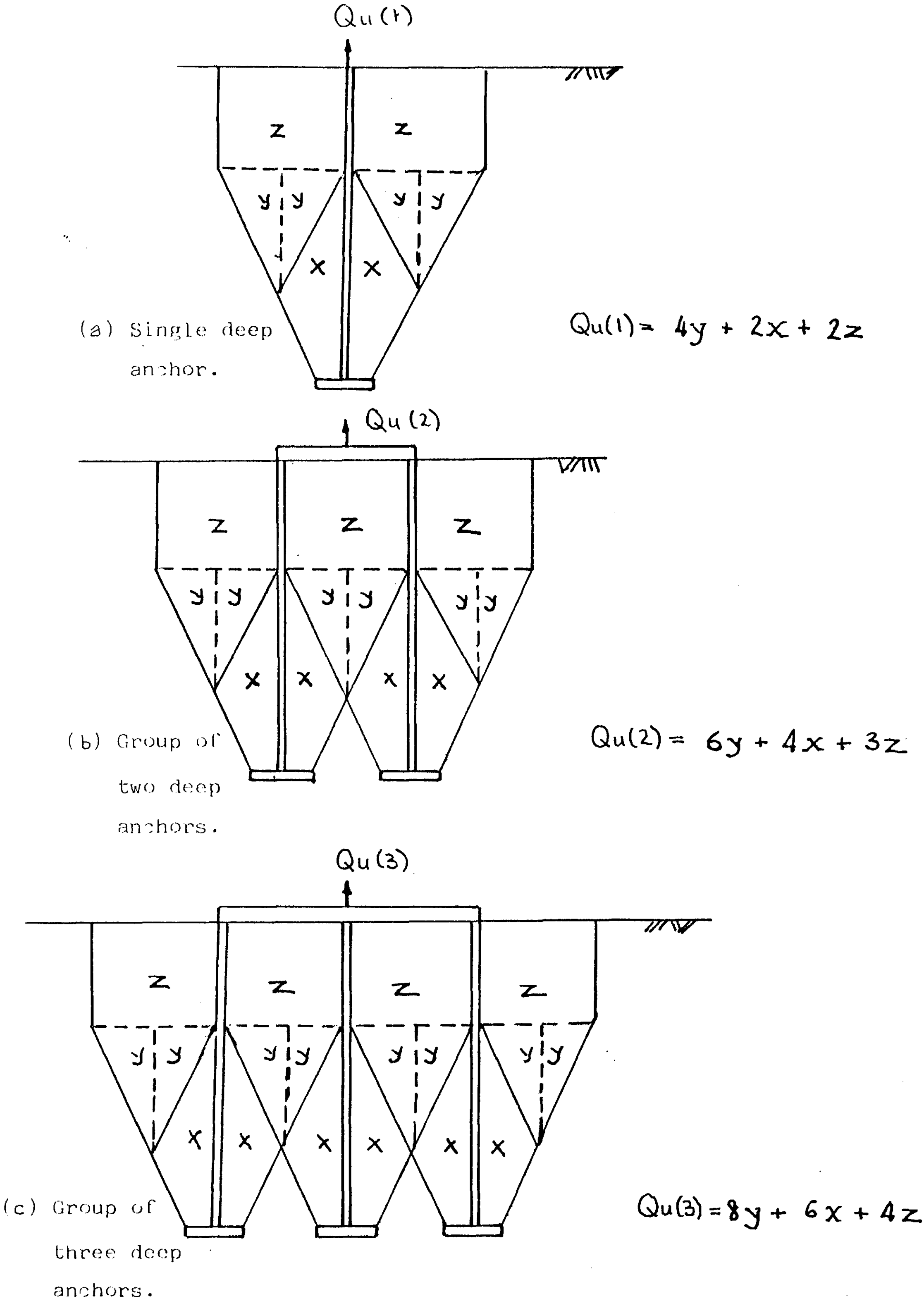


FIG. 3.00. THE INTERACTION OF THE PROPOSED FAILURE ZONES OF DEEP ANCHORS IN LINEAR GROUPS.

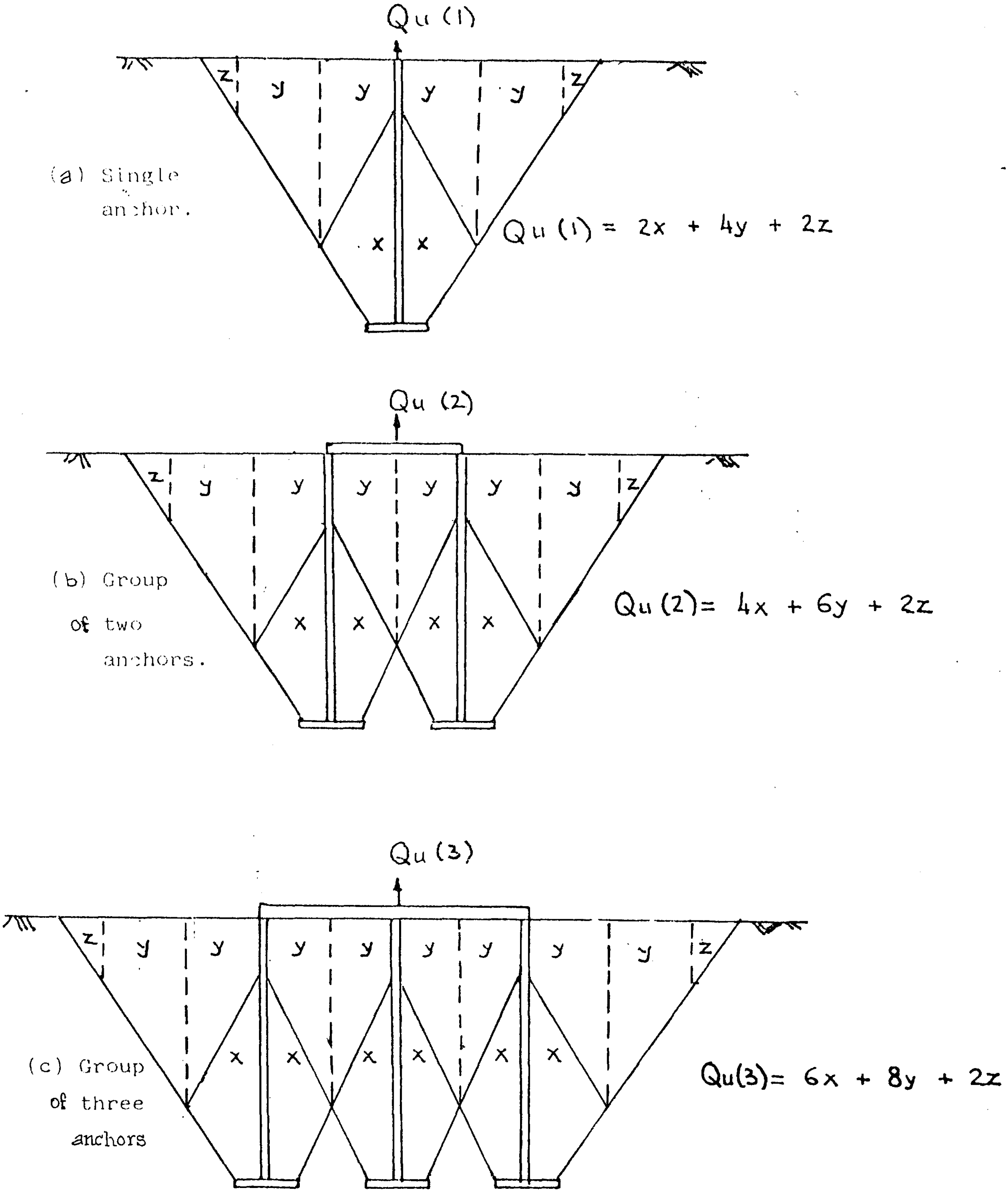


FIG. 3.01. THE INTERACTION OF THE PROPOSED FAILURE ZONES OF SHALLOW ANCHORS IN LINEAR GROUPS.

failure zones for an arrangement of a single, two and three deep anchors, respectively, and shows the interaction between the failure zones of closely spaced anchors for the two latter cases.
In this simple analysis it is assumed that the volume within the failure zone of a single anchor can be divided into volumes x, y, z, each of which represents a proportion of the single anchor failure load Qu(1), as shown in FIG. 3.00. (a). Therefore it can be stated from FIG. 3.00. (a) that:

$$Qu(1) = 4y + 2x + 2z \qquad \text{.... (E.3.01.)}$$

and from FIG. 3.00. (b).

$$Qu(2) = 6y + 4z + 3z \qquad \text{.... (E.3.02.)}$$

which can be represented as:

$$2Qu(1) - Qu(2) = 2y + z \qquad \text{.... (E.3.03.)}$$

The ultimate uplift load of the three anchors in a line group is given from FIG. 3.00. (c) as:

$$Qu(3) = 8y + 6x + 4z \qquad \text{.... (E.3.04,)}$$

Rearranging to:

$$Qu(3) = 3 (4y + 2x + 2z) - 4y - 2z \qquad \text{.... (E.3.05.)}$$

and substituting equations (E.3.02.) and (E.3.03.) into (E.3.05.) gives:

$$Qu(3) = 3Qu(1) - 2 \left[2Qu(1) - Qu(2) \right] \qquad \text{.... (E.3.06.)}$$

Which can also be expressed as:

$$Qu(3) = 2Qu(2) - Qu(1) \qquad \text{.... (E.3.07.)}$$

This final equation is in similar form to that for ultimate uplift capacity of three "shallow" anchors in a line group, see equation (E.2.05.). Hence the equation (E.3.07.) can be used to predict the ultimate static uplift capacity of three "shallow" and "deep" anchors installed in a line group, at anchor spacings ranging from B to 8B.

By following the same principles as before it can be shown that:

$$Qu(4) = 3Qu(2) - 2Qu(1) \qquad \text{.... (E.3.08.)}$$

where Qu(4) = ultimate uplift capacity of a line group of four anchors.

Similiarly,

$Qu(5) = 4Q(2) - 3Q(1)$ .... (E.3.09.)

and

$Qu(6) = 5Q(2) - 4Q(1)$ .... (E.3.10.)

where

$Qu(5)$ = ultimate uplift capacity of a line group of five anchors.

$Qu(6)$ = ultimate uplift capacity of a line group of six anchors.

From these equations, a general empirical equation to predict the ultimate static uplift capacity of "shallow" and "deep" vertically embedded anchors in line groups at spacings ranging from B to 8B is:

$Qu(n) = (n-1)\,Qu(2) - (n-2)\,Qu(1)$ .... (E.3.11.)

where

$Qu(1)$ = ultimate uplift capacity of an isolated anchor.

$Qu(2)$ = ultimate uplift capacity of two anchors in a group.

n = number of anchors in a group.

$Qu(n)$ = ultimate uplift capacity of n number of anchors in a line group.

The assumptions made in developing this general equation (E.3.11.) are as follows:

(a) The soil is homogeneous, isotropic, dry and cohesionless.

(b) The anchors are installed with their axes in a vertical orientation.

(c) A "shallow" anchor develops a failure zone as shown in FIG. 3.01. It is assumed that the volume within the rupture zone can be divided into volumes x, y, z, each of which represents a proportion of the failure load.

(d) A "deep" anchor develops a failure zone as shown in FIG. 3.00. Again each of the volumes x, y, z, represents a proportion of the failure load.

(e) The interaction of the volumes of adjacent failure zones of n anchors in a group results in a smaller failure load capacity than a group of n isolated anchors with no interaction between their failure zones. This concept is reflected in the proposed analysis as shown in equations E.3.01. E.3.02. and E.3.04.

(f) The model test parameters used in obtaining Qu(1) and Qu(2) (in equation E.3.11.) should be the same. These test parameters are anchor spacings, anchor depth of embedment, anchor group configurations, shape of anchor body, breath or diameter of anchor body, method of installation of anchors, soil conditions and rate of application of static uplift loads. Hence predicted Qu(n)(in equation E.3.11.) will have the same test parameters as Qu(1) and Qu(2).

3.3. APPLICATION OF DIMENSIONAL ANALYSIS TO GROUP ANCHOR RESEARCH:

Dimensional analysis, introduced by Buçkingham in his π - theorem , has been used by numerous researchers to determine the functional relationships between the primary physical constants involved in physical phenomena. In this respect, dimensional analysis is often helpful in establishing similitude between small-scale model tests and full-scale prototypes.

The π - theorem states that a physical phenomenon which is a function of n physical quantities involving m fundamental units can be described in the functional form (Baker & Kondner, 1966.)

$$F\ (\pi_1, \pi_2, \pi_3, \ \ldots\ldots, \pi_{n-m}) = 0 \qquad \ldots.(E.3.12.)$$

where the π - terms are the (n-m) independent dimensionless products of the n physical quantities.

The primary physical quantities for the uplift capacity of a circular plate anchor buried in sand are listed in TABLE 3.02. using the force, length, and time system of fundamental units. Using the Buckingham π - method the physical quantities yield the functional relationship:

$$Qu = f_1\ (S, Wg, D, B, \emptyset, R.D., \psi) \qquad \ldots.(E.3.13.)$$

PHYSICAL QUANTITIES	SYMBOLS	FUNDAMENTAL UNITS.
UPLIFT CAPACITY.	Qu	F
DIAMETER OF ANCHOR.	B	L
DEPTH OF EMBEDMENT.	D	L
UNIT WEIGHT OF SOIL.	Wg	FL^{-3}
ANGLE OF FRICTION	ϕ	$F^{o}L^{o}T^{o}$
RELATIVE DENSITY.	R.D.	$F^{o}L^{o}T^{o}$
ANGLE OF INCLINATION OF ANCHOR.	ψ	$F^{o}L^{o}T^{o}$
ANCHOR SPACING	S	L

TABLE. 3.02. PRIMARY PHYSICAL QUANTITIES INFLUENCING THE ULTIMATE UPLIFT CAPACITY OF CIRCULAR PLATE ANCHORS EMBEDDED IN SAND.

Using algebraic transformations, alternative sets of independent π - terms can be obtained which yield the functional relationships:

$$\frac{Qu}{\frac{\pi}{4}\, B^2 DWg} = f_2 \left(S/B \; , \; D/B \; , \; \phi \; , \; R.D. \; , \; \psi \right) \qquad \ldots.(E.3.14.)$$

$$\frac{Qu}{DB^2 Wg} = f_3 \left(S^2/B^2 , \; D^2/B^2 , \; \phi \; , \; R.D. \; , \; \psi \right) \qquad \ldots.(E.3.15.)$$

$$\frac{Qu}{B^3 Wg} = f_4 \left(S/B \; , \; D/B \; , \; \phi \; , \; R.D. \; , \; \psi \right) \qquad \ldots.(E.3.16.)$$

Hence the conclusions that can be drawn from (e.g. equation E.3.14.) are that for a given group of circular anchor plates embedded in cohesionless soil with known ϕ, R.D., ψ :-

(a) $\dfrac{Qu}{\frac{\pi}{4}\, B^2 DWg}$ depends on S/B and D/B.

(b) $\dfrac{Qu}{\frac{\pi}{4}\, B^2 DWg}$, S/B, D/B are constants which can provide a simple basis for the possible correlation of the results of small - scale model tests with that of full - scale prototypes.

The findings from the dimensional analysis e.g. equation (E.3.14.) will be used to present the author's experimental results (in Chapter 5).

CHAPTER 4

EXPERIMENTAL INVESTIGATION

CHAPTER 4

EXPERIMENTAL INVESTIGATION

4.1. INTRODUCTION:

This chapter describes the materials and equipment used in the experimental programme, which comprised 10 uplift tests on single anchors and 41 tests on various arrangements of groups of anchor embedded in sand. The 51 tests covered a range of anchor spacings and depths as described in section 4.4.4.

The sand used was "standard" Leighton Buzzard sand, since this would enable comparisons to be drawn where appropriate with the work of previous investigators. The sand was placed at medium density of 1638 kg/m^3 (R.D. = 0·58). The particle size distribution and shear strength of the sand are described. The equipment used for placing the sand along with the other equipment for measuring and recording the experimental results are reported.

The discussion of the experimental results and their comparison with the theoretical predictions are presented in Chapter 5.

4.2. PROPERTIES OF THE SAND USED.

4.2.1. BASIC CHARACTERISTICS:

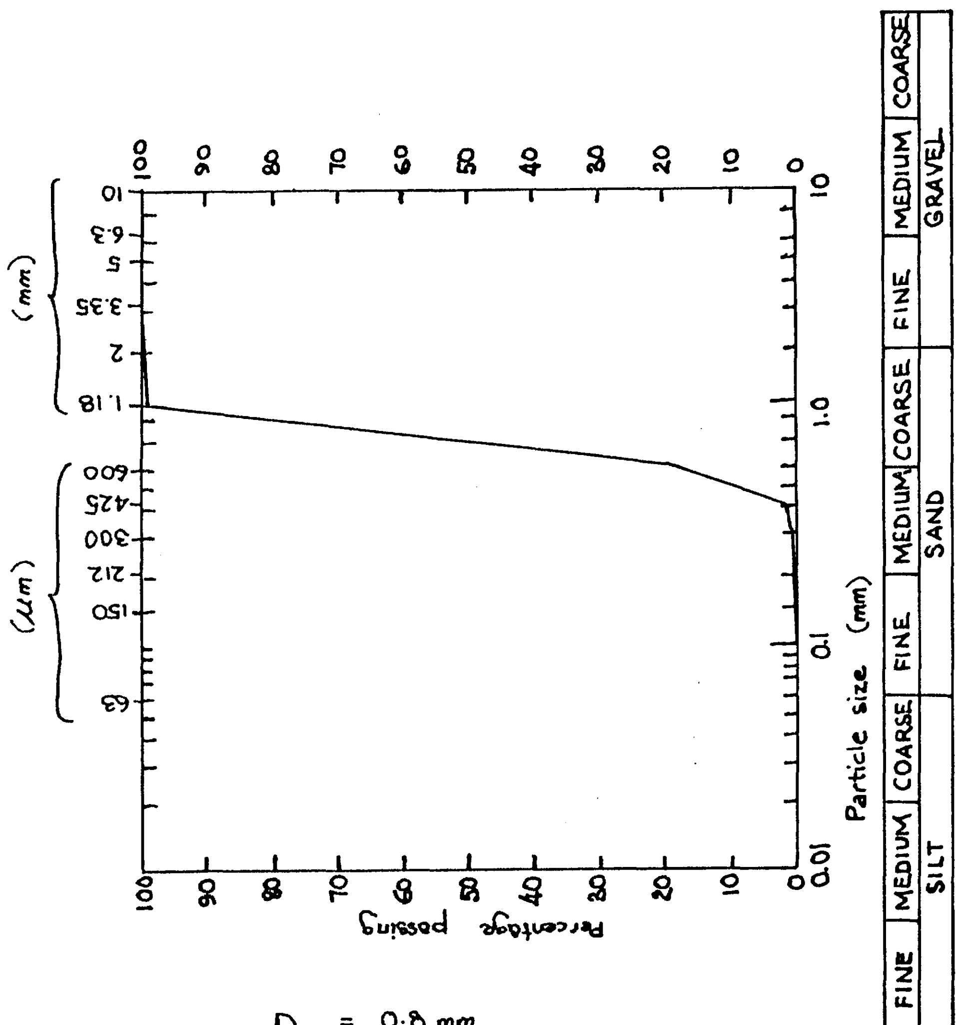
The Leighton Buzzard sand used throughout the testing programme was air dried. The moisture content of the samples was measured to be 0.1% throughout the whole testing period.

The Particle Size Distribution (P.S.D.) curve of the sand in FIG. 4.00., was obtained using the standard wet sieving method (B.S. 1377) : 1975. It can be observed that the sand particle size ranged from 2.0 to 0.2mm (equivalent B.S.S. 10 - 170). The uniformity coefficient Cu was 1.76 and the mean particle diameter, D_{50} was 0.8mm. The sand particles had a subrounded shape.

Using the test method for fine grained soil described in B.S. 1377 : 1975, the specific gravity of the sand was found to be 2.65. The mineral composition of the sand was mainly quartzite.

The determination of maximum and minimum densities of the sand was carried out by using the techniques developed by Kolbuszewski (1948). To determine the minimum porosity, the

FIG. 4.00. PARTICLE SIZE DISTRIBUTION CHART OF DRY LEIGHTON BUZZARD SAND.



$D_{50} = 0.8$ mm

$\frac{D_{60}}{D_{10}} = 1.76$

test technique involved using an electric hammer ("Kango hammer") to compact the sand sample in a compaction cylinder placed in a container full of water. The results of three minimum porosity tests for Leighton Buzzard and were 0.329, 0.336 and 0.318, the average being 0.328. In the loosest sand conditions the maximum porosity of the sand was determined by allowing a sample of dry sand to fall freely in a 2000 cc glass cylinder. The average result of several tests was 0.444.

4.2.2. SHEAR STRENGTH:

Drained triaxial tests were performed on dry sand samples of dimensions 102mm diameter x 203mm length, having bulk densities ranging from 1561 - 1748 kg/m^3 (R.D. ranging from 32% - 90%). The vacuum lateral pressure applied ranged from 10 - 60 kN/m^2. The detailed procedure can be found in Bishop & Henkel[9]. FIG. 4.01. shows a diagrammatic representation of the triaxial test conducted. The relationship between bulk density (ρ), relative density (R.D.) and effective angle of friction (ϕ') of dry Leighton Buzzard sand found from the tests conducted is shown in FIG. 4.02.

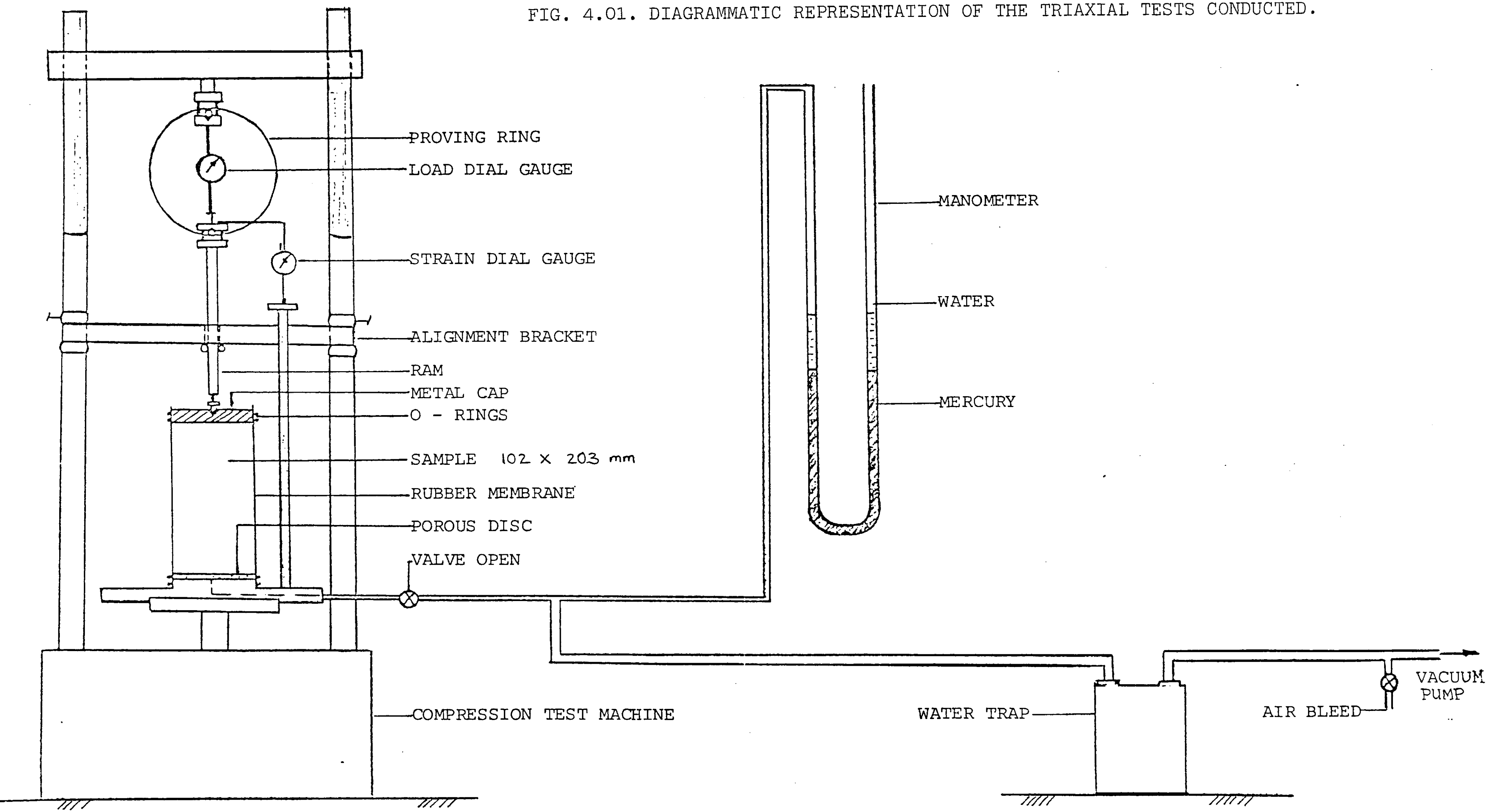
4.3. EXPERIMENTAL EQUIPMENT:

4.3.1. APPARATUS FOR LAYING UNIFORM SAND BEDS:

In this series of model tests, it was important that the sand beds used were uniform and isotropic, since all theoretical work is based on idealised conditions. The sand beds should be reproducible and the method of deposition should not induce any lateral stress that would give rise to a coefficient of earth pressure at rest, Ko, that was higher than the coefficient for an undisturbed, unconsolidated sand bed.

The technique of sand bed deposition used was similar to that used by Hutchison (1982)[25] and as shown in FIG. 4.03. This sand placing technique was adopted in this present investigation, which is part of a continuing research programme into anchor group behaviour, to be compatible with the technique used in Hutchison's (1982) previous investigation. It was hoped that the author's test results could be compared directly with Hutchison's (1982) test results to produce a more comprehensive report than

FIG. 4.01. DIAGRAMMATIC REPRESENTATION OF THE TRIAXIAL TESTS CONDUCTED.



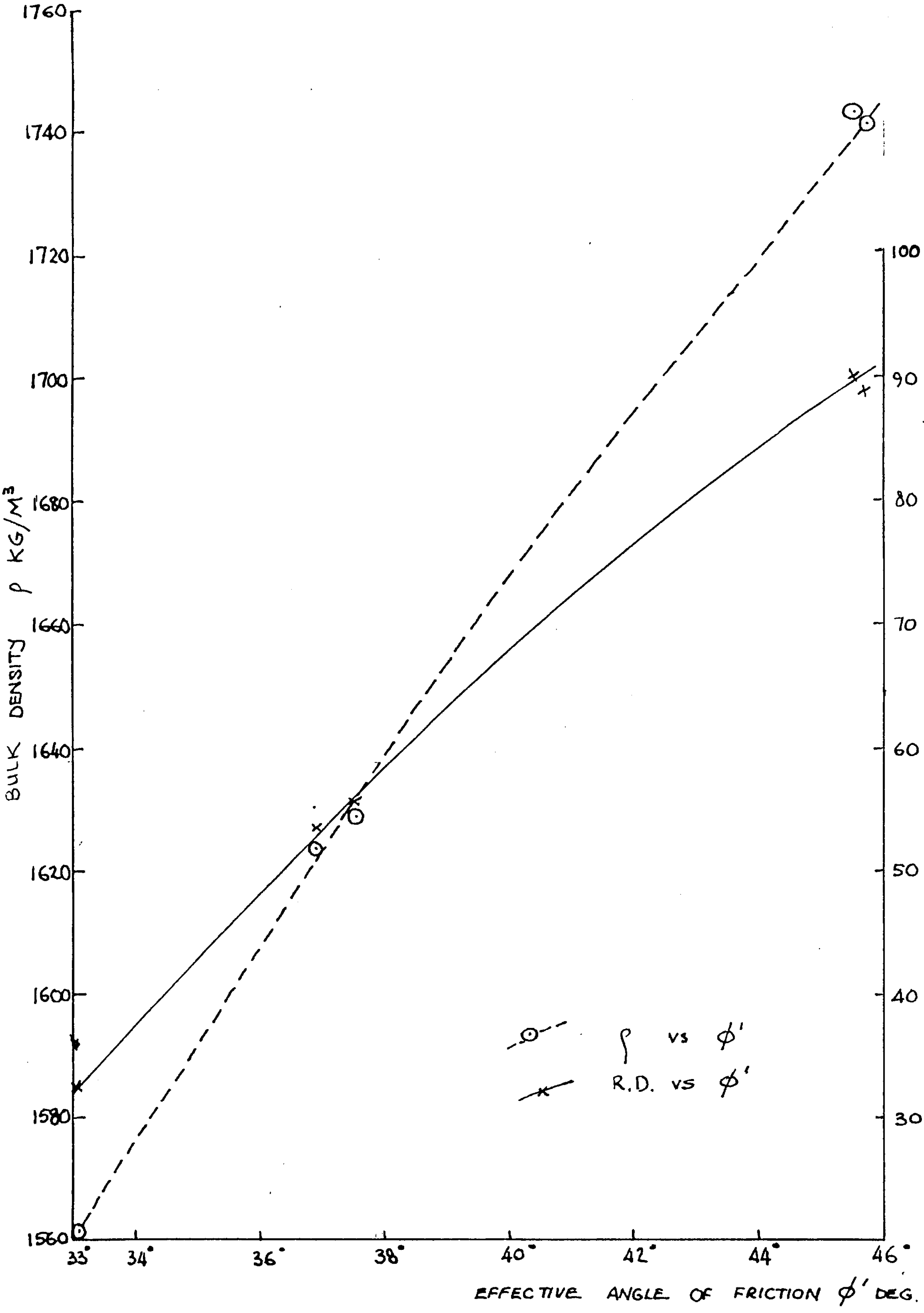


FIG. 4.02. RELATIONSHIP BETWEEN ρ AND Ø'; R.D. AND φ'

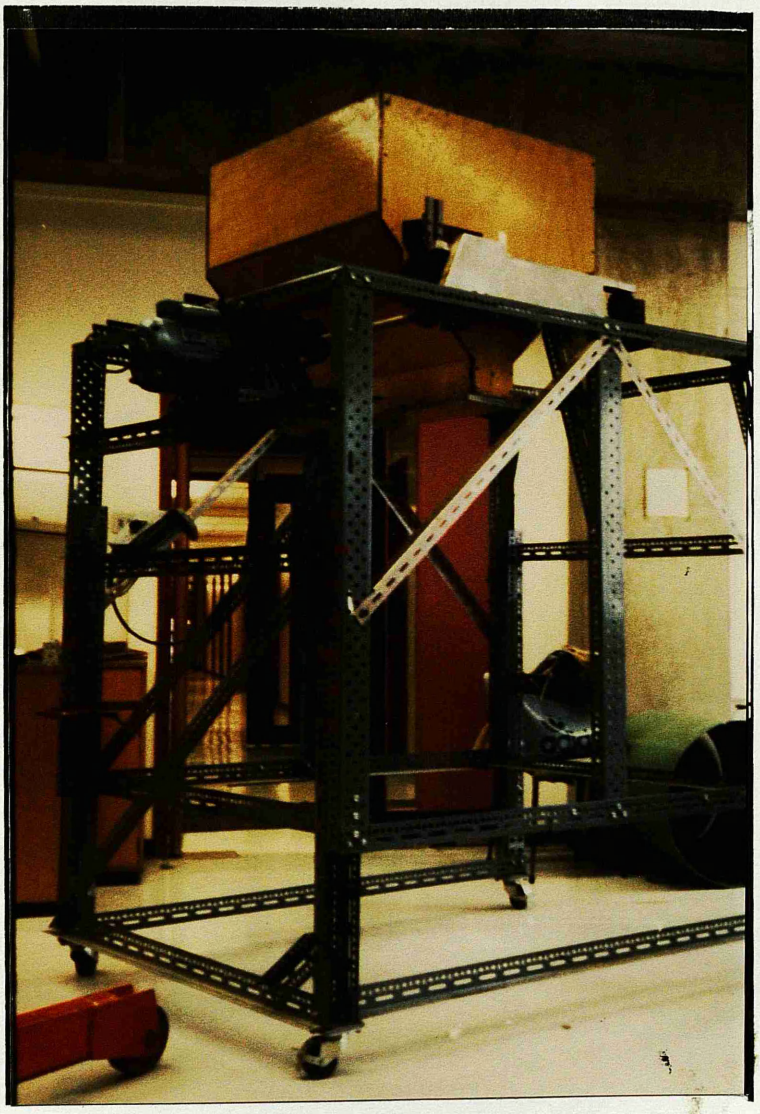


FIG. 4.03. SAND RAINING MACHINE.

the present report on anchor group behaviour. However, Hutchison's (1982) work is still not complete and hence will not be discussed in this investigation.

In this sand placing technique, the sand bed was built up of thin layers each of which was produced from a uniform rain of sand, discharged from a travelling hopper. The hopper was driven by a motor, and a system of chains and gears on rail tracks. A shutter plate could fully open or close the apertures of the hopper's perforated base. The supporting frame of the hopper could be adjusted to any height directly above the sand receiver tank.

According to Kolbuszewski and Jones (1961)[30], the density, of the sand is a function of the intensity of fall of the sand and the height of free fall of sand from the hopper to the surface of the sand layer in the receiver tank. The sand raining technique used could control both these factors. The intensity of fall of the sand was controlled by the aperture size of the sieve through which the sand fell. It was noted that the larger the aperture size of sieve, the higher the intensity of fall of sand resulting in a correspondingly lower density of sand. As for the free fall height, the higher the height of free fall the denser the sand. The uniformity of the sand beds was checked using density pots and results obtained showed insignificant variation throughout the beds. A range of loose to dense density sand beds could be deposited using the above technique.

In this investigation, the model tests were conducted at one sand density of 1638 kg/m^3 (R.D. = 0.58). No attempt was made to produce a lower density for the model tests since the sand placing technique used does not prepare consistently uniform beds of loose sand. However, denser sand beds of above 1638 kg/m^3 (maximum density of 1750 kg/m^3) could be reproduced satisfactorily if required.

4.3.2. STRAIN GAUGES:

Strain gauges were used to measure the individual loads applied to each anchor unit arranged in a group. Four strain gauges were connected in a typical full bridge circuit on each anchor unit (See FIG. 4.04.). Before using the strain gauge unit,

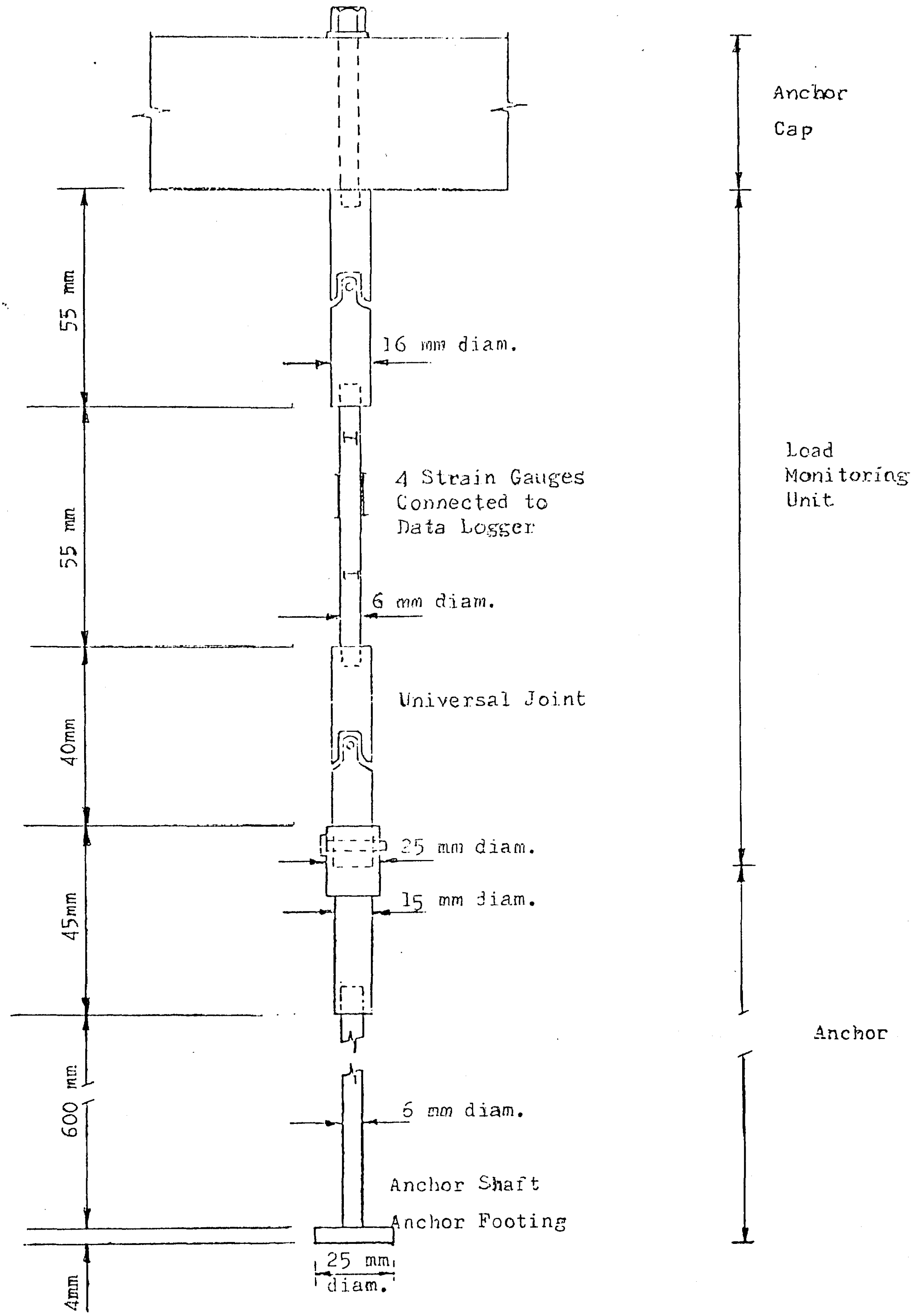


FIG.4.04. DETAILS OF AN ANCHOR UNIT.

it was calibrated in accordance with the manufacturer's recommendations.

4.3.3. LOAD CELL:

A "calibrated" load cell of 2224 N capacity (FIG 4.05) was used to measure the total load applied to a group of anchor units. The difference between the strain gauge and load cell readings over a range of 0 - 600 N was $\pm$ 2% (which was acceptable).

4.3.4. DISPLACEMENT TRANSDUCERS:

The "calibrated" displacement transducers (FIG. 4.06.) were used to measure the vertical linear displacement of a single anchor or group of anchors when subjected to a static uplift load. The transducers performed satisfactorily throughout the tests and regular checks on their calibrations required only minor adjustments (errors of $<$ 1%).

4.3.5. OTHER EQUIPMENT:

The other equipment used in the experiments is shown in FIGs. 4.07, 4.08, 4.09. The equipment was regularly maintained according to the manufacturer's specifications throughout the experimental investigation.

4.4. EXPERIMENTAL PROCEDURE:

4.4.1. ALIGNMENT OF THE SAND TANK:

A supporting frame for the wooden sand tank (1000mm x 1000mm x 647mm deep) was bolted into position. The sand tank was then seated on the supporting frame and screwed down. The anchor was aligned in the centre of the tank and in turn the piston arrangement was aligned directly above the anchor (see FIG 4.09).

4.4.2. PREPARATION OF SAND BED:

To obtain the medium density sand (1638 kg/m^3), a suitable aperture size (10mm) of perforated plate was used, and the height of free fall of sand into the sand tank was kept at a chosen constant (700mm) through the deposition of every layer. The required density of the sand beds could be reproduced each time with an accuracy of $\pm$ 0.5%.

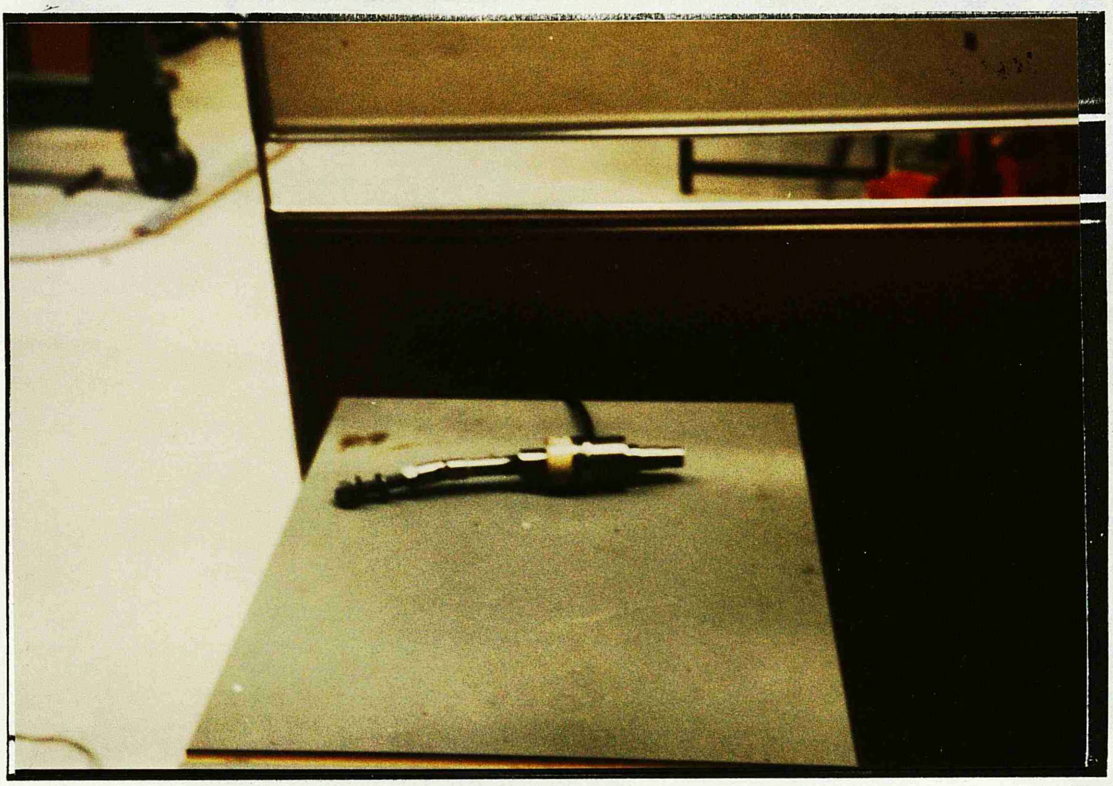


FIG. 4.05. LOAD CELL OF 2224 N CAPACITY.

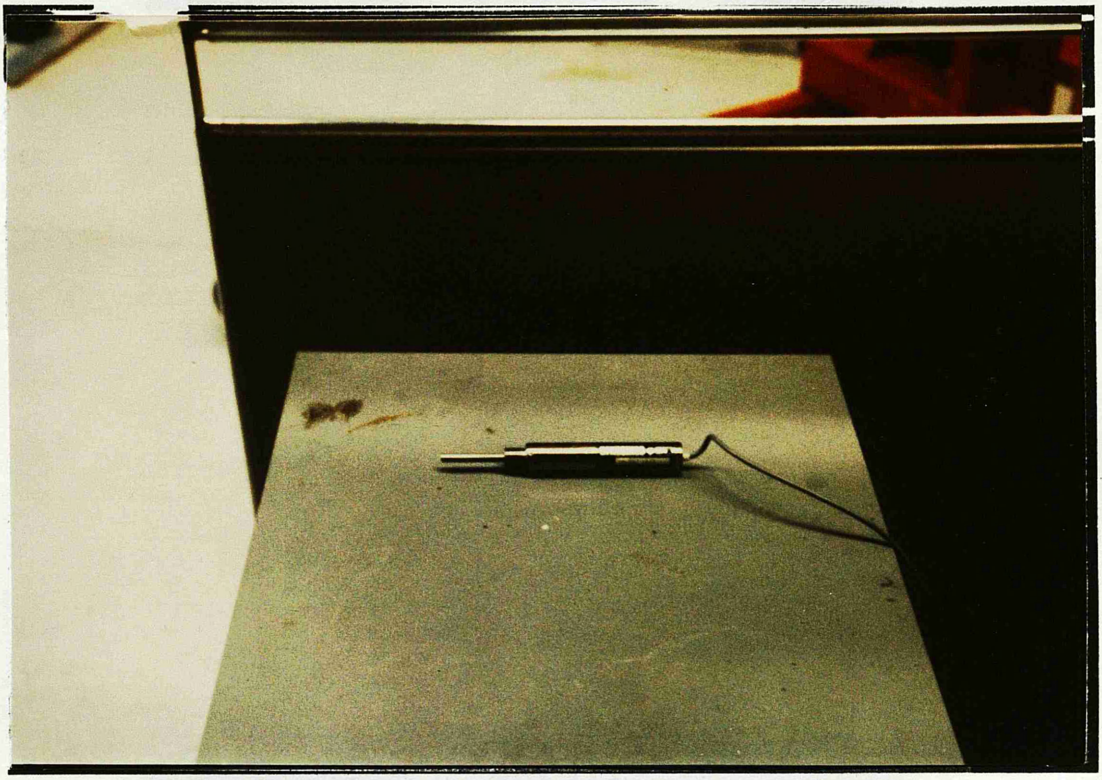


FIG. 4.06. DISPLACEMENT TRANSDUCER.

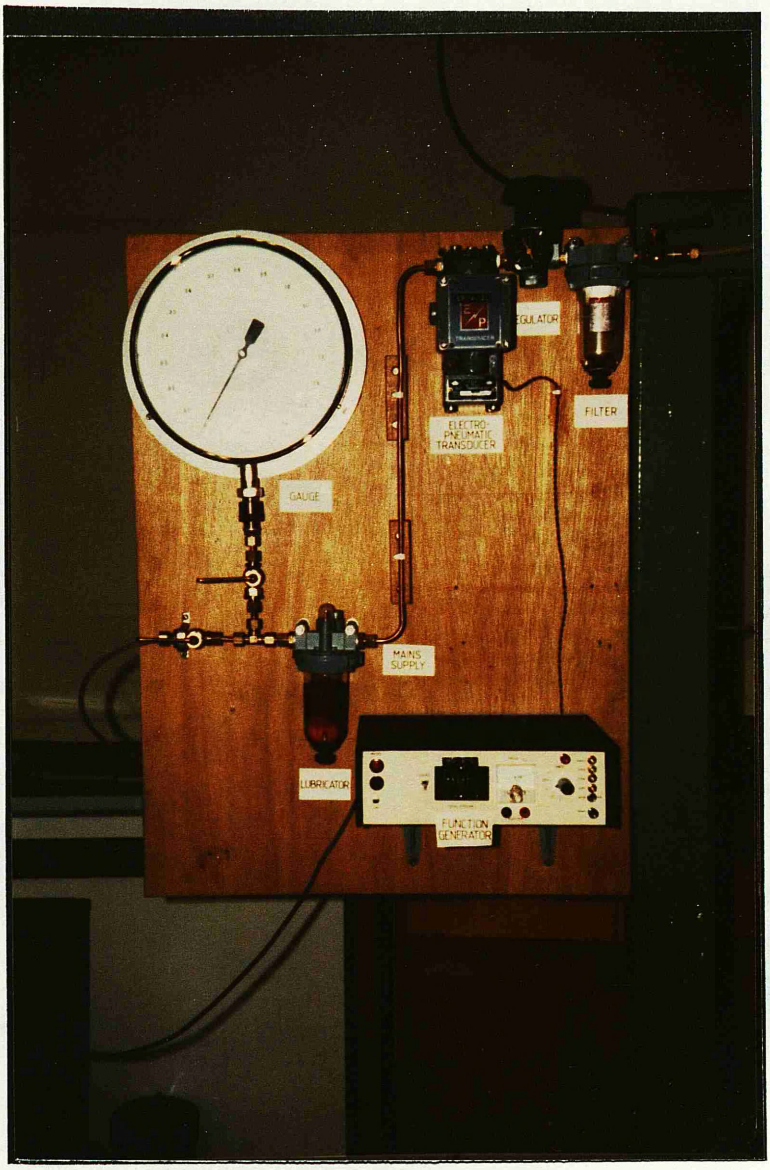
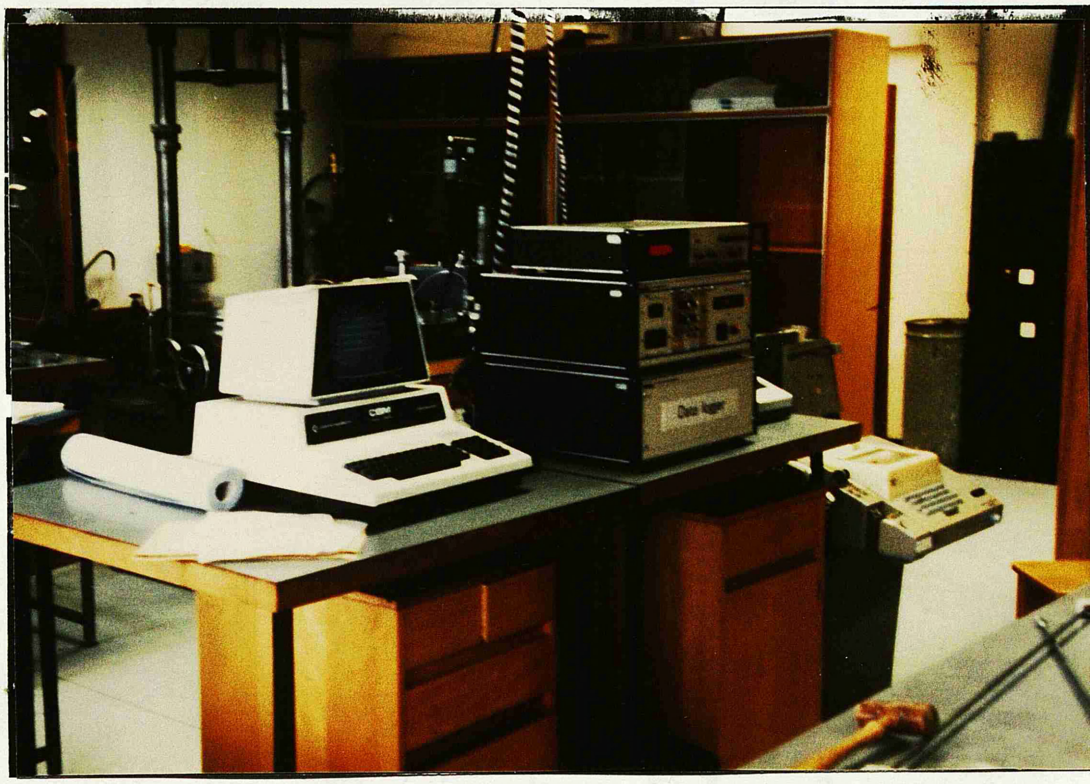


FIG. 4.07. AIR PRESSURE GAUGE, LUBRICATOR, ELECTOR-PNEUMATIC TRANSDUCER, FUNCTION GENERATOR, AIR REGULATOR, AIR FILTER.



F IG. 4.08. DATA LOGGER AND TELEPRINTER.

1. "Reaction" Frame
2. Loading Yoke
3. Piston
4. Supporting Connection
5. Displacement Transducer
6. Load Cell
7. Rigid Anchor Cap
8. Electric Strain Gauges
9. Anchor < Anchor Shaft / Anchor Footing
10. Sand Tank
11. "Supporting" Frame
12. Data Logger and Auxiliary Equipment
13. Teleprinter and Paper Tape Punch Equipment

---- Denotes Power Cables

(All Dimensions in mm)

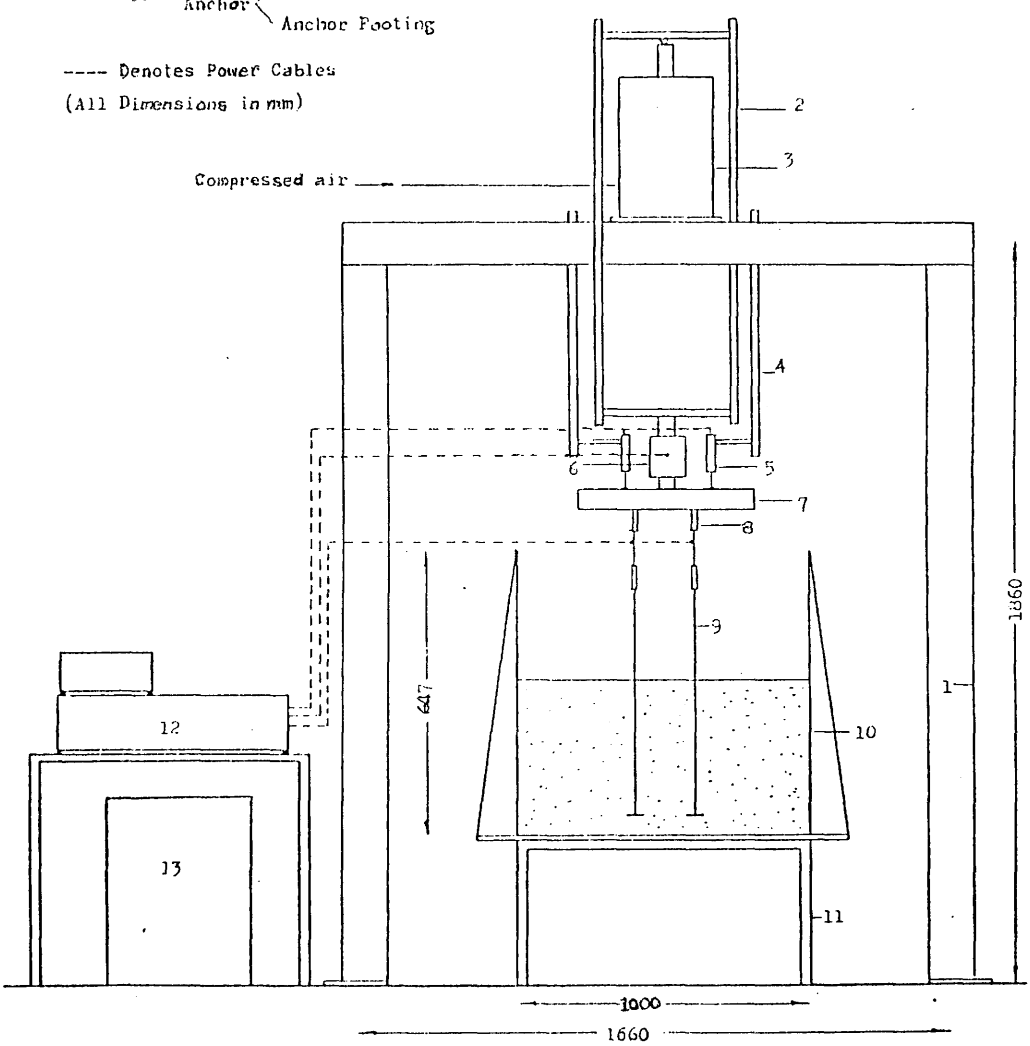


FIG. 4.09. GENERAL LAYOUT OF THE UPLIFT TEST APPARATUS.

4.4.3. POSITIONING OF ANCHOR IN SAND FOR TESTING:

The anchor was positioned on a 100mm "foundation" layer of sand and held in position at the centre of the sand container by brass rod holders. Layers of medium dense sand were then deposited to the required depth D. The final depth of sand deposited was measured to give the actual depth D. The experiment was now ready to proceed to test for ultimate static uplift load once the measuring devices and piston arrangement were in place and operational.

4.4.4. ANCHOR MODEL TESTS:

The ultimate static uplift capacity on single and groups of 25mm diameter model plate anchors arranged in various group configurations, number of anchors in the group, the anchor spacing S and embedded in various depths of anchor embedment D in dry medium dense sand (R.D. = 0.58) were investigated. The group configurations considered were single line arrays of up to 6 anchors with anchor spacings of 50mm to 150mm. The range of depth/diameter ratios considered was from 3 to 15. A summary of the uplift capacity tests conducted is as shown in FIG 4.10.

FIG. 4.10. A SUMMARY OF THE AUTHOR'S UPLIFT CAPACITY TESTS.

GROUP CONFIGURATIONS	ANCHOR SPACINGS RATIO S/B	DEPTH OF EMBEDMENT RATIO S/B	NUMBER OF TESTS
SINGLE	–	6; 9; 12	10
2 x 1	2; 4; 6	3; 6; 9; 12; 15	11
3 x 1	2; 4; 6	6; 9; 12	9
4 x 1	2; 4; 6	6; 9; 12	11
6 x 1	2; 4; 6	6; 9; 12	10
			TOTAL 51

NOTE: B = DIAMETER OF AN ANCHOR.

CHAPTER 5

DISCUSSIONS AND COMPARISON OF RESULTS

CHAPTER 5

DISCUSSIONS AND COMPARISON OF RESUTLS

5.1. INTRODUCTION:

Based on the fifty-one experimental results obtained from the present investigation, the influence of various test parameters on the behaviour of group anchors is discussed. The model test parameters varied were the number of anchors in the group, the group configuration, the anchor spacing S and the depth of anchor embedment D. Spacings of S = 2B, 4B, 6B (B = diameter of an anchor) and D ranging from 3B to 15B were investigated. The group configurations considered were single line arrays of up to 6 anchors. All of the static uplift capacity tests conducted were done under simple, load-controlled loading. The ultimate static uplift capacity is the failure load reached when the anchor displacement increased without further load increase. It is convenient to refer to the efficiency and the displacement ratio of groups, since these values serve as indices of the interference due to group action. Efficiency is defined as the ratio of the ultimate load on a group of n anchors to n times the ultimate load on a single isolated anchor.

The displacement of a group of anchors at failure is compared with that of a single isolated anchor at the same D/B ratio by means of a displacement ratio. This is the ratio of the displacement of the group at failure to the displacement of a single isolated anchor at failure.

The present results and previous theories are compared and discussed for the line groups of anchors. The previous theories considered are Meyerhof and Adam's theory (1968) and Yilmaz's first analysis (1971). Yilmaz's second analysis (1971) for groups of circular anchors was omitted because in this investigation the many prerequisite physical parameters were not measured and calibrated.

The results derived from the proposed analysis are compared with the present experimental results. To further investigate the adequacy of the proposed analysis, comparisons between the observed results from other previous experiments and those derived from the proposed analysis are carried out. The previous experimental results presented are from Larnach and McMullan's (1975) and Yilmaz's (1971) investigations.

5.2. INFLUENCE OF DEPTH EMBEDMENT RATIO (D/B).

5.2.1. GENERAL:

The ultimate static uplift factors ($P/\gamma D$) to D/B ratios relationships for single anchor and line groups at anchor spacings ranging from 2B to 6B are shown in FIG. 5.00, 5.01 and 5.02. (where P = pressure exerted on each anchor in a group; γ = unit weight of dry soil; D = depth of embedment; and B = diameter of each anchor footing. It can be observed that for all sizes of groups, the ($P/\gamma D$) values will increase as D/B ratios increase.

5.2.2. LOAD DISTRIBUTION:

The phenomenon of unequal load distribution of each anchor installed at close spacings in groups was observed by Yilmaz (1971).

The distribution of loading amongst the individual anchors within a group is shown for typical line groups in FIG. 5.03. The curves relate to individual loads achieved at the instant when the group load is at its peak. A close examination of these test results reveals a difference in the behaviour of the "inside" anchor as the ultimate load is reached. Just before failure condition, the two "end" anchors of the line groups carry more load than the "inside" ones. The general trend is that the proportion of the total ultimate group load carried by an anchor in the group increases as its distance increases from the point of load application. The non-uniform load distribution at failure can be partially explained by the occurrence of greater interaction of failure zones amongst the "inner" anchors than the "end" anchors. Hence the load carried by the "end" anchors is proportionally larger.

5.2.3. GROUP EFFICIENCY.

FIG. 5.04. shows the relationship of ultimate group efficiency

FIG. 5.00. THE AUTHOR'S TESTS RESULTS OF ANCHORS IN LINEAR GROUPS.

-GRAPHS OF P/γD vs D/B (AT S/B = 2)

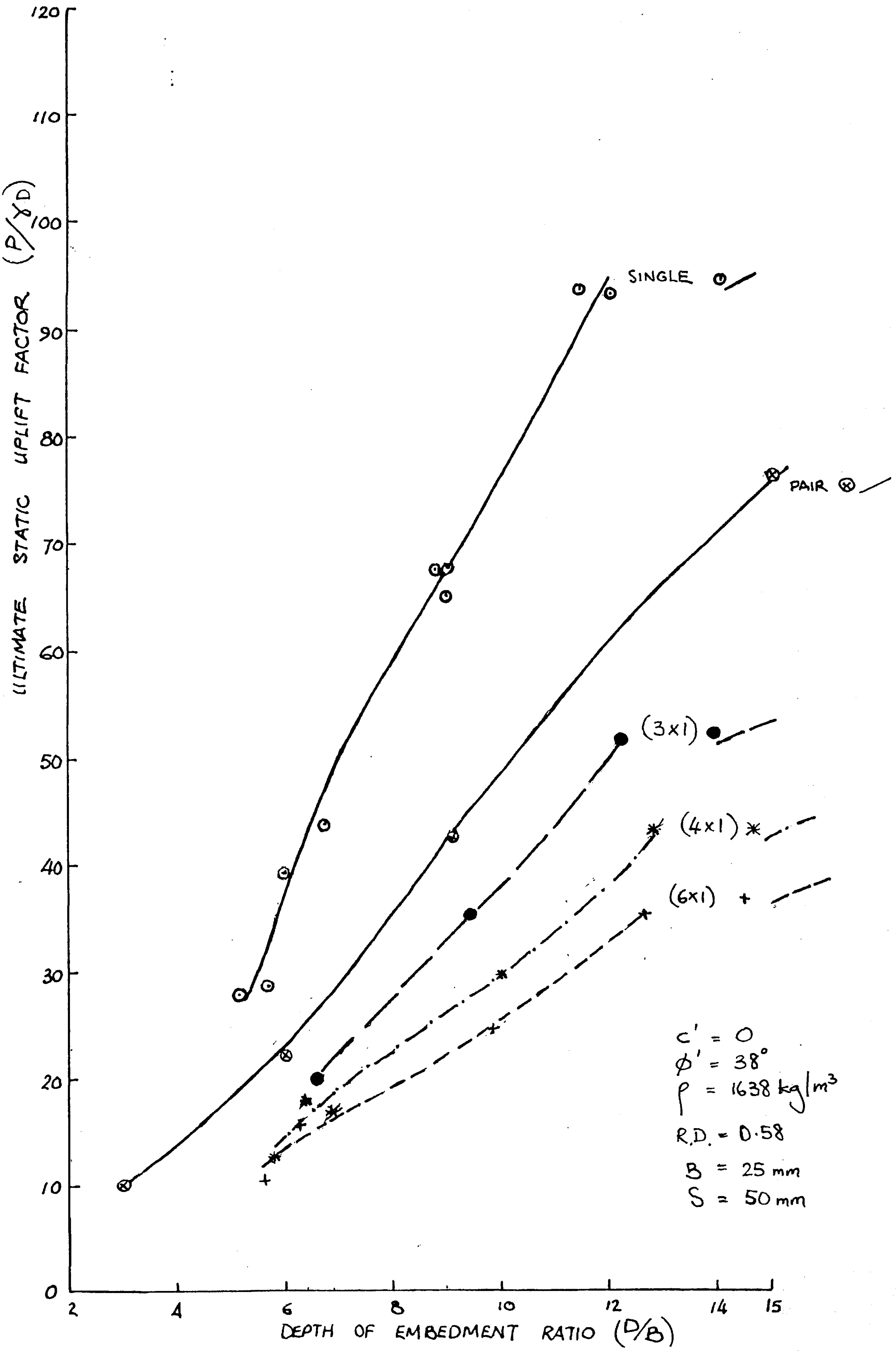


FIG. 5.01. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.
-GRAPHS OF P/γD vs D/B (AT S/B = 4)

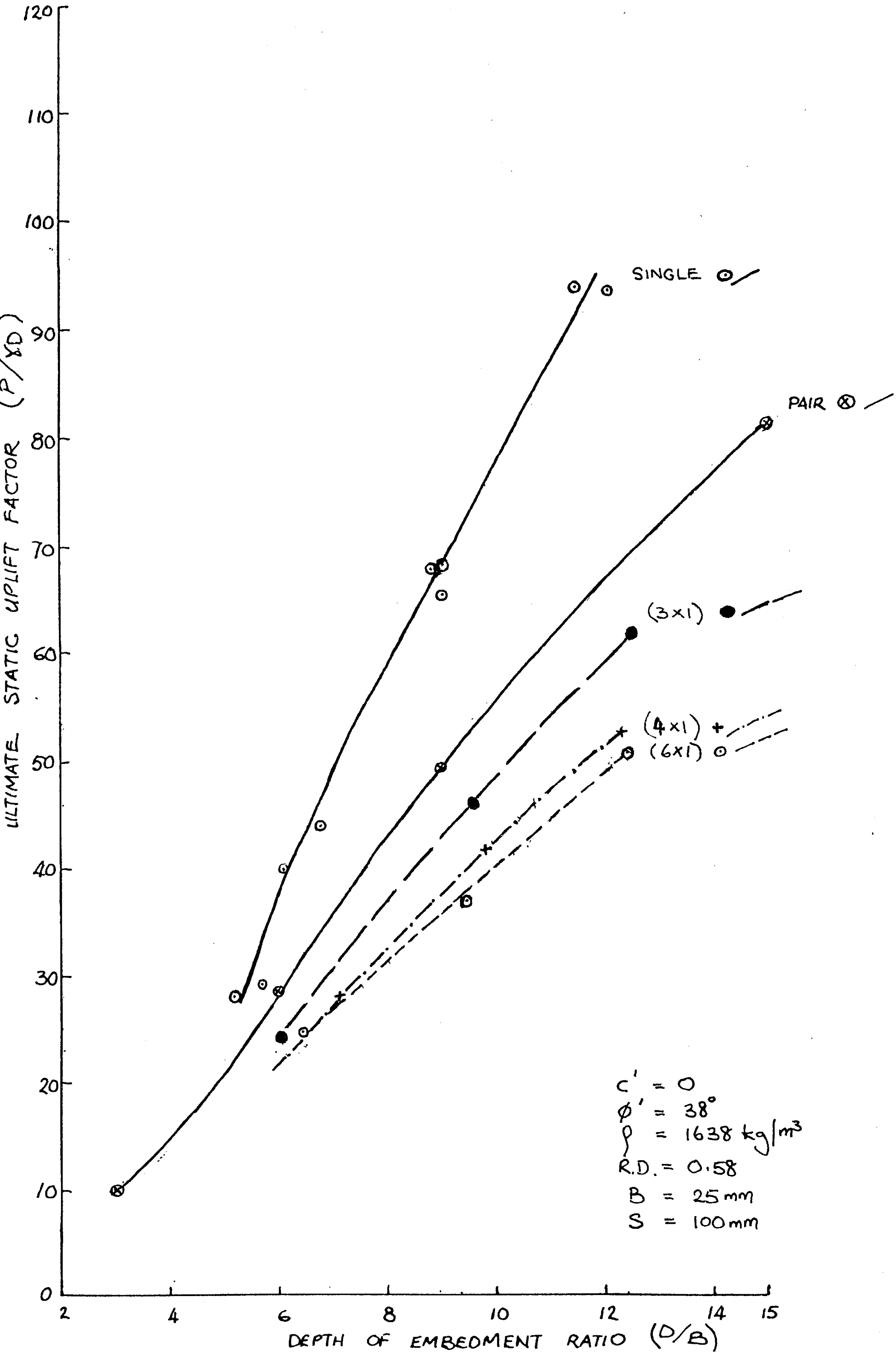


FIG. 5.02. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.

-GRAPHS OF P/γD vs D/B (AT S/B = 6)

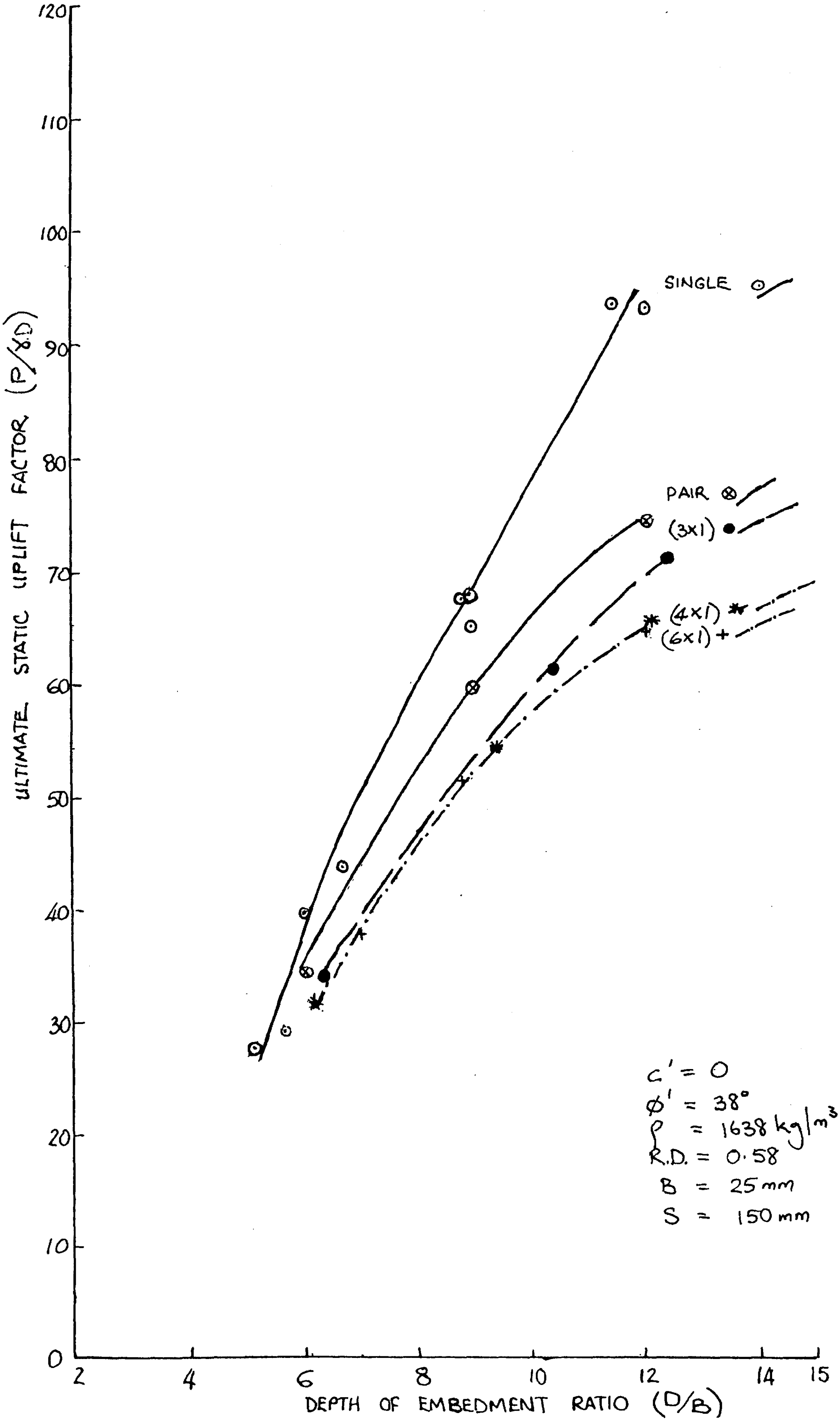
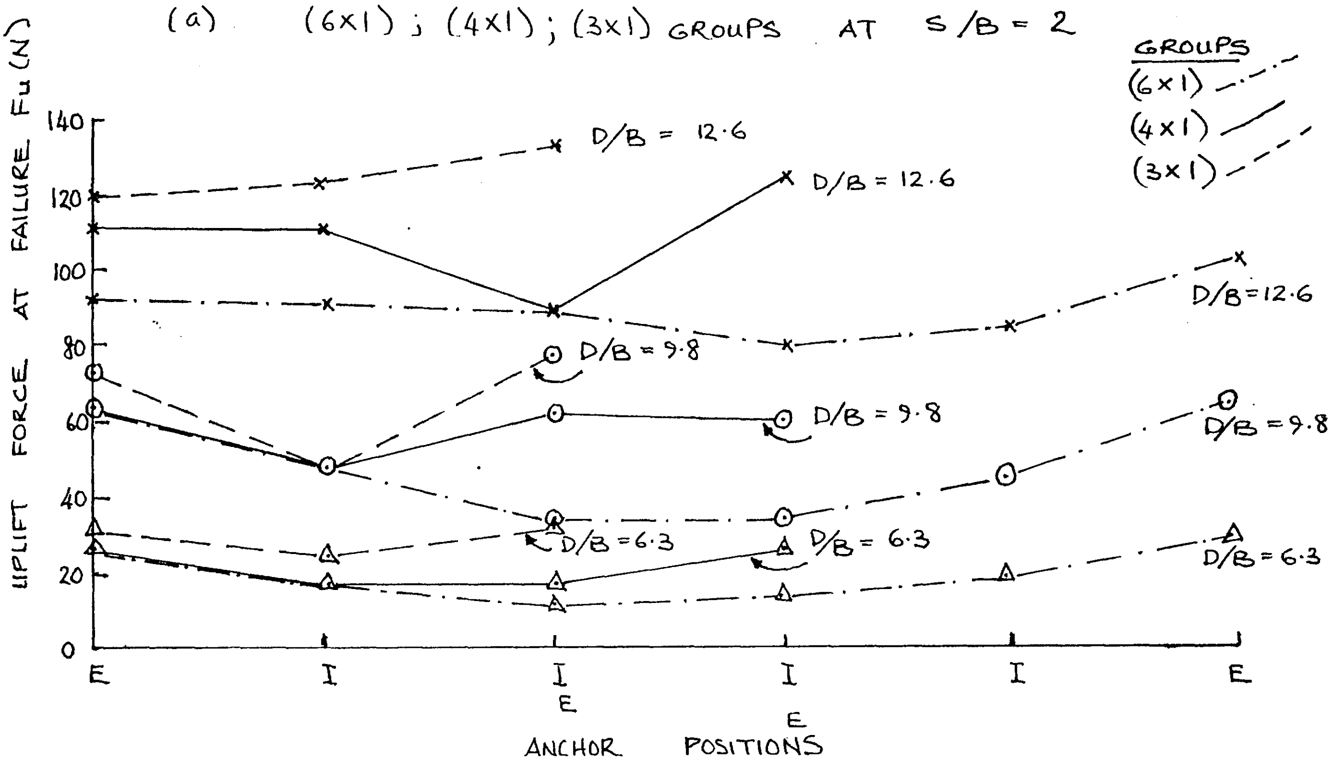


FIG. 5.03. THE AUTHOR'S TEST RESULTS.

-GRAPHS SHOWING DISTRIBUTION OF UPLIFT FORCE AT FAILURE OF EACH ANCHOR WITHIN A LINEAR GROUP.

(a) (6x1) ; (4x1) ; (3x1) GROUPS AT S/B = 2



(b) (6x1) ; (4x1) ; (3x1) GROUPS AT S/B = 6

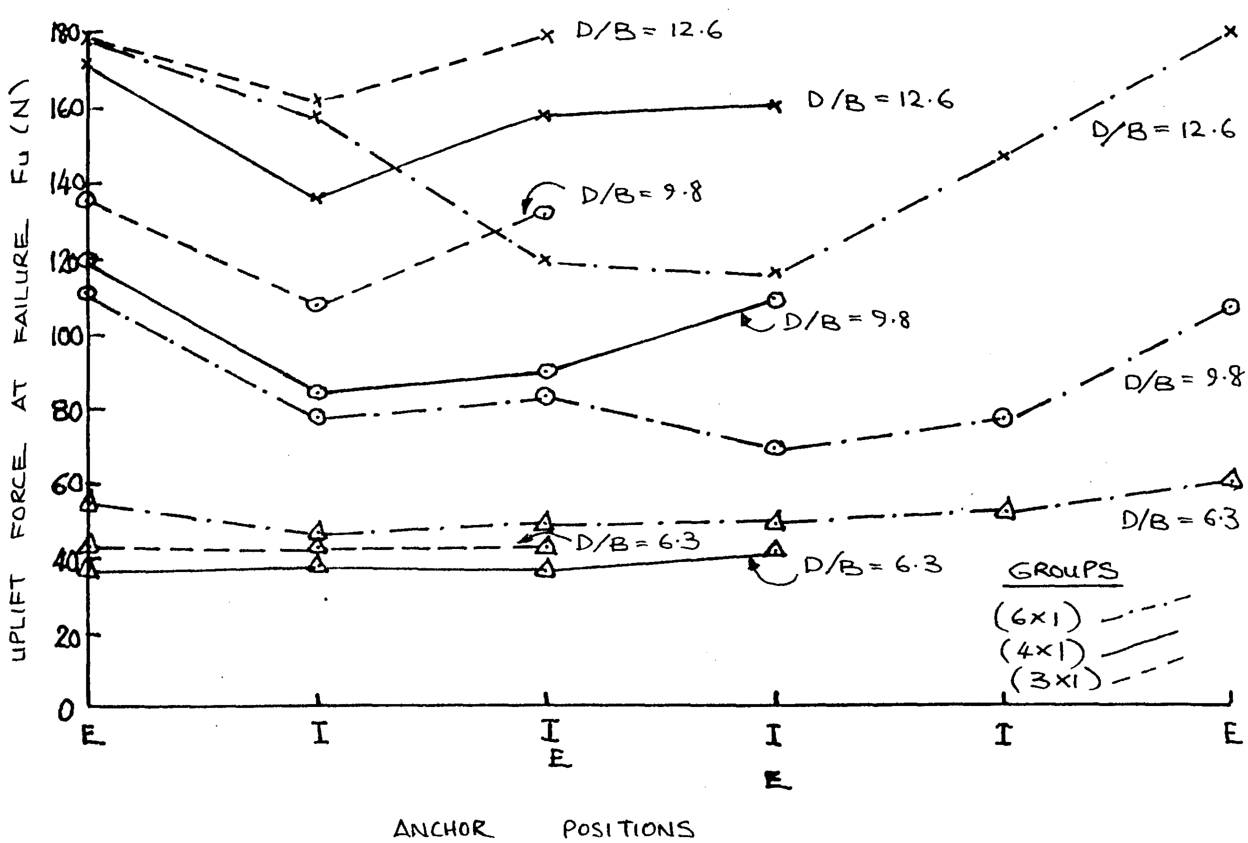
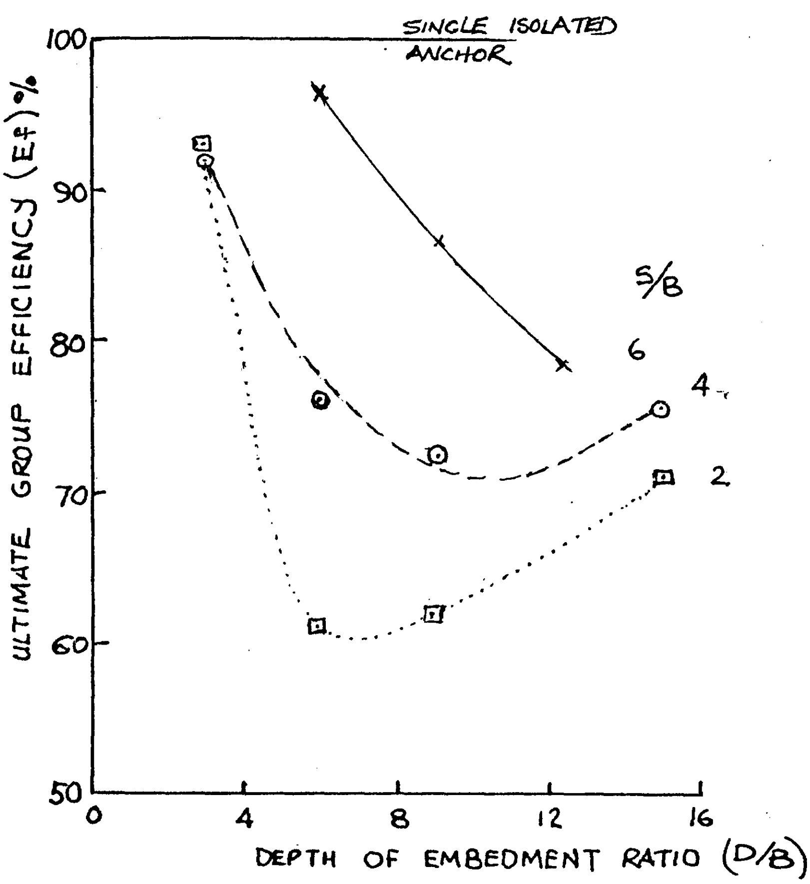


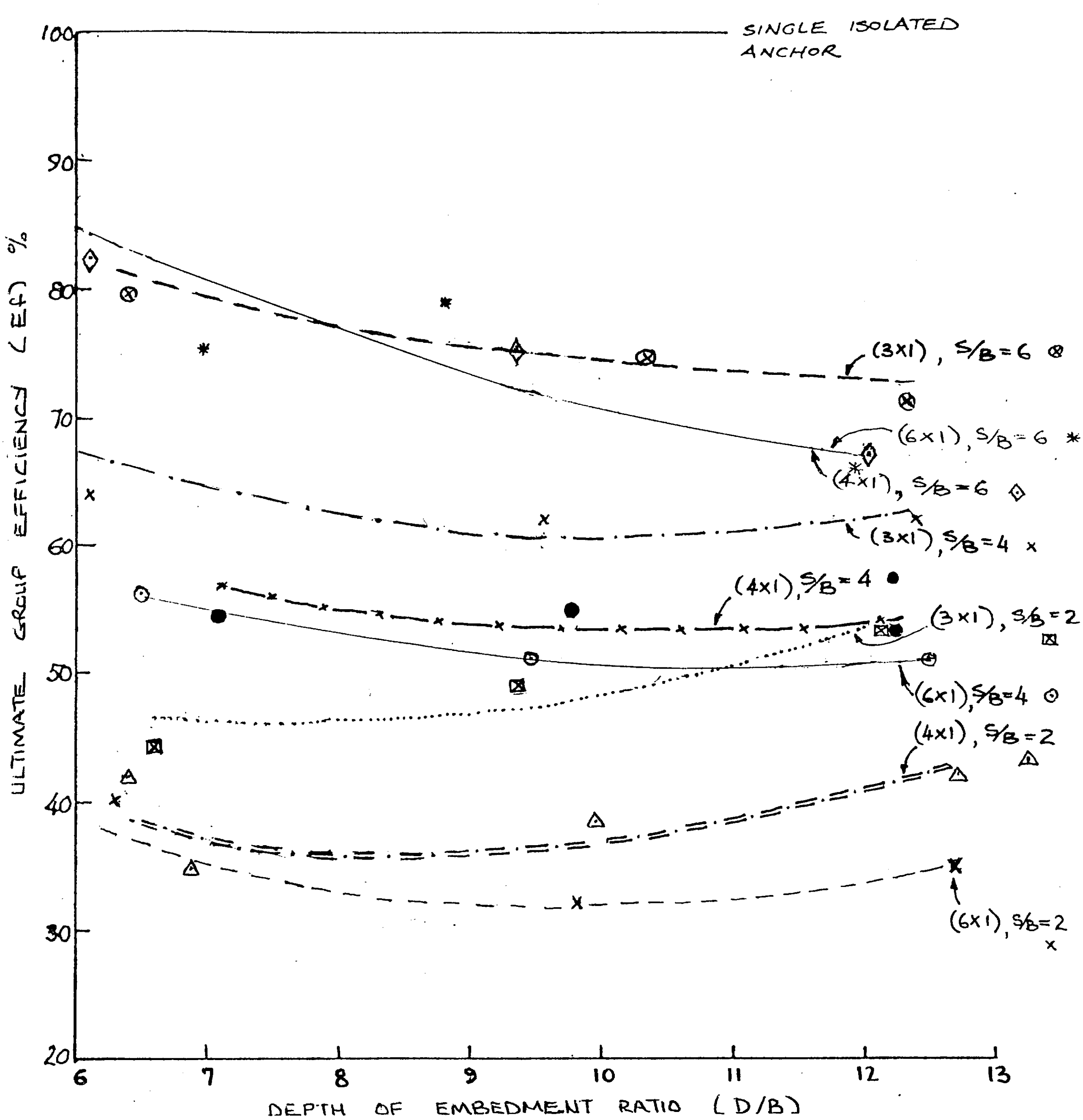
FIG. 5.04. THE AUTHOR'S TEST RESULTS OF ANCHORS IN (2X1) GROUP. -GRAPHS OF Ef vs D/B.



to D/B ratios for a typical (2x1) group at various spacings. The ultimate group efficiency (Ef) decreases to a minimum with increasing D/B ratios, and then increases to as large a value as that found with small values of D/B ratios. There appears to be a distinct critical D/B ratio where Ef is a minimum value. For example, from the author's test results for a (2x1) anchor group at S/B = 4, the (Ef) value decreases to a minimum of about 70% with increasing D/B ratios, and then increases to as large a value as about 80%. This critical depth ratio has been reported in previous research work by McMullan and Larnach (1975) (see section 2.4.) to be the transition depth separating "deep" and "shallow" anchor failures. To explain this phenomenon of varying efficiencies, it is suggested that the interpenetration of failure zone factor and the single anchor factor occur concurrently at any instance to give various degrees of combined influence. The interpenetration of failure zone factor is defined as the factor which decreases the efficiencies and results from the multi-interpenetration of failure zones of individual anchors within a group. The single anchor factor is defined as the factor which increases the group efficiency and is a result of the tendency of the anchors in a group to act as a single large anchor. As the D/B ratios increase (from very shallow) to the critical depth ratio, the degree of influence of the single anchor factor is increasingly dominated by the interpenetration of failure zone factor. As the D/B ratio increases beyond the critical value, the degree of influence of the single anchor factor increasingly dominates.

FIG. 5.05. shows the relationship of ultimate group efficiency (Ef) and D/B ratio for (3x1), (4x1), (6x1) groups at spacings ranging from 2B to 6B. It was found that for a given anchor spacing, as the D/B ratio increases from about 6 the ultimate efficiency decreases to a minimum, and then increases with increasing D/B ratio. For a given anchor group size, at anchor spacings of 2B, 4B and 6B, the minimum ultimate group efficiency corresponded to critical D/B ratios of about 8, 10 and 12 respectively. Hence at anchor spacing of 2B, the author's definition of "shallow" anchors in groups with D/B $\leqslant 8$ and "deep" anchor groups have D/B > 8. At anchor spacings of 4B and 6B, "shallow" and "deep" anchor groups can be

FIG. 5.05. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.
-GRAPHS OF Ef vs D/B.



similarly classified. It must be remembered that for very shallow depths of embedment, although very large efficiencies for the anchor groups are obtained, the actual load resisted by the group is relatively small as can be seen in FIG. 5.03. (a).

5.2.4. DISPLACEMENT RATIO:

The author defined the displacement ratio of a group as the ratio between the displacement of a group of anchors and that of a comparable single anchor when both carry the same fraction of their failure load. From the author's experiments, the relationship of ultimate displacement and D/B ratios for a single isolated anchor as shown in FIG. 5.06. is used as a base reference for the calculations of ultimate displacement ratios of anchors in groups. FIG. 5.07. which presents the displacement ratio - D/B ratio relationships for line groups shows that displacement ratios will decrease as D/B increases. The displacement ratios will increase to unity as D/B ratios decrease.

5.3. INFLUENCE OF ANCHOR SPACING RATIO (S/B):

5.3.1. GENERAL:

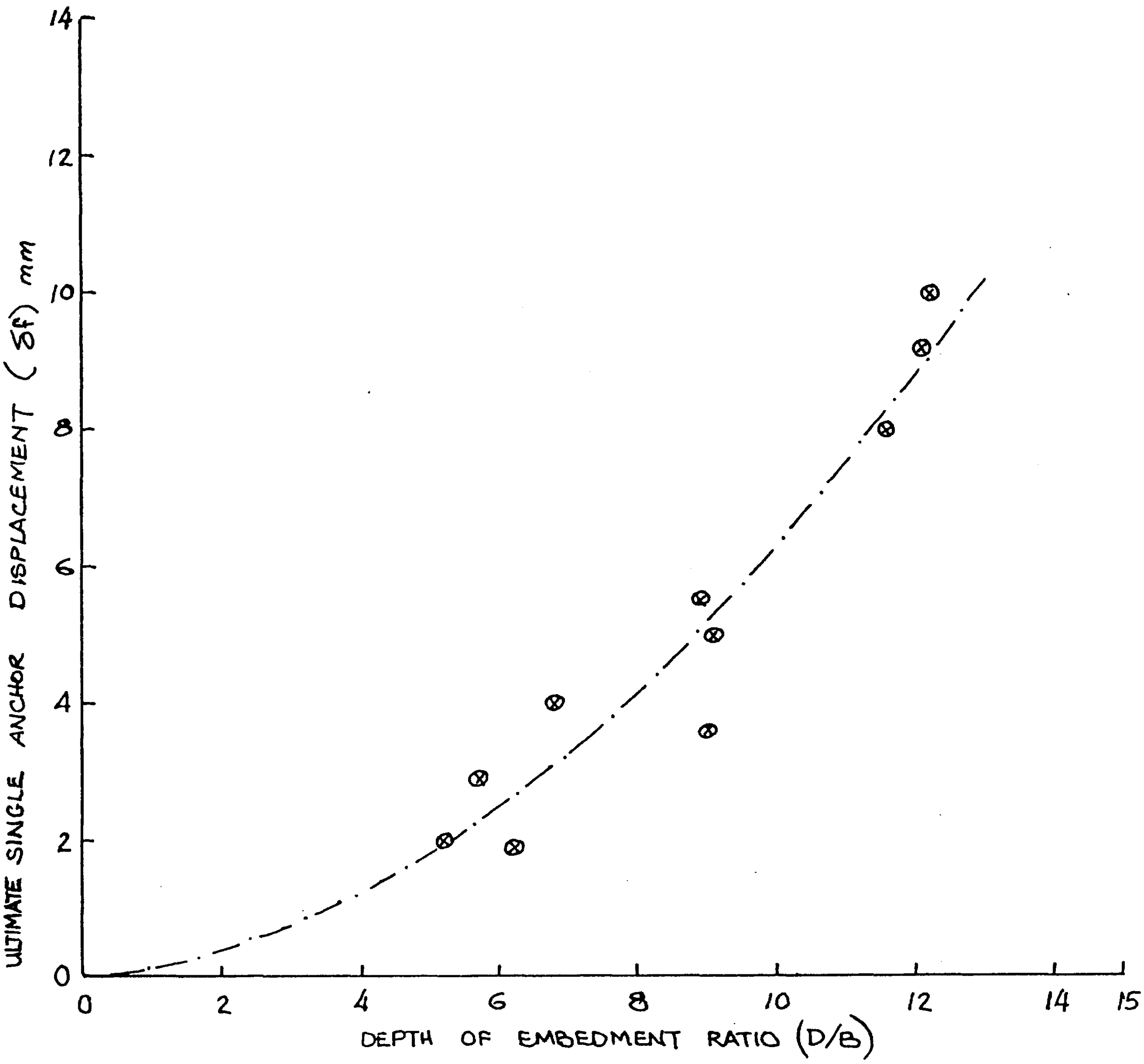
In FIG. 5.08. (A).(i), 5.08. (A).(ii) and 5.08. (A).(iii), it can be seen that for a given sand density and depth of embedment the uplift resistance of a group is dependent upon the spacing between the anchors in the group. It can be observed that the uplift factor ($P/\gamma D$) will decrease as the anchor spacing decreases, for a group at a given depth ratio (D/B). This is probably because the interaction of the failure zones of the anchors in the group is more intense at close anchor spacings, than at large anchor spacings.

5.3.2. LOAD DISTRIBUTION:

The unequal load distribution at failure become less pronounced as the anchor spacings increase in the shallower anchor groups (e.g. D/B = 6) (see FIG. 5.03. (b)). As the anchor spacing in a group approaches "isolation" spacing, each anchor within the group will act as an isolated anchor with no interaction. So at "isolation" spacing the ultimate uplift capacity of the group will equal the summation of the ultimate capacity of each anchor in the group.

FIG. 5.06. THE AUTHOR'S TEST RESULTS OF SINGLE ISOLATED ANCHORS AT FAILURE LOADS.

-GRAPH OF δf vs D/B.



F IG. 5.07. THE AUTHOR'S TEST RESULTS OF LINEAR GROUPS.

-GRAPHS OF Δ_f vs D/B.

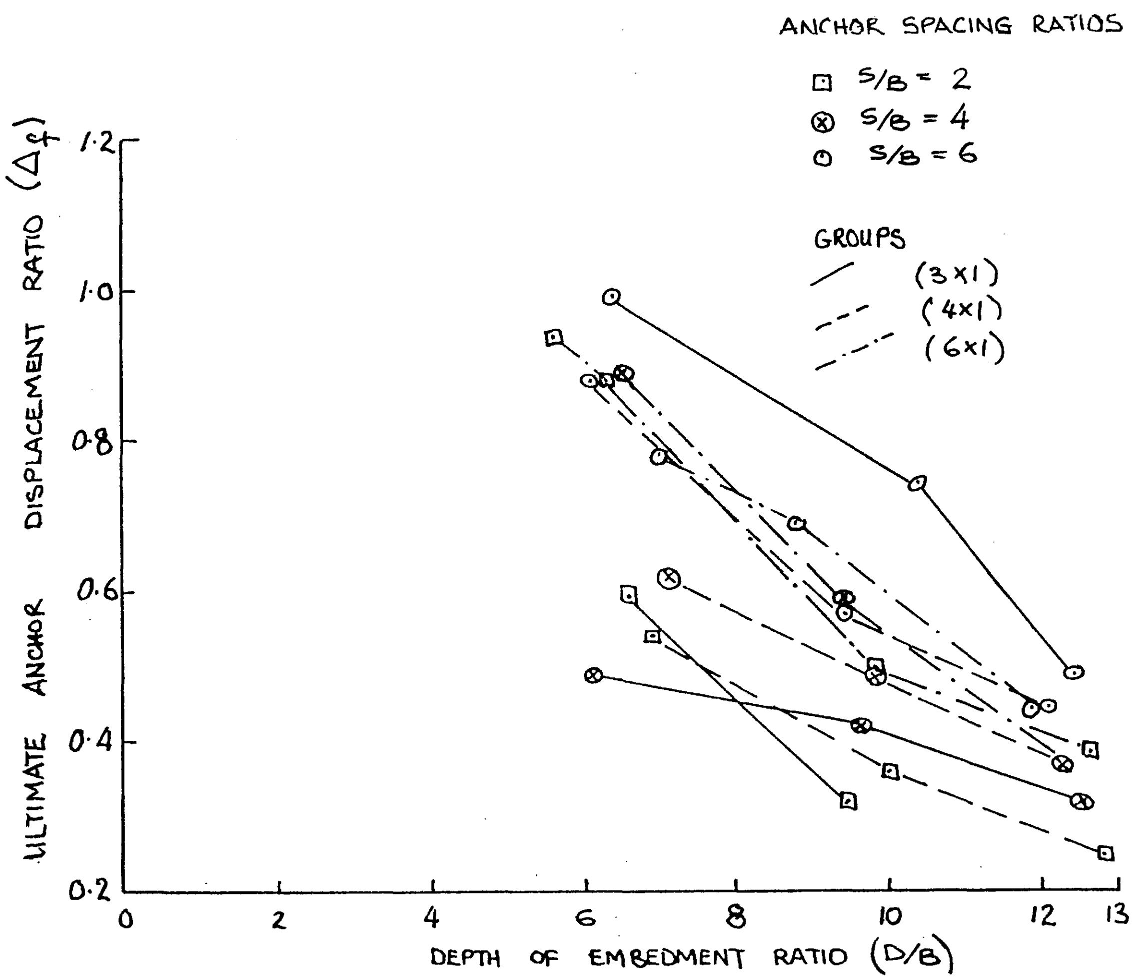
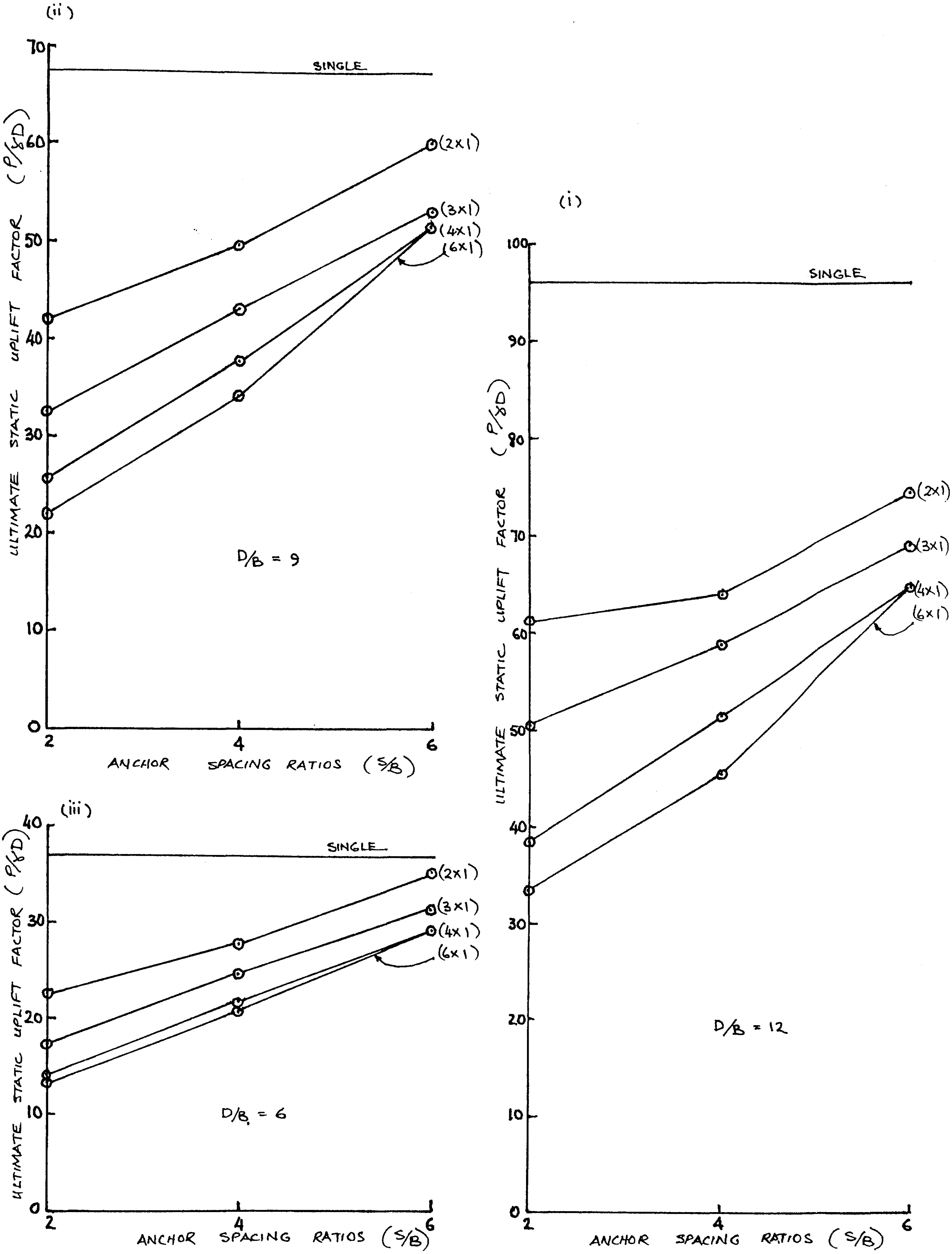


FIG. 5.08.(A). THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.
-GRAPHS OF P/γD vs S/B (AT D/B= 12 , 9 , 6)



5.3.3. GROUP EFFICIENCY:

The efficiency variation with spacing for anchors in line groups is plotted in FIG. 5.08. and shows that at a given density and depth of embedment, large efficiency values can be obtained by increasing the spacing between anchors in the group. When the anchor spacings between each anchor within a group is equal to the "isolation" spacing (Si), the group efficiency is 100%. It can be observed from FIG. 5.08. that the "isolation" spacing (Si) is greater than 6B. Although the author's test results did not allow direct measurement of the "isolation" spacing, nevertheless, by extrapolating values of S/B, the "isolation" spacing for a (3x1) group (for example) is about 7.5B (for D/B = 6).

5.3.4. DISPLACEMENT RATIO:

The displacement ratios for line groups of anchors is shown in FIG. 5.09. where it can be seen that the displacement at failure will increase as the anchor spacing increases. The displacement ratio at failure is unity when the spacings between each anchor in a group is at the spacings where each anchor within the group can act as a single anchor. This is because at "isolation" anchor spacing there is probably no interference of failure zones within the group and the average load on each anchor in the group is equal to that on a single isolated anchor at the same D/B ratio. It can be seen from FIG. 5.09. that the isolation spacing is greater than 6B.

5.4. INFLUENCE OF GROUP SIZE:

It has been shown in FIG. 5.00 - 5.02. that the loads resisted by anchor groups varies with the group size (i.e. number of anchors in the group). By increasing the number of anchors in the group the ultimate uplift load capacity of the anchor group will also increase, provided the anchor spacing remain unchanged.

The influence of group size on the distribution of load within the group is not apparent. The non-uniform load distribution at failure with the "inside" anchor carrying the least load and "end" anchors the highest load, is unaffected by the group size.

FIG. 5.08. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.

-GRAPHS OF Ef VS S/B.

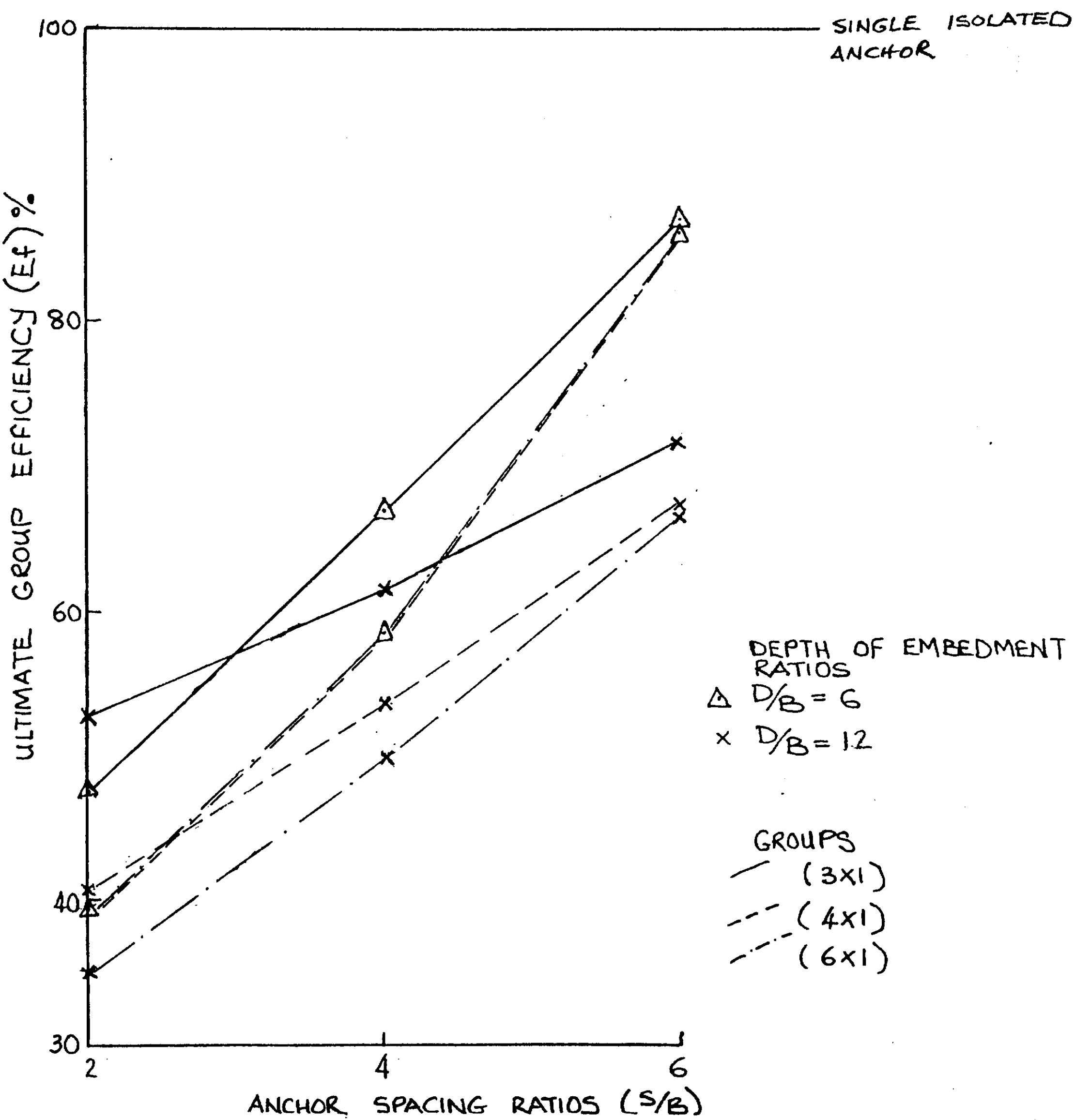
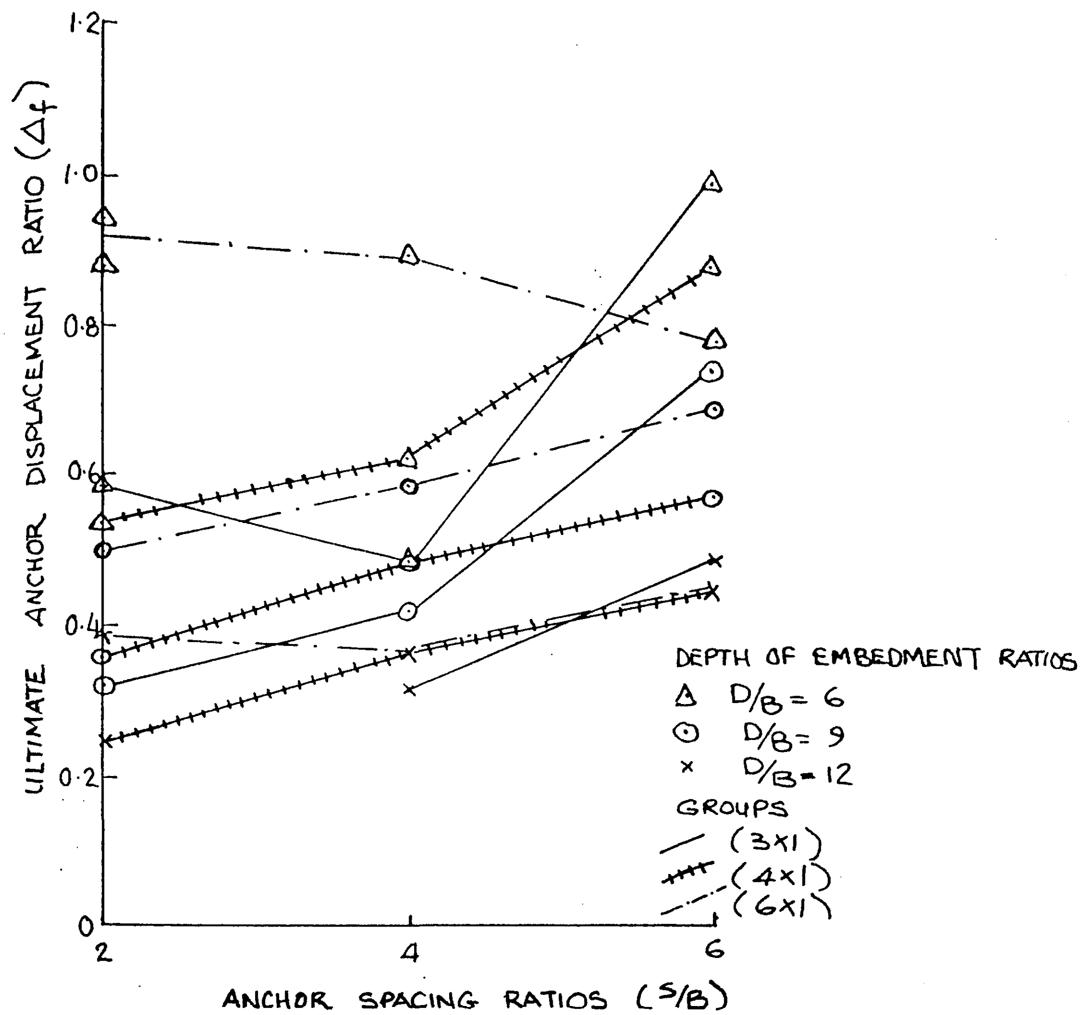


FIG. 5.09. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.
-GRAPHS OF Δ_f vs S/B.



It is observed that at close spacings the ultimate group efficiency of the anchor groups will decrease as the number of anchors in the group increase (see FIG. 5.10.).

At various anchor spacings, the ultimate displacement ratio will increase as the number of anchors increase. FIG. 5.11. shows the relationships of ultimate displacement ratios and group size for line groups at various D/B ratio and anchor spacing. It is observed that the ultimate displacement ratios decrease as the D/B ratios increase. For large groups with shallower depth of embedment, the ultimate displacement ratios will approach unity.

5.5. COMPARISON OF EXPERIMENTAL AND THEORETICAL RESULTS:

5.5.1. PRESENT RESULTS AND PREVIOUS THEORIES:

In this section, the author's experimental (Ef) results are compared with the predicted (Ef) results derived from Meyerhof and Adam's theory, and Yilmaz's first analysis. As mentioned earlier in Chapter 2, Meyerhof and Adam stated that the ultimate uplift load of a footing group was the smaller value of either the sum of the uplift loads of the individual footings or the uplift loads of an equivalent pier foundation consisting of the footings and enclosed soil mass. Both values are estimates, the former method tends to overestimate the ultimate uplift capacity whilst the latter method tends to underestimate the failure load. The predicted failure loads adopted are the smaller values calculated by the two methods. In this investigation, the smaller values derived from Meyerhof and Adam's methods are presented.

In FIG. 5.12. the predicted (Ef) results derived from Meyerhof and Adam's theory are compared with the author's observed (Ef) results for line groups of (3x1), (4x1) and (6x1). The most common feature in the figure is that the predicted efficiencies derived from Meyerhof and Adam's theory are overestimated. It can be observed that the author's (Ef) results will decrease as anchor spacing ratios increase. The trend of this observation is reflected in Meyerhof and Adam's theory. FIG. 5.13. shows the relationship between efficiencies and the number of anchors in line groups at

F IG. 5.10. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.
-GRAPHS OF Ef vs GROUP SIZE.

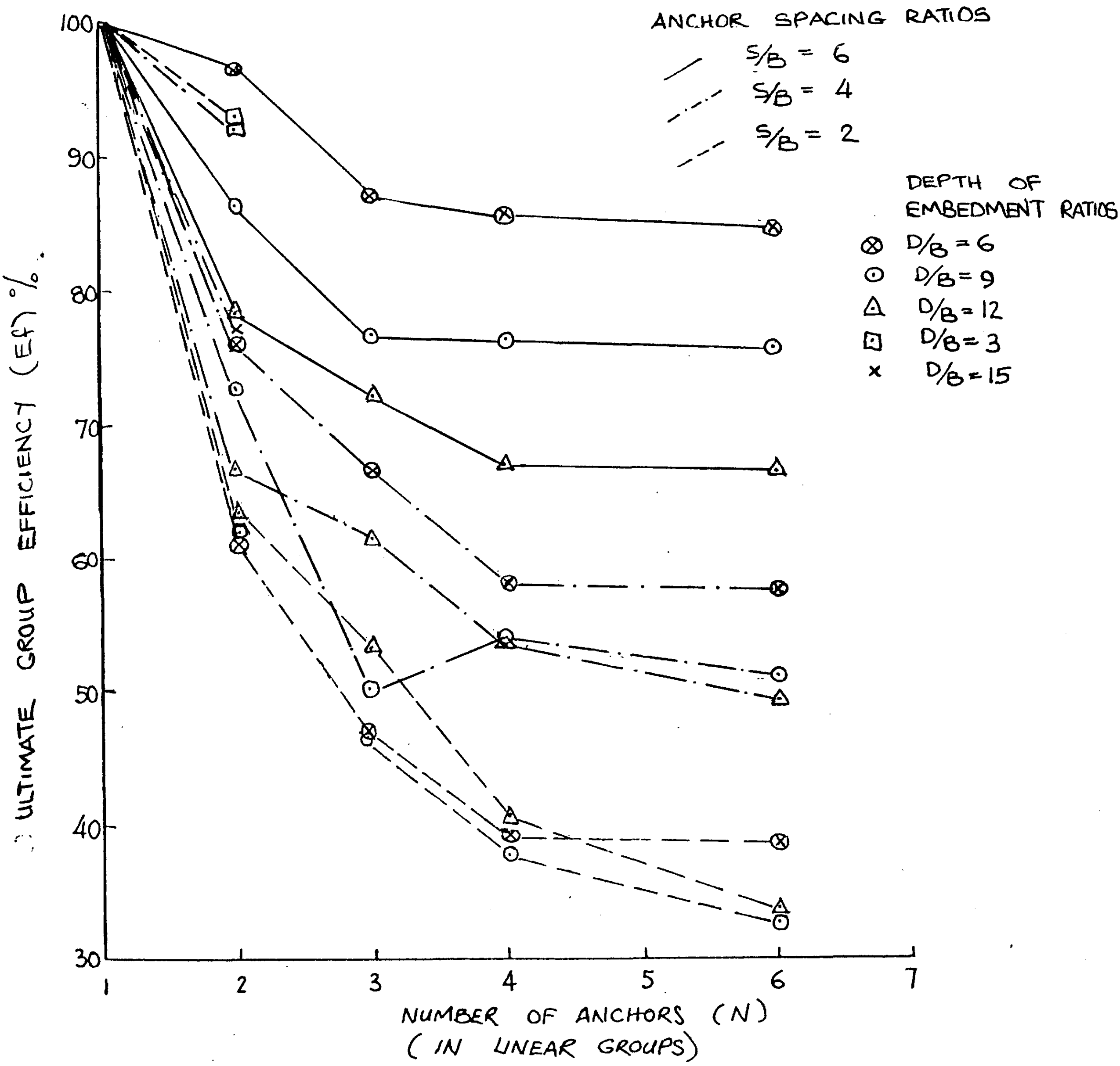
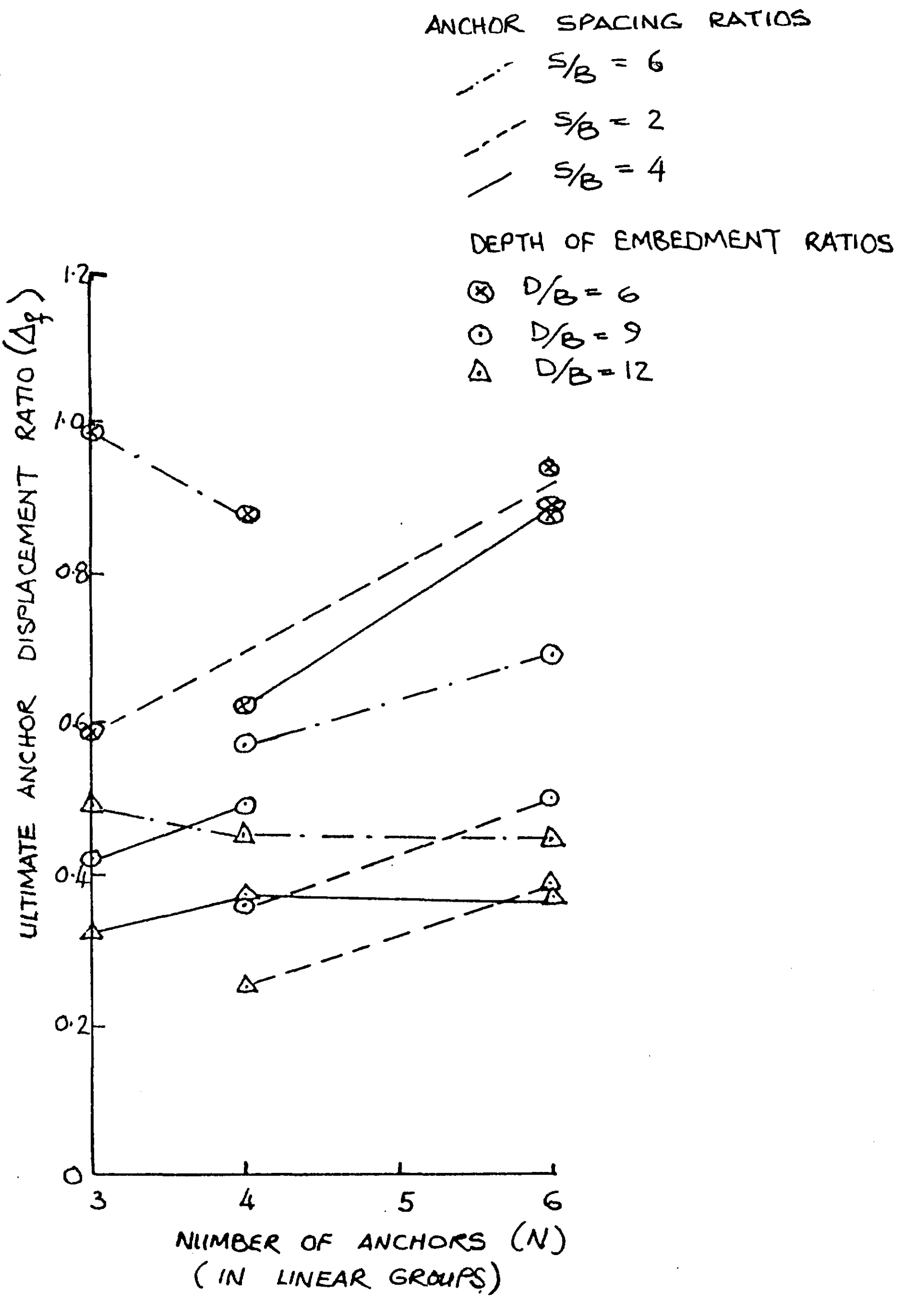


FIG. 5.11. THE AUTHOR'S TEST RESULTS OF ANCHORS IN LINEAR GROUPS.
-GRAPHS OF Δ_f vs GROUP SIZE.



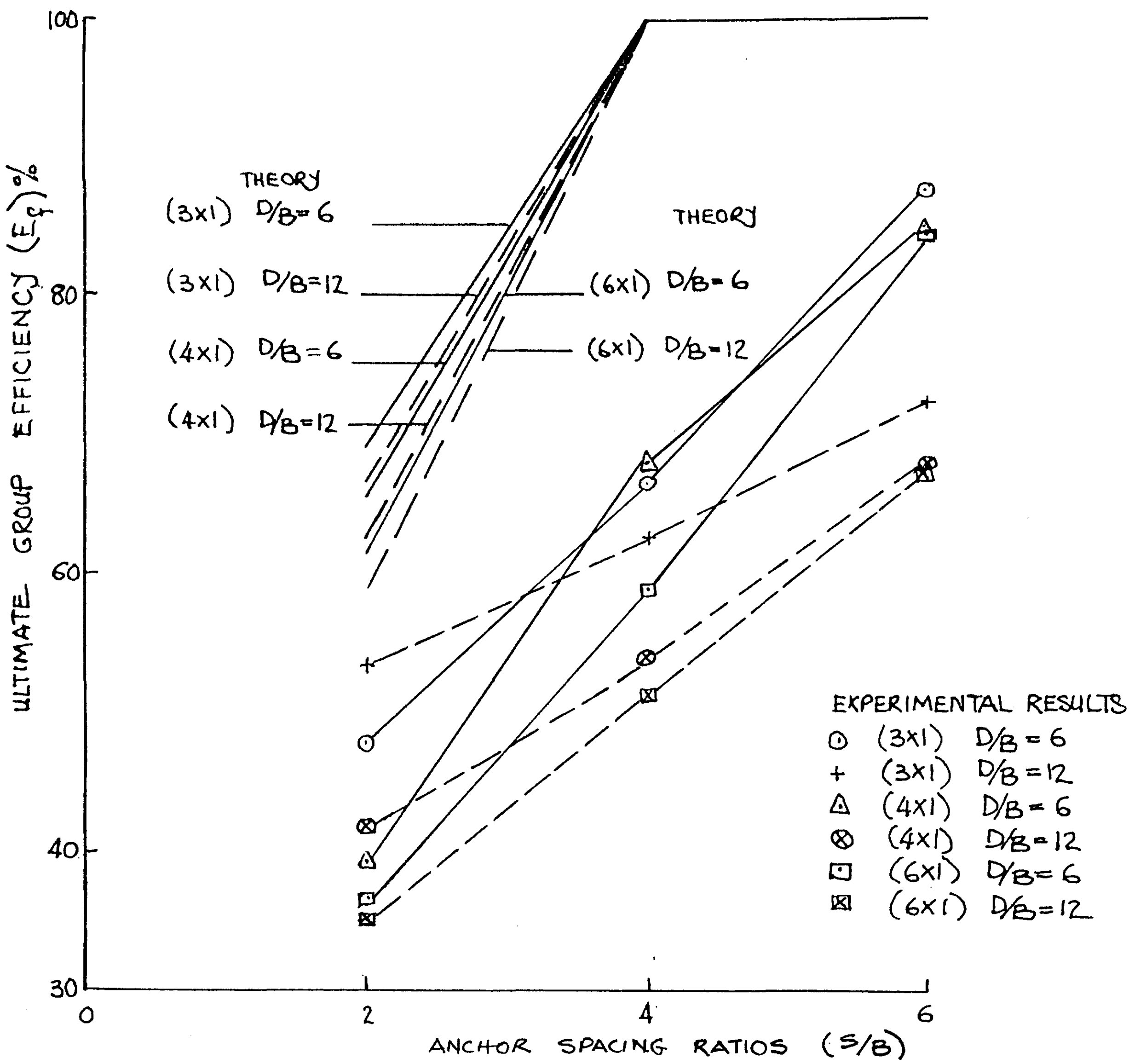


FIG. 5.12. THE COMPARISON OF THE AUTHOR'S TEST RESULTS WITH MEYERHOF AND ADAM'S THEORY.

-GRAPHS OF Ef vs S/B.

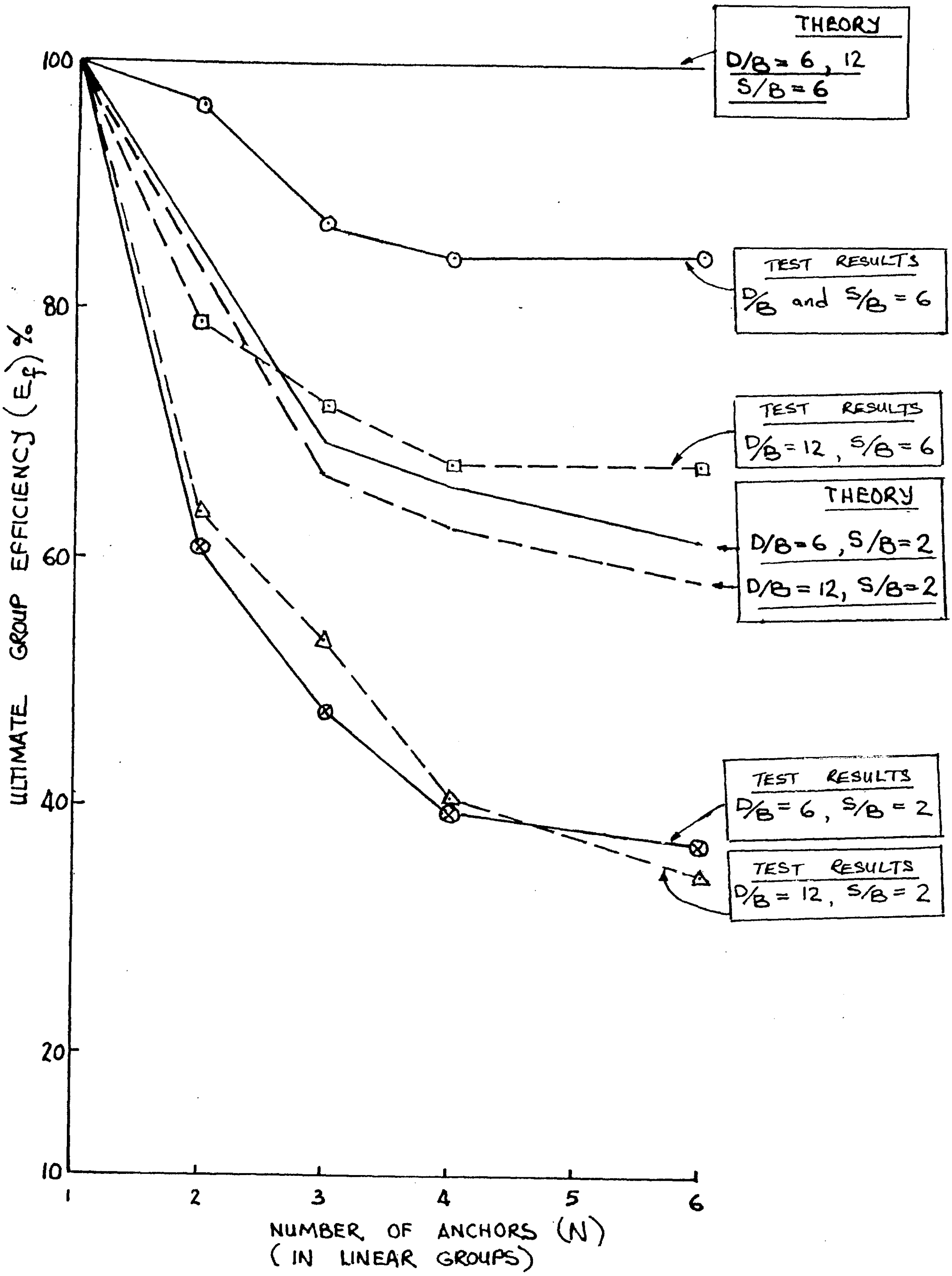


FIG. 5.13. THE COMPARISON OF THE AUTHOR'S TEST RESULTS WITH MEYERHOF AND ADAMS THEORY.
-GRAPHS OF Ef vs GROUP SIZE.

various spacings. It can be observed that while the experimental trend of decreasing efficiencies with increasing group size is reflected in Meyerhof and Adam's theoretical predictions (for $S/B < 4$), the agreement between Meyerhof and Adam's predicted and the author's observed efficiencies is not entirely satisfactory.

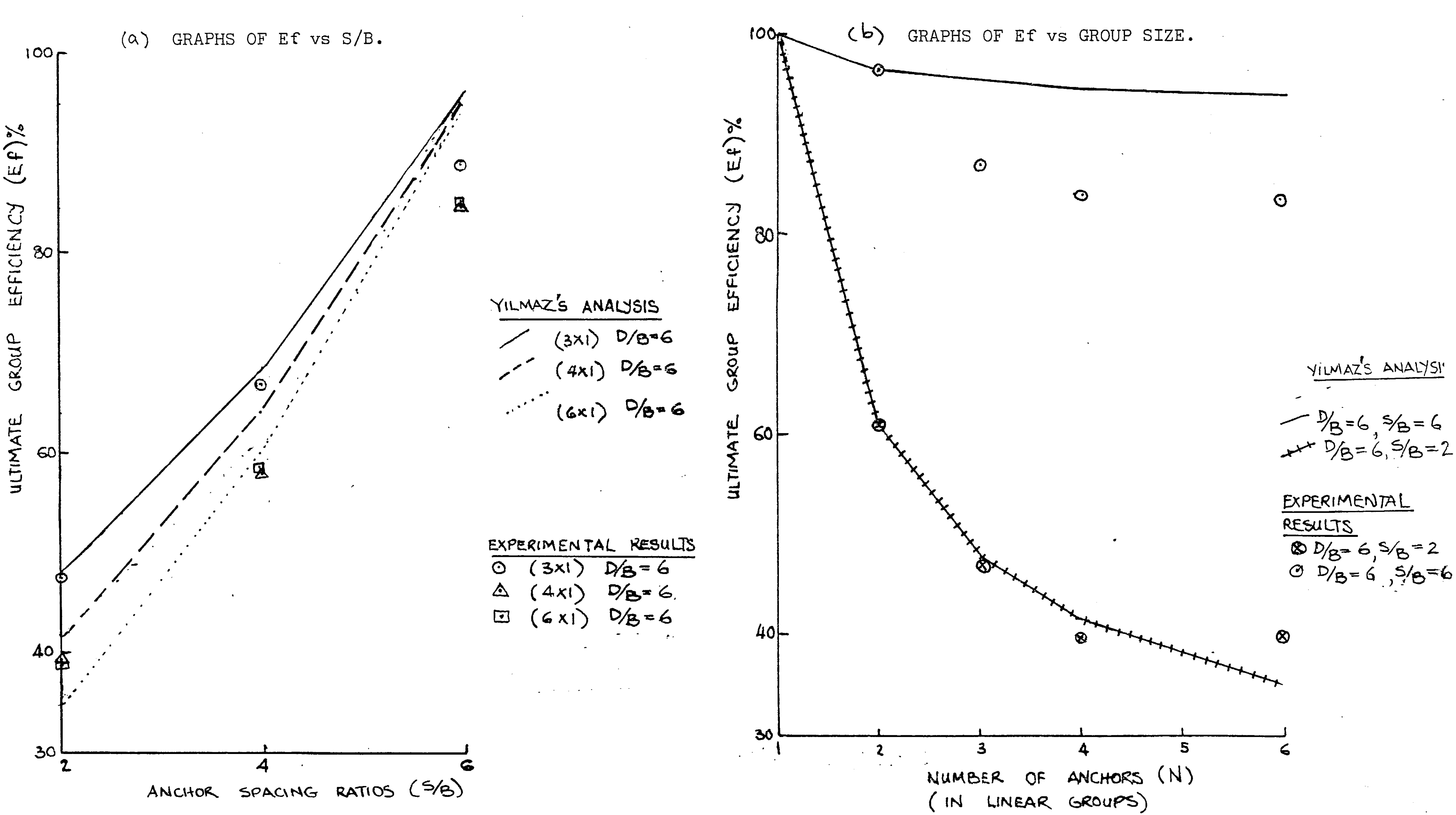
The difference between Meyerhof and Adam's predicted (Ef) results and the author's observed (Ef) results are discussed. In this case, Meyerhof and Adam's predicted (Ef) results ranged from 12.7% greater to 49.8% greater than the author's observed (Ef) results. The difference may probably be due to the reasons discussed in section 2.3. (pp.16.).

Yilmaz's (1971) second analysis as described in Chapter 2 was omitted because in this investigation the many prerequisite physical parameters were not measured and calibrated. In FIG. 5.14. (a) the predicted (Ef) results derived from Yilmaz's first analysis are compared with the author's observed (Ef) results for line groups of (3x1), (4x1) and (6x1). The classification of "shallow" and "deep" anchors in Yilmaz's investigation was based on the previous work done by Meyerhof and Adam (1968). The predicted efficiencies reflect the observed trend of increasing efficiencies as the anchor spacing increases. The difference between the predicted (Ef) results from Yilmaz's first analysis and the author's observed (Ef) results are discussed. In this case, Yilmaz's predicted (Ef) results ranged from 6.5% greater to 7.3% less than the author's observed (Ef) results. In FIG. 5.14. (b) the ultimate group efficiencies to number of anchors in line groups relationships are presented. It is observed that efficiencies will decrease as the number of anchors in a line group increases. The trend of this observation is reflected in Yilmaz's first analysis. The limitations of Yilmaz's analysis are discussed in section 2.3.

5.5.2. PRESENT RESULTS AND PROPOSED ANALYSIS:

In this section, the author's experimental (Ef) results are compared with the predicted (Ef) results derived from the proposed analysis. As described in Chapter 3, the author's experimental results of ultimate uplift loads for a single and a group of two circular plate anchors are used in the proposed analysis. The

FIG. 5.14. THE COMPARISON OF THE AUTHOR'S TEST RESULTS WITH YILMAZ FIRST ANALYSIS.



proposed analysis also incorporate the use of the simple interaction phenomenon to predict the ultimate uplift capacity for line groups of anchors.

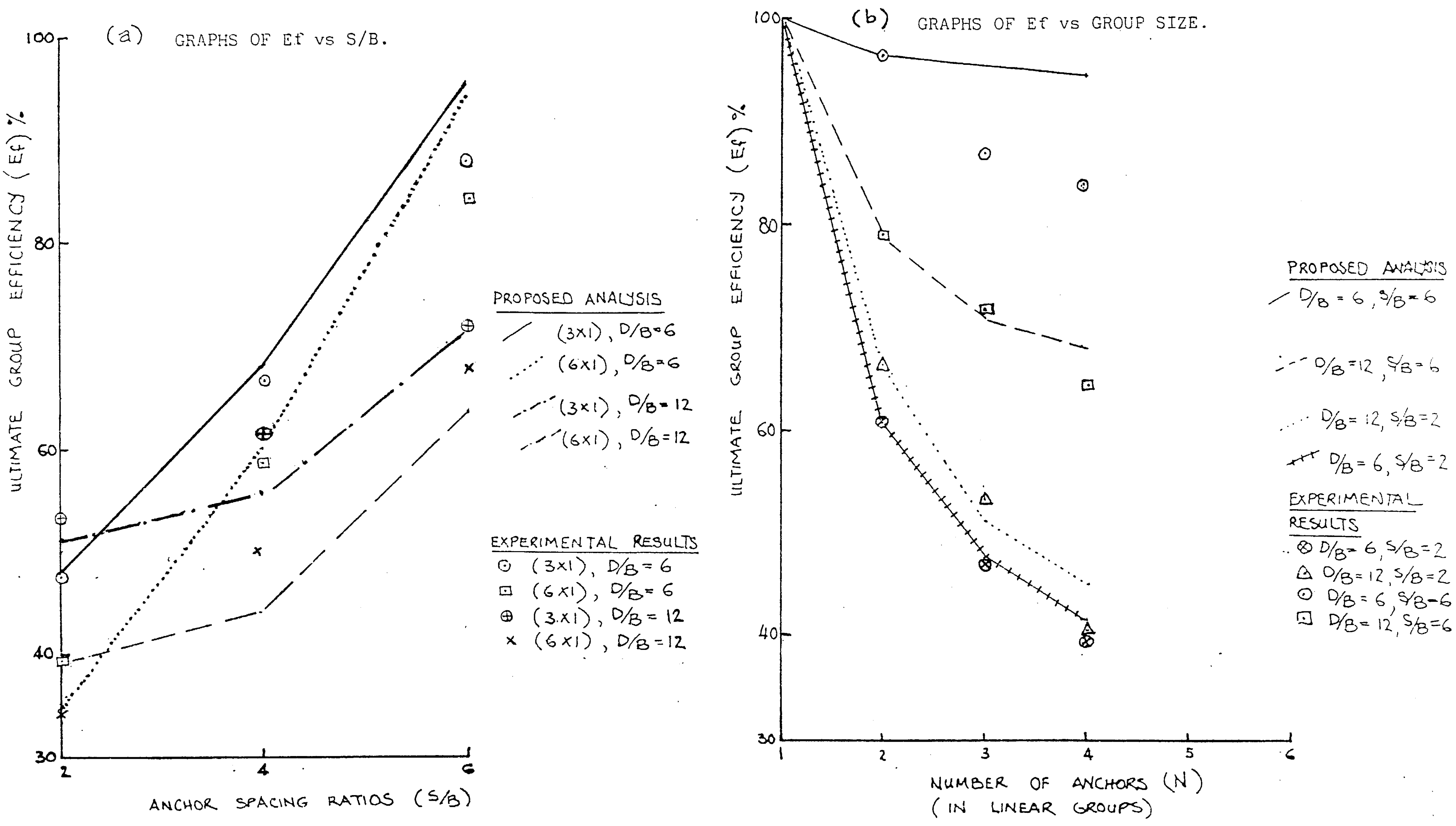
In FIG. 5.15. (a), proposed theoretical efficiencies are compared with the author's experimental efficiencies for line groups of (3x1) and (6x1). The experimental efficiencies will increase as anchor spacings increase. The trend of this observation is reflected in the proposed analysis. In FIG. 5.15. (b) the ultimate group efficiencies to number of anchors in line groups relationships are presented. The experimental efficiencies will increase as the number of anchors in a group increase. The trend of this observation is reflected in the proposed analysis. The difference between the proposed predicted (Ef) results and the author's observed (Ef) results are discussed. In this case, the proposed predicted (Ef) results ranged from 9.3% greater to 6.1% less than the author's observed (Ef) results. Any small inaccuracy of the observed results is reflected in the predicted results. The author's experimental failure loads can be reproduced with an average accuracy of $\pm$ 5%.

The predicted ultimate group efficiency of anchors in line groups derived from the proposed analysis give a better estimate than that derived from Meyerhof and Adam's theory. The predicted (Ef) results derived from the proposed analysis (for "shallow" anchor in groups - with D/B = 6) are the same as that derived from Yilmaz's first analysis. To further investigate the validity of the proposed analysis, comparisons between the observed results from other previous experiments and that derived from the proposed analysis are discussed in the following section.

5.5.3. PROPOSED ANALYSIS AND OTHER PREVIOUS EXPERIMENTAL RESULTS:

The experimental (Ef) results from Larnach and McMullan's investigation (1975) are compared with the predicted (Ef) results derived from the proposed analysis. The single and pair anchor ultimate uplift capacities used in the proposed analysis are Larnach and McMullan's observed uplift capacities. The relationship of ultimate group efficiencies and number of anchors in line groups

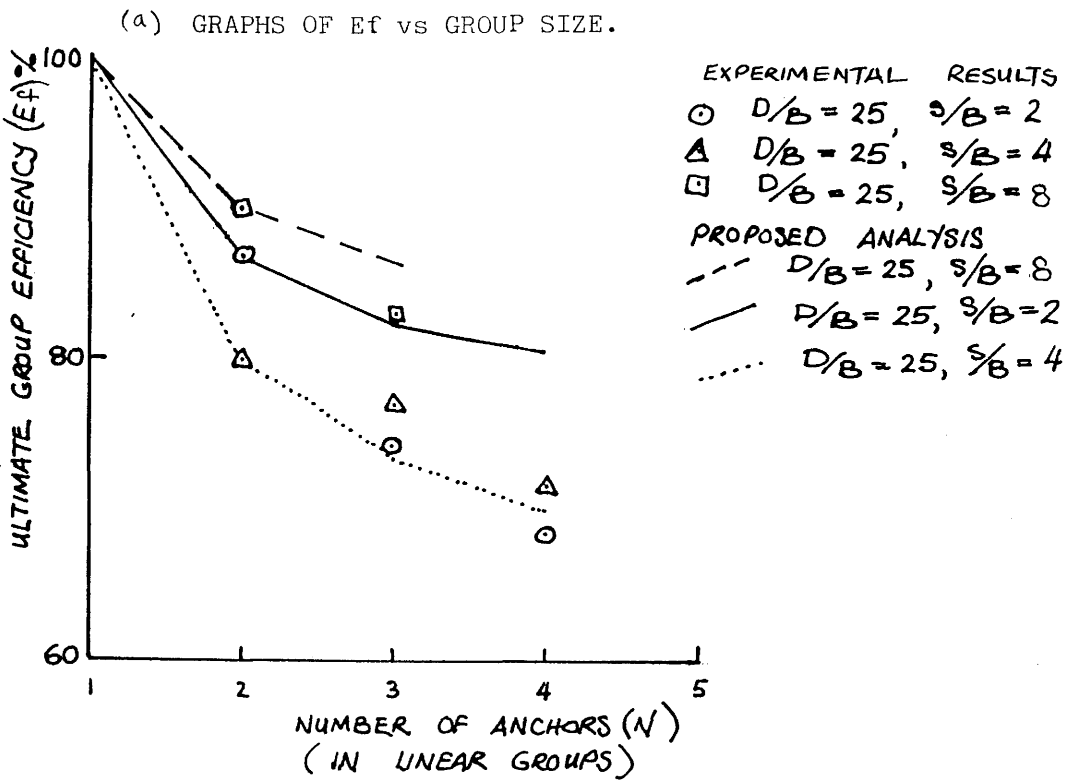
FIG. 5.15. THE COMPARISON OF THE AUTHOR'S TEST RESULTS WITH PROPOSED ANALYSIS.



is shown in FIG. 5.16. (a). For closer spacings (e.g. S/B = 2), the difference between the proposed predicted and, Larnach and McMullan's observed ultimate group efficiency will increase as the number of anchors in a line group increases. In FIG. 5.16. (b) the ultimate group efficiency-anchor spacing ratios relationships are presented. The difference between the proposed predicted and, Larnach and McMullan's observed efficiencies will decrease as the anchor spacing ratios increase. The difference between the proposed predicted (Ef) results and, Larnach and McMullan observed (Ef) results are discussed. In this case, the proposed predicted (Ef) results ranged from 12.1% greater to 3.8% less than Larnach and McMullan's observed (Ef) results.

Another set of test (Ef) results presented by Yilmaz (1971) is compared with the predicted (Ef) results derived from the proposed analysis. All Yilmaz's experimental results shown in FIG. 5.17. are from "shallow" anchors (D/B = 6) in line groups at various anchor spacings. In FIG. 5.17. (a), the ultimate group efficiencies to anchor spacing ratios relationships are shown. The proposed predicted and Yilmaz's observed efficiencies match up satisfactorily. It can also be observed that Yilmaz's experimental efficiencies will increase as anchor spacing ratios increase. The trend of this observation is reflected in the proposed analsyis. FIG. 5.17. (b), shows the relationships of the ultimate group efficiencies and number of anchors in line groups. The difference between the proposed predicted and Yilmaz's observed efficiencies will increase as number of anchors increase, especially for closer anchor spacing ratios (e.g. S/B = 2). It can also be observed that Yilmaz's experimental efficiency will decrease as the number of anchors in a line group increases. The trend of this observation is reflected in the proposed analysis. The difference between the proposed predicted (Ef) results and Yilmaz's observed (Ef) results are discussed. In this case, the proposed predicted (Ef) results ranged from 6.5% greater to 7.3% less than Yilmaz's experimental (Ef) results.

The limitations of the proposed analysis, based on the evidence compiled from the comparisons of the predictions and



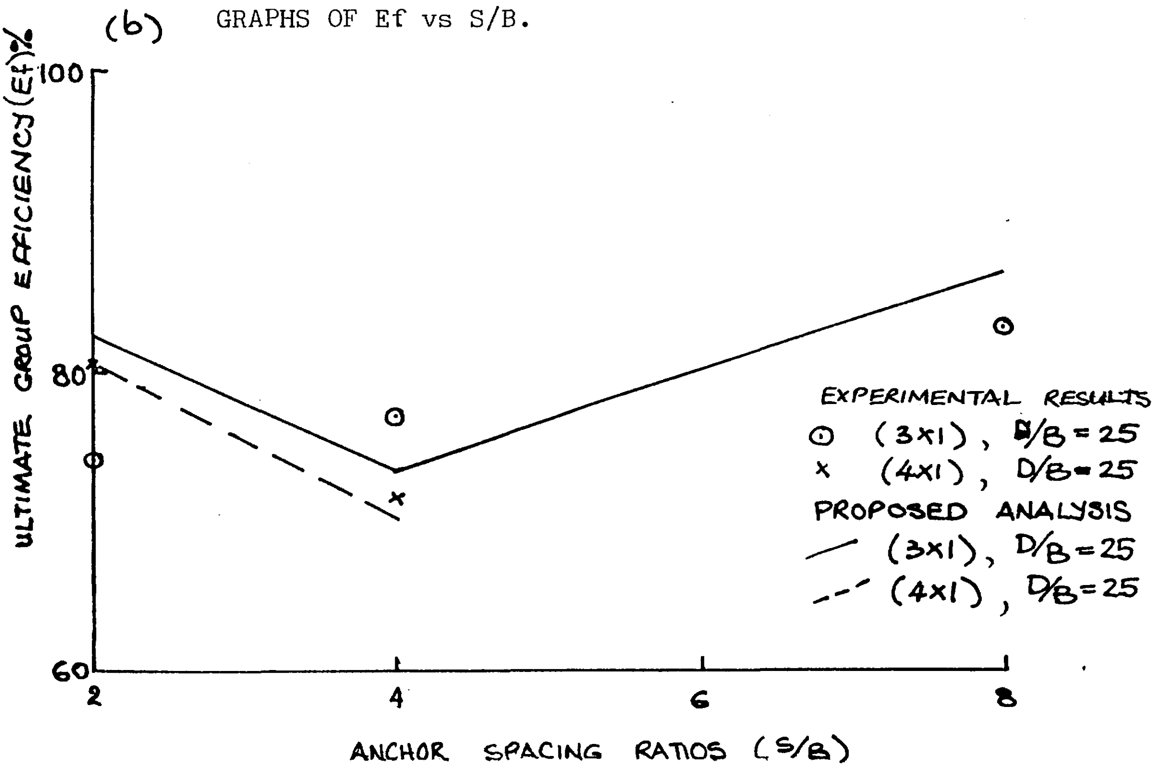
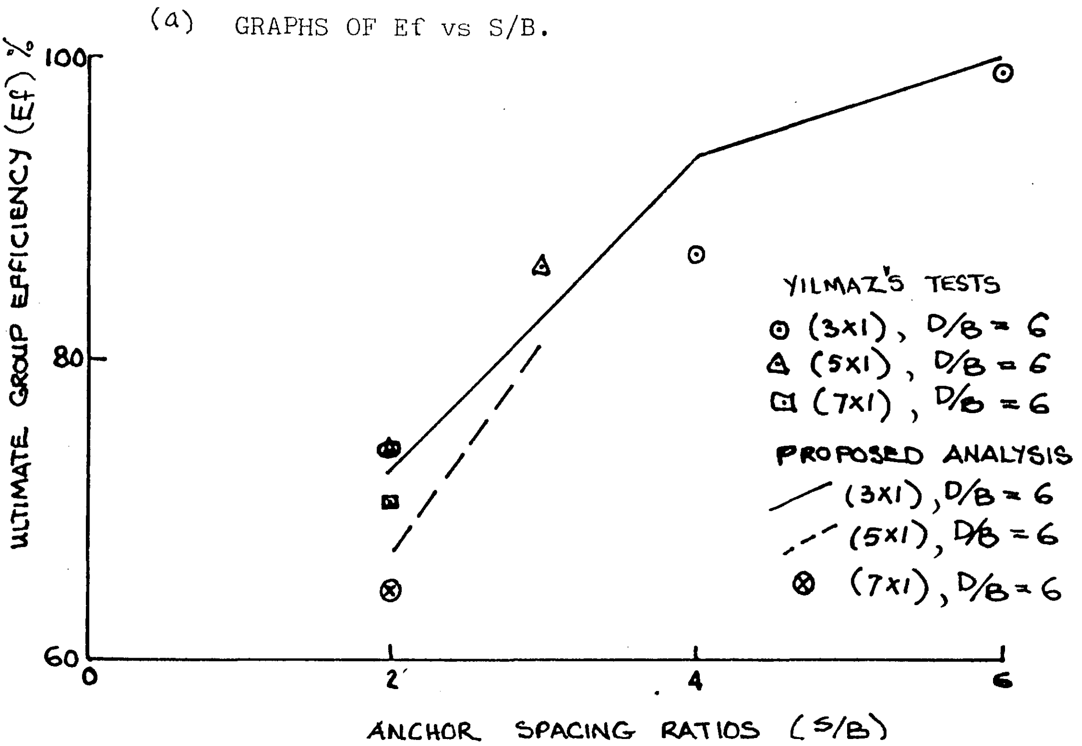


FIG. 5.16. THE COMPARISON OF LARNACH AND McMULLAN'S TEST RESULTS WITH PROPOSED ANALYSIS.



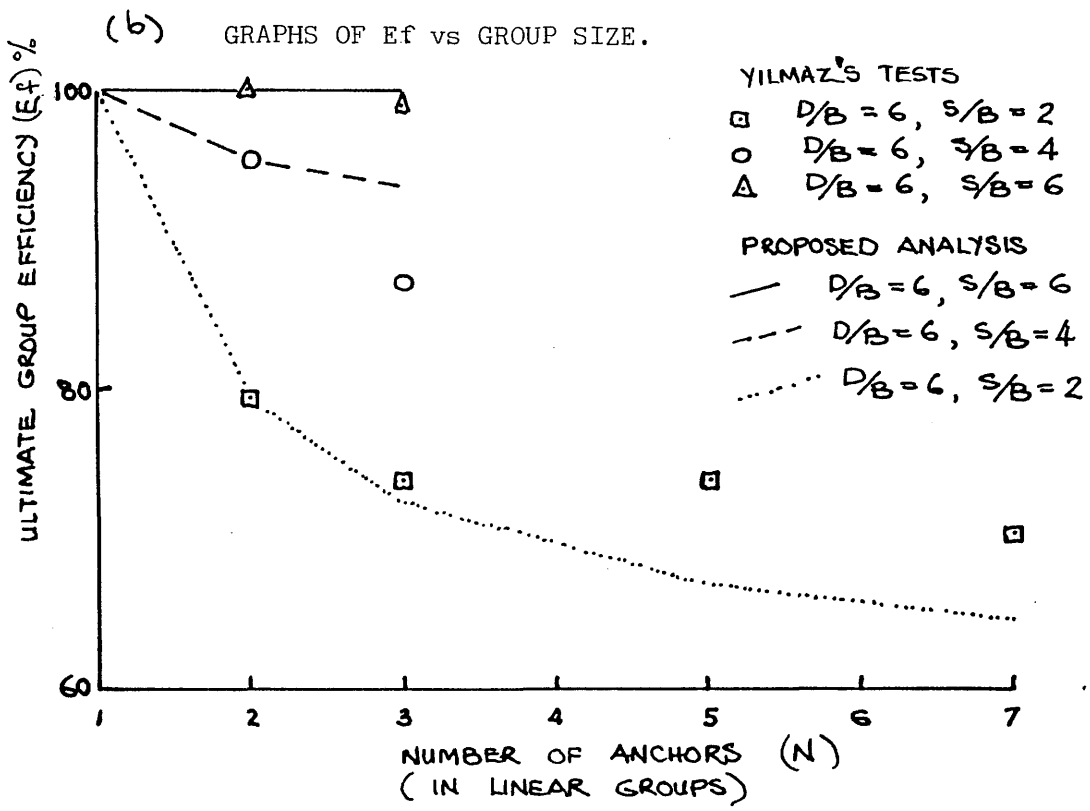


FIG. 5.17. THE COMPARISON OF YILMAZ'S TEST RESULTS WITH PROPOSED ANALYSIS.

various sets of experimental results, are as follow:

(a) The predicted (Ef) of anchors in line groups derived from the proposed analysis gives a better estimate than that derived from Meyerhof and Adam's theory (1968) (see pp. 38-40). The predicted (Ef) results derived from Yilmaz's first analysis (1971) for "shallow" anchors in line groups are the same as that derived from the proposed analysis (for "shallow" anchor in groups). (see FIGs. 5.14. and 5.15.).

(b) The difference between the predicted and experimental results will increase as the number of anchors in a line group increase. This is probably because on increasing the anchor group size, the (n-1) and (n) factors (see equation E.3.11. - Chapter 3) will increase. These factors will in turn magnify the inherent experimental errors in the failure loads of single and pair anchors used in the proposed analysis.

(c) The proposed analysis can be used to predict the ultimate uplift capacity of "shallow" or "deep" anchors in line groups installed in dry cohesionless soil. The classification of "shallow" and "deep" anchors is a prerequisite in the application of both Yilmaz's first analysis and, Meyerhof and Adam's theory. However, in the proposed analysis the classification procedure is not required.

5.5.4. COMPARISON OF RESULTS ON THE BASIS OF ULTIMATE GROUP CAPACITY:

Although in sections 5.5.1. to 5.5.3. the theoretical and experimental results are discussed in terms of ultimate group efficiency, it has been thought worthwhile to express the results in terms of ultimate group capacity (Qu). A tabulated summary of the comparison of theoretical and experimental results on the basis of ultimate capacity of anchor groups is shown in TABLE. 5.18.

Meyerhof and Adam's (1968) theoretical (Qu) results are compared with the author's experimental (Qu) results. The ratio

TABLE. 5.18. COMPARISON OF THEORETICAL AND TEST RESULTS ON THE BASIS OF ULTIMATE GROUP CAPACITY.

GROUPS	S/B	D/B	T.WANG			LARNACH & McMULLAN		YILMAZ			MEYERHOF & ADAM			
			T.ANALYSIS Qu(N)	T.TEST RESULT Qu(N)	T.ANALYSIS / T.TEST	LM.TEST RESULT Qu(N)	T.ANALYSIS / LM.TEST	Y.ANALYSIS Qu(N)	Y.TEST RESULT Qu(N)	T.ANALYSIS / Y.TEST	MA.THEORY Qu(N)	MA.THEORY / Y.TEST	T.ANALYSIS / MA.THEORY	T.ANALYSIS / Y.ANALYSIS
3x1	2	6	62.6	62.1	1.00	-	-	-	-	-	84.2	1.36	0.74	-
	2	9	180.1	173.0	1.04	-	-	-	-	-	167.0	0.97	1.08	-
	2	12	348.6	362.0	0.96	-	-	-	-	-	250.6	0.69	1.39	-
	4	6	88.6	87.0	1.02	-	-	-	-	-	121.5	1.40	0.73	-
	4	9	230.1	228.9	1.01	-	-	-	-	-	247.8	1.08	0.93	-
	4	12	377.6	418.8	0.90	-	-	-	-	-	374.8	0.89	1.01	-
	6	6	124.6	113.6	1.10	-	-	-	-	-	121.5	1.07	1.03	-
	6	9	300.1	282.2	1.06	-	-	-	-	-	247.8	0.88	1.21	-
	6	12	485.6	489.8	0.99	-	-	-	-	-	374.8	0.77	1.30	-
4x1	2	6	72.2	68.6	1.05	-	-	-	-	-	106.1	1.55	0.68	-
	2	9	209.2	184.6	1.13	-	-	-	-	-	209.2	1.13	1.00	-
	2	12	409.7	369.1	1.11	-	-	-	-	-	313.4	0.85	1.31	-
	4	6	111.2	101.8	1.09	-	-	-	-	-	162.0	1.59	0.69	-
	4	9	284.2	266.2	1.07	-	-	-	-	-	330.4	1.24	0.86	-

CONTINUED:

GROUPS	S/B	D/B	T.WANG T.ANALYSIS Qu(N)	T.WANG T.TEST RESULT Qu(N)	T.WANG T.ANALYSIS / T.TEST	LARNACH & McMULLAN LM.TEST RESULT Qu(N)	LARNACH & McMULLAN T.ANALYSIS / LM.TEST	YILMAZ Y.ANALYSIS Qu(N)	YILMAZ Y.TEST RESULT Qu(N)	YILMAZ T.ANALYSIS / Y.TEST	MEYERHOF & ADAM MA.THEORY Qu(N)	MEYERHOF & ADAM MA.THEORY / T.TEST	T.ANALYSIS / MA.THEORY	T.ANALYSIS / Y.ANALYSIS
4x1	4	12	453.2	487.5	0.93	-	-	-	-	-	499.8	1.03	0.91	-
	6	6	165.2	149.1	1.11	-	-	-	-	-	162.0	1.09	1.02	-
	6	9	389.2	372.7	1.04	-	-	-	-	-	330.4	0.89	1.18	-
	6	12	615.2	610.5	1.01	-	-	-	-	-	499.8	0.82	1.23	-
6x1	2	6	91.4	102.9	0.89	-	-	-	-	-	149.9	1.46	0.61	-
	2	9	267.4	239.6	1.12	-	-	-	-	-	293.6	1.23	0.91	-
	2	12	531.9	468.7	1.13	-	-	-	-	-	439.0	0.94	1.21	-
	4	6	156.4	152.6	1.02	-	-	-	-	-	243.0	1.59	0.64	-
	4	9	392.4	378.0	1.04	-	-	-	-	-	495.6	1.31	0.79	-
	4	12	604.4	681.5	0.89	-	-	-	-	-	749.6	1.10	0.81	-
	6	6	246.4	220.1	1.12	-	-	-	-	-	243.0	1.10	1.01	-
	6	9	567.4	559.0	1.02	-	-	-	-	-	495.6	0.89	1.14	-
	6	12	874.4	915.8	0.95	-	-	-	-	-	749.6	0.82	1.17	-

CONTINUED:

GROUPS	S/B	D/B	T.WANG T.ANALYSIS Qu(N)	T.WANG T.TEST RESULT Qu(N)	T.WANG T.ANALYSIS / T.TEST	LARNACH & McMULLAN LM.TEST RESULT Qu(N)	LARNACH & McMULLAN T.ANALYSIS / LM.TEST	YILMAZ Y.ANALYSIS Qu(N)	YILMAZ Y.TEST RESULT Qu(N)	YILMAZ T.ANALYSIS / Y.TEST	MEYERHOF & ADAM MA.THEORY Qu(N)	MEYERHOF & ADAM MA.THEORY / T.TEST	T.ANALYSIS / MA.THEORY	T.ANALYSIS / Y.ANALYSIS
3x1	2	25	1614.6	-	-	1448.9	1.11	-	-	-	-	-	-	-
	4	25	1430.0	-	-	1503.5	0.95	-	-	-	-	-	-	-
	8	25	1690.0	-	-	1616.6	1.04	-	-	-	-	-	-	-
4x1	2	25	2096.9	-	-	1783.6	1.18	-	-	-	-	-	-	-
	4	25	1820.0	-	-	1856.4	0.98	-	-	-	-	-	-	-
3x1	2	6	130.7	-	-	-	-	130.7	134.0	0.97	-	-	-	1.00
	4	6	168.6	-	-	-	-	168.6	156.8	1.08	-	-	-	1.00
	6	6	184.8	-	-	-	-	184.8	177.5	1.04	-	-	-	1.00
5x1	2	6	201.4	-	-	-	-	201.4	223.3	0.90	-	-	-	1.00
	3	6	243.1	-	-	-	-	243.1	260.2	0.93	-	-	-	1.00
7x1	2	6	272.1	-	-	-	-	272.1	295.3	0.92	-	-	-	1.00

of Meyerhof and Adam's theoretical (Qu) predictions to the author's experimental (Qu) results ranged from 1.59 to 0.69. The range of (M.A. theory / T.test) ratios is wide and this suggests that Meyerhof and Adam's theory for predicting (Qu) values is not entirely satisfactory.

The predicted (Qu) values derived from Yilmaz's first analysis (1971) for "shallow" anchors in groups are the same as that derived from the proposed analysis (for "shallow" anchor in groups). The ratio of Yilmaz's first analysis (Qu) predictions to the author's experimental (Qu) results for "shallow" anchor groups (at D/B = 6) ranged from 1.12 to 0.89. The range of the (Y. analysis / T.test) ratios is narrower than that of the (M.A. theory / T.test) ratios. This suggests that the (Qu) results derived from Yilmaz's first analysis gives a better estimate than that derived from Meyerhof and Adam's theory.

The proposed analysis is basically an improved version of Yilmaz's first analysis. The proposed analysis can be used to predict the (Qu) values of "deep" and "shallow" anchors in groups (see Chapter 3). The predicted (Qu) results derived from the proposed analysis are compared with the author's experimental (Qu) results. The ratio of the predicted (Qu) results derived from the proposed analysis to the author's experimental (Qu) results ranged from 1.13 to 0.89. To further investigate the validity of the proposed analysis, the observed (Qu) test results from Larnach and McMullan, and Yilmaz investigations are compared separately with the predicted (Qu) results derived from the proposed analysis. The ratio of the predicted (Qu) results derived from the proposed analysis to Larnach and McMullan's experimental (Qu) results ranged from 1.18 to 0.95. The ratio of the predicted (Qu) results derived from the proposed analysis to Yilmaz's experimental (Qu) results ranged from 1.08 to 0.90. Based on these above observations, the predicted (Qu) results derived from the proposed analysis give a better estimate than that derived from Meyerhof and Adam's theory.

In Appendix A is an example of how the proposed analysis can be used to design an anchor group system at each stage of the design procedure.

CHAPTER 6

CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

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CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

6.1. CONCLUSIONS:

The author conducted 10 static uplift tests on single anchors and 41 tests on anchors in various line groups embedded in dry sand. The 51 tests were conducted using 25mm diameter model plate anchors. Anchor spacings of S = 2B, 4B, 6B, (B = diameter of an anchor footing) and depth of anchor embedment ranging from 3B to 15B were studied. The line groups considered were up to 6 anchors. On the basis of the tests conducted, the conclusions are as follows:

(a) At close spacing ratio (e.g. S=2B), the ultimate capacity of a group of n anchors is not equal to n times the ultimate load of a single isolated anchor. This is because of the interaction of the failure zones in the soil above and around each anchor within the group. As the spacing between the anchors in a group increase, the ultimate load of the group will increase. The scope of author's test results show that the "isolation" anchor spacing (Si) was observed to be greater than 6B for the circular plate anchors in line groups.

(b) For a given anchor spacing, the ultimate group efficiency (Ef) decreases to a minimum with increasing D/B ratios, and then increases to as large a value as that found with small values of D/B ratios. There appears to be a distinct critical D/B ratio where Ef is a minimum value. It was observed from the author's test results that for a given anchor group size, at anchor spacings of 2B, 4B, 6B, the minimum ultimate group efficiency corresponded to the critical D/B ratios of 8, 10, 12, respectively. Hence, at anchor spacing of 2B, the author's definition of "shallow" anchors in groups are anchor groups with $D/B \leqslant 8$ and "deep" anchor groups have $D/B > 8$. At anchor spacing of 4B and 6B, "shallow" and "deep" anchor groups can be similarly classified.

(c) As anchor spacings increase the (Ef) values will increase. However, as group sizes increase the (Ef) values will decrease.

(d) The distribution of load amongst a group of anchors is non-uniform at failure. For anchors in a line group, the proportion of the total ultimate group load carried by an anchor in the group increases as its distance increases from the point of load application.

(e) The displacement ratios (Δr) for an anchor group installed in a given density of sand were dependent on the definition of displacement ratio, the depth of embedment ratio, the anchor spacing, group size and group configuration. According to the author's definition of displacement ratio, the displacement ratios will increase as the anchor spacings increase. At close anchor spacings ($S < 4B$), increasing the depth of embedment and group sizes, the displacement ratios will decrease. The displacement ratio at failure is unity when the spacing between each anchor in a group is at the spacing where each anchor within the group can act as a single anchor.

The previous theoretical and experimental work on anchor group behaviour was discussed in Chapter 2. A summary of the previous experimental work done on anchors in groups in sand is shown in TABLE. 2.06. The experimental results from each reported case are difficult to compare because of difference in testing conditions. However the general similarity with previous test results are reported in section 2.5. The analysis suggested by Meyerhof and Adams (1968), and Yilmaz (1971) for predicting the ultimate uplift capacity of anchors in groups were found not completely satisfactory for various reasons (see sections 2.3. and 5.5.). Based on the author's experimental results, a simple analysis was derived to predict the ultimate uplift capacity of "shallow" or "deep" anchors in line groups embedded in dry sand (see Chapter 3). The limitations of the proposed analysis, and the difference between the proposed analysis and the previous analysis for predicting the behaviour of anchors in groups were discussed in sections 5.5.3. and 5.5.4.

This investigation has hopefully improved the understanding of the behaviour of line groups of model plate anchors in sand subjected to static uplift loading. The proposed analysis can be considered as another useful tool in the preliminary design of anchors in line groups embedded in sand.

6.2. SUGGESTIONS FOR FUTURE WORK:

To further the understanding of the behaviour of anchors in groups, the following research suggestions are considered to be worthy of pursuing.

(a) A study of the failure zones of anchors in groups subjected to static and cyclic loadings:
The objective of this investigation will be to establish a "standard" failure surface for anchors in groups. If this problem is solved, an analytical solution to predict precisely the behaviour of group anchors could possibly be achieved.

(b) Research work to investigate the behaviour of anchors subjected to a combination of static and cyclic loading:
The previous experiments were conducted with anchors subjected to static loading, not a combination of both types of loadings. The purpose of this research would be to establish a theory to predict the ultimate uplift load of anchors in groups subjected to the combination loading.

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APPENDIX A

APPENDIX A

PROBLEM:

Consider the uplift load to be resisted by a line group of anchors to be 1300 kN. The embedded anchors used consist of a shaft and circular bottom plate with diameter (B) of 0.5m. The group of anchors is to be installed at spacings (S) of 4B which is considered feasible from the superstructure design criteria. The anchors are to be installed vertically by placing the anchor plates in the base of an excavated shallow trench (of depth D = 4m), attaching the anchor shafts, and then backfilling with dry sand above the anchor plates to the original ground surface. After installation, the backfill is compacted by transmitting high frequency vibrations through the anchor assembly. The dry sand used as backfill should then achieved the following characteristics --- $\phi = 38^{\circ}$, R.D. = 0.58, $\gamma = 16.0$ kN/m^3. Find the number of anchors required.

SOLUTION:

A.1. THEORETICAL - PRELIMINARY DESIGN

A.1.1. ULTIMATE UPLIFT CAPACITY OF A SINGLE ANCHOR

The ultimate capacity of a single anchor installed in sand can be derived from various previous analysis. In this investigation, Meyerhof and Adam's [40] and Fadl's [18] theoretical (Qu) predictions for a single anchor are considered.

(a) Meyerhof and Adam's theory (1968).
For a single anchor with D/B = 8, hence
$P/\gamma D = 41.5$, then a value of Qu(1) = 535 kN.

(b) Fadl's theory (1981).
For a single anchor with D/B = 8, hence
$P/\gamma D = 41$, then a value of Qu(1) = 528 kN.

A.1.2. ULTIMATE UPLIFT CAPACITY OF A PAIR ANCHOR

Based on a survey of experimental results of (2x1) groups of anchors from Meyerhof and Adam; Yilmaz; Larnach and McMullan; and Wang (author) investigations, it can be observed that there is a relationship of the form as shown in FIG.A.00. So for a (2x1) group at S/B = 4, hence the average (Ef) = 84%.

(a) For Qu(1) = 535 kN, then

$$Qu(2) = 535 \times 2 \times \frac{84}{100} = 899 \text{ kN}$$

(b) For Qu(1) = 528 kN, then

$$Qu(2) = 528 \times 2 \times \frac{84}{100} = 887 \text{ kN}$$

A.1.3. PROPOSED ANALYSIS

The proposed analysis is then used to determine the number of anchors required.

(a) For Qu(1) = 535 kN and Qu(2) = 899 kN and assuming a factor of safety of 3,
Try (11x1) group,
Qu(11) = 10(899) - 9(535) = 4175 kN.
Adopting a F.O.S. of 3,
therefore Qu(11) = 1392 kN (OK).
Therefore the number of anchors required in a line group is 11.

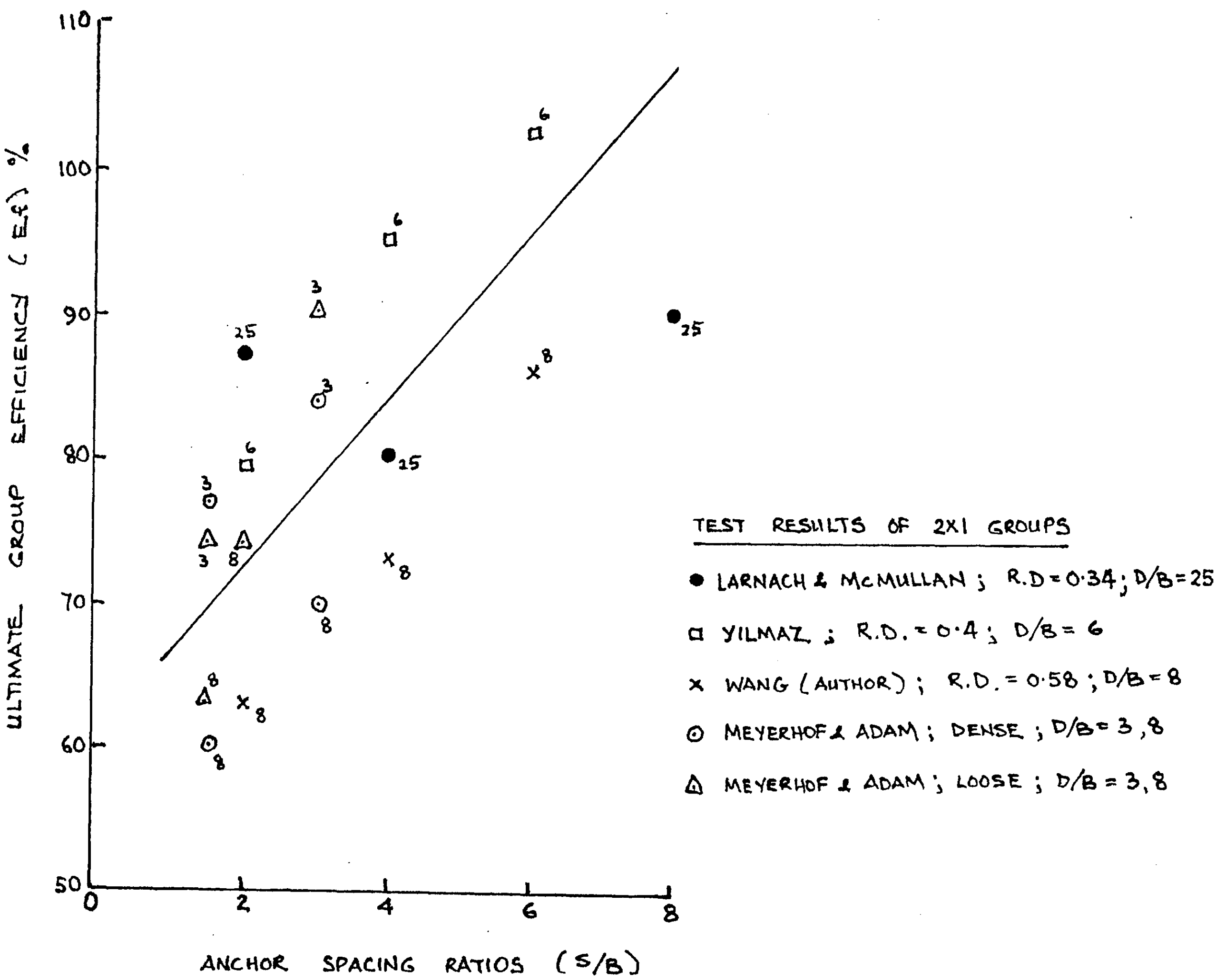
(b) For Qu(1) = 528 kN and Qu(2) = 887 kN and assuming a factor of safety of 3,
Try (11x1) group,
Qu(11) = 10(887) - 9(528) = 4118 kN.
Adopting a F.O.S. of 3,
therefore Qu(11) = 1372 kN (OK).
Therefore the number of anchors required in a line group is 11.

A.2. LABORATORY EXPERIMENTS

If laboratory facilities were available, then a more accurate prediction could be made by model tests (for single anchor and pair anchors) similar to those carried out by the author.

FIG. A.00. TEST RESULTS OF (2x1) GROUPS

÷ GRAPH OF Ef vs S/B



A.2.1. ULTIMATE CAPACITIES OF A SINGLE ANCHOR AND A PAIR ANCHOR

In this example, the field site has the same soil properties as the sand used in the author's experimental investigation. Hence, based on the author's experimental results (see FIG. 5.01.) the following failure loads can be obtained.

(a) For a single anchor with D/B = 8 and at S/B = 4, hence $P/\gamma D = 59$, then a value of Qu(1) = 760kN.

(b) For a pair anchor with D/B = 8 and at S/B = 4, hence $P/\gamma D = 43.5$, then a value of Qu(2) = 2(560) = 1120 kN.

A.2.2. PROPOSED ANALYSIS

For Qu(1) = 760 kN and Qu(2) = 1120 kN and assuming a factor of safety of 3,
Try (10x1) group,
Qu(10) = 9(1120) - 8(760) = 4000 kN.
Adopting a F.O.S. of 3,
therefore Qu(10) = 1333 kN (OK).
Therefore the number of anchors required in a line group is 10.

A.3. <u>SITE TESTS</u>

If job is big enough to justify, then field tests to determine the failure loads of a single anchor and a pair of anchors can be conducted. These field test results are then input into the proposed analysis to determine the number of anchors required.

APPENDIX B

APPENDIX B. AUTHOR'S EXPERIMENTAL RESULTS.

$\rho = 1638\ kg/m^3$, $\phi = 38.0^o$, R.D. = 0.58, B = 25mm.

GROUP CONFIGURATION	S/B	D/B	Qu (N)	qu (N)	Ef (%)	P/γD	δ_f (mm)	Δf (mm)	Δr
SINGLE AND ISOLATED.	-	5.2	-	29.3	100	28.5	2.00	-	-
	-	5.7	-	31.9	100	29.0	2.90	-	-
	-	6.2	-	49.2	100	40.1	1.93	-	-
	-	6.8	-	58.9	100	44.0	4.00	-	-
	-	8.9	-	119.0	100	67.8	5.50	-	-
	-	9.0	-	121.9	100	68.7	3.60	-	-
	-	9.1	-	117.0	100	65.3	5.00	-	-
	-	11.6	-	216.6	100	94.4	5.80	-	-
	-	12.1	-	256.5	100	108.0	9.20	-	-
	-	12.2	-	225.9	100	93.8	10.00	-	-
(2x1)	2	3.2	13.1	-	92.9	10.2	0.90	0.89	0.99
	2	15.0	453.2	-	70.9	76.7	13.30	6.00	0.45
	4	3.2	14.7	-	91.8	10.1	0.90	0.90	1.00
	4	15.0	485.1	-	75.9	82.0	13.30	5.16	0.39

CONTINUED.

GROUP CONFIGURATION	S/B	D/B	Q_u (N)	q_u (N)	Ef (%)	$P/\gamma D$	δ_f (mm)	Δf (mm)	Δr
(2x1)	2	6.0	53.0	–	61.0	22.4	–	–	–
	2	9.0	151.0	–	61.9	42.5	–	–	–
	4	6.0	66.0	–	76.0	27.9	–	–	–
	4	9.0	176.0	–	72.2	49.6	–	–	–
	6	6.0	84.0	–	96.8	35.5	–	–	–
	6	9.0	211.0	–	86.5	59.4	–	–	–
	6	12.0	356.0	–	78.6	75.2	–	–	–
(3x1)	2	6.6	78.5	–	44.2	20.1	3.00	1.77	0.59
	2	9.4	197.5	–	49.3	35.5	5.60	1.77	0.32
	2	12.2	377.5	–	53.6	52.3	–	–	–
	4	6.1	90.2	–	64.1	25.0	2.60	1.27	0.49
	4	9.6	261.2	–	62.1	46.0	5.80	2.43	0.42
	4	12.5	458.5	–	62.3	62.0	9.50	3.05	0.32
	6	6.4	129.7	–	79.6	34.5	2.80	2.76	0.99
	6	10.4	376.9	–	75.6	61.4	6.80	5.06	0.74
	6	12.4	522.3	–	71.9	71.2	9.30	4.60	0.49

CONTINUED.

GROUP CONFIGURATION	S/B	D/B	Qu (N)	qu (N)	Ef (%)	$P/\gamma D$	δf (mm)	Δf (mm)	Δr
(4x1)	2	5.8	57.2	–	36.8	12.5	–	–	–
	2	6.4	90.9	–	41.9	18.0	–	–	–
	2	6.9	95.2	–	35.0	17.5	3.20	1.72	0.54
	2	10.0	234.3	–	38.3	29.7	6.30	2.28	0.36
	2	12.8	438.2	–	42.1	43.4	9.90	2.51	0.25
	4	7.1	156.8	–	54.4	28.0	3.40	2.10	0.62
	4	9.8	323.1	–	55.4	41.8	6.10	2.96	0.49
	4	12.3	511.3	–	53.5	52.7	9.20	3.43	0.37
	6	6.1	154.4	–	82.3	32.1	2.60	2.28	0.88
	6	9.4	404.8	–	75.8	54.6	5.60	3.21	0.57
	6	12.1	626.1	–	68.3	65.6	9.00	4.04	0.45
(6x1)	2	5.6	69.6	–	37.4	10.5	2.20	2.06	0.94
	2	6.3	124.5	–	39.8	16.7	2.70	2.37	0.88
	2	9.8	289.9	–	33.1	25.0	6.10	3.05	0.50
	2	12.6	538.2	–	35.6	36.1	9.60	3.71	0.39
	4	6.5	193.7	–	56.0	25.2	2.90	2.59	0.89
	4	9.4	410.4	–	51.2	36.9	5.60	3.30	0.59
	4	12.4	745.3	–	51.1	50.8	9.30	3.40	0.37
	6	7.0	315.5	–	75.4	38.1	3.30	2.57	0.78
	6	8.8	537.2	–	79.4	51.6	5.00	3.45	0.69
	6	11.9	903.9	–	67.6	64.2	8.70	3.91	0.45