# The Effect on the Uplift Resistance of Anchors of Ground Disturbance During Placing

by

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## SUMMARY

In civil engineering works, anchors are sometimes used to support uplift of tensile forces. The installation of an anchor in the ground will inevitably cause a certain degree of disturbance to the soil around it. From a practical point of view this zone of disturbance does exist to a lesser or greater extent depending upon the types and shapes of anchor used and the surrounding soils.

The work described in this thesis was primarily concerned with the effect of installation disturbance on the pullout capacity of a vertical circular plate anchor embedded in sand. As far as the author is aware, this particular anchor problem has not received much attention.

Because of this no literature review which had a direct relevance to the present investigation was available. To date only Kulhawy ( 1985 ) has proposed tentative guidelines for the design of spread anchors embedded in a soil zone which had a density different from the surrounding soil mass. Nevertheless a brief summary of the available pullout theories on circular vertical plate anchor is presented.

To investigate the effect of installation disturbance on the anchor pullout capacity, two types of sand bed were required. Firstly to offer a standard of comparison with disturbed sand beds, homogeneous sand beds were prepared in a sand container in order to carry out pullout tests on anchors with depth/diameter ( D/B ) ratios ranging from 3 to 15. Three states of homogeneous sand bed of unit weight 17.14, 16.40 and 15.75 $KN/m^3$ of relative densities 92%, 70% and 49% respectively were used in this investigation.

These sand beds represented the dense, medium and loose sand states in the context of the research. Three model circular anchors were used namely 25, 50 and 75mm diameter.

The pullout tests on the anchors were performed under load-control and displacement-control so that a comparison could be made of the ultimate load on the anchor between these two types of test. It was found that under the load-controlled test the post peak load on the anchor could not be observed.

The anchor pullout capacity curves expressed in terms of the breakout factor ( $Pu/\gamma D$ ) against the depth/diameter ratio ( D/B ) between the dense and loose homogeneous sand beds were used as the upper and lower limits of the pullout capacity available.

Secondly to simulate the effect of the installation procedure disturbed beds were prepared or formed in such a way that a volume of loose sand in the form of a cylinder was formed above the anchor position within a container filled with homogeneous sand beds of varying unit weights deposited from a spreader. The depths of anchor embedment D, were varied to produce a range of D/B ratios from 3 to 15. The unit weight of the loose homogeneous sand bed i.e. 15.75 $KN/m^3$ was used as a basis for the formation of the cylindrical disturbed zone above the anchor plate. The width of the disturbed zone $B_z$, was varied in proportion to the anchor diameter B over the range of $B_z/B$ = 1,2 and 3.

Results from the pullout tests in the homogeneous sand beds showed that the author's tests were consistent with Fadl's ( 1981 ) experimental results. Particular attention was drawn to Fadl's work because his theory was used by the author as a preliminary step to establish his ( the author's ) theoretical uplift resistance for an anchor in a disturbed zone.

From the tests carried out in the disturbed beds, the installation disturbance significantly affected the anchor pullout capacity especially when the ratio of $B_z$ /B was greater than 1. The tests also showed that the value of $B_z$ /B should be kept to a minimum and for a plate anchor the minimum possible value of $B_z$ /B was 1. When the width of the zone of disturbance was increased to three anchor diameters, the results showed that the anchor pullout capacity embedded in a disturbed zone surrounded by a dense homogeneous sand bed, was similar to that anchor as if it were being pulled out from a bed which was wholly disturbed throughout the sand mass.

A simple expression for the anchor uplift resistance in a disturbed zone derived from Fadl's equations is presented. The theoretical uplift resistance showed a reasonable agreement with the test results.

Conclusions were drawn from the test results and the theoretical analysis. Due to the absence of other theoretical analyses and published works, it was not possible to make a comparison with the existing data. Finally suggestions were made and further works were proposed for the future.

# CONTENTS

## NOMENCLATURE

Unless otherwise stated the following notations are used throughout the Thesis.

$B$ = Size or Diameter of Anchor, mm

$D$ = Depth of Embedment of Anchor, mm

$D/B$ = Depth to Diameter ratio

$B_z$ = Width or Diameter of Disturbed Zone,mm

$B_z/B$ = Ratio of Disturbed Zone to Anchor Diameter

$K$ = Coefficient of Lateral Earth Pressure

$\gamma$ = Unit weight of Soil , $KN/m^3$

$\gamma_z$ = Unit Weight of Soil in Disturbed Zone , $KN/m^3$

$P$ = Ultimate Load on Anchor, N

$Pu$ = Ultimate Pressure on Anchor , $N/mm^2$

$Pu/\gamma D$ = Uplift Capacity ratio or Break-out Factor

$\delta$ = Displacement of anchor , mm

$\varphi$ = Angle of Internal Friction in Degree

$D_z$ = Height of Inclined Failure Plane at the Sand Interface

$\alpha$ = Cone Break-out angle from the Vertical through th Anchor Edge

D.R. = Relative Density of Sand

$n$ = Porosity of Sand

$H/B$ = Critical Embedment Ratio of Anchor

# CHAPTER 1

## TYPES OF ANCHORS AND METHODS OF INSTALLATION

### 1.1 Introduction

In civil engineering works a foundation is sometimes required to withstand tensile forces or loads and the stability of such a foundation will depend on the strength of soil above it just as a foundation under a compressive load depends for its stability on the soil underneath . An anchor is a form of foundation which is primarily used to support uplift or tensile forces.

In this chapter various types of anchor and methods of installation will be discussed. The types of anchors and their installation will influence their ultimate holding capacity especially in regions where the soil conditions are unfavourable and they are susceptible to external disturbance.

The installation of an anchor in the ground will inevitably result , to a certain extent , in a degree of disturbance around it . To maximise the pullout capacity it is important to keep the disturbance to a minimum especially in areas where the ground conditions are not favourable . Naturally the wider the disturbed zone the lesser will be the anchor holding capacity.

Traditionally anchors were constructed of a mass concrete or stone block tied to a structure in question but as the need for larger capacity anchors increased it became uneconomical and unfeasible to use this type of anchor . With the advent of new techniques and machines in this field more and more types of anchor are being designed and produced in a large scale to meet the increasing demand . Nowadays in virtually all aspects of civil engineering works many different types of anchor are widely used . Some of the commonly used types of anchor are shown in Fig. 1.1 .

## 1.2 Types_of_Anchors

Broadly speaking anchors can be divided into three main groups viz;

1.Ground Anchors

2.Rock Anchors

3.Marine Anchors

### 1.2.1 Ground_Anchors

A ground anchor is a structural member which transmits the tensile forces to a 'competent' ground ( Hanna,1980 ). The soil parameters and types of anchor govern the strength of the anchor . Some examples of ground anchor are shown in Fig. 1.2.

#### a).Dead-weight_Anchor

This is the simplest and crudest form of anchor.Dead-weight anchors depend on their weight for stability . To mobilise the load the anchor has to be moved relative to the surrounding ground . These types of anchor are normally used in onshore and offshore operation but as their load carrying capacities are limited and their construction is no longer feasible because they are massive, they have become obsolete . Fig.1.2(a) shows a typical example of a dead-weight anchor.

#### b).Plate_Type_Ground_Anchor

The stability of a plate anchor depends on the weight and shear strength of the soil above it and the weight of the anchor itself . Depending upon its size a considerable amount of soil has to be excavated to a certain depth to install the anchor and this results in the soil around it being disturbed as shown in Fig 1.2(b). To compensate for any loss of soil strength , the backfill has to be compacted so that the uplift capacity can be maximised.

c).Bored or Under-reamed Type Anchor

This type of anchor is similar in shape to a bored pile having an expanded base at the bottom end . The anchor together with its shaft ( Fig.1.2(c) ) makes full use of the frictional resistance of the soil . It is normally constructed of concrete ( may be reinforced ).

d). Helix Anchor

This form of anchor as illustrated in Fig.1.2(d) consists of a long shaft with a number of helical-shaped circular steel plates welded to the shaft . The anchor is installed in the ground by mounting it to a flight auger equipment . It can be used as a cost effective means of providing tension anchorages for foundations where the soil conditions permit its installation . Loads upto 60 tons ( 534 KN ) can be developed by using large multi-helix anchors ( Mitsch and Clemence,1985 ).

### 1.2.2 Rock Anchors

Rock anchors are normally grouted in position and the anchorage is obtained by bond between the grout and the surrounding rock . Anchorage length may vary with rock type , being short for granite . Fig.1.3 (a) and Fig.1.3(b) show some available types of rock anchors.

### 1.2.3 Marine Anchors

Marine anchors are widely used in onshore and offshore operations where large tensile loads are to be supported . In view of this the anchors should also be capable of resisting dynamic and cyclic loads . They are used to provide uplift resistance for oil rigs , ships , barges etc . Some common types of marine anchor are shown in Fig. 1.4.

## 1.3. Methods of Installing Anchors

With the introduction of new methods and equipment there are various ways of installing an anchor in the ground . The method of installation will depend on the type of anchor , ground conditions , cost etc.New applications of anchors have also given rise to many descriptive terms (Littlejohn,1973) such as multi-underream , ground placement , lost point multi-helix and so on . Some of the available methods today are;

1).Vibration

2).Augering

3).Drilling

4).Driving

5).Excavation

6).Blasting

1).Vibration

The installation is suitable for deep water. The anchor is attached to a long metal construction consisting of a fluke-shaft assembly and a vibrator . The vibrator, which consists of counter-rotating eccentric masses, drives the anchor into the ground . The anchor performance depends on the vibrator power , the length of the shaft and the soil properties .

2).Augering

A slender shaft having one or more single turn helical surfaces fixed to the shaft is screwed into the soil at a pre-determined depth . Penetration can be monitored very easily . Originally used on land as a guy anchor for electrical transmission lines .

3). Drilling

The hole for placing the anchor is first drilled to the required depth . A provision for extracting water and soil during drilling is available in some machines. This type of installation permits the use of an anchor in the form of a pile with a straight or under-reamed base. Grouted anchors can also be installed by this method .

4). Driving

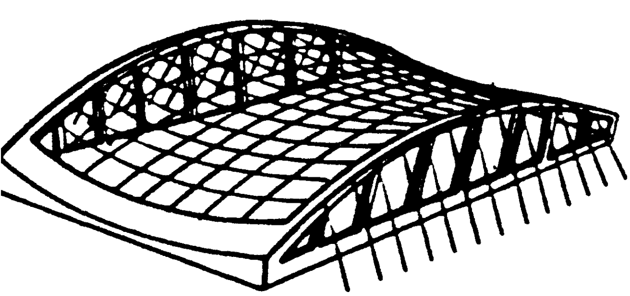
During installation the anchor is forced into the ground by repeated or impulsive loads , usually from a hammer . Several types of anchor such as the umbrella pile and the stake pile are installed in this manner ( Fig.1.4(b)). A single plate anchor can also be placed in this way by driving with a mandrel and follower and then the plate is opened up by applying a pullout load ( Fig. 1.4 (e)).

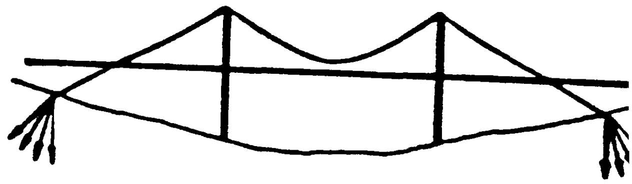
5). Excavation

This is the most traditional and the simplest way to install an anchor designed primarily as a slab or plate . The ground is excavated to the anchor level and is then backfilled after the anchor has been placed in position . A considerable amount of excavation may be needed depending on the anchor size and shape . The uplift capacity depends on the strength of the backfill.

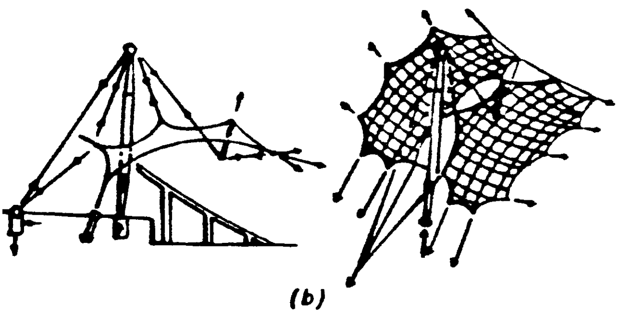
6). Blasting

In harder ground such as rocks or shales a hole or cavity is sometimes created to install an anchor . The hole is formed by blasting and a cement grout is injected under pressure into the hole to provide a base for the anchor rod or cable . However as a result of the explosion cracks may form in the rock surfaces which in turn will reduce the anchor capacity.

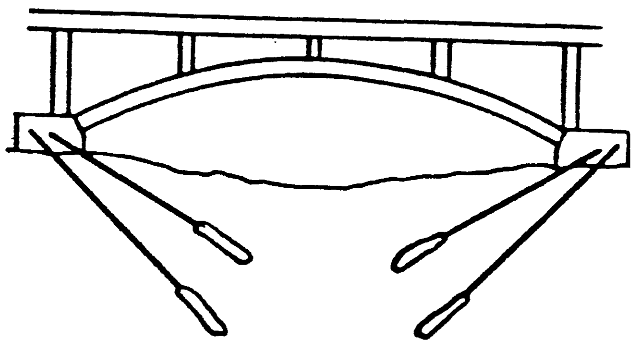




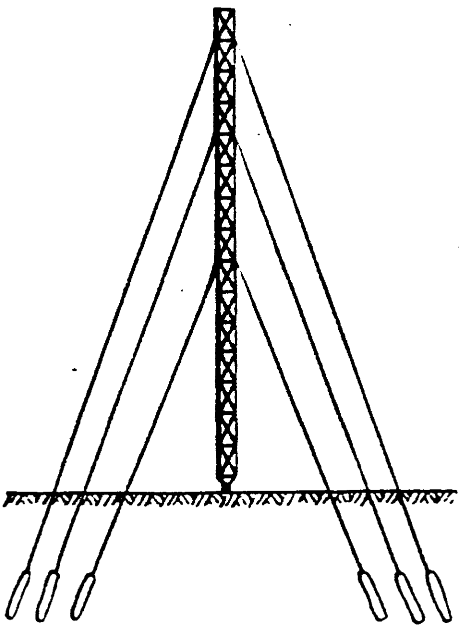
(a) Cable suspension



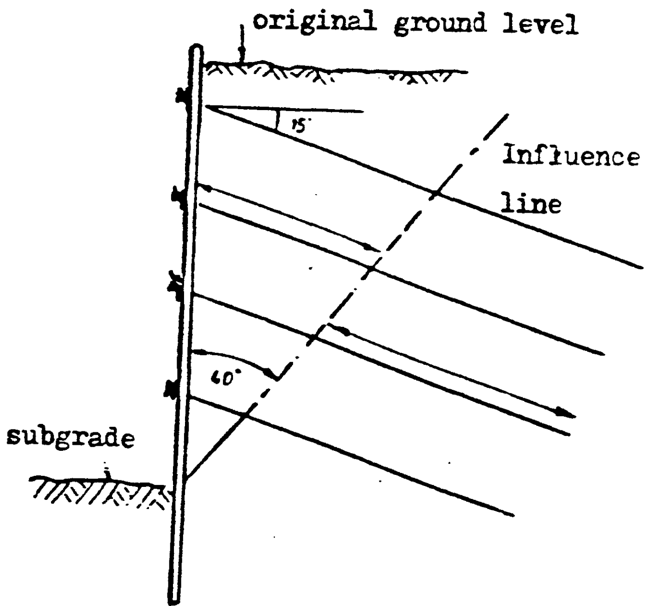

(b) Tension roofs for sports stadia and aircraft hangers



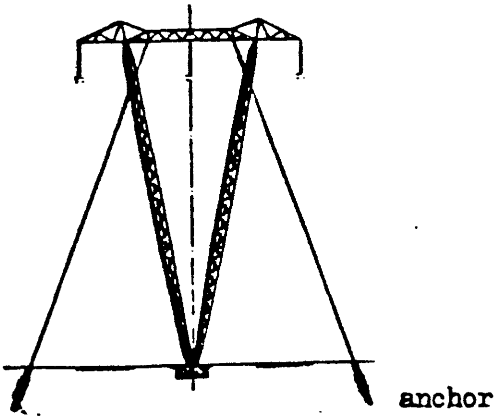
(c) Arch bridge



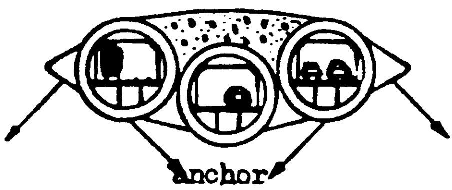
(d) Radio transmission mast




(e) Supporting deep excavation

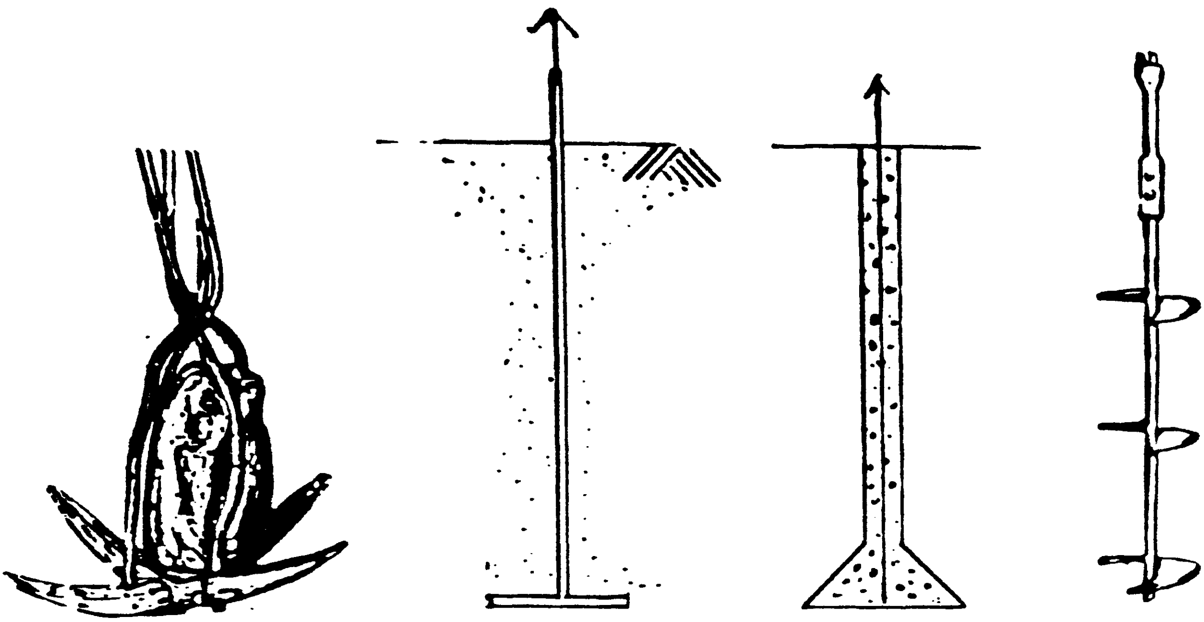



(f) Power transmission tower




(g) Submerged tunnel

Fig.1.1 Some uses of anchor in practice



(a) Dead-weight Anchor (b) Plate Anchor (c) Under-reamed Anchor (d) Helix Anchor

Fig. 1.2 Types of Ground Anchor

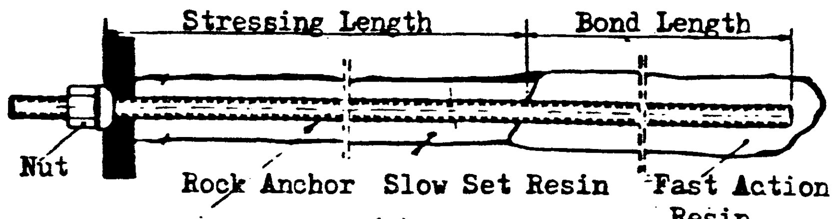
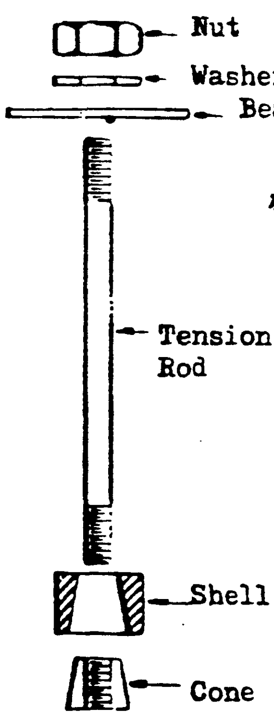



Fig. 1.3 (a) Grout Anchor



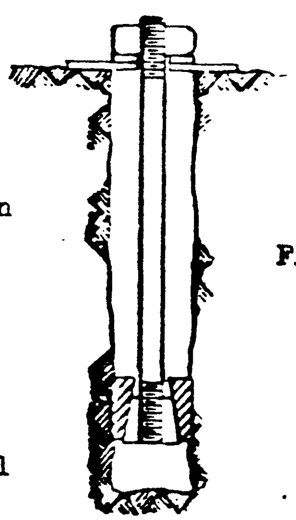
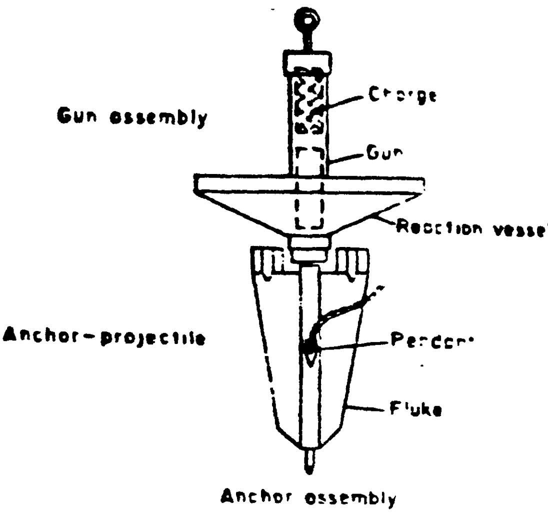
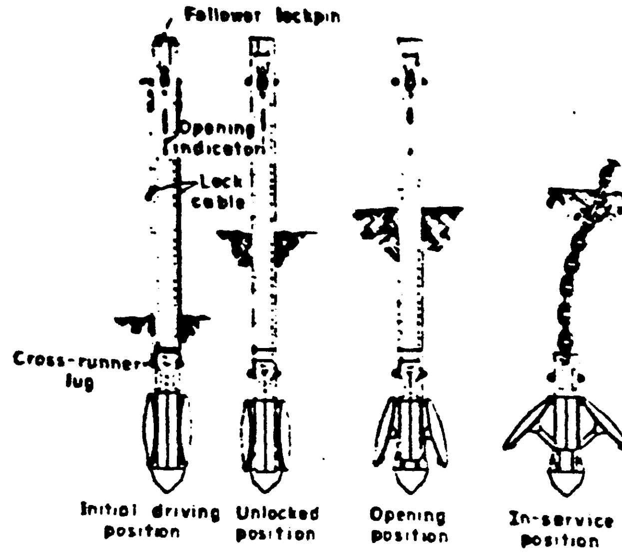



Fig. 1.3 (b) Rawl Bolt

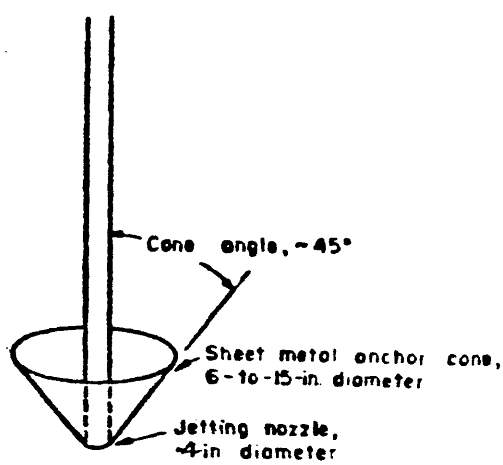
Fig. 1.3 Rock Anchors



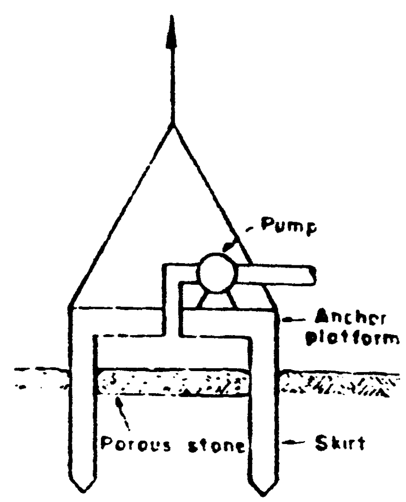

(a) Propellent-actuated
Anchor



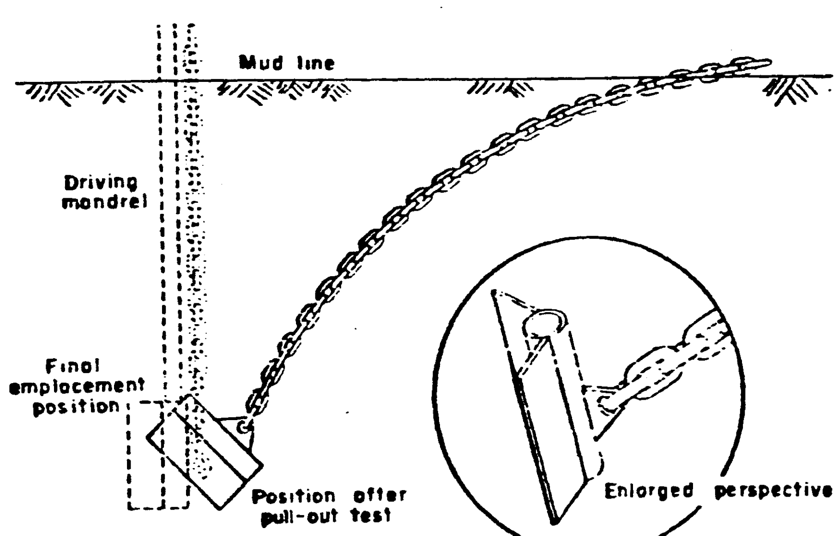

(b) Umbrella-pile Anchor




(c) Jetted Anchor




(d) Hydrostatic Anchor




(e) Rotating Plate Anchor ( after McCormick,1979)

Fig. 1.4 Some Types of Marine Anchor

# CHAPTER 2

## PREVIOUS THEORETICAL STUDIES AND EXPERIMENTAL INVESTIGATIONS

### 2.1 Introduction

Various theories on vertical circular anchor pullout capacities have been proposed by many investigators such as Balla ( 1961 ), Matsuo ( 1964 ), Meyerhof and Adams (1968 ), Mariupol'skii ( 1965 ), Fadl ( 1981 ) etc. These theories were all based on homogeneous soil conditions either in the model tests or in the prototypes.

As far as the author is aware, studies of the pullout capacity of an anchor embedded in a loose sand zone surrounded by a dense homogeneous sand bed are rare. To date only Kulhawy ( 1985 ) has discussed the variation in the horizontal stress on a backfilled spread anchor and has proposed guidelines for the design of such an anchor. Since this particular problem has not received much attention, no literature review which has a direct relevance to the present investigation is available.

In this chapter only a brief summary of uplift capacity theories on vertical anchors will be given. These are presented because of their possible relevance to the problem of the disturbance effect.

### 2.2 Balla's Theory ( 1961 )

The modern theory on anchor uplift capacity was started by Balla who showed that the breaking out earth body of a mushroom-shaped foundation resembled a solid of revolution with a rupture surface making an angle $(\pi/4 - \varphi^{\circ}/2)$ with the horizontal as shown in Fig. 2.1. Balla proposed a theoretical analysis based on a curvilinear failure surface and showed that the resistance was proportional to the third power of the depth of embedment.

Balla method's showed good agreement between his analysis and the full-scale test results on shallow anchors.

Baker and Kondner ( 1966 ) however showed that for depth/diameter ratio greater than 6, Balla's method was not applicable because it gave higher pullout load than that actually developed. They also reported that Balla's theoretical method was in good agreement with their experimental results for shallow anchors ( depth/diameter ratio less than 6 ).

Sutherland ( 1965 ) also indicated that Balla's method gave errors on the unsafe side for denser sands and errors on the safe side for looser sands.

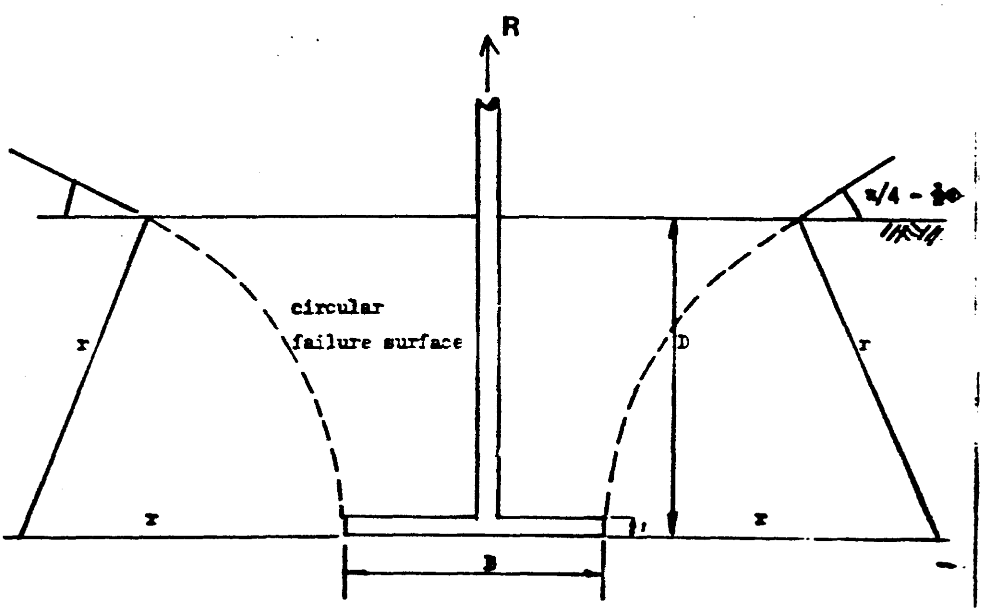



Fig. 2.1 Balla's Theory

## 2.3 Matsuo's Theory ( 1964 )

Matsuo assumed that the failure surface of the soil was composed of a logarithmic spiral at the lower part and its tangential straight line at the upper part as shown in Fig. 2.2. The upper straight line made an angle$(\frac{\pi}{4} - \frac{\varphi}{2})$at the ground surface.

Matsuo showed that the method was in good agreement with his model tests as well as in the prototypes. He pointed out that Balla's method gave higher results than his test results because in the Balla's method only the vertical component of the shearing forces was taken into account and the vertical component of the normal forces to the shear plane was neglected.

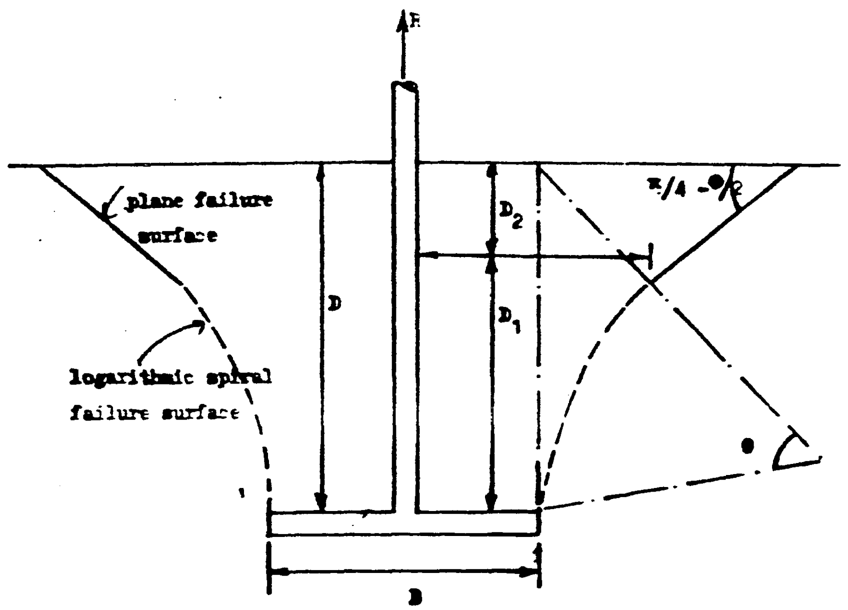



Fig. 2.2 Matsuo's Theory

## 2.4 Mariupol'skii ( 1965 )

Mariupol'skii assumed that the extracting force required to pull a vertical circular anchor from a soil mass gradually increased to a certain value R followed by a formation of a conical wedge above the anchor plate and the wedge pushed the soil above it apart to the sides allowing the anchor to move upwards under constant load as shown in Fig. 2.3.

He determined the state of stresses in the soil wedge above the anchor by assuming that the maximum shear stress was mobilised in every vertical cylindrical surface such as LM around the anchor axis as shown in Fig. 2.3 (a) and that failure occurred in tension at different points along a surface such as OMP at any time when the vertical shear force exceeded the shearing strength along the vertical cylindrical surface over which it was to be transmitted.

However Vesic ( 1972 ) pointed out that the assumptions made by Mariupol'skii in analysing the state of stresses in the soil wedge were in contradiction with the elementary theory of earth pressure.

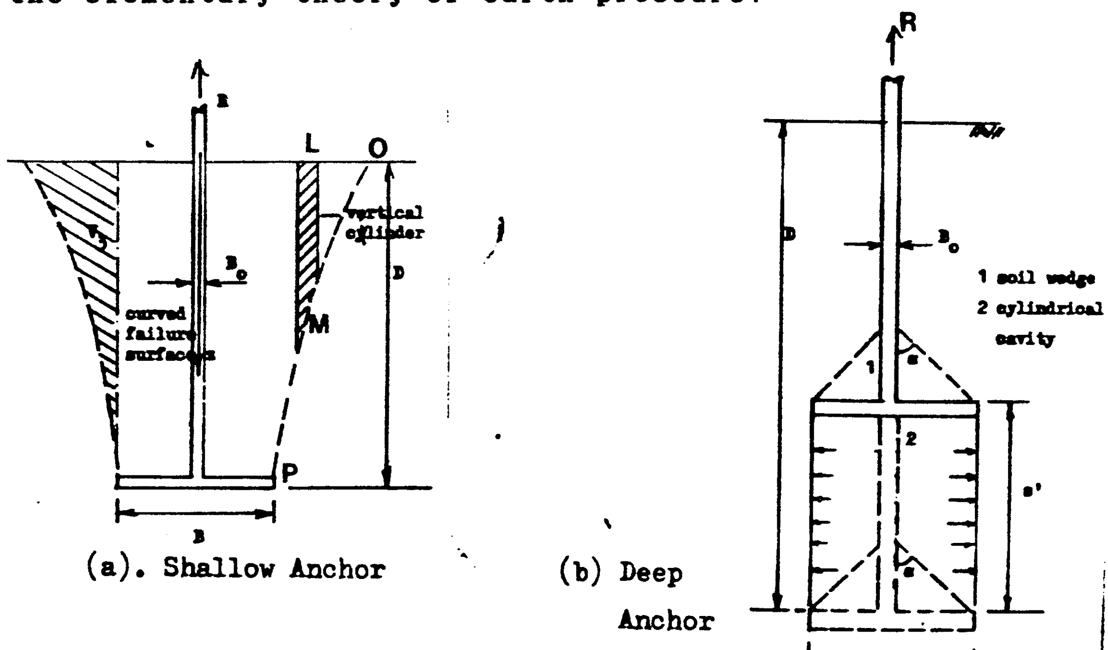



Fig.2.3 Mariupol'skii's Theory

## 2.5 Meyerhof and Adams's Theory ( 1968 )

The theory proposed by Meyerhof and Adams was originally developed for a strip footing but it was modified for circular and square footings by introducing shape factors to take into account the three-dimensional effect of individual footings. The shape of failure surface adopted by them is shown in Fig.2.4. Meyerhof and Adams introduced certain factors to be used in their theoretical uplift resistance from the experimental investigations. They also reported that there was a limiting value of Q which was equal to the bearing capacity of a footing under downward load. For a shallow vertical circular anchor in sand the formula was given as,

$$Q = \frac{\pi}{2} B \gamma D \; s \; k_{\mu} \tan\varphi + W$$

and for a deep vertical anchor the formula was given as,

$$Q = \frac{\pi}{2} ( 2D - H ) \gamma B \; H \; s \; k_{\mu} \tan\varphi + W$$

where,

Q = Uplift Resistance

D = Depth of Embedment

B = Diameter of Anchor

$\gamma$ = Unit Weight of Soil

$\varphi$ = Angle of Internal Friction

s = 1 + mD/B ( with a maximum value for deep footing of 1 + mH/B )

H = Vertical extent of the failure surface which was determined empirically and it was a function of $\varphi$ and B. The values of H/B are tabulated in Table 2.1.

$k_\mu$ = coefficient of lateral earth pressure during uplift

= $k_{pv}$ $\tan\varphi$ where $k_{pv}$ is the vertical component of the coefficient of passive earth pressure $k_p$

$k_{pv}$ = $k_p$ $\tan\delta$ and $\delta$ was approximated as $\delta = 2/3\varphi$

m is a coefficient depending on $\varphi$

| | | | | | | |
|---|---|---|---|---|---|---|
| Friction angle, $\varphi^\circ$ | 20 | 25 | 30 | 35 | 40 | 45 |
| Depth H/B | 2.5 | 3 | 4 | 7 | 9 | 11 |

Table 2.1 Vertical Extent of Failure Surface for Different Values of $\varphi$

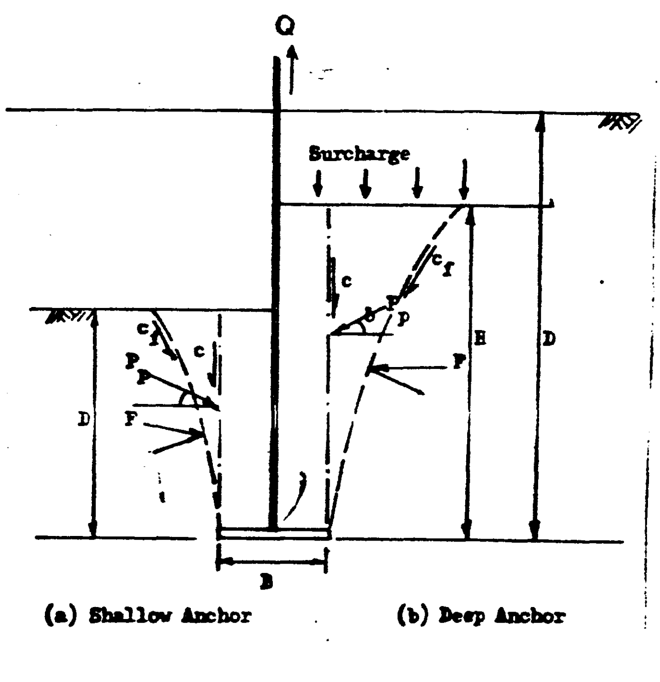



Fig. 2.4 Meyerhof and Adam's Theory

2.6 Khadilkar et al ( 1971 )

Khadilkar et al showed that the shape of the failure surface for circular under-reamed piles subjected to uplift loads was in the form of a log-spiral. The radius of the rupture surface zone on the ground surface calculated by their theory was close to Balla's approximation. The method proposed by Khadilkar et al was particularly useful for estimating the uplift resistance of under-reamed pile anchors at shallow embedment. However this type of anchor foundation was suitable in cohesive soils.

2.7 Wang and Wu ( 1980 )

From the tests carried out on vertical rectangular anchors in sand Wang and Wu found that for $\varphi = 35^{\circ}$, the peak anchor resistance was significantly greater than the residual anchor resistance up to a depth/ height ( D/h ) of about 10 where h was the least lateral dimension of the anchor. Above this ratio all anchor pressure displacement curves exhibited no peaks and approached a maximum value. They also showed that beyond the above ratio the anchor capacity was independent of the depth of embedment. Wang and Wu's method was consistent with Meyerhof and Adams's theory.

2.8 Fadl ( 1981 )

Fadl reported that no comprehensive series of tests had been carried out to study the anchor uplift capacity covering a wide range of relative densities and angles of orientation in a cohesionless soil. From his study using Leighton-Buzzard sand he adopted a simplified failure surface ( which was a straight line ) as shown in Fig. 2.5 making an angle $\alpha$ with the vertical through the anchor edge for a shallow and a deep anchor. $\alpha$ was defined in terms of relative density and $\varphi$ .

For a shallow vertical anchor the uplift resistance was given as,

$$P = \frac{\pi D \gamma}{12} \left( 8D^2 \tan^2\alpha + 12\ BD \tan\alpha + 3B^2 \right)$$

For a deep vertical anchor the the uplift resistance was given as,

$$P = \frac{\pi \gamma}{12} \Big[ 8H^2 ( 3D - 2H ) \tan^2\alpha + 12HB ( 2D - H ) \tan\alpha + 3DB + 6K_o ( D - H )^2 (B + 2H \tan\alpha) \tan \bar{c}\varphi \Big]$$

where,

$B$ = diameter of anchor

$D$ = depth of anchor embedment

$H$ = vertical distance of failure surface ( deep anchor )

$\gamma$ = unit weight of soil

$\alpha$ = $M\varphi$ and $M = 1/2[D_r(1 + \cos^2\varphi ) + (1 + \sin^2\varphi )]$

$\bar{C}$ = $D_r \cos\varphi$ and $K_o \simeq 1 - \sin\varphi$

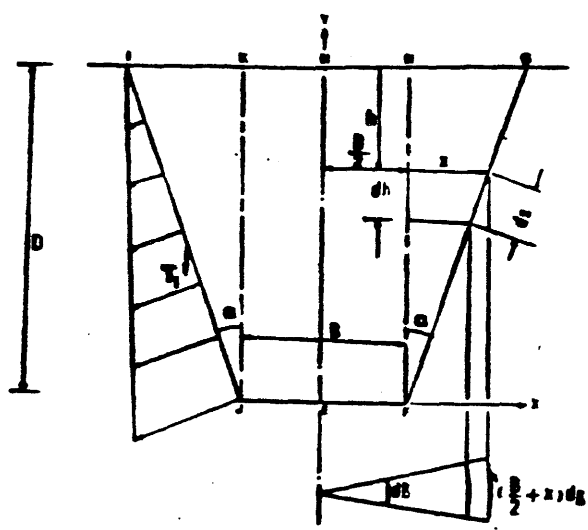
$\varphi$ = angle of internal friction

$D_r$ = relative density of sand

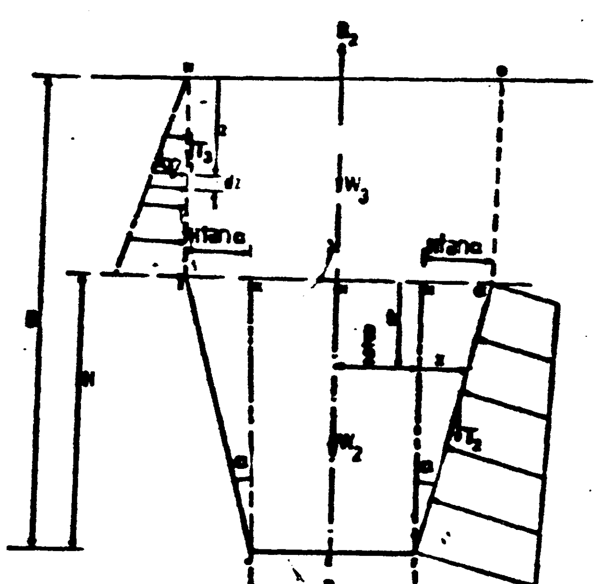
He made a clear distinction between a shallow anchor and a deep anchor the transition of which was defined by the critical depth ratio ( H/B ) where H was the vertical extent of the inclined failure surface as shown in Fig.2.5 .Fadl also emphasised the importance of relative density and critical depth ratio in his analysis which as he pointed out were not considered by previous investigators.

The correct value of $\varphi$ from the triaxial tests should also be used in the analysis because in the model tests the overburden stresses were lower than in the field.

Fadl's method was checked against previous model and field tests for shallow and deep anchors and good agreement was found. Comparison with other published data which took into account all the factors particularly to those of Bemben and Kupferman ( 1975 ) and Esquivel-Diaz ( 1967 ) showed that reasonable agreement was obtained.



(a) Shallow Anchor



(b) Deep Anchor

Fig. 2.5 : Simplified Failure Surfaces Adopted by Fadl

## 2.9 Vesic's Theory ( 1963, 1965, 1971, 1972 )

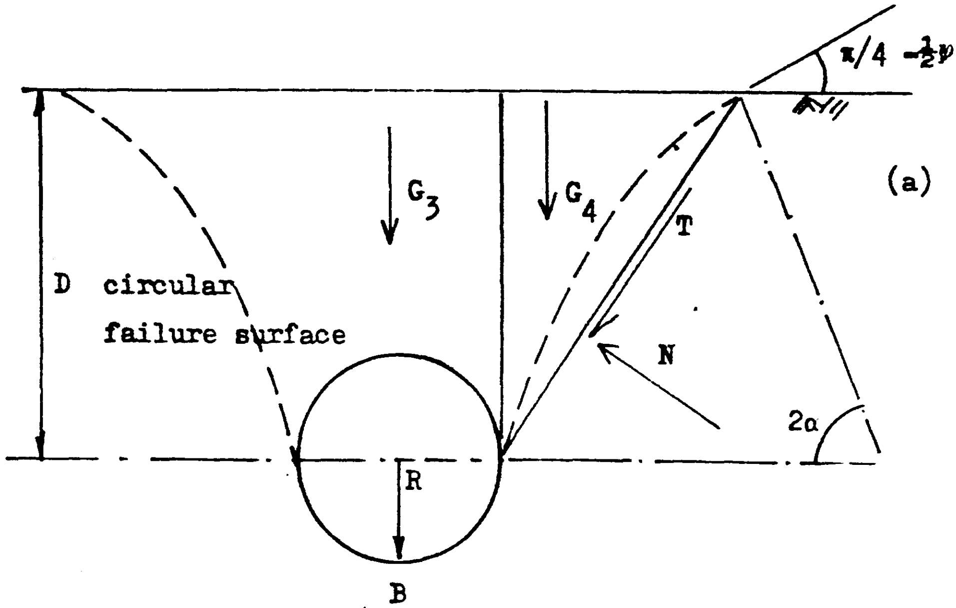
Vesic treated the breakout resistance of objects embedded in a soil mass as analogous to the expansion of cavities in an infinite soil mass. But as he pointed out this approach was not uncommon because Bishop et al (1945) Gibson ( 1950 ) and Chadwick ( 1959 ) had also used a similar technique.

Vesic assumed that as the anchor was being pulled an expansion of cavity was formed in an infinite homogeneous, isotropic soil mass and the extent of the cavity would depend whether the anchor was buried at shallow or great depth. The cavity was formed as the pressure was increased until equilibrium was achieved whereby the cavity would have an enlarged radius $r_u$ sustained by an internal pressure $P_u$ as given in Fig.2.6. The ultimate pressure $P_{uc}$ was given in terms of spherical cavity expansion factors $F_c$ and $F_q$ which could be obtained from charts.

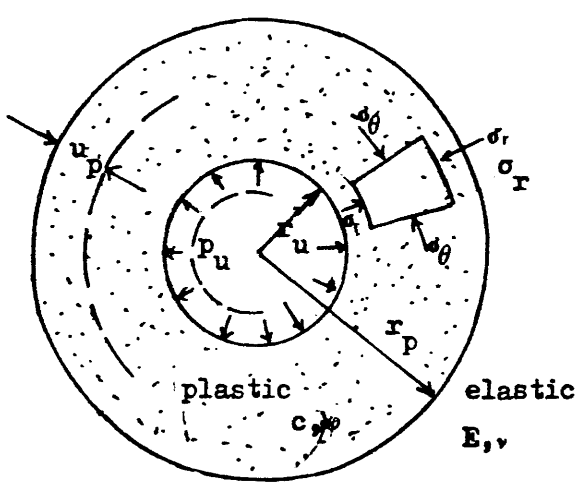
$P_{uc} = cF_c + DF_q$ where D was the anchor depth.

Vesic showed that the characteristic depth D/B ( B = anchor diameter ) beyond which the anchor plate started behaving as a deep anchor increased with relative density from about 3 for loose sands to over 10 for dense sands.

He also noted that the breakout factors for deep anchors were practically equal to the corresponding point bearing capacity factors for deep foundations. For example $F_q$ increased from 6 in loose sand to about 90 in dense sand as proposed by him earlier. The only chief disadvantage of this analysis was that the time effects on breakout were introduced only indirectly through strength and deformation parameters of soil.




(a). Expansion Of Spherical Cavity Close to Ground Surface




(b). Expansion of a Deep Spherical Cavity

Fig. 2.6 Vesic's Theory

## 2.10 Finite Element Analysis

The finite element method is fast becoming a powerful tool for the engineer to find solutions to many engineering problems encountered today. It was originally developed for use in structural analysis but further improvements and modifications have been made in the technique that it is now possible to solve a wide range of geotechnical problems by the finite element technique.

A vertical circular plate anchor is normally treated as an axi-symmetric problem. So far the problem of uplift anchor capacity in cohesionless soils has been concentrated on the basis that the soils were homogeneous and isotropic throughout the soil mass. The effects of boundary conditions were normally ignored because they were taken some distance away from the anchor axis. However these effects exist in the present investigation because the cylindrical loose sand zone is sandwiched between the dense homogeneous sand bed in any lateral direction. In this case the analysis requires different material idealisations within the affected zone.

Rowe and Davis ( 1982 ) proposed a numerical solution for a vertical anchor by considering an elasto-plastic finite element technique using the soil structure-interaction theory as given by Rowe et al ( 1976 ). The approach allowed the consideration of plastic failure within the soil, anchor breakaway from the soil and shear failure at a frictional, dilatant soil structure interface without the introduction of special joint of interface elements.

For deep anchors it was found that the deformation due to local yield might be sufficently large to necessitate the adoption of a practical definition of failure at loads below the collapse load. Comparison with their data showed that the theoretical analysis slightly overestimated the anchor capacity in most cases but it was generally small and agreement was considered acceptable.

The method was also used for different types of anchor such as grout anchors as attempted by Desai et al ( 1986 ) in relation to anchor-soil interaction analysis. Yap ( 1979 ) carried out a similar finite element analysis on the uplift resistance of a grout anchor in rock.

## 2.11 Dimensional Analysis

A dimensional analysis technique can be used to investigate the physical relationship between the parameters which govern the uplift capacity problem in the model as well as in the field.

Sutherland ( 1965 ) used this technique to study the problem of shaft raising through cohesionless soil at the Sizewell Power Station. By studying the parameters governing the uplift problem he obtained a relationship in the form ,

$$P_u/\gamma D = f\ ( D/B\ ,\ \varphi\ )$$

where $P_u$ was the ultimate pressure on the anchor. It follows from the above equation that for a particular value of $\varphi$ , the breakout factor $P_u/\gamma D$ depends only on depth/diameter ratio ( D/B ). Laboratory pullout tests on anchors were carried out by using circular plates with diameters ranging from 25mm to 150mm and depth/diameter ratios from 1 to 5. The anchors were tested in dry and submerged sand in both dense ( $\varphi = 45^{o}$ ) and loose ( $\varphi = 31^{o}$)

states . From the plots of $Pu/\gamma D$ against D/B the jacking force required to raise the shaft was predicted. The field tests gave consistent results when plotted on the same plots of $Pu/\gamma D$ against D/B for $\varphi$ of $42^{o}$ and $35^{o}$.

Baker and Kondner ( 1966 ) defined a shallow circular earth anchor in dense sand as the one which had a depth/diameter ratio of less than 6 and they gave a separate dimensionless relationship for a shallow and a deep anchor. Their method was in good agreement with Balla's method for shallow anchors ( depth/diameter < 6 ).

For a shallow anchor the dimensionless relationship was expressed in the form,

$$R/DB^2\gamma = 3 + 0.67 ( D/B )^2$$

For a deep anchor it was given as,

$$( R/B^3\gamma - 170 ) B/b = -2800 + 470 D/B$$

where,

R = ultimate load on anchor

B = diameter of anchor

D = depth of anchor

b = thickness of anchor plate

$\gamma$ = unit weight of soil

2.12 <u>Centrifugal Model Test</u>

A centrifugal model test is a new technique of establishing a relationship between a model and a prototype whereby the model when subjected to the correct scale under similar boundary conditions, the model experiences the same stresses as the prototype.

Centrifugal test methods provided a useful correlation between a model test and a prototype in sand as shown by Dickin and Leung ( 1983 ). In this test the dynamic similarity such as force between the model and the prototype was predicted more accurately. Perhaps the method can be extended to an anchor uplift resistance in a disturbed zone.

## 2.13 Laboratory and Field Studies of Anchor Resistance

Hanna et al ( 1972 ) reported that for a shallow anchor, the sand movements were near vertical but as the depth increased, the zone of sand movement was within the sand mass with no surface movement and the displacement radiated outwards from the anchor. Sand movement near to a loaded dead anchor and a prestressed anchor extended several anchor diameters away from the anchor, the magnitude and the extent of the movements being greatest in the prestressed anchor case. The study was however not extended to the effect of installation disturbance on the anchor.

Hanna and Carr ( 1971 ) also reported that at large burial depth to diameter ratio ( D/B ) the anchor ultimate load increased linearly with depth. This suggested that the bearing capacity of a deep anchor was similar to the bearing capacity of a deep foundation. Vesic (1972 ) had also showed a similar behaviour earlier for a deep anchor.

Robinson and Taylor ( 1969 ) showed that plate anchors were expensive to install because of the compactive effort and greater quality control needed during construction.

From a series of tests on statically loaded circular anchors Andreadis et al ( 1981 ) found that the mode of failure was governed by relative depth of embedment, soil density and anchor shape. The breakout factor and relative anchor movement to failure increased for a shallow anchor but approached an approximate constant for a deep anchor.

They also showed that significant horizontal stresses were generated at some distances from the anchor. The stresses changed from zero at the soil surface to maximum value and then decreased again to zero just below the level of embedment of the anchor. At distances greater than five anchor diameters, maximum horizontal stresses developed at a depth equal to half the embedment depth of the anchor.

Healy ( 1971 ) found that at a ratio of depth/diameter ( D/B )= 5 the pullout resistance of small anchors in the field ( i.e 150mm-diameter ) in sand varied directly with the overburden stress provided that the anchors were at least six anchor diameters below the surface in dense sand and two anchor diameters in loose sand.

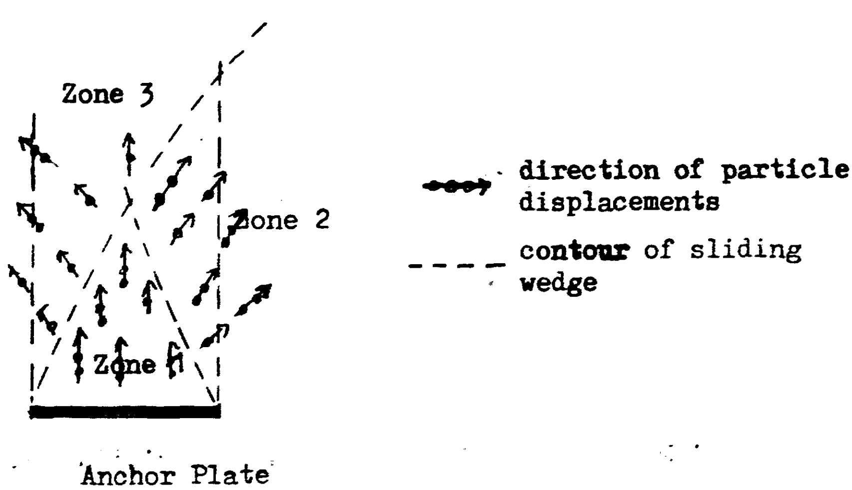
From a series of laboratory tests on a circular anchor in fine sand Kanayan ( 1966 ) found that when loads were about 50% of the ultimate loads, compaction extended almost through the founding depth. At loads close to the ultimate loads ( 70% - 80% ) heaving of the soil occurred preceded by the formation of radial cracks near the column of the anchor.

Koslov ( 1966 ) reported that as an anchor was being pulled out of a sand bed a compacted zone of sand was formed on the anchor plate cutting through the soil mass.

Kostyukov ( 1967 ) carried out similar tests in sand

and had divided the sand above the anchor plate into three sand zones according to their density as illustrated in Fig.2.7. The density of the sand and the directions of sand movement were monitored by radioisotope $Co^{60}$ and a radiometer device.

From his investigation, a triangular compacted zone of sand was formed above a vertical plate anchor. The sand particles radiated outwards as that triangular zone was being pulled upward.




N.B. Anchor shaft is omitted for clarity

Zone 1: zone of compacted sand

Zone 2: soil lying above the compacted zone

Zone 3: ground lying above the surface of sliding of the wedge

Fig. 2.7 Shape of Sliding Wedge and Kinematics of Sand Motion ( After Kostyukov, 1967)

Kulhawy (1985 ) has laid down tentative guidelines as shown in Table 2.2 for the evaluation of the horizontal stress for backfilled spread anchors. For neat excavation where the diameter of anchor was equal to the width of excavation the lower strength of either the backfill of the host soil governed the anchor capacity. He suggested the overall value of K ( coefficient of lateral earth pressure ) should be used for design purposes to evaluate the resultant horizontal stress where $K = K_h K_b$ ; $K_h$ and $K_b$ were the coefficient of lateral earth pressure of the host soil and the backfill respectively. For over-excavation where the width of excavation was greater than the anchor diameter the backfill normally controlled the anchor capacity.

| Type of Excavation | Backfill Compaction | Coeff. of Horizontal Soil Stress K: Host Soil | Coeff. of Horizontal Soil Stress K: Backfill |
|---|---|---|---|
| Neat excavation | Loose | $2/3\ K_o$ | $K_a$ to $K_{onc}$ |
| | Medium | $K_o$ | $K_{onc}$ to 1 |
| | Dense | $5/4\ K_o$ | 1 to $K_p$ |
| Over excavation | Loose | Normally does not control | $K_a$ to $K_{onc}$ |
| | Medium | | $K_{onc}$ to 1 |
| | Dense | | 1 to $K_p$ |

Note: $K_a = \tan^2(45-\varphi/2)$; $K_{onc} = 1-\sin\varphi$ ; $K_p = \tan^2(45+\varphi/2)$

$K_o$= in-situ horizontal stress coefficient

Table 2.2 Tentative Guidelines to Evaluate Horizontal Stress for Backfilled Spread Anchors.

2.14 Comments on Previous Theories on Anchor Pullout

1). It can be shown that the theories do not show good agreement and contradict each other. In Fadl's ( 1981 ) method for example, the pullout load obtained from his predictions was higher than the load obtained from Meyerhof and Adams's ( 1968 ) theory although the same anchor and soil parameters were used. This may be well understood because each method was put forward by using a particular type of sand under specific test conditions. Each theoretical method might agree well with a particular series of tests conducted. For example in Balla's method, his experimental results showed good agreement with his theoretical analysis for a very limited range but was in contradiction with Sutherland's findings.

2). Fadl's theoretical analysis gave results which were in reasonable agreement with the experimental results of El-Rayes( 1965), Sutherland( 1965), Bemben and Kupferman (1975) etc.

3). Although agreement was sometimes obtained between some methods it only applied to a limited extent. For example Baker and Kondner reported that Balla's theory only showed reasonable agreement when applied to their experimental results up to a depth/diameter ratio of 6.

4). Various approaches have been used by different investigators to derive their own theoretical analysis but it appears that the assumed failure surface ( especially in the form of a curve ) was the most commonly used such as Balla, Matsuo, Mariupol'skii etc. Perhaps Vesic's theory was an exception in that he considered the breakout resistance of an anchor to be analogous to the expansion of cavities in an infinite soil mass.

5). There was no common numerical basis for establishing the difference between a shallow and a deep anchor although generally it varied with relative density.

6). Centrifugal model tests provided a useful correlation between a model and a prototype. The use of this technique might have further applications with regard to the anchor uplift problem.

7). Recent developments have shown that the so-called 'conventional' anchor uplift analyses such as proposed by Balla, Matsuo, Meyerhof and Adams, Fadl and so on are being attempted by the use of computers. The use of computers has been proved useful and encouraging. A Computer-oriented approach had been adopted by Rowe and Davis (1982), Desai et al ( 1986), Yap ( 1979), Stewart ( 1973 ) etc.

8). No consideration has previously been given to the effect of installation disturbance on the anchor in relation to its load carrying capacity and displacement behaviour.

## 2.15 Scope of the Present Work

A review of the available literature on anchors revealed that, although tests have been carried out on models and at full scale, and various theories have been proposed for various designs, the role of the zone of disturbance set up on installing certain types of anchor has not received much attention. From a practical point of view this zone must exist to a greater or lesser extent, and the purpose of the present investigation was to determine the effect of the zone of disturbance created around a vertical plate anchor in sand on the pullout capacity.

The investigation was divided into two parts. Firstly pullout tests on an anchor were carried out in homogeneous sand beds at unit weights of 17.14,16.40 and 15.75 $KN/m^3$ of relative densities 92.0%, 70.0% and 49.0% respectively. The unit weight of the loose sand bed ( R.D. = 49.0% ) was used as a basis for the formation of the cylindrical disturbed sand zone within the dense bed itself. The aim of the tests was to provide the upper and lower limits of anchor pullout capacity between the dense and loose sand. The depth/diameter ratios ( D/B ) were in the range from 3 to 15.

To simulate the possible disturbance caused by the installation procedure, zones of disturbance were created by forming a loose volume of sand above the anchor position within a tank filled with dense sand.

In the second part of the investigation ( which was the major part ) pullout tests were carried out on an anchor embedded at a depth D in a disturbed zone of width $B_z$ as shown in Fig. 2.8. The width of the disturbed zone was increased to a certain proportion of the anchor diameter and in this investigation it was increased from 1 to 3 anchor diameters. The anchor was embedded at ratios similar to the above. The loose sand volume in the zone was kept at approximately 15.75 $KN/m^3$ throughout the course of the investigation which in fact represented the unit weight of the loose, homogeneous sand bed used previously.

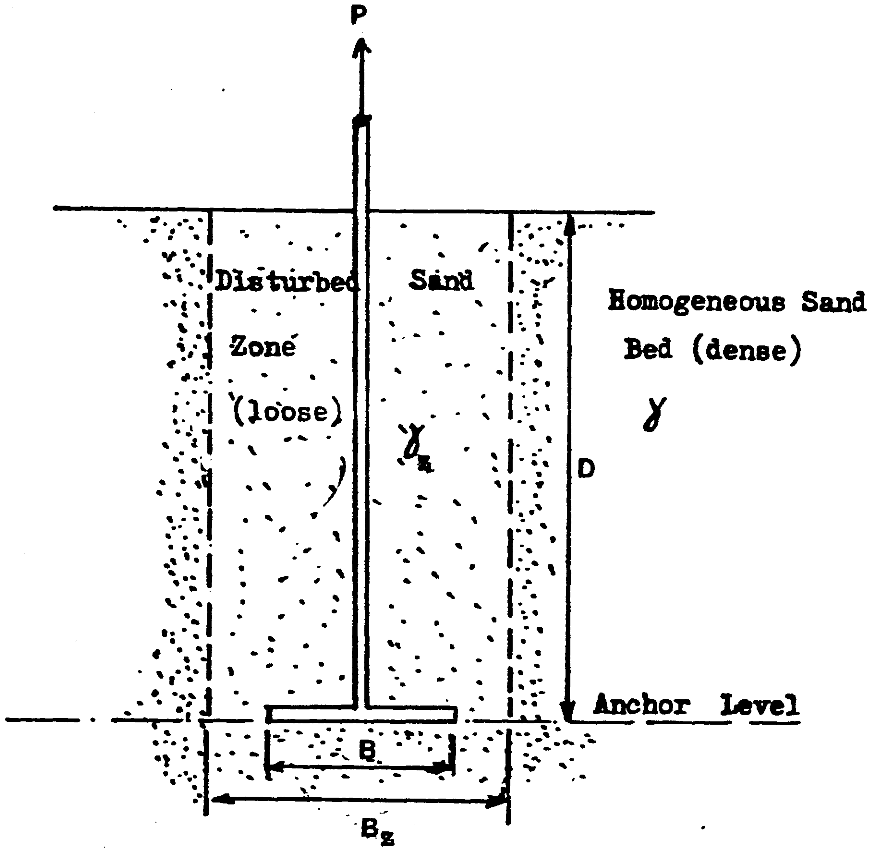



Fig. 2.8 The Problem

# CHAPTER 3

## PROPERTIES OF SAND AND SAND BED FORMATION

### 3.1 Introduction

In a soil mechanics laboratory it is sometimes necessary to prepare a uniform sand bed of granular material for testing model piles,anchor foundations etc .As granular materials are frictional and composed of individual and discrete particles, it is essential that the parameters governing the soil properties be kept constant during such an operation. The factor which governs the uniformity of the sand bed i.e. porosity should be taken very seriously (Butterfield and Andrawez,1970). In the present research the porosity of the bed was controlled during deposition as will be described later in this chapter.

### 3.2 Properties of Sand Used

In the research Leighton-Buzzard sand was used throughout as it is a common sand used in research (Fadl,1981). A granular material was selected because the method used to simulate the effect of disturbance in the laboratory was convenient and simple. A bed of such material can be produced in a variety of densities without much difficulty and the uniformity and porosity can be determined quite accurately. Furthermore any undue disturbance on the anchor during its installation in the model test could be minimised.

In the laboratory the properties determined were those which were relevant to the present investigation.

### 3.2.1 Particle Size Analysis

The test was carried out in accordance with BS 1377:1977. Fig. 3.1 shows the particle size distribution curve of the sand. The sand had a particle size range of 2.0 - 0.2mm., a uniformity coefficient $C_u$ = 1.46 and a mean diameter $D_{50}$ = 0.875mm. The grains were predominantly rounded.

### 3.2.2 Specific Gravity

The specific gravity $G_s$ of the sand particles was found to be 2.65. The major constituent of the sand was quartzite.

### 3.2.3 Porosity Limit

Following the methods suggested by Kolbuszewski (1948) the maximum and minimum porosities in the loosest and densest states were found to be $n_{max}$ = 44.30% and $n_{min}$ = 32.70%.

### 3.2.4 Shear Strength of Sand

The shear strength of the sand was determined by carrying out quick undrained triaxial tests as per Bishop and Henkel (1962). The purpose of the tests was to find the graphical relationship between the porosity and angle of internal friction of the sand as shown in Fig. 3.2.

## 3.3 Apparatus for Forming Uniform Sand Bed

### 3.3.1 Factors Affecting the Uniformity of a Sand Bed

Kolbuszewski and Jones (1961) showed that the density of a sand bed produced was a function of the intensity of deposition and the height of free fall of sand from a hopper into the sand surface in a receiver tank. In the

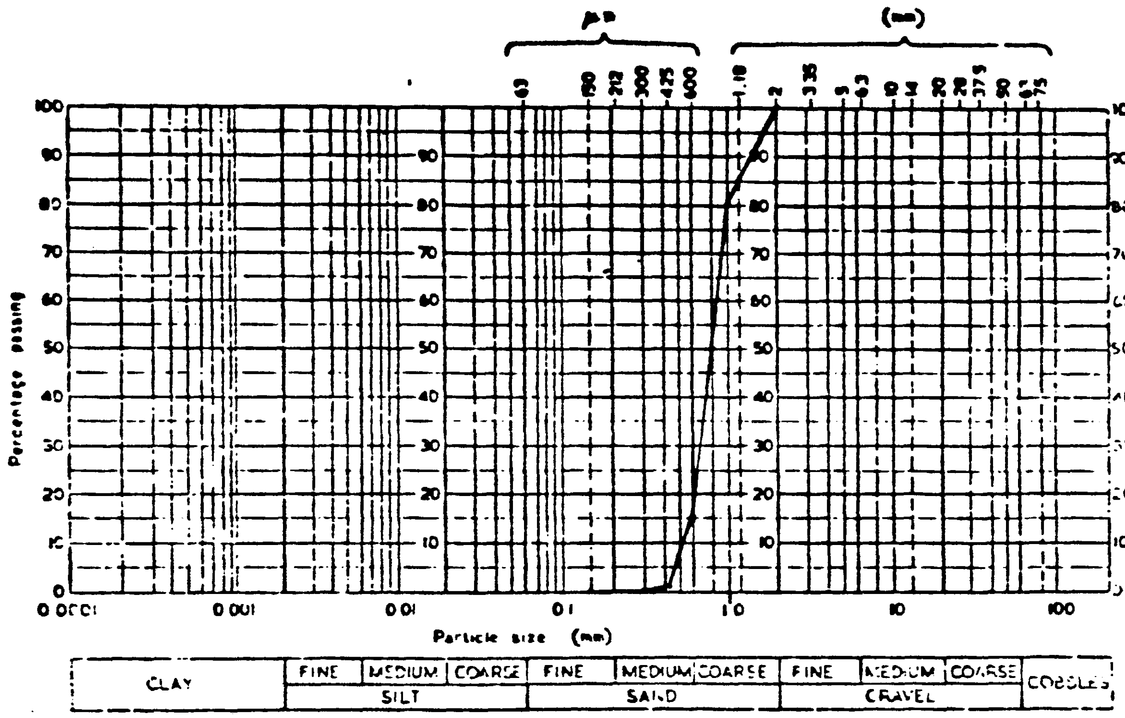



Fig.3.1 Particle Size Distribution of the Sand

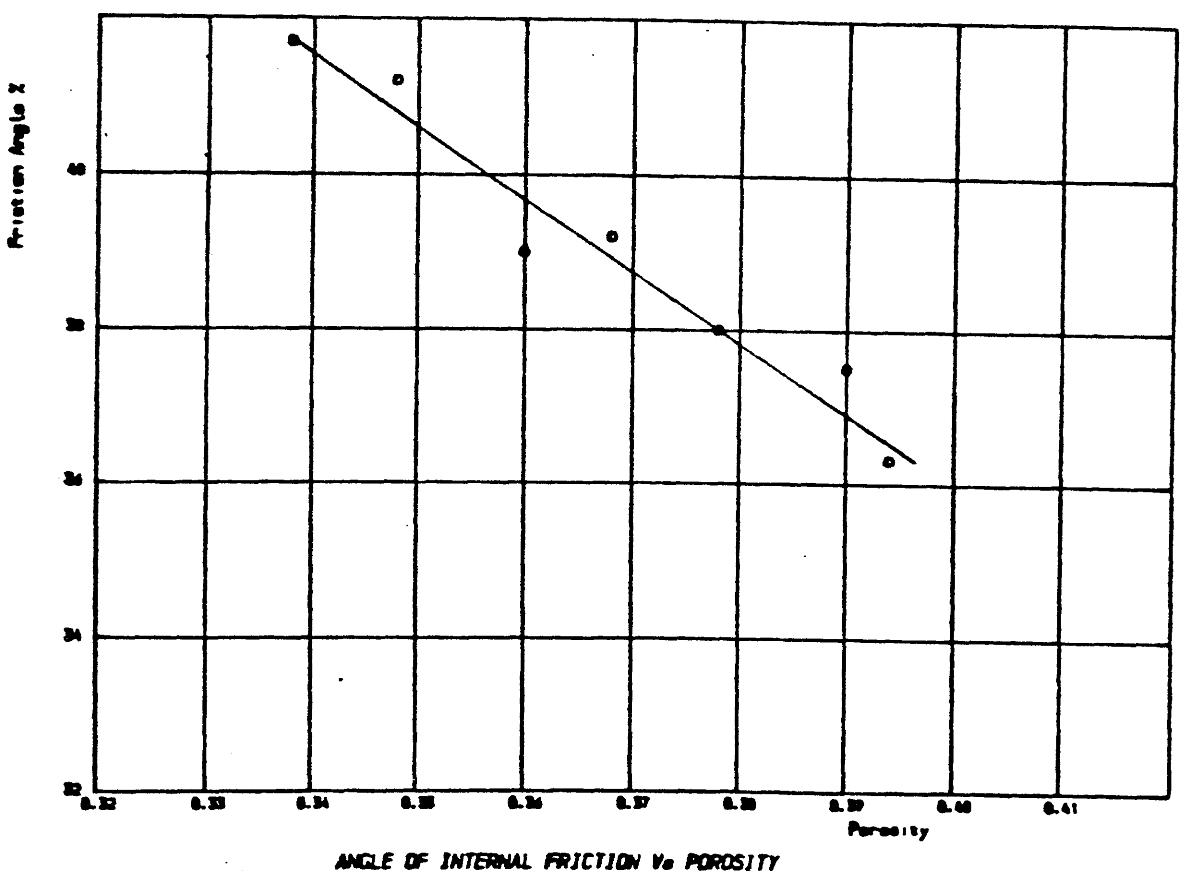



Fig. 3.2 Variation of Angle of Internal Friction With Porosity

present investigation the intensity of deposition was varied by using various plate apertures while the height of fall was adjusted by lowering or raising the frame supporting the hopper (see Fig. 3.3 ).

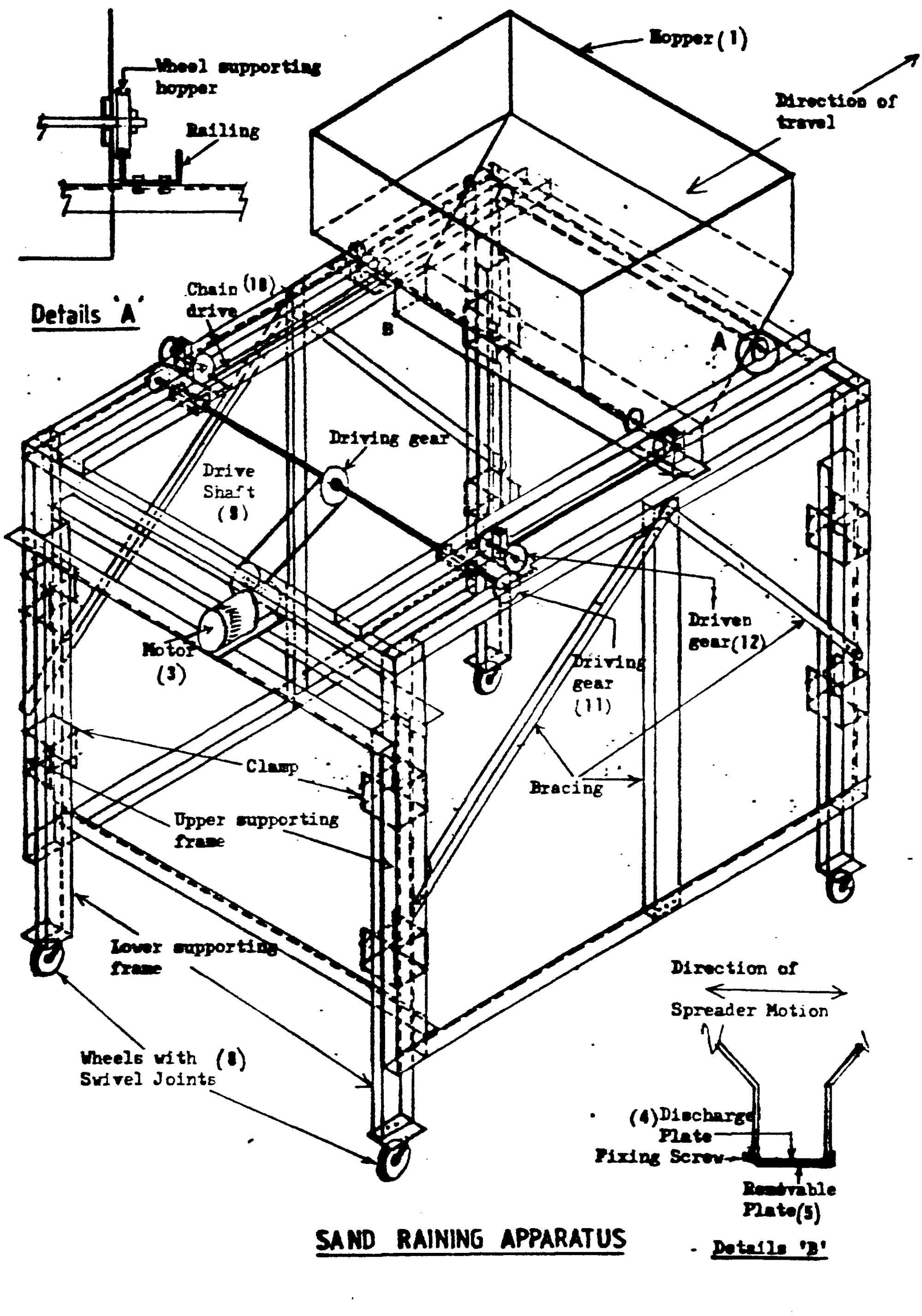
The rate of deposition could be increased by using larger aperture sizes. Although sufficient energy was available for dense packing when the velocity was high ( at high deposition ) there was insufficient time available for the dense packing due to the 'locking' action of the newly arrived particles (Kolbuszewski , 1948).

With regard to the height of fall of sand under gravity , there is a limit for the velocity of fall called the terminal velocity beyond which the increase in height will have no significant effect on the velocity of fall.

### 3.3.2 Construction of Apparatus

The sand raining apparatus is as shown in Fig. 3.3.

1. Hopper to discharge sand
2. Sand tank, dimensions 800mm square section and 650mm deep.
3. Motor drive, endless chain drive to enable hopper to move forward and backward.
4. Rectangular perforated discharge plates measuring 100mm wide wide and 820mm long drilled on 20mm grids fixed to the bottom of the hopper to produce sand rain. Three discharge plates were used each having perforations of 4mm, 7mm and 10mm diameter and giving three rates of deposition namely low, medium and high which produced dense, medium dense and loose beds in the context of the research.
5. Removable plate to retain sand while the hopper was




SAND RAINING APPARATUS

Fig. 3.3

being filled.

6. Collector boxes mounted on the opposite sides of the tank in the direction of spreader motion to collect overspill during filling of the tank.
7. Jacking system allowing the height of the hopper to be adjusted to maintain a constant height of fall of sand.
8. Wheels with swivel joints supporting the whole frame.
9. Drive Shaft
10. Chain Drive
11. Driving Gear
12. Driven Gear

The present apparatus was formerly used by Whiteford,1983 and Wang (1986) to form a uniform bed in a prepared container. The hopper was mounted on a prefabricated angle iron frame which was made up of two parts, upper and lower. The height of the upper frame which supported the hopper could be adjusted by turning each in turn a nut placed inside the support pillars of the frame.

A system of endless chain driven by gears moved the hopper forward and backward at a speed of about 0.1m/sec. A shaft which drove the gears extended the full width of the frame and was operated by an electric motor. The whole assembly was supported on wheels with swivel joints so that it could be moved easily. A detailed calibration for the apparatus is described in the next section.

### 3.4 Calibration of Sand Spreader

In order to assess the reproducibility and repeatability of the density of the sand bed it was important to calibrate the spreader. There are several ways in which the calibration of a sreader can be performed. While the design of each apparatus may vary, the factor which determines the satisfactory performance of such a spreader is the ability to produce a consistent density bed throughout its depth in a particular container.

By virtue of the construction of the apparatus the maximum height of the sand bed that could be produced was limited to 375mm excluding the 25mm thick of sand layer permanently laid at the bottom of the tank. The minimum height of drop of sand available was 66mm for ease of operation. If a greater drop was adopted then the total height of bed that could be produced would subsequently be reduced because there was limitation in the height by which the spreader could be raised.

There were two ways to measure the density of the sand bed.

1.Weight Method

2.Volume Method

#### 3.4.1 Weight Method

Three rectangular steel discharge plates were used to form the bed in the tank as given in Section 3.3. The perforations were drilled on a 20mm grid so that the sand would be equally discharged into the tank. The size of the perforations gave beds of dense, medium dense and loose sand of relative densities 92.0% , 70.0% and 49.0%

Table 3.1 Summary of Density Test Results for a Height of Fall of 660mm by Weight Method

| | Pot No. | Unit Weight ( $KN/m^3$) | |
|---|---|---|---|
| Test No. 1 | 13 | 17.10 | |
| Plate Aperture | 7 | 17.10 | |
| = 4mm-Diameter | 14 | 17.15 | |
| | 4 | 17.14 | Mean Value = |
| | 11 | 17.13 | 17.11 $KN/m^3$ |
| | 8 | 17.12 | |
| | 1 | 17.11 | |
| | 2 | 17.14 | |
| | 3 | 17.13 | |
| Test No. 2 | 11 | 16.72 | |
| Plate Aperture | 13 | 16.80 | |
| = 7mm-Diameter | 14 | 16.78 | |
| | 7 | 16.80 | Mean Value = |
| | 8 | 16.78 | 16.40 $KN/m^3$ |
| | 9 | 16.77 | |
| | 1 | 16.66 | |
| | 2 | 16.63 | |
| | 3 | 16.58 | |
| Test No. 3 | 1 | 15.63 | |
| Plate Aperture | 2 | 15.57 | |
| = 10mm-Diameter | 3 | 15.70 | |
| | 4 | 15.75 | Mean Value = |
| | 7 | 15.89 | 15.75 $KN/m^3$ |
| | 9 | 15.84 | |
| | 11 | 15.67 | |
| | 13 | 15.99 | |
| | 14 | 15.80 | |

respectively in the context of this research. The height of drop was set at 660mm i.e. the minimum available for ease of operation. A series of pots were placed on the sand layer at certain points forming a grid of equally spaced pots in the direction of spreader motion.The pots had a diameter of 76mm and internal depth of 51mm with a knife-edged upper rim to prevent bouncing of sand particles into them. After depositing the sand the pots were carefully removed from the bed and the excess sand removed. Knowing the mass of the sand collected in the pots the density of that layer was evaluated.

To proceed to the next layer the previous layer had to be made good and the same pots were placed on the current surface. The spreader was accordingly raised by 75mm to correspond to the increase in sand layer depth in the tank so that a constant height of drop could be maintained.The height of drop was taken as the vertical distance between the discharge plate to the mid-height of every layer produced. The procedure was repeated until the required depth of bed was achieved. The summary of the results is shown in Table 3.1.

3.4.2 Volume Method

Knowing the mass of the sand put into the tank and measuring the volume of the sand produced, then by simply dividing the mass by the volume measured the density of the whole bed could be evaluated. The advantage of this method over the previous one was that it allowed the calculation of density of the bed as it built up in the tank without disturbing the bed at any time. Provided the bed was fairly flat this method could be used satisfactorily.

The plan area of the tank was divided into several grid lines 100mm apart orthogonal to each other. The initial vertical distance of the grid points on the bottommost layer from a reference datum, in this case, the top of the tank was taken. When the next layer was laid the vertical distance of the same grid points was again taken from the reference datum. By substracting the two sets of values and using Simpson's Rule the volume of the newly laid layer could be found. Knowing the mass needed to produce the layer the density of that layer could be determined. By continuing the process the density of the whole bed could be evaluated as well as the individual layers. Depending upon the number of points taken the calculations involved were quite lengthy.

As the rate of deposition increased the bed produced an undulating surface especially at the edges of the tank where the spreader changed its direction of motion. Under these conditions the method became unreliable.

| Layer | Cumulative Thickness(mm) | Mass (kg) | Cumulative Volume ($cm^3$) | Unit Wt ($KN/m^3$) |
|---|---|---|---|---|
| 1 | 76.29 | 85.597 | 48828 | 17.20 |
| 1 to 2 | 153.41 | 171.421 | 98178 | 17.13 |
| 1 to 3 | 229.43 | 256.618 | 146829 | 17.14 |
| 1 to 4 | 305.96 | 342.093 | 195811 | 17.14 |
| 1 to 5 | 382.52 | 427.352 | 244808 | 17.13 |
| 1 to 6 | 443.24 | 495.500 | 283672 | 17.14 |

Table 3.2: Variation in Unit Weights of Bed by Volume Method (Dense Bed only )

Table 3.2 gives the the summary of unit weights obtained by the volume method for the dense bed only. In the volume method the final density of the whole bed was calculated as 17.14 $KN/m^3$.

The final unit weight obtained for the whole bed by the volume method was 17.14 $KN/m^3$ while the corresponding value obtained by the weight method was 17.11 $KN/m^3$ a difference of 0.17%. However it should be noted that the unit weight obtained by the weight method was the average value produced from a height of 660mm. On the other hand, in the volume method each unit weight given in Table 3.2 showed the unit weight of the current bed as it built up in the tank. Therefore provided the sand surface was fairly flat especially for the dense bed in this investigation, the volume method was more accurate than the weight method.

## 3.5 Preparation of Undisturbed Sand Bed

The supporting cross-beam together with the pullout machine mounted on it was removed temporarily from the support column to provide space for the spreader. An angle steel frame was specially fabricated to provide support for the beam and it was very convenient since the frame could be easily moved about on the floor.The spreader was placed vertically over the tank so that during filling it would travel the whole width of the tank and avoid unnecessary overspilling at the edges.

The sand bed was laid in layers and to form a 75mm thick of bed in the tank approximately five buckets of sand were required. The supply of sand was obtained from a large container placed beside the test rig. The cover plate was securely attached while the hopper was being filled. Carefully and quickly the cover plate was removed and the drive motor was switched on simultaneously to enable the sand to drop down while the spreader was in the forward and backward motion.

The first layer thus formed was meant for the foundation of the anchor so that the bed extended slightly below the anchor level. After determining the exact spot for the anchor (approximately at the centre) the anchor together with its shaft was placed at that position. It was very important to have the anchor placed at that position otherwise after the bed had been completed the anchor shaft could not be attached to the connecting rod of the load cell vertically and accurately.

After each batch of filling a little sand was still left on the discharge plate by virtue of the arrangement of the holes. This sand was removed immediately after each deposition. The spreader was again filled with sand and the method of laying was repeated until the required depth of anchor embedment was achieved. After the bed was completed the spreader was pushed aside to provide room for fixing the loading arrangement for the testing of the anchor.

### 3.6 Method of Forming Disturbed Zone

The disturbed zone was prepared in such a way that its density was always less than the surrounding bed. The density of the zone would always correspond to the density of the minimum possible loose undisturbed sand bed used in the earlier part of the research i.e. assuming the bed was disturbed throughout.

Three methods were investigated for creating the disturbed zone around the anchor.

Method_1

The homogeneous sand bed was first laid to the required depth as described in Section 3.5. A knife-edged tube of specific length (greater than the anchor depth anticipated) was driven or pushed into the bed to the anchor level at the right position. The sand in the tube was removed and the anchor was then placed in position.Sand was poured into the tube by means of a perforated pot.The tube was then withdrawn from the bed.

In this method the soil surrounding the anchor had already been disturbed during driving of the tube even prior to placing the anchor and the loose sand. It resulted in the disturbed zone being larger than the actual width expected. Correlation between $B_z$ and B could not be determined accurately.

Method_2

The first layer for the anchor foundation was laid and a tube long enough to extend beyond the anticipated depth of embedment was placed on the layer at the anchor position. Its open end was covered to prevent the falling sand from entering the tube while the bed being laid. When the laying was completed the anchor was placed in the tube which was then filled with loose sand as above. The tube was then withdrawn.

Since the anchor was embedded at a certain depth it was difficult to pull the tube without disturbing the sand around it as lateral pressure and frictional forces built up. The tube needed to be turned a little in order to pull it out but in so doing the sand around it was disturbed.

Comment

In view of the shortcomings of the previous two methods, a third method was tried and eventually adopted as being the one which gave the desired results i.e. a well-defined boundary between the disturbed and undisturbed zone around the anchor. The method involved building up the loose sand volume in layers as described below and illustrated in Fig. 3.4.

The foundation for the anchor was prepared first. After deciding on the position of the anchor, the anchor attached to its shaft was placed at that position. An open-ended tube about 100mm long was placed on the foundation enclosing the anchor completely. A similar tube called a collecting tube having a hole at its bottom end to accommodate the anchor shaft was placed on the lower tube already positioned to prevent the sand from embedding the anchor.

Sand was rained from the spreader to form the undisturbed bed in the tank but leaving only the anchor unembedded. The sand retained in the collecting tube was removed by suction to prevent any disturbance to the anchor. The collecting tube was removed temporarily .

A similar tube to the collecting tube but with perforations in its base was used to form a loose volume of sand in the tube enclosing the anchor. In this way the level of the loose sand in the tube could be checked

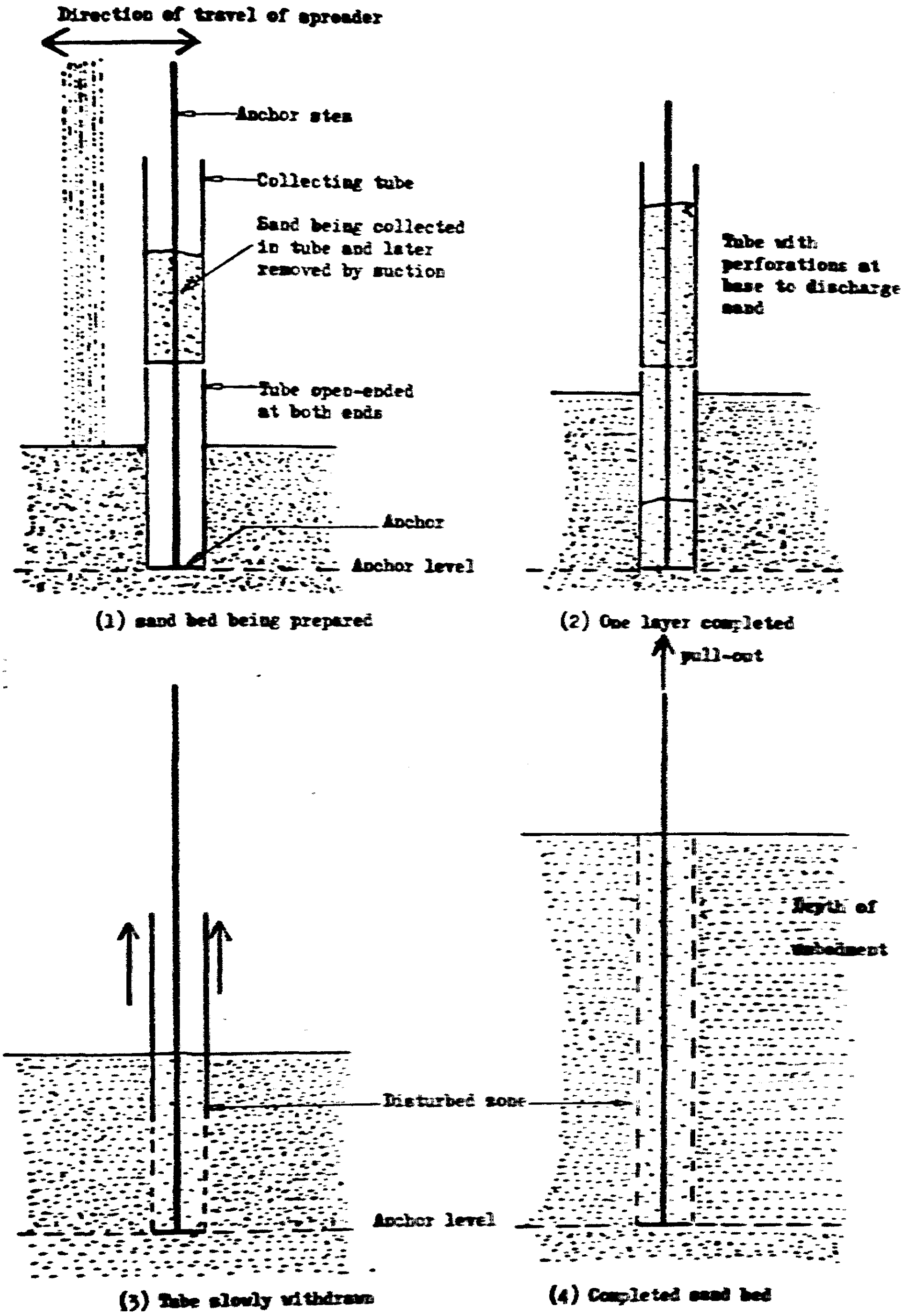



Fig.3.4 Method of Forming Disturbed Zone

against the surrounding bed. It was then slowly removed and the tube enclosing the shaft was raised to the next surface as illustrated in Fig.3.4.

The procedure was repeated until the depth of embedment was achieved. The tube was then completely withdrawn from the bed leaving only the embedded anchor in a 'loose' sand zone surrounded by the dense homogeneous sand bed. The anchor was now ready for testing.

<u>Note:</u>

1. The length of the tube enclosing the anchor was between 75mm and 100mm long and was as light as possible. A longer tube would have been difficult to withdraw from the bed as lateral pressure developed in the sand mass and it would inevitably cause undue disturbance to the sand bed.

2. In addition, the tube was as thin as possible and its inner and outer surfaces were smooth so as to reduce friction in the sand. A thick tube was seen to form a distinct layer between the loose sand and the homogeneous sand bed while it was being withdrawn. In the experiment all the tubes used had a thickness of less than 0.5mm.

3. The collecting tube was not removed while sand was still in it because in so doing, friction developed between the shaft and the sand especially at the upper threaded end of the shaft. It was found that it was very difficult to perform the task unless the sand was removed first by suction.

### 3.7 <u>Controlling the Density of the Disturbed Zone</u>

It was essential to control the unit weight of the loose sand volume (disturbed zone) so that it always corresponded to the unit weight of the loose homogeneous sand bed which served as the lower limit of the pullout curve. In this case the tube or pot used to form the loose sand volume was calibrated so that whatever the size of the tube used it would produce the same density. If the same aperture had been used as the tube increased in diameter it would have produced a denser sand volume (Kolbuszewski,1948).

In this connexion a few trial and error tests were carried out using various apertures/openings for the tubes having diameters equal to that of the intended disturbed zone. Accordingly as the tubes increased in diameter the apertures had to be enlarged in order to give the same unit weight for the disturbed zone. It was assumed that if in those tests the unit weight obtained was approximately 15.75 $KN/m^3$ i.e. which corresponded to unit weight of the loosest homogeneous sand bed available, then by simulating the same principle at the time the disturbed zone was being formed, it would give the unit weight. Nevertheless it was difficult to actually measure or check the density of the disturbed zone in the tank after it had been laid and the anchor had been installed.

# CHAPTER 4

# TESTING OF ANCHORS

## 4.1 Introduction

The methods of forming undisturbed and disturbed sand beds have been discussed in Chapter 3. It has also been shown that the spreader was capable of producing consistent density beds throughout its depth in the tank within a range of relative densities from 49.0% to 92.0% for a height of fall of 660mm. In this chapter two methods of load test on the anchors i.e. load-controlled and displacement-controlled with their respective advantages and disadvantages are presented.

## 4.2 Anchor Test Programme

The load tests on the anchors were divided into two parts and they are:

1). Tests in Undisturbed Sand Bed

In these tests the anchors were pulled out from homogeneous sand beds of relative densities 92.0%, 70.0% and 49.0%. The purpose of the tests was to obtain the upper and lower limits of the anchor pullout capacity between the dense and loose sand beds. The anchor planned test programme is as shown in Table 4.1.

| D (mm) / B (mm) | 75 | 112.5 | 150 | 187.5 | 225 | 300 | 337.5 | 375 |
|---|---|---|---|---|---|---|---|---|
| 25 | 3 | | 6 | 7.5 | 9 | 12 | | 15 |
| 50 | | | 3 | | | | | 7.5 |
| 75 | | | | | 3 | | 4.5 | |

Table 4.1 Anchor Planned Test Programme

Table 4.1 shows the planned anchor test programme and indicates that a range of D/B values from 3 to 15 was covered using B values ranging from 25 to 75mm.

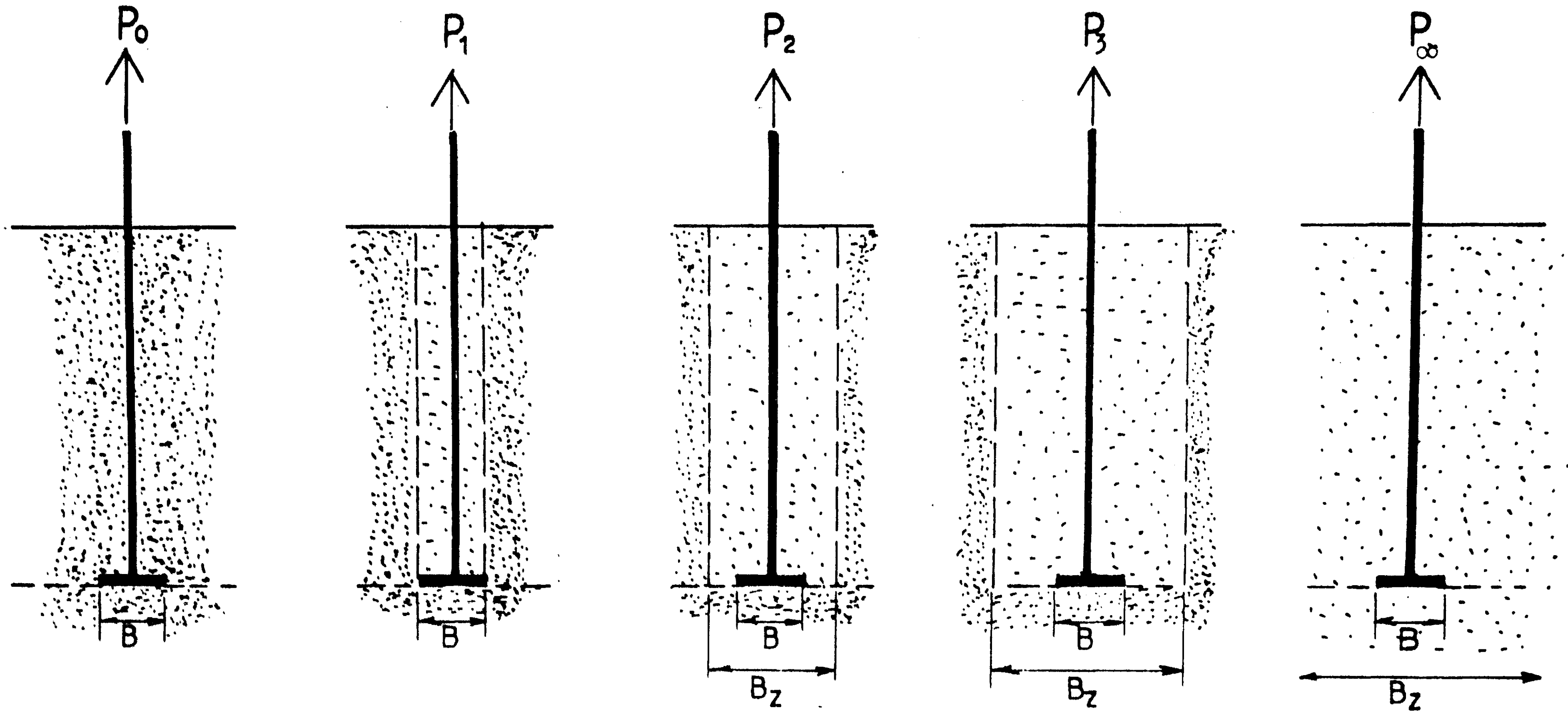
2). Tests in Disturbed Sand Bed

In this part of the tests (which was the major part) the anchors were pulled out from cylindrical loose sand zones which were surrounded by dense and medium dense homogeneous sand beds. The loose sand zone which formed the disturbed zone was varied in width to certain proportions of the anchor diameter and in this research the width of zone of disturbance, $B_z$ was increased from 1 to 3 anchor diameters. The range of D/B values used for the anchors embedded in the disturbed zone was similar to those given in Table 4.1. The schematic procedure for the anchor pullout tests in the disturbed zone is shown in Fig. 4.1.

## 4.3 Calibration of Equipment

A 1112N-capacity ( 250 lb ) Type D Sangamo Load Cell was used to record the load on the anchor. The load cell was calibrated to its maximum capacity against dead load. The calibration curve for the load cell is shown in Fig. 4.2. The load cell was connected to a data logger satellite and the readings of the load were displayed from a digital voltmeter.

A 25mm-travel Sensonics displacement transducer whose calibration curve given in Fig. 4.3 was used to record the displacement of the anchor during testing.




(a) Dense Homogeneous Bed, $\frac{Bz}{B} = 0$

(b) Disturbed Bed $\frac{Bz}{B} = 1.0$

(c) Disturbed Bed $\frac{Bz}{B} = 2.0$

(d) Disturbed Bed $\frac{Bz}{B} = 3.0$

(e) Bed Wholly Disturbed $\frac{Bz}{B} = \infty$

Fig. 4.1.Variation in Width of Zone of Disturbance With Respect to Anchor Diameter

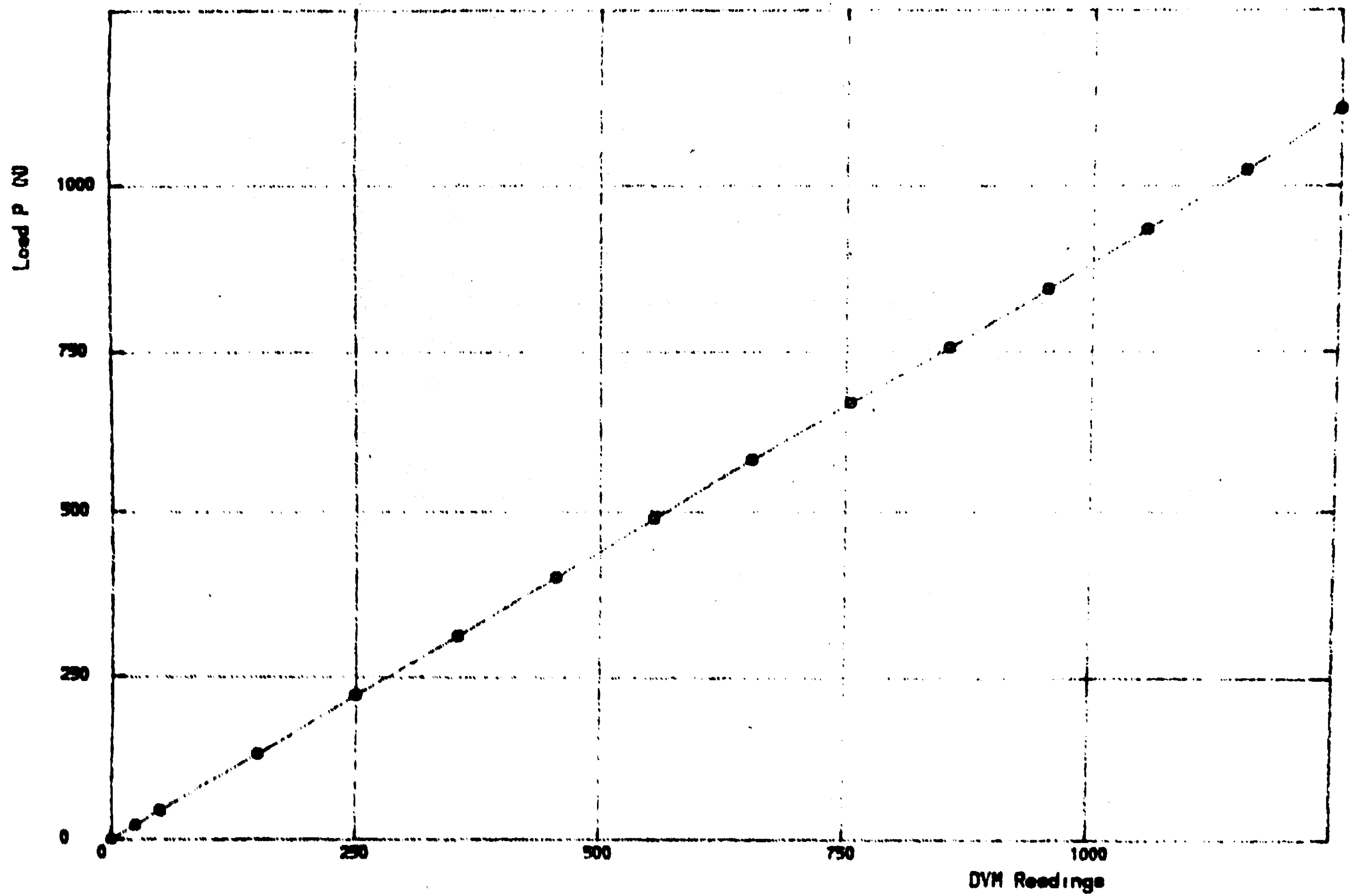



Fig. 4.2 Calibration Curve for Load Cell ( Capacity 1112 N )

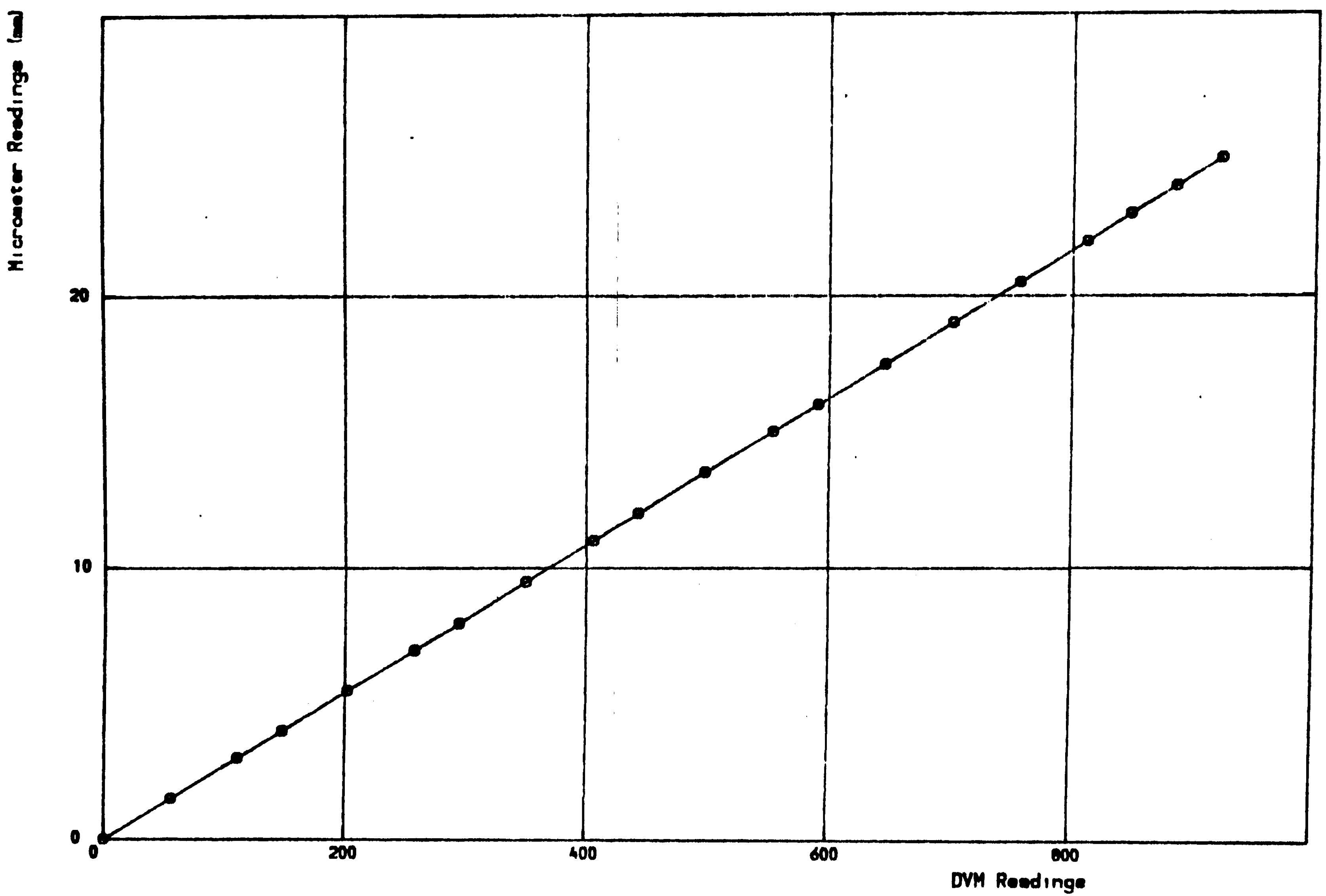



Fig. 4.3 Calibration Curve for Displacement Transducer

## 4.4 Anchor Test Rig

Except for the machine providing the pullout on the anchor the equipment used in both modes of test was similar. Referring to Fig. 4.4 and 4.6 the test equipment could be divided into two parts viz;

1). Anchor Assembly

2). Loading Assembly

### 4.4.1 Anchor Assembly

The anchor assembly consisted of the following items;

1). Circular anchor plate

2). Anchor shaft

3). Pin

4). Anchor cap

5). Anchor support cap

6). Extension rod to load cell

7). Displacement datum

8). Centering device

The circular anchor plates were made of brass the surface of which was smooth so that any friction with the sand particles was neglected. The plates were rigid so that their deformation during testing could be ignored. Great care was taken when installing the anchor so that any disturbance caused to the anchor was kept to a minimum.

### 4.4.2 Loading Assembly

The loading assembly was devised in such a way that it was convenient and simple to install while at the same time avoiding any undue disturbance to the anchor. The loading assembly consisted of the following items;

1). Motor drive or air cylinder piston

2). Load cell

3). Yoke

4). Tie bars

5). Displacement transducer

The motor drive or air cylinder piston was securely placed on the reaction frame. The support columns for the reaction frame were bolted to the floor. The bolt connection was strong enough to prevent the columns from moving or swaying to one side while the test was in progress. The reaction frame together with the pullout machine were temporarily removed while the sand bed was being formed in the receiver tank. A support rig was specially built for the reaction frame when it was detached from the columns.

## 4.5 Load Controlled Test on Anchor

The diagrammatic layout of anchor pullout test under the load-control is shown in Fig. 4.4.

### 4.5.1 Method of Assembly

A centering rod which had a ring-like connection at the centre was carefully placed around the anchor shaft to minimise any lateral effect which might occur while the loading frame was being assembled.

A displacement datum was attached to the shaft as a means of recording the vertical movement of the anchor by a LVDT.

An anchor cap was screwed onto the upper threaded end of the shaft. A second and bigger cap called a supporting cap suspended from a load cell through an extension rod

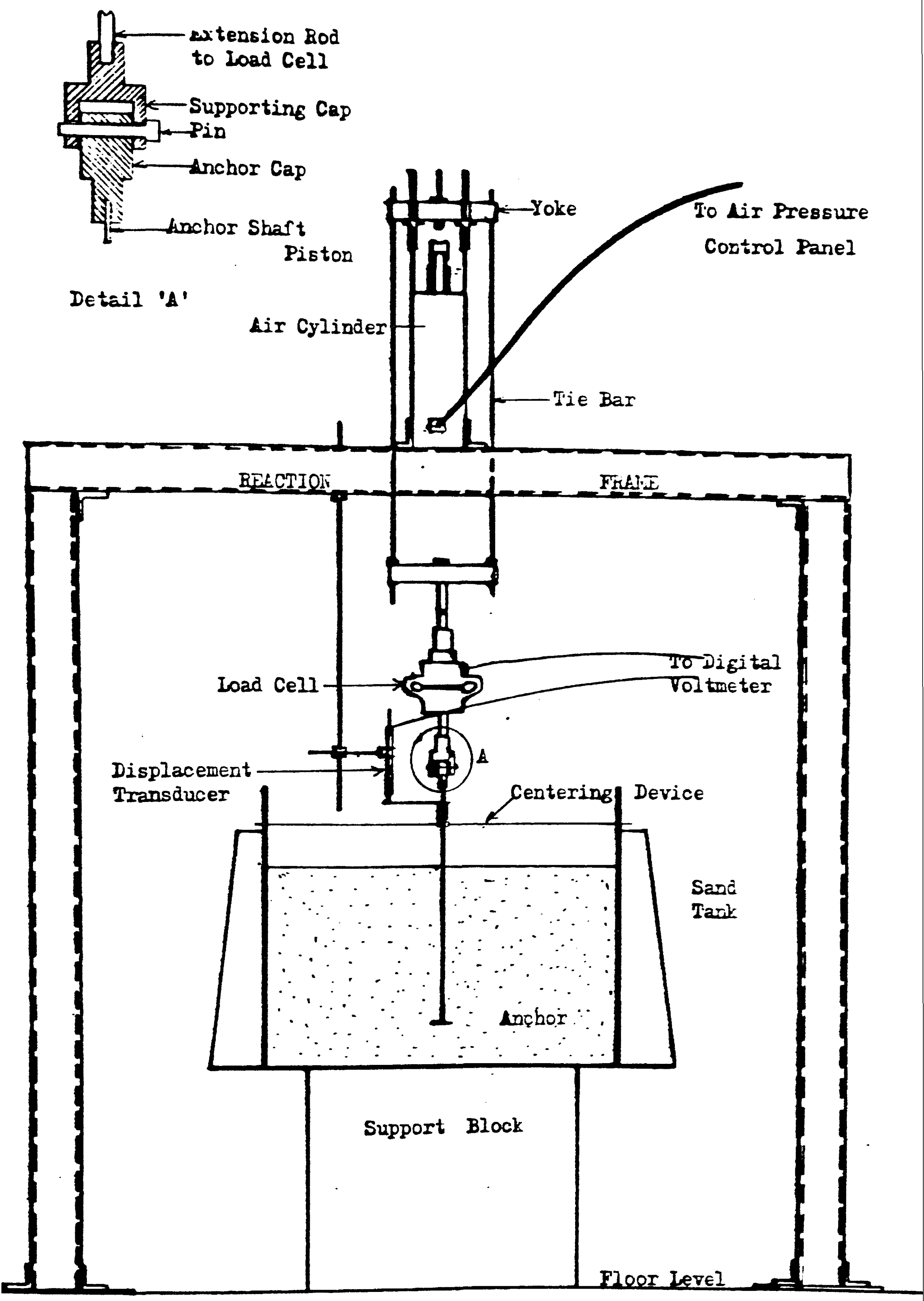



Fig. 4.4 Diagrammatic Layout of Anchor Pullout Test in Sand

( Load Controlled )

was sleeved into the anchor cap so that a pin could be inserted across them through a coaxial hole as shown in Fig. 4.4.

### 4.5.2 Method of Loading

At the start of the test, pressure was slowly applied to the piston until the weight of the loading assembly was counter-balanced. The piston pressure was then increased until the anchor failed ; failure being indicated by a disproportionately larger displacement followed by complete pullout failure. The rate of loading was 0.007 mm/min and the tests on the deep anchors sometimes had to be discontinued overnight with the anchor load held. One disadvantage of the load-controlled test was that the post-peak behaviour of the anchor could not be observed. Digital voltmeters were used to record the load on the anchor and its displacement.

### 4.5.3 Air Pressure Control Panel

The air pressure control panel was fixed to one of the support columns of the reaction frame. The air pressure control panel is shown in Fig. 4.5. The air supply from the central air compressor system passed through an air filter and could be channelled to an electro-pneumatic transducer or if only a static load test was required, the air was directly passed through a manually controlled air regulator and read by a heavy duty air pressure gauge. To ensure the air was free from foreign materials, it was then passed through a lubricator in its final stage before entering the air cylinder.

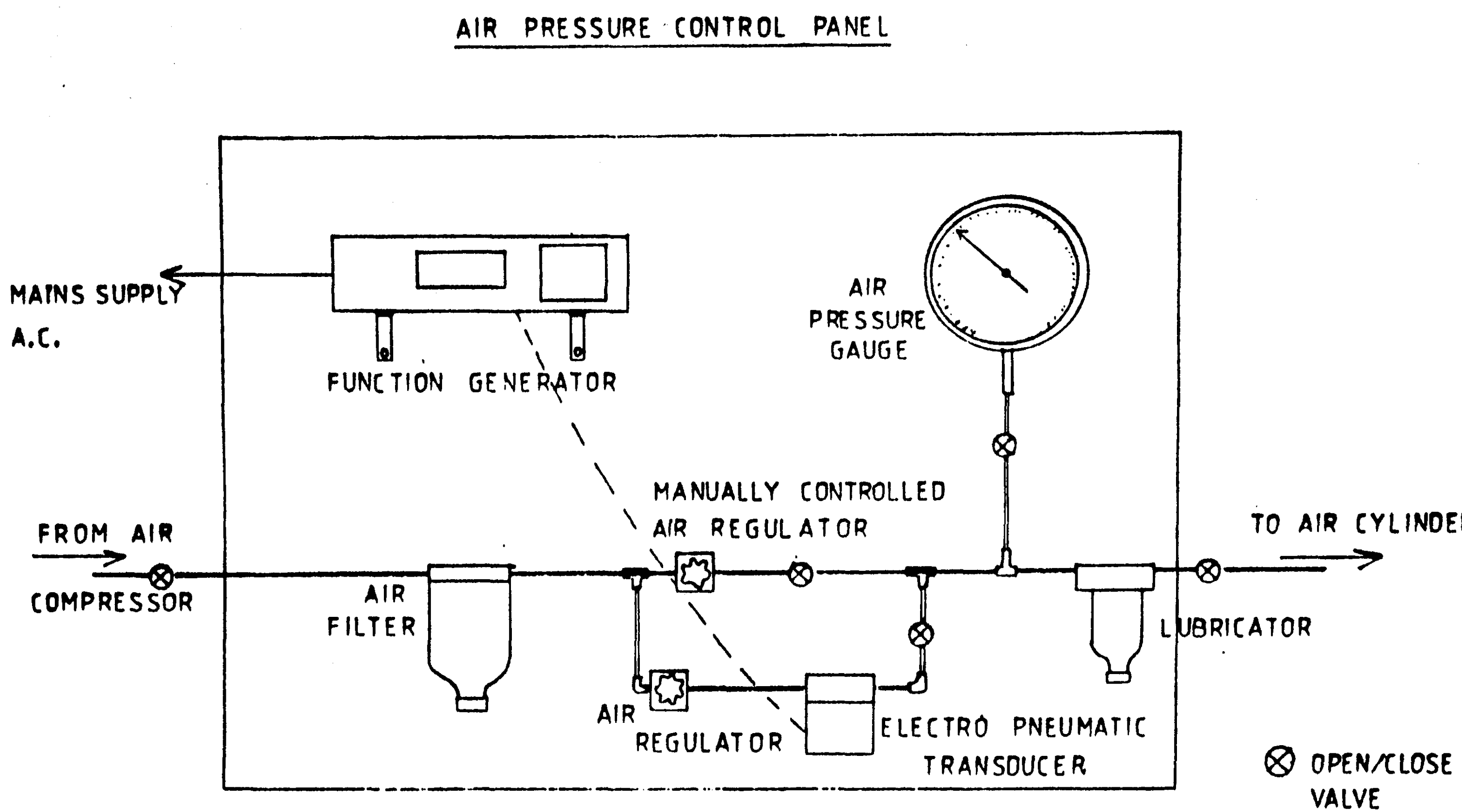

AIR PRESSURE CONTROL PANEL
MAINS SUPPLY
A.C.
FUNCTION GENERATOR
AIR
PRESSURE
GAUGE
MANUALLY CONTROLLED
AIR REGULATOR
FROM AIR
COMPRESSOR
AIR
FILTER
TO AIR CYLINDER
LUBRICATOR
AIR
REGULATOR
ELECTRO PNEUMATIC
TRANSDUCER
OPEN/CLOSE
VALVE

Fig. 4.5

## 4.6 Displacement Controlled Test on Anchor

This mode of test was easier to control compared to the previous one. As its name implied, the anchor was displaced at a constant rate of strain or displacement and the increase in load depends on the state of sand.

### 4.6.1 Method of Assembly

A 1-ton capacity motor drive unit was mounted on the reaction frame as shown in Fig. 4.6 in place of the air cylinder. The unit had multiple-choice gear ratios giving altogether 25 different rates of strain. The rate of strain was selected to give a reasonable time for the anchor to reach failure. If a fast rate was used the anchor would fail in a very short time and it was not sufficient to get enough information about the behaviour of the anchor. In all tests under this mode a strain of 0.375mm/min was used.

### 4.6.2 Method of Loading

The general set-up for the strain-controlled test is shown in Fig. 4.6. Note that except for the motor drive unit, both modes of test had a similar loading arrangement. As usual the load cell and LVDT were connected to digital voltmeters for taking the readings. Since only two variables were monitored during the test the recording of readings could be performed manually without much difficulty.

During the initial stage of the anchor testing program the values of D/B given in Table 4.1 were used. Different anchor sizes and depths of embedment but having

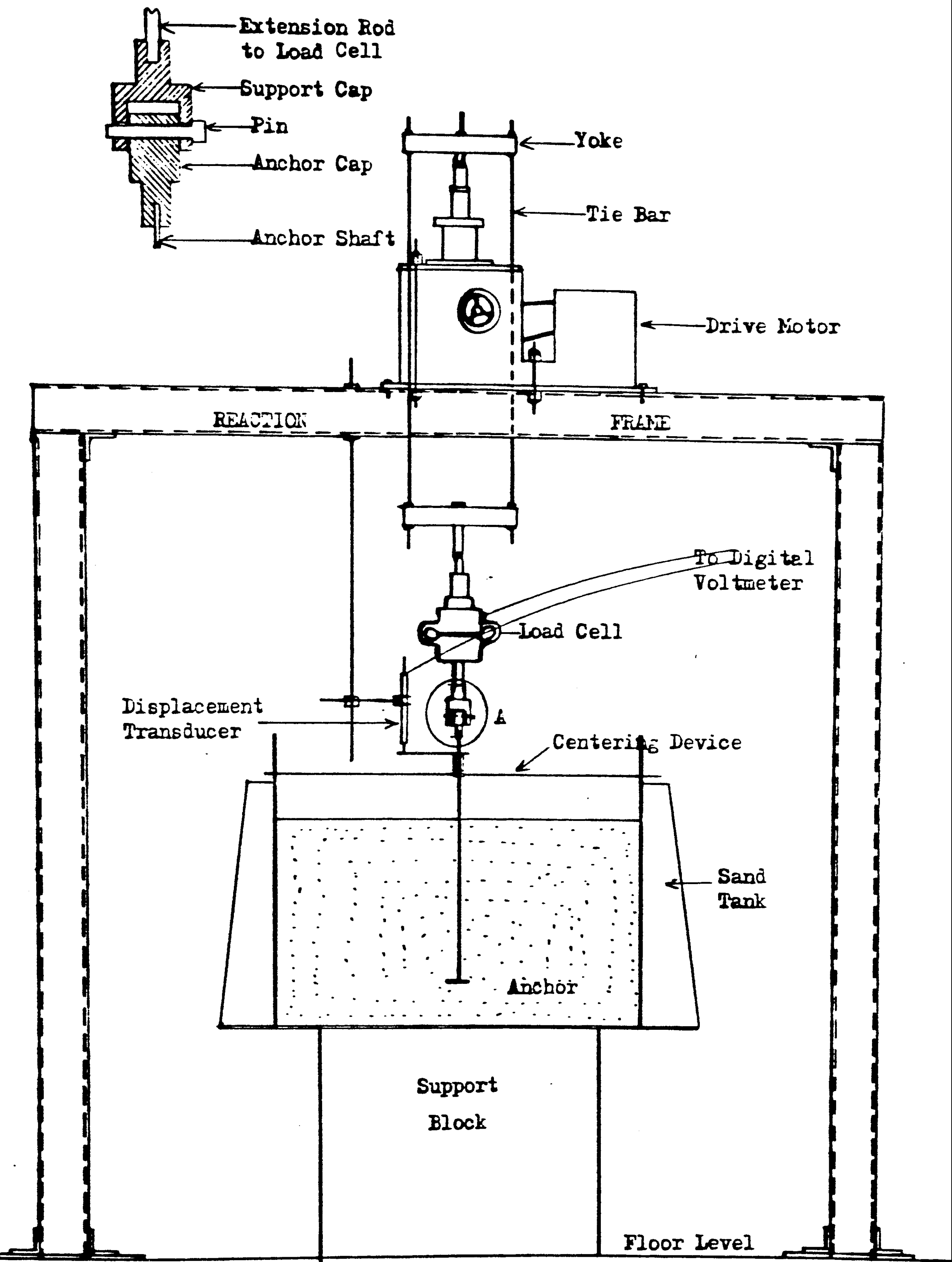



Fig. 4. 6 Diagrammatic Layout of Anchor Pullout Test in Sand

( Strain Controlled )

the same D/B ratio were initially used to assess the repeatability and reproducibility of the test method. Such tests were mainly performed in the dense homogeneous bed.

From the tests it was found that the break-out factors of the anchor at a particular D/B ratio were almost identical. So in the later stage of the testing program not all anchor sizes were used as shown in Table 4.2, 4.3 and 4.4.

A total of 71 tests were carried out in the investigation. The load-controlled tests were initially performed because the test set-up was already installed in the laboratory and was used by previous investigators under the anchor research programme carried out in the department.

The test method was employed for anchors embedded in the undisturbed as well as in the disturbed sand beds. It was found that by using the load-controlled tests the post peak load on the anchor after failure could not be observed because the anchor failed suddenly after that peak load was achieved. The author thought that it was worth using the displacement-controlled mode for the remainder of the tests so that the difference ( if any ) between the two test methods could be noted. The displacement-controlled tests contributed about 66% of the total number of tests in the research.

Table 4.2 Summary of Anchor Pullout Tests in Homogeneous Sand Bed

| Test No. | Bed Unit Wt. $KN/m^3$ | Anchor Dimensions D (mm) | B (mm) | D/B | Anchor Loading P (N) | Pu ($Nmm^{-2}$) | Pu/$\gamma$D | Mode of Test | Anchor Displ. at Fail. $\delta_f$ (mm) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 17.14 | 155 | 50 | 3.1 | 75.0 | 0.04 | 13.77 | lc | 1.81 |
| 2 | | 225 | 75 | 3.0 | 223.0 | 0.05 | 13.39 | lc | 2.54 |
| 3 | | 338 | 75 | 4.5 | 659.0 | 0.15 | 26.06 | lc | 3.13 |
| 4 | | 155 | 25 | 6.2 | 63.5 | 0.14 | 52.40 | lc | 1.65 |
| 5 | | 160 | 25 | 6.4 | 59.2 | 0.13 | 47.00 | lc | 1.30 |
| 6 | | 188 | 25 | 7.5 | 104.5 | 0.23 | 70.63 | sc | 2.29 |
| 7 | | 225 | 25 | 9.0 | 197.0 | 0.43 | 111.00 | lc | 3.86 |
| 8 | | 277 | 25 | 11.1 | 333.0 | 0.73 | 152.40 | lc | 5.00 |
| 9 | | 328 | 25 | 13.1 | 495.2 | 1.08 | 191.30 | lc | 5.59 |
| 10 | | 380 | 25 | 15.2 | 627.5 | 1.36 | 209.00 | lc | 5.75 |
| 11 | 16.40 | 75 | 25 | 3.0 | 9.2 | 0.02 | 16.34 | sc | 1.35 |
| 12 | | 155 | 50 | 3.1 | 60.0 | 0.03 | 12.73 | sc | 2.67 |
| 13 | | 150 | 25 | 6.0 | 44.8 | 0.10 | 39.63 | sc | 2.43 |
| 14 | | 300 | 50 | 6.0 | 369.0 | 0.19 | 38.77 | sc | 4.78 |
| 15 | | 185 | 25 | 7.4 | 80.4 | 0.17 | 57.68 | sc | 2.97 |
| 16 | | 230 | 25 | 9.2 | 153.0 | 0.33 | 88.20 | sc | 4.50 |
| 17 | | 300 | 25 | 12.0 | 291.0 | 0.63 | 126.60 | sc | 4.62 |
| 18 | | 380 | 25 | 15.2 | 451.0 | 0.98 | 158.00 | sc | 6.07 |
| 19 | 15.75 | 232 | 75 | 3.1 | 133.3 | 0.03 | 8.39 | lc | 3.78 |
| 20 | | 165 | 50 | 3.3 | 45.0 | 0.02 | 8.85 | lc | 3.16 |
| 21 | | 300 | 50 | 6.0 | 237.0 | 0.12 | 25.32 | lc | 5.78 |
| 22 | | 160 | 25 | 6.4 | 27.0 | 0.06 | 23.30 | lc | 3.81 |
| 23 | | 235 | 25 | 9.4 | 87.0 | 0.18 | 51.10 | lc | 3.94 |
| 24 | | 300 | 25 | 12.0 | 147.0 | 0.32 | 61.63 | lc | 4.83 |
| 25 | | 380 | 25 | 15.2 | 201.0 | 0.44 | 73.00 | lc | 7.13 |

Note:

lc = load-controlled test

sc = strain or displacement-controlled test

Table 4.3 Summary of Anchor Pullout Tests in Disturbed Zone

Basic Medium Dense Bed, $\gamma$ = 16.40 KN/m$^3$

| Test No. | Anchor Dimensions | | | Anchor Loading | | | Mode of Test | Anchor Displ. at Failure $\delta_f$ (mm) |
|---|---|---|---|---|---|---|---|---|
| | B (mm) | $B_z/B$ | D/B | P (N) | Pu ($Nmm^{-2}$) | $Pu/\gamma D$ | | |
| 26 | 25 | 1.0 | 3.0 | 6.5 | 0.01 | 12.11 | sc | 2.16 |
| 27 | 50 | | 3.1 | 50.5 | 0.03 | 10.70 | sc | 2.19 |
| 28 | 25 | | 6.2 | 45.0 | 0.10 | 40.08 | sc | 1.81 |
| 29 | 25 | | 7.4 | 81.3 | 0.18 | 60.70 | sc | 5.05 |
| 30 | 25 | | 9.0 | 123.0 | 0.27 | 75.40 | sc | 4.13 |
| 31 | 25 | | 9.2 | 127.0 | 0.28 | 76.20 | sc | 4.45 |
| 32 | 25 | | 12.0 | 213.0 | 0.46 | 103.50 | sc | 5.97 |
| 33 | 25 | | 15.0 | 333.0 | 0.72 | 122.6 | sc | 6.67 |
| 34 | 25 | 2.0 | 3.0 | 5.7 | 0.01 | 10.40 | sc | 2.43 |
| 35 | 25 | | 3.0 | 4.8 | 0.01 | 8.80 | sc | 1.98 |
| 36 | 25 | | 6.0 | 33.0 | 0.07 | 30.30 | sc | 3.46 |
| 37 | 25 | | 7.4 | 56.2 | 0.13 | 32.20 | sc | 3.48 |
| 38 | 25 | | 9.0 | 93.0 | 0.20 | 57.10 | sc | 4.29 |
| 39 | 25 | | 12.0 | 183.0 | 0.40 | 84.20 | sc | 6.00 |
| 40 | 25 | | 15.0 | 249.0 | 0.54 | 91.6 | sc | 7.05 |
| 41 | 25 | 3.0 | 3.1 | 6.6 | 0.01 | 11.70 | sc | 1.46 |
| 42 | 25 | | 6.0 | 27.0 | 0.06 | 24.80 | sc | 4.67 |
| 43 | 25 | | 7.2 | 41.3 | 0.09 | 31.60 | sc | 5.35 |
| 44 | 25 | | 8.8 | 57.0 | 0.12 | 36.00 | sc | 4.94 |
| 45 | 25 | | 11.8 | 128.5 | 0.28 | 56.80 | sc | 6.18 |
| 46 | 25 | | 14.8 | 165.5 | 0.35 | 60.00 | sc | 7.02 |
| 47 | 25 | | 15.0 | 169.0 | 0.37 | 63.00 | sc | 10.26 |

Table 4.4 Summary of Anchor Pullout Tests in Disturbed Zone

Basic Dense Bed, $\gamma$ = 17.14 KN/m$^3$

| Test No. | Anchor Dimensions | | | Anchor Loading | | | Mode of Test | Anchor Displ. at Failure $\delta_f$ (mm) |
|---|---|---|---|---|---|---|---|---|
| | B (mm) | $B_z/B$ | D/B | P (N) | Pu (Nmm$^{-2}$) | Pu/$\gamma$D | | |
| 48 | 75 | 1.0 | 3.0 | 241.8 | 0.05 | 15.05 | lc | 3.19 |
| 49 | 50 | | 3.1 | 69.0 | 0.03 | 15.18 | lc | 1.65 |
| 50 | 50 | | 5.9 | 419.0 | 0.22 | 47.20 | lc | 3.08 |
| 51 | 50 | | 6.0 | 411.0 | 0.21 | 45.10 | lc | 3.24 |
| 52 | 25 | | 7.6 | 120.6 | 0.25 | 84.90 | sc | 3.51 |
| 53 | 25 | | 9.0 | 201.0 | 0.44 | 123.0 | lc | 3.59 |
| 54 | 25 | | 9.1 | 198.0 | 0.43 | 120.0 | sc | 3.62 |
| 55 | 25 | | 12.0 | 342.1 | 0.74 | 157.5 | sc | 5.40 |
| 56 | 25 | | 15.0 | 464.0 | 1.01 | 170.8 | sc | 6.13 |
| 57 | 50 | 2.0 | 3.1 | 39.0 | 0.02 | 8.39 | lc | 1.75 |
| 58 | 25 | | 3.2 | 5.4 | 1.01 | 9.21 | sc | 1.16 |
| 59 | 25 | | 5.9 | 33.3 | 0.07 | 31.12 | sc | 3.08 |
| 60 | 25 | | 6.7 | 63.5 | 0.14 | 33.80 | lc | 3.25 |
| 61 | 25 | | 7.4 | 70.6 | 0.15 | 52.5 | sc | 4.03 |
| 62 | 50 | | 7.3 | 633.0 | 0.33 | 57.70 | lc | 4.87 |
| 63 | 25 | | 9.0 | 132.1 | 0.29 | 81.00 | sc | 5.40 |
| 64 | 25 | | 12.0 | 225.5 | 0.49 | 103.7 | sc | 5.86 |
| 65 | 25 | | 15.0 | 329.6 | 0.72 | 121.4 | sc | 6.10 |
| 66 | 25 | 3.0 | 3.0 | 4.8 | 0.01 | 8.80 | sc | 1.35 |
| 67 | 25 | | 6.0 | 20.8 | 0.04 | 15.10 | sc | 3.24 |
| 68 | 25 | | 7.4 | 56.4 | 0.12 | 39.50 | sc | 4.05 |
| 69 | 25 | | 9.0 | 65.3 | 0.14 | 40.10 | sc | 6.16 |
| 70 | 25 | | 12.0 | 141.8 | 0.31 | 65.30 | sc | 6.29 |
| 71 | 25 | | 15.0 | 193.5 | 0.42 | 71.10 | sc | 7.24 |

As stated previously a series of tests were carried out in homogeneous sand beds of different unit weight. A total of 25 such tests were made as shown in Table 4.2 at unit weights of 17.14 , 16.40 and 15.75 $KN/m^3$ .

Table 4.3 shows the results of 22 tests carried out in a basic test bed of medium density through a range of $B_z/B$ values from 1 to 3.

Table 4.4 records 24 tests where the same range of $B_z/B$ values were covered in a basic dense bed of sand.

To check the reproducibility of the results, duplicate tests were performed in both modes. For example tests no. 61 and 62 for D/B = 7.3 and D/B = 7.4. Although the breakout factor from the load-controlled test was slightly higher than the displacement-controlled test ( which was about 9% ) the difference was attributed to experimental errors. Tests no. 53 and 54 showed a difference in the $Pu/\gamma D$ values of about 2% under different modes of test conditon.

As can be seen from the tables of results, several tests were sometimes run under exactly the same conditions to assess the reproducibility of the test method. Duplicate tests were carried out in 14 cases i.e. tests no. 1,2, 4,5, 11,12, 13,14, 19,20, 21,22, 26,27, 30,31, 34,35, 48,49, 50,51, 53,54, 57,58, and 61,62. In those tests they indicated good agreement in the breakout factors ( on average to 7% ).

# CHAPTER 5

## DISCUSSION OF RESULTS AND CONCLUSIONS

### 5.1 Introduction

From the experiments as described in Chapter 4 it was shown that the installation procedure as simulated by the formation of a disturbed sand zone around the anchor had siginificant effects on the anchor uplift capacity especially when the width of zone of disturbance was greater than the anchor diameter.

In this chapter the discussions of results are divided into three main themes and are arranged in the following order;

1). Behaviour of anchor in homogeneous sand bed

2). Comparison with Fadl's works

3). Behaviour of anchor in a disturbed zone

Lastly conclusions are drawn from the discussions and are given at the end of the chapter.

Discussions were concentrated on Fadl's works because he carried out similar tests in Leighton -Buzzard sand under similar tests conditions. Attention was drawn to Fadl's theoretical analysis because his analysis was used by the author as a preliminary step to establish his ( the author ) theoretical method on the anchor uplift capacity in a disturbed sand zone which will be given in Chapter 6. Fadl's theoretical analysis was used in preference to the others described in Chapter 2 because a study of the other theories indicated that Fadl's method gave results which were in reasonable agreement with the experimental results of El-Rayes (1965 ), Balla ( 1961 ), Sutherland ( 1965 ), Bemben and Kupferman ( 1975 ), Harvey and Burley ( 1973 ) etc.

## 5.2 Ultimate Load of Anchor Under Different Test Conditions

As it was pointed out in Chapter 4 the tests described herein were conducted under the load-control and displacement-control. Although there was no difference in the ultimate pullout load on the anchor between both types of test ( El-Rayes, 1965 ) the definition of the failure load under each test should be understood. A displacement-controlled test in sand normally gives a distinct peak or failure load and it can be readily distinguished from a load-displacement curve.

On the other hand in a load-controlled test as adopted in the test series the anchor failed suddenly giving a total failure when the peak load was achieved. In this connexion, Matsuo ( 1968 ) had defined the ultimate resistance of the anchor under this mode of test to be equal to the stage load immediately before the total failure occurred although he did not specify the range within which the stage load should fall. In a load-controlled test the load increment could be varied with time. The anchor could fail in a short time had the load been increased at larger intervals and vice versa. In the experiments the stage load was taken as the last load increment immediately before failure occurred as illustrated in Fig. 5.8.

## 5.3 Ultimate Loads of Anchor in Homogeneous Sand Beds

The results from these tests are presented in dimensionless form as shown in Fig. 5.1. As shown by previous investigators ( Fadl, Meyerhof and Adams, Vesic, Matsuo, El-Rayes etc. )the relationship between the breakout factor, $Pu/\gamma D$ and the depth/diameter ratio, D/B of an anchor in a homogeneous sand bed is a marked increase in $Pu/\gamma D$ as the ratios increase reaching the peak value at a certain D/B ratio and then remaining fairly constant after that peak value even though the D/B ratio increases.

From an examination of Fig. 5.1 the following comments can be made,

1). The shape of the curves follow similar trends to others notably to Fadl (1981) as illustrated in Fig. 5.2 and 5.3.

2). The curves of $Pu/\gamma D$ against D/B are initially concave up, change to concave down and eventually flatten out to give no increase in $Pu/\gamma D$ with an increase in D/B. Although this generally agreed with Wang and Wu's ( 1983 ) and Meyerhof and Adams's ( 1968 ) in that the trends of the plots were followed, their conclusion that for dense sand ( $\varphi = 35^o$ ) beyond D/B ratio of about 10, the breakout factors were independent of the depth/diameter ratios the author's test results did not agree with their theories. The disagreement was due to the difference in the anchor shape used by the author and by them. Wang and Wu and Meyerhof and Adams had used rectangular anchors and strip footings respectively although Meyerhof and Adams did some modifications in their theory to take into account of circular shape anchors.

3). Change in anchor diameter does not affect the

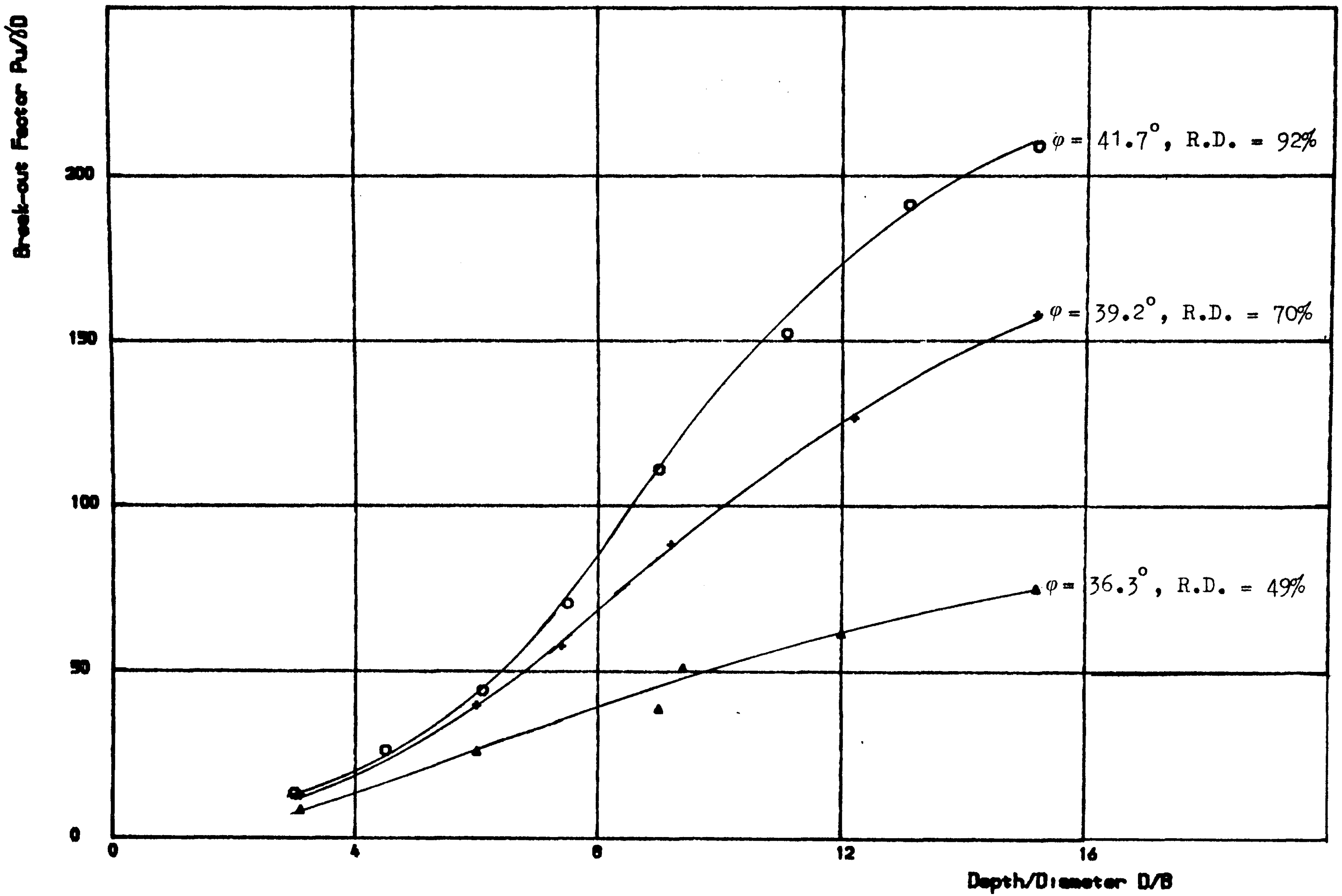



Fig. 5.1 $P_u/\gamma D$ VS D/B for Anchors Embedded in Dry Sands at Different Relative Densities

shape of the curves. This agreed with the principles of dimensional analysis as proposed by Sutherland ( 1965 ) where he showed that for a particular $\varphi$ value the breakout factor $Pu/\gamma D$ only depended on the ratio of depth/ diameter ( D/B ).

4). Transition between concave up and concave down is the transition between shallow and deep anchors. From the curves it can be observed that the transition occurs at D/B ratios of approximately 7.2, 8.0 and 9.0 for the loose , medium dense and dense sand beds respectively in this test series. This did not agree with Baker and Kondner,s ( 1966 ) theory because for dense sand they had defined a shallow anchor when D/B < 6. As shown by Meyerhof and Adams ( 1968 ), Vesic ( 1972 ) and Wang and Wu ( 1983 ) the transition between a shallow and a deep anchor varied with relative density. The values quoted above were consistent with their findings although the numerical values were different.

5). The breakout factor changes with relative density.This was because from the relationship $Pu/\gamma D$ = f(D/B, $\varphi$ ), if D/B was kept constant,the breakout factor was also a function of $\varphi$. Thus if $\varphi$ was low the relative density would also be low. For example at D/B = 12, the breakout factors at relative densities 92.0%, 70.0% and 49.0% were 172, 124 and 62 respectively.

## 5.4 Comparison With Fadl's Experimental Investigation

Fig. 5.2 shows a comparison between Fadl's experimental results and the author's. Fadl carried out similar tests in Leighton-Buzzard sand under displacement-

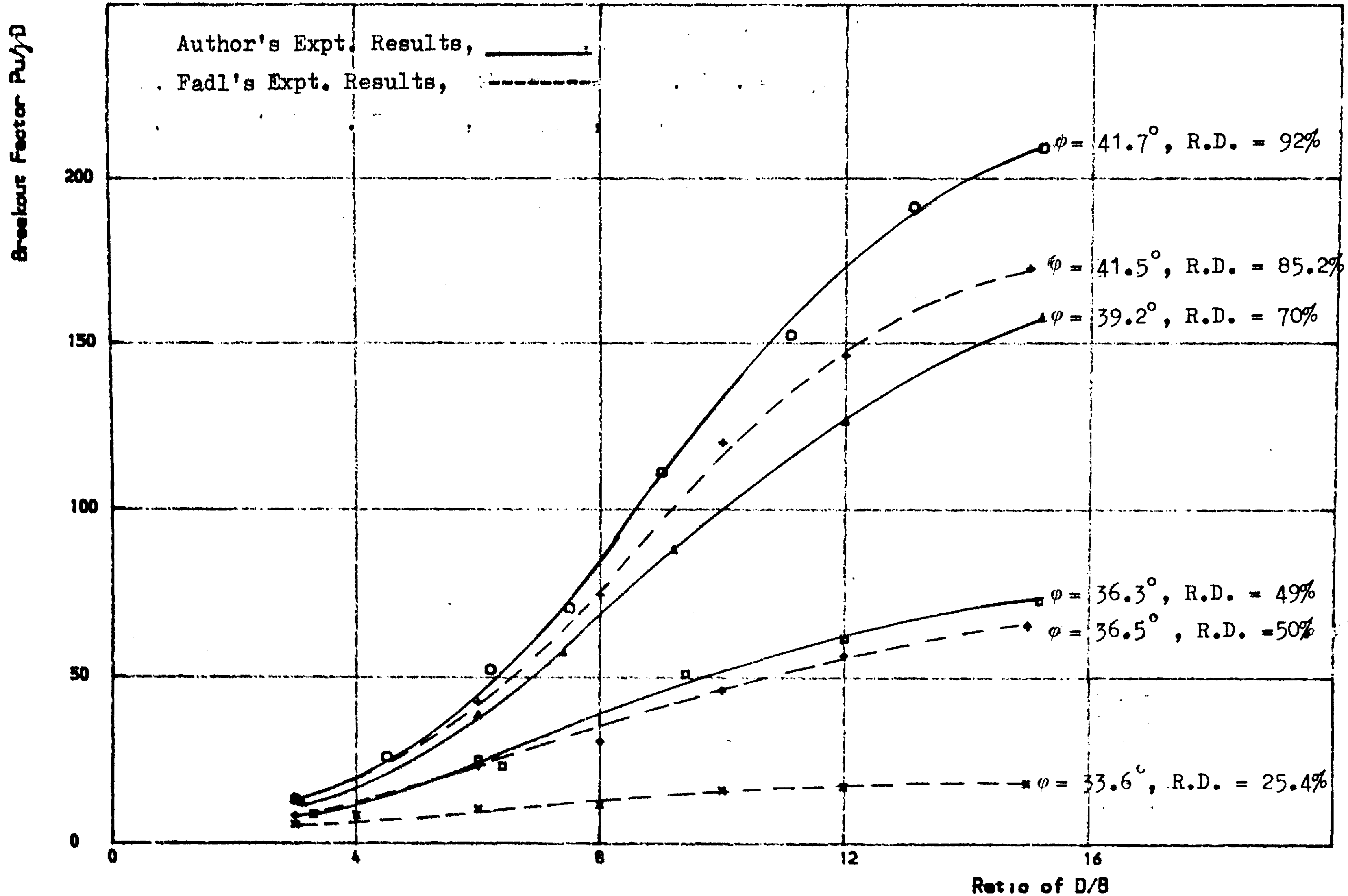



Fig. 5.2 Comparison between Author's Experimental Results and Fadl's Experimental Results

control at relative densities 85.5%, 50% and 25.4% respectively. From the test results and an observation of the failure surface he proposed expressions for the anchor uplift resistance which have already been discussed in Section 2.8. It is seen from the plots that the author's experimental results are consistent with Fadl's test results.

A change in the relative density of about 22% in the author's tests ( from dense to medium ) gave a change in the breakout factor $Pu/\gamma D$ value of about 40% at D/B = 15. Whereas a change of about 21% in the relative density ( from medium to loose ) gave a corresponding value of nearly 100%. In Fadl's tests a change of 35.2% in the realtive density ( from dense to medium ) resulted in a change in the $Pu/\gamma D$ of about 62% while a change of about 25% in the relative density ( from medium to loose ) resulted in a change of 71% of the breakout factor $Pu/\gamma D$. This showed that the breakout factors were very sensitive to a change in the relative density towards the looser state.

It might suggest that for a loose sand which was sufficiently weak and compressible, the anchor might just punch through the soil upward in a bearing capacity type failure instead of mobilising the shear strength of the soil along a distinct failure surface resulting in a sudden drop of the anchor pullout load.

The author's curve for R.D. = 49.0% slightly overestimated the Fadl's curve for R.D. = 50.0% but generally reasonable agreement was obtained.

5.5 Comparison With Fadl's Theoretical Analysis

Fig. 5.3 shows a comparison between the predictions from Fadl's theoretical analysis and the author's experimental test results. It is seen from the graph that in all states of the sand the author's experimental results are higher than Fadl's theoretical method. For R.D. = 92.0%, Fadl's method overestimated the author's experimental results in the range of D/B values from 3 to about 7.5. Beyond this point the author's results were higher than the method proposed by Fadl. The difference in Pu/ $\gamma$D value for D/B = 15 at relative density 92.0% was about 8%.

The results obtained from the experiments were higher than Fadl's predictions over the whole range of D/B values in both cases of sand relative densities i.e. 70.0% and 49.0%. The difference in Pu/$\gamma$D value for D/B = 15 at relative density 70.0% was about 32.0% while the corresponding value at relative density 49.0% was about 12.7%.

Fadl showed that his predictions in the breakout factors differed by 4% at D/B = 15 compared to his test data at relative density 85.2%. The difference in the values of Pu/$\gamma$D from his predictions at relative densities 50% and 25.4% gave corresponding values of 7% and 5% respectively.

It could be concluded that Fadl's predictions also agreed with the author's experimental results except at author's relative density 70% where Fadl's predictons were about 32% higher at the same D/B ratio ( Fig. 5.3 ). It was likely that the difference remained constant beyond

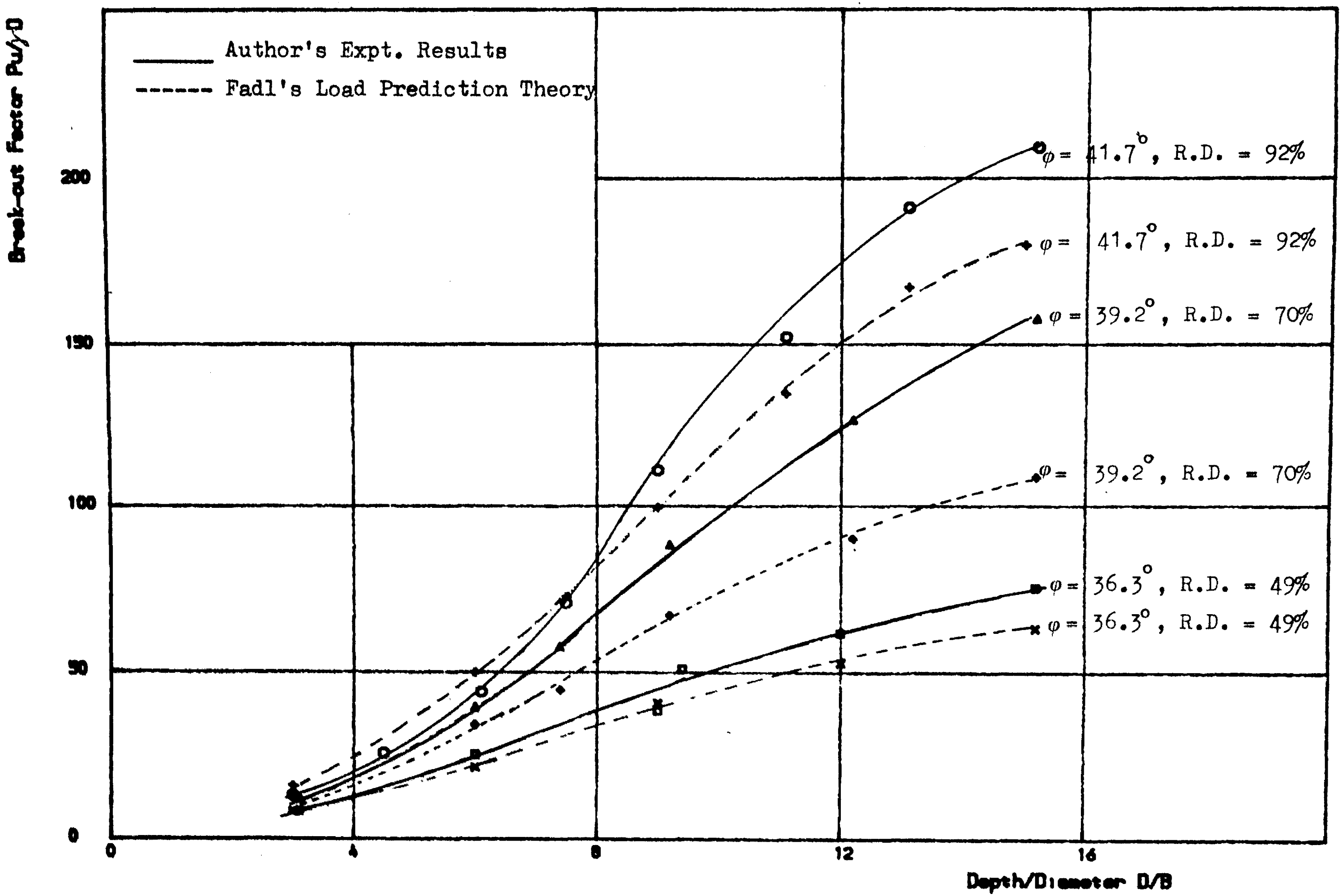



Fig. 5.3 Comparison between Fadl's Load Prediction Theory and Author's Expt. Results

D/B = 15, as the curves showed a linear pattern beyond that ratio.

5.6 <u>Ultimate Load of Anchor in Disturbed Zone</u>

Fig. 5.4 and 5.5 depict the effect of installation disturbance on the breakout factor $Pu/\gamma D$ in the basic dense and medium dense sand bed. Generally the curves of $Pu/\gamma D$ versus D/B for anchors embedded in a disturbed zone lay between the upper and lower limits of the anchor pullout capacity curves in the dense and loose sand states available in the test series. It is interesting to note from Fig. 5.4 that the anchor pullout capacity or its breakout factors in the disturbed zone for $B_z/B = 1.0$ was initially higher than its capacity in the basic dense homogeneous bed upto to a value of D/B of about 9.7. This phenomenon was not obvious in the case of medium dense bed.

These anomalies could have been resulted from the higher localised density and degree of sand interlocking at the disturbed zone/sand bed interface. Under these conditions ( $B_z = B$ ) the anchor was pulled out within a narrow cylindrical zone. Had the bed been homogeneous, an inclined failure surface ( straight or curved ) making an angle $\alpha$ or any other angle with the vertical through the anchor edges would have been developed as discussed in Chapter 2. When the anchor was being pulled out from the disturbed zone ( $B_z = B$ ) some sand grains directly above the anchor plate were pushed outwards ( Hanna, 1971 and Kostyukov, 1967 ) but as the surrounding bed was denser than the disturbed zone, there was a higher degree of

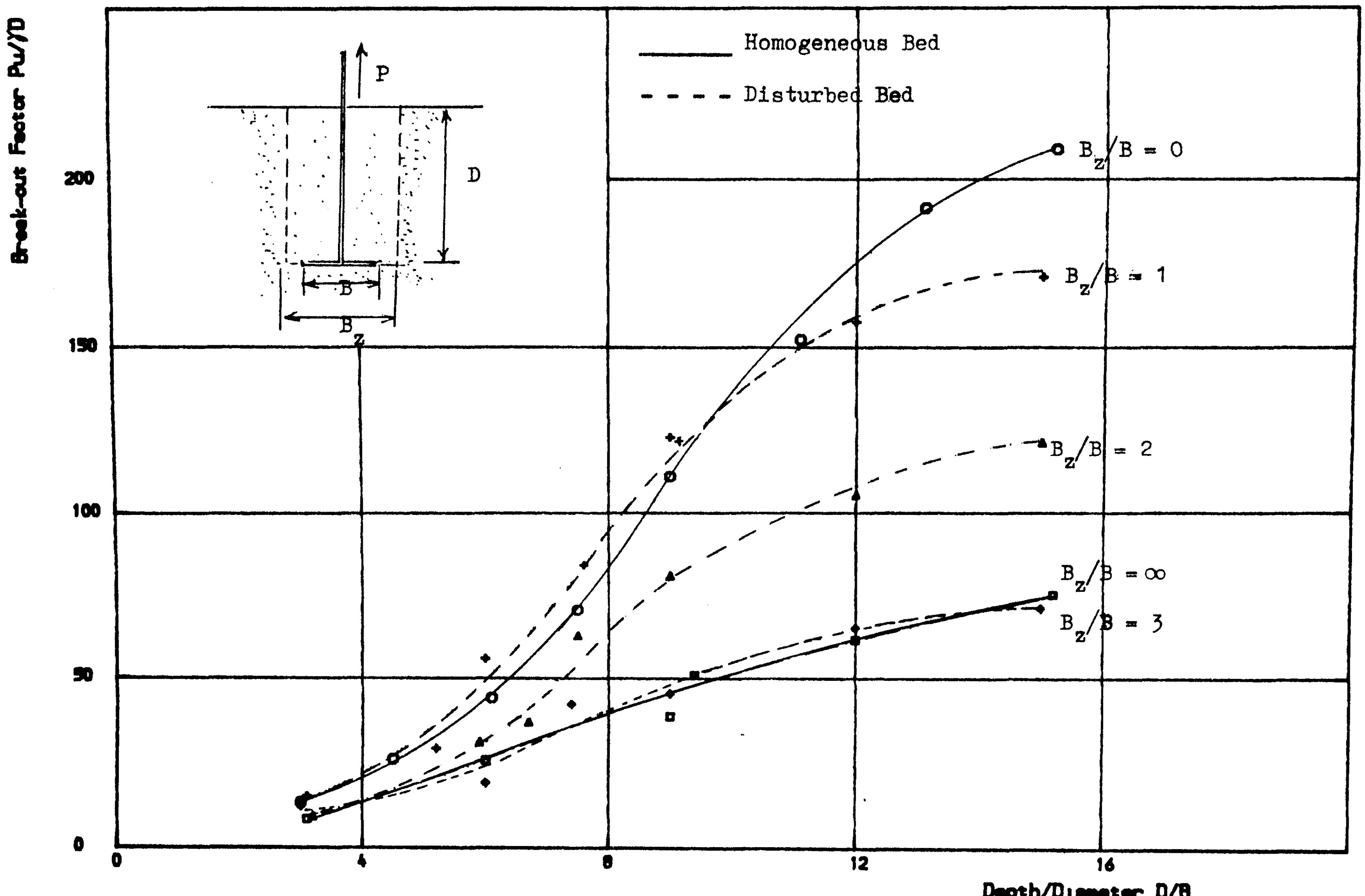



Fig.5.4 Effect of Disturbed Zone on Anchor Breakout Factor in Dry Sand ( Basic Dense Bed )

concentration of particles from the disturbed zone pushing the interface. The presence of these particles might increase the degree of sand interlocking and the density of the sand within the regions in the vicinity of the anchor edges. Thus a higher localised density was created and it resulted in a higher anchor pullout load. However it was difficult to check this densification phenomenon because no such facilities were available.

Another factor which might contribute to this higher breakout factor was the value of $\gamma$ used to calculate the vertical stress. The value of $\gamma$ ( unit weight ) of the disturbed zone was found to be 15.75 KN/$m^3$ ( R.D.= 49% ) i.e. a difference of about 8% compared to the dense homogeneous bed which was 17.14 KN/$m^3$ ( R.D.= 92% ). So when the pullout pressure $P_u$ ( which might have the same magnitude as in the dense homogeneous bed ) was divided by the vertical stress ( = $\gamma D$ ) in the disturbed zone, the overall result would yield a higher breakout factor.

When $B_z/B$ was increased from 2 to 3 anchor diameters the pullout factor in the disturbed zone was roughly equal to the pullout factor in the loose homogeneous bed. This means that the effect of disturbance was significant upto a ratio of $B_z/B = 3$. Beyond this ratio it made no difference between pulling out an anchor in a loose homogeneous bed and pulling out the same anchor in a disturbed zone surrounded by a dense homogeneous bed having a $B_z/B$ ratio greater than 3 or precisely $3<B_z/B<\infty$.

In the case of basic medium dense bed ( Fig.5.5 ) for $B_z/B = 1.0$ the anchor capacity in the disturbed zone was equal to its capacity in the medium dense bed ( $B_z/B$ =

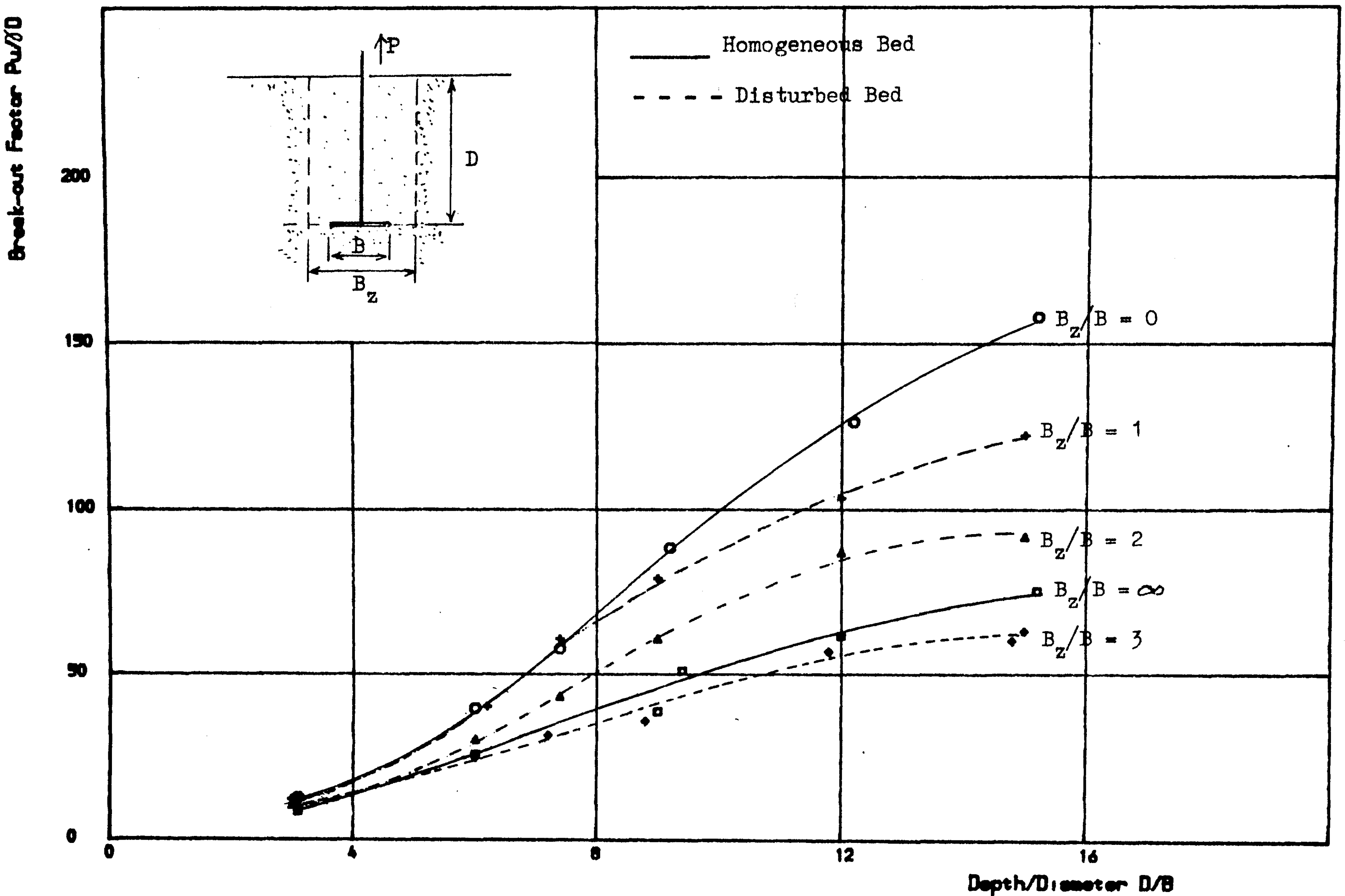



Fig. 5.5 Effect of Disturbed Zone on Anchor Breakout Factor in Dry Sand ( Basic Medium Dense Bed )

0 ) upto a ratio of D/B of about 7.5 but the effects discussed above were not pronounced because at no instant did the anchor pullout in the disturbed zone for $B_z/B = 1$ exceed its capacity in the medium dense bed.

The figure also shows that when $B_z/B$ was increased to 3 the anchor pullout load was lower than its own load in the loose homogeneous bed. This implied that the density or unit weight of the disturbed zone did not correspond to the unit weight of the loose homogeneous bed which was used as a basis for the formation of the disturbed zone as given in Section 3.6.

Although great care was taken to form the cylindrical loose sand volume ( disturbed zone ) so that its unit weight corresponded to that loose sand bed available above the anchor position, it seemed that as the tube became wider the method of forming the disturbed zone became inefficient. Nevertheless the present method adopted was superior than the other two methods discussed earlier.

### 5.7 Reduction in Anchor Breakout Factors Caused by Installation Disturbance

By using Fig. 5.4 as a basis the change in the breakout factors with respect to the breakout factors in the dense homogeneous bed ( $B_z/B = 0$ ) against the ratio of $B_z/B$ could be plotted. In this case the breakout factor curve for the anchor in the dense homogeneous bed ( $B_z/B = 0$ ) was used as a reference. For example, for D/B = 12 the breakout factors at the $B_z/B$ ratios with their respective percentage changes are given as follows,

| $B_z/B$ | $Pu/\gamma D$ | % change in $Pu/\gamma D$ ( cf. $B_z/B$ = 0 ) |
|---|---|---|
| 0 | 174.0 | 0 |
| 1 | 160.0 | 8.0 |
| 2 | 107.0 | 38.5 |
| 3 | 62.0 | 64.4 |
| $\infty$ | 65.0 | 62.6 |

The difference between the breakout factors obtained from the reference curve and the rest of the curves was expressed as percentage and presented in Fig.5.6 for the basic dense bed. Similarly Fig. 5.7 was obtained by using the data from Fig. 5.5 as a basis for the basic medium dense bed.

Fig. 5.6 and Fig.5.7 represented the effects of disturbance on the anchor pullout expresed in terms of the percentage change with respect to its pullout capacity in the basic dense homogeneous bed and basic medium dense homogeneous bed respectively.

Referring to Fig.5.6 when $B_z/B$ was increased from 0 to 1, the percentage change in the pullout factor increased in three cases of anchor embedment i.e. at D/B = 3, 6 and 9. For example when the disturbed zone was first created with $B_z/B = 1$, the percentage change in the pullout factor increased from 0 to about 13% at D/B = 9 i.e there was a gain in the anchor uplift capacity of about 13%. Thus to take advantage of the anchor capacity in a disturbed zone the excavation should be kept as close as possible to the anchor diameter. The same argument as given earlier in Section 5.6 applied to this higher percentage change in the pullout factor in the disturbed

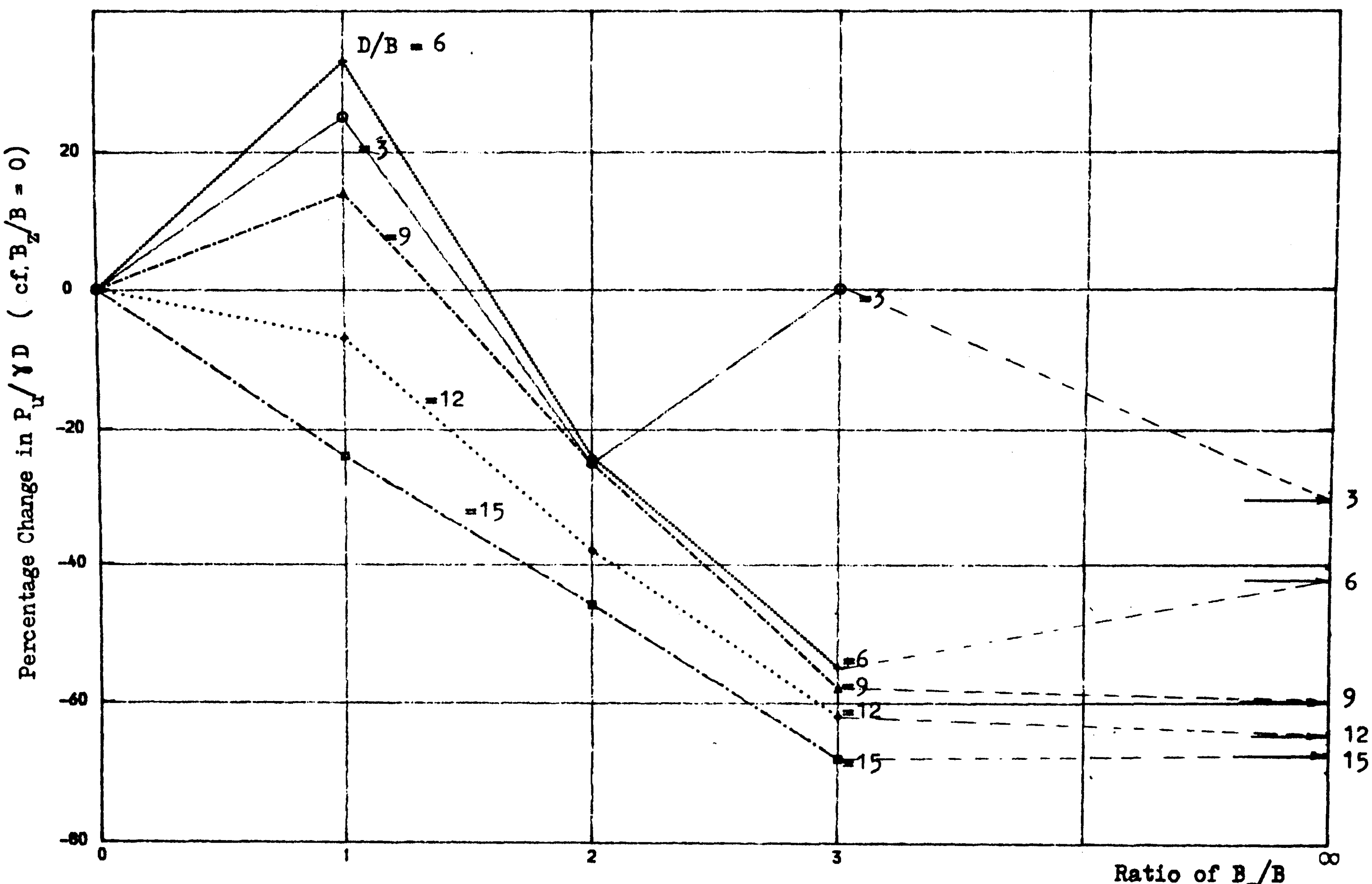



Fig.5.6 Percentage Change in Pullout Factor Caused by Installation Disturbance in Dense Bed

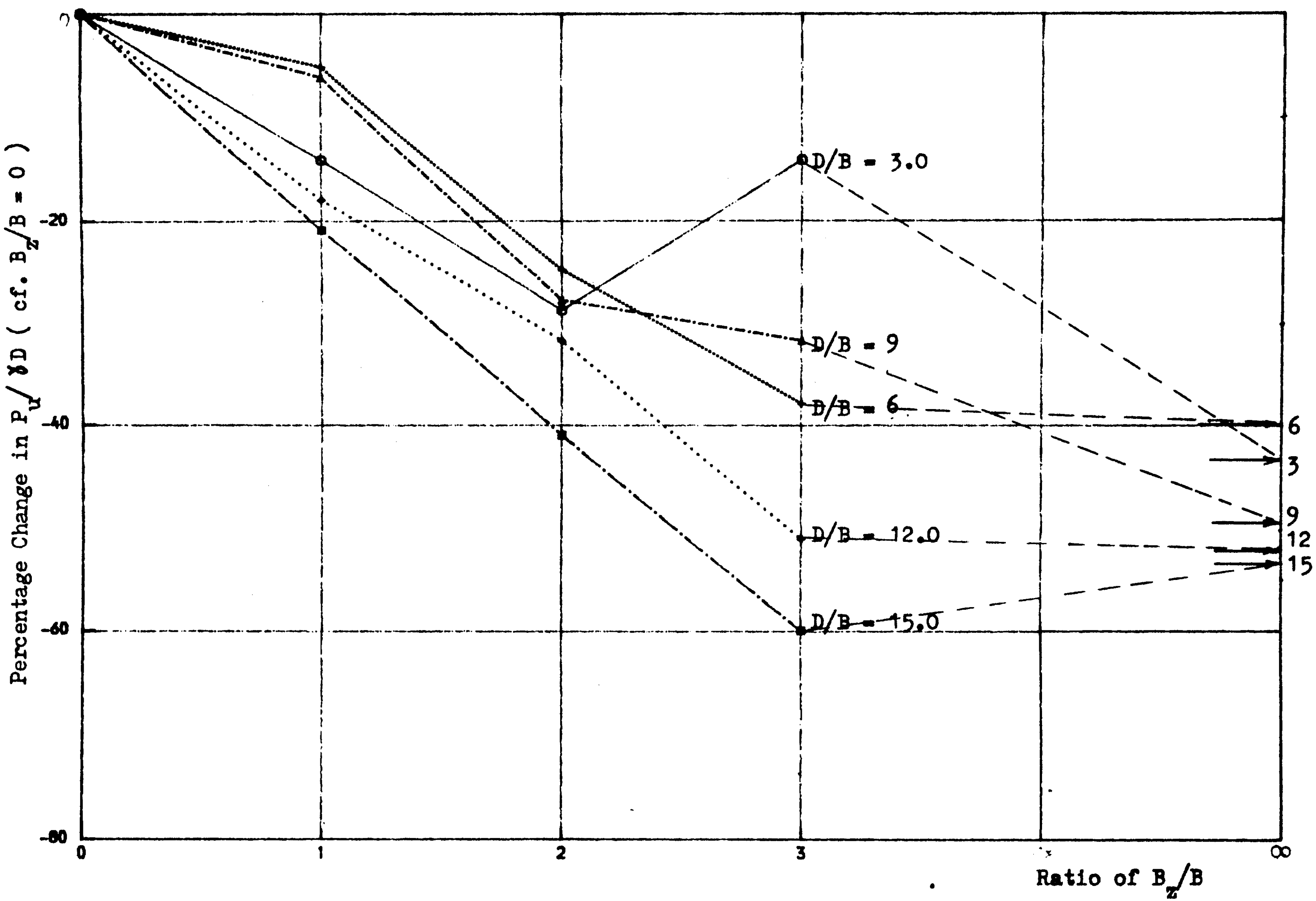



Fig. 5.7 Percentage Change Pullout Factor Caused by Installation Disturbance in Medium Dense Bed

zone.

However there was a net loss of about 37% ( from +13% to -24% ) when $B_z/B$ was further increased from 1 to 2. Beyond the ratio of D/B of about 9.7 i.e. at the point where the curves for $B_z/B = 0$ and B /B = 1 met, the load on the anchor continued to drop in the disturbed zone and reaching a maximum value at D/B ratio of approximately 15.

## 5.8 Load Displacement Behaviour of Anchor in a Disturbed Zone

Fig.5.8 shows a load-displacement behaviour curve for a 25mm-diameter anchor embedded at D/B = 15 in a homogeneous and disturbed bed. It was not possible to show the curve for each of the load-displacement behaviour of the anchor because of the large number of tests involved. It was a representative curve and it served as a comparison in the ultimate load between a load-controlled and a displacement-controlled test.

The uppermost curve shows a load-controlled test on the anchor and it clearly indicated that at its highest load obtained by the anchor i.e. at failure the displacement was significantly large with a nominal decrease in the load. Thus according to Matsuo ( 1964 ), the load at failure which occurred at a displacement of about 4mm should not be taken as the ultimate load. Instead a stage load which occurred immediately before failure at a displacement of about 3.9mm should be taken as the ultimate load of the anchor as shown in the figure.

The rest of the curves was obtained by displacement-controlled tests on the anchor. As was expected a distinct

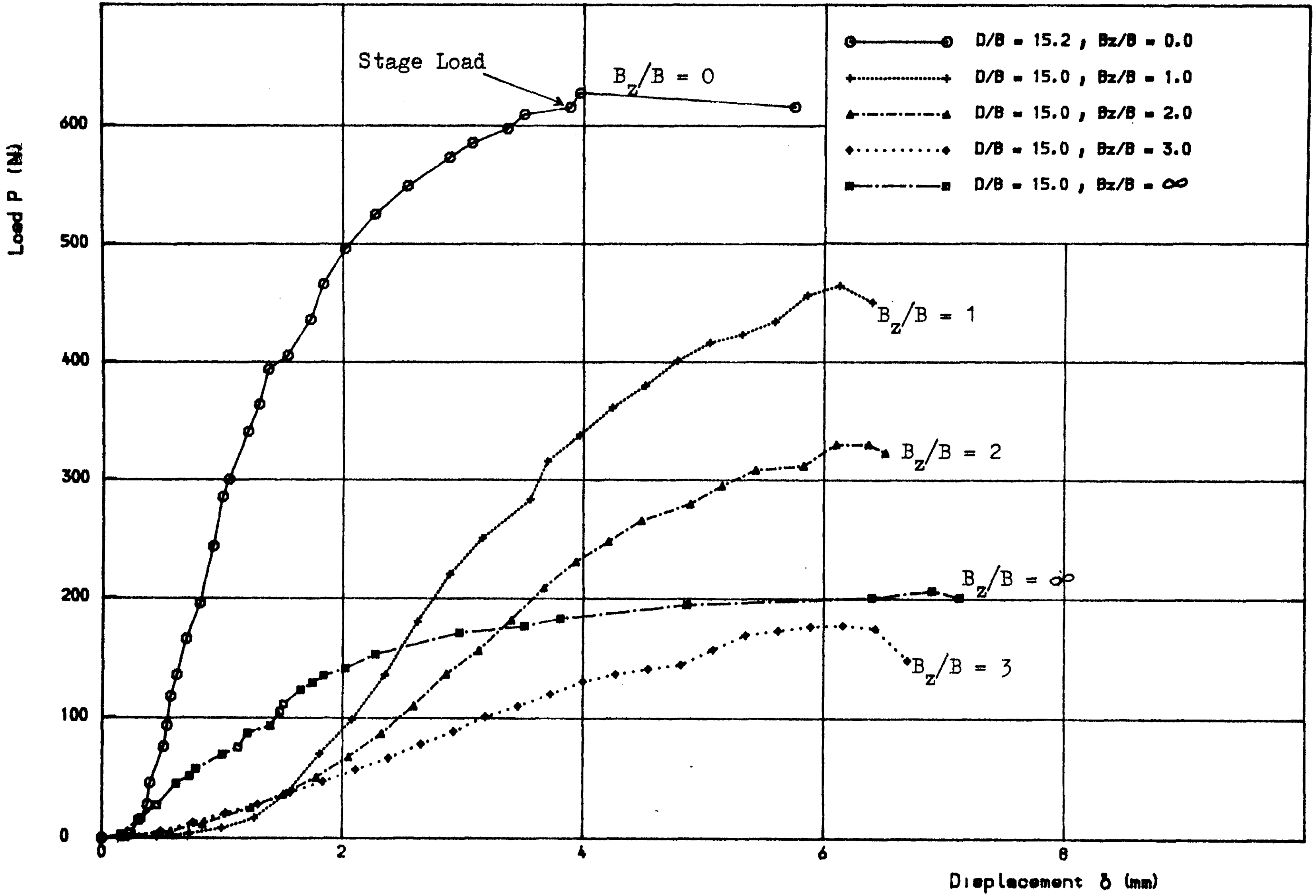



Fig. 5.8 Load-Displacement Curve for 25mm-∅ Anchor in Homogeneous and Disturbed Bed

peak load was visible and it was taken as the ultimate load of the anchor.

In a dense homogeneous sand bed ( $B_z/B = 0$ ) the load-displacement curve was initially steep. The increase in load was faster at smaller displacement intervals. This was because the shear strength of the sand was immediately mobilised when the anchor was pulled out from the bed. As the sand was dense and its degree of interlocking was high the load from the anchor transferred to the sand was only required to break out the soil mass rather than to first densify the soil mass above the anchor plate.

On the other hand as the ratio of $B_z/B$ increased the anchor was actually pulled out from a loose sand zone although the surrounding bed was denser. At the beginning of the test the load was initially required to compact or densify the sand region directly above the anchor plate as shown by Koslov ( 1966 ) and Kostyukov ( 1967 ). As displacement increased the shear strength of the densified sand zone started to be mobilised and the load on the anchor started to pick up until failure occurred.

In a homogeneous sand bed the particles were displaced upward as well as in the lateral direction as shown by Kostyukov ( 1966 ) and Hanna ( 1971 ). But in a disturbed zone the sand particles could only move in the vertical direction because the sand was weak to continue its movement into the denser sand bed surrounding it.

## 5.9 Conclusions

From the preceding discussions the following conclusions can be made;

(1). <u>Pertaining to Tests in Homogeneous Sand Beds</u>

a). The experimental results were consistent with Fadl's test series. Close agreement was achieved between Fadl's relative density 50% and the author's relative density 49% throughout the range of D/B ratios from 3 to 15.

b). Consistency was also obtained with Sutherland's and El-Rayes's test results at shallow depths.

c). The author's results agreed with the principles of dimensional analysis as proposed by Sutherland and Baker and Kondner.

d). There was a critical depth embedment ratio (H /B) beyond which an anchor should behave as a deep anchor. In the test series the H /B ratios were found to be 9.0, 8.0 and 7.2 at relative densities 92%, 70% and 49% respectively.

(2). <u>Pertaining to Tests in Disturbed Sand Beds</u>

a). Installation disturbance had siginificant effects on the anchor pullout capacity.

b). For $B_z$ /B = 1 the pullout capacity was not significantly reduced. This means that the width of excavation should be kept to a minimum. For a plate anchor the minimum possible value of $B_z$/B is 1.

c). When the excavation width in a dense and medium dense sand bed was increased to three anchor diameters, the anchor pullout capacity obtained was similar to that anchor as if it were pulled out from a bed which was wholly disturbed throughout the soil mass.

d). There was no way to check the validity and consistency of the test results because no published data was available for comparisons and comments.

e). In no way was the claim made that the method of forming the disturbed zone was perfect. It was certain that there were local effects associated with the method which might have not been eradicated indirectly.

# CHAPTER 6

# GENERALISED METHOD OF DETERMINING THE ANCHOR PULLOUT CAPACITY IN A DISTURBED ZONE.

## 6.1. Introduction

The influence of installation disturbance on anchor pullout has been mentioned in Section 5.6. No existing method appears to take into account the effect of disturbance, and in particular no attempt appears to have been made to quantify anchor pullout in a disturbed zone and established its relationship to pullout capacity in a homogeneous sand bed.

In this chapter an attempt will be made to give an insight into the response mechanism of an anchor embedded in a disturbed zone. The method proposed for the pullout capacity is different from the normally embedded anchor in a homogeneous bed, in that for backfilled anchors and neat excavation ( $B_z/B = 1$ ) the lesser properties of either the loose sand ( disturbed zone ) or host soil ( homogeneous sand bed ) govern. For backfilled and over-excavation ( $B_z/B > 1$ ) the backfill ( loose sand ) properties govern the anchor pullout.

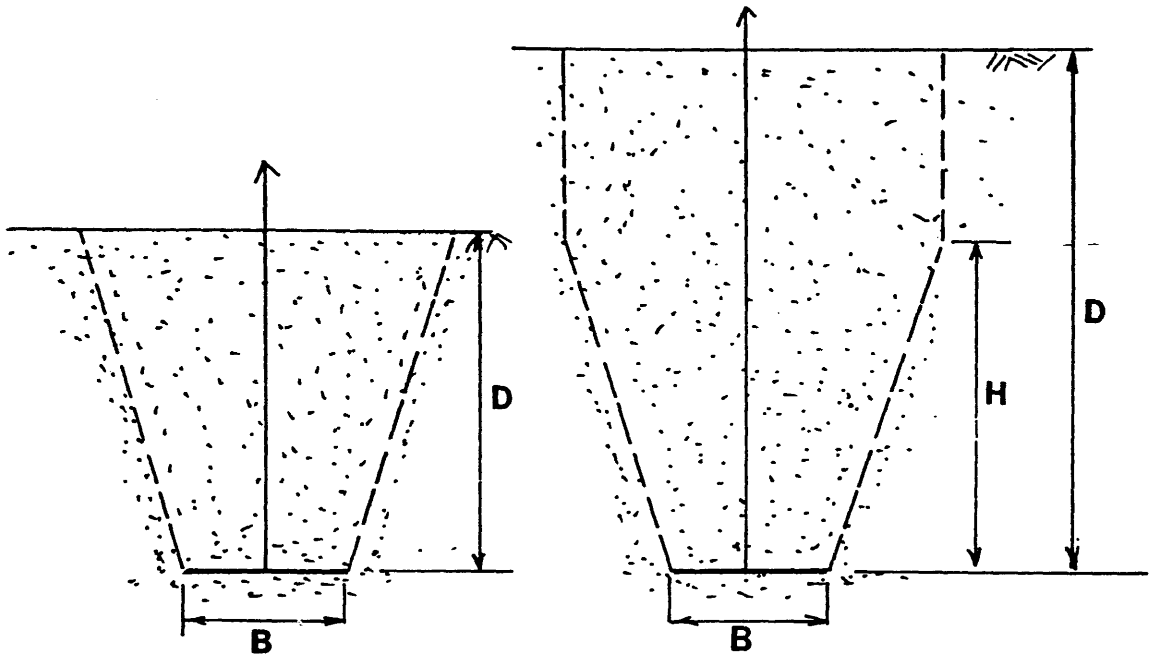
For an anchor embedded in a homogeneous bed, various theories have been put forward by many investigators such as Balla (1961), Matsuo (1964), El-Rayes (1965), Mariupol'skii (1965), Meyerhof and Adams (1968), Khadilkar (1971), Vesic (1972), Fadl (1981), Rowe and Davis (1982), and others. Most of the pullout theories recognise the difference between shallow anchor failure and deep anchor failure as shown in Fig. 6.1 with the transition depth between the two mechanisms occurring at a value between 4 and 8 i.e. 4 < D/B < 8. The method proposed for the pullout capacity in a disturbed zone assumes that the failure surface is curtailed at the disturbed zone boundary because as discussed above the lower strength of the loose sand volume governs the anchor pullout and the failure surface. Thus the failure surface for shallow and deep anchor will then conform to the anchor geometry as shown in Fig. 6.3.

## 6.2 Behaviour of Sand Bed During Anchor Installation

Due to the restriction imposed by the inclusion of the loose sand volume as shown in Fig. 6.2, the failure surface for the shallow and deep anchor will be identical. At the outset it is assumed that the density of the loose sand (disturbed zone ) will always be less than the surrounding homogeneous sand bed (host soil ). Thus the lower strength of the infill (loosened material ) will govern the strength of the anchor pullout capacity (Kulhawy 1985).

In the host soil (basic dense bed ) the stresses will relax upon excavation. If the infill is placed or dumped loose the host soil will have a relaxed stress and the infill will have a stress ranging from active ( if dumped ) to normally-consolidated ( if lightly compacted ). For an over-excavation the infill ( loose material )properties normally will control because the shear surface will be within the the infill and generally will not be influenced by the host soil ( dense homogeneous bed ).

In the laboratory the sand was prepared in such a way




(a) Shallow Anchor (b) Deep Anchor

Fig.6.1 Failure Surface in Homogeneous Sand Bed

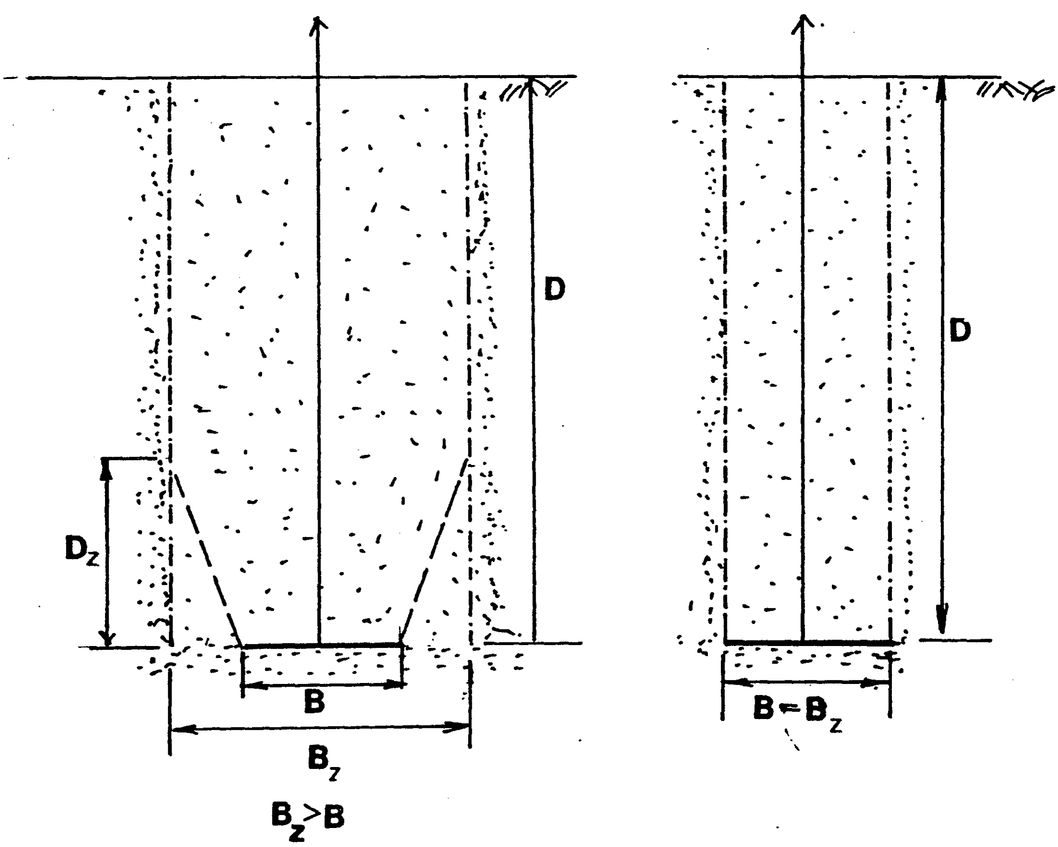



Fig. 6.2 Suggested Failure Surface in Disturbed Zone

that an at-rest condition was induced. The bed was laid in layers and the sand fell vertically onto the surface at a low velocity and so it could be assumed that no lateral stresses greater than the at-rest condition were set up. But in the formation of the loose sand volume i.e. the disturbed zone, as the tube was being withdrawn layer by layer a change in the lateral stresses could occur. In the present problem as the width of the disturbed zone $B_z$ becomes larger while the anchor diameter B remains constant there is a decrease in the effect of the lateral stresses from the dense sand bed on the region in the vicinity of the anchor plate. Therefore the anchor resistance in the disturbed zone will be reduced. However as the ratio of $B_z/B$ becomes infinite the anchor capacity in the disturbed zone should not be less or more than its capacity in the loose homogeneous sand bed available.

## 6.3 Assumptions

In the proposed method, provided the ratio of the disturbed zone to the anchor diameter ( $B_z/B$ ) is greater than one i.e. $B_z/B > 1$, the initial failure surface is assumed to be similar to that proposed by Fadl (1981) because Fadl carried out similar tests programme in Leighton-Buzzard sand but at different relative densities. As the boundary between the disturbed zone is reached the failure surface will occur along the boundary interface. Fig. 6.3 shows the generalised failure surface for the anchor embedded at a shallow and great depth, and the following conditions are assumed.

1). The density of the loose sand volume is always constant.

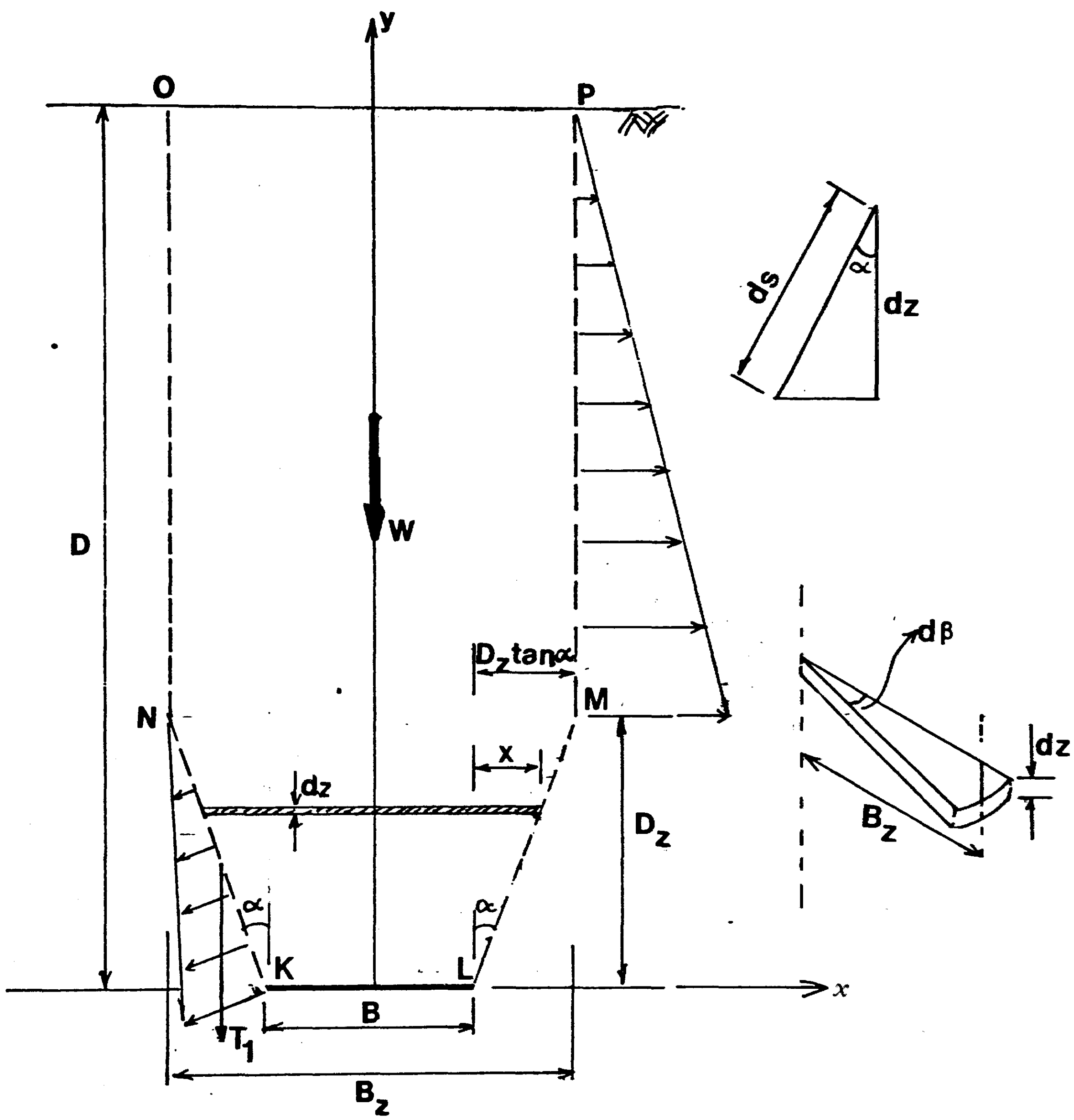



Fig.6.3 Generalised Failure Surface for Anchor in Disturbed Zone

2). The soil within the disturbed zone is homogeneous, isotropic and cohesionless.

3). The initial inclined failure surface makes an angle $\alpha$ with the vertical through the anchor edge in both cases of anchor embedment. $\alpha$ was defined in terms of relative density and $\varphi$ by Fadl ( 1981 ) as given in Section 2.8.

4). The final failure surface occurs along the pre-determined cylindrical boundary between the disturbed zone and the homogeneous sand bed.

5). The interface between the two states of sand is considered rough to allow full mobilisation of friction.

## 6.4 Determination of the Uplift Resistance of an Anchor in a Disturbed Zone

The uplift resistance of an anchor in a disturbed zone can be given as the sum of,

1). The weight of soil within the failure surface

2). The shear resistance that develops along the failure surface. This failure surface consists of two parts. Initially the failure surface is inclined making an angle $\alpha$ with the vertical through the anchor edge. But as the surface reaches towards the interface the shear resistance is then mobilised along the vertical cylindrical surface of the interface which is vertical. The uplift resistance can be written as,

$P = G + W + T$ where

$P$ = Ultimate Uplift Resistance

$G$ = Weight of Anchor and Accessories

W = Weight of Soil within the failure surface

T = Vertical Shear Resistance along the failure surface

6.4.1 Weight of Soil KLMN

Referring to Fig. 6.3 the soil components which resist the pullout force consist of two parts whatever the depth of the anchor i.e. the soil in the truncated cone at the bottom and the soil in the vertical cylindrical zone above it.

1). Weight of soil in truncated cone

Referring to Fig. 6.3 between the limits of intergration, the volume of the truncated cone can be written as,

$$V = \int_0^{D_z \tan\alpha} \int_0^{2\pi} \left(\frac{B}{2} + x\right)^2 d\beta \; dz \quad \text{where} \quad D_z = \frac{1}{2}(B_z - B)\cot\alpha$$

$$= \int_0^{D_z \tan\alpha} \int_0^{2\pi} \left(\frac{B}{2} + x\right)^2 d\beta \; \frac{dx}{\tan\beta} \quad \text{where } dz = \frac{dx}{\tan\alpha}$$

$$= \frac{\pi}{\tan\alpha} \int_0^{D_z \tan\alpha} \left(\frac{B}{2} + x\right)^2 dx$$

On integrating it can be shown that the weight of the soil is,

$$W = \frac{\pi \gamma D_z}{12} \left[ 3B^2 + 6BD_z \tan\alpha + 4D_z^2 \tan^2\alpha \right]$$

2). Weight of soil in cylindrical section MNOP

By taking an element of soil of thickness dz at a depth z above MN, the elemental volume can be given as,

$$dV = \int_0^{D-D_z} \int_0^{2\pi} (B_z/2)^2 \, d\beta \; dz = \frac{B_z^2}{8} \int_0^{D-D_z} dz \int_0^{2\pi} d\beta$$

$$V = \frac{\pi}{4} B_z^2 \; ( D - D_z )$$

$$W = \frac{\pi}{4} B_z^2 \gamma \; (D - D_z )$$

### 6.4.2 Frictional Resistance

Again the frictional resistance consists of two parts. Firstly shear resistance occurs along an inclined plane extending from the anchor edge to the boundary of the loose sand zone and the homogeneous sand bed making an angle $\alpha$ with the vertical Fig.6.3. Beyond this point as the backfill is weak, the inclined failure plane is not able to mobilise into the dense bed. Instead the failure surface will be in the form of a vertical curved plane.

1). Shear Resistance along inclined failure plane

Kotter's equation for the variation of shear resistance along a curved surface can be given as,

$$\frac{\partial P}{\partial s} + \frac{\partial \theta}{\partial s} \partial P \tan\varphi = \gamma \sin\theta$$

For a simplified failure surface(straight line) $\frac{\partial \theta}{\partial s}$ $=0$ therefore for the present problem, $\frac{\partial P}{\partial s} = \gamma \sin\alpha$

Matsuo (1968) used the Kotter equation to deduce an expression for the shear resistance along the rupture surface.

$$P_1 = \gamma (y - D) \sin\alpha$$

Total shearing resistance,

$$T_1 = \int_0^{D_z \tan\alpha} \int_0^{2\pi} P_1 \left(\frac{B}{2} + x\right) d\beta \; ds \quad \text{and} \quad ds = \frac{dx}{\sin\alpha}$$

$$\text{But} \quad z = D - y = \frac{D_z \tan\alpha - x}{\tan\alpha} \quad \text{and} \quad ds = \frac{dx}{\sin\alpha}$$

$$P_1 = \gamma z \sin\alpha \text{ ( downwards)}$$

$$T_1 = \int_0^{D_z \tan\alpha} 2\pi \sin\alpha\, \gamma \left(\frac{D_z \tan\alpha - x}{\tan\alpha}\right)\left(\frac{B}{2} + x\right)\frac{dx}{\sin\alpha}$$

$$T_1 = \frac{\gamma B \pi D_z^2 \tan\alpha}{6} ( 2 + B_z/B )$$

2). Shear Resistance along vertical cylindrical plane

As shown in Fig. 6.3 the vertical shear plane occurs along the sand interface. Taking an element of soil at depth z below the surface the vertical stress is $\gamma z$. The lateral stresses acting on the interface is $K\gamma z$ where K is the lateral earth pressure coefficient. This lateral stress acts on an element of area of $\frac{1}{2} B_z d\beta dz$.

The lateral force acting perpendicular to the surface is,

$$dS = K\gamma z \frac{1}{2} B_z \, d\beta \, dz \tan\varphi$$

The total shear force acting on the surface can be given as,

$$S = \int_0^{D-D_z} \int_0^{2\pi} K\gamma z \frac{1}{2} B_z \, d\beta \, dz \tan\varphi$$

On integrating it can be shown that,

$$S = \frac{\pi}{2} K\gamma B_z \tan\varphi \, (D - D_z)^2$$

The total anchor resistance can thus be given as,

$$P = \frac{\pi\gamma D_z}{12}\left[3B^2 + 6BD_z\tan\alpha + 4D^2\tan^2\alpha\right] + \frac{\pi}{4} B_z \gamma (D - D_z)^2 + \frac{\pi\gamma D_z \tan\alpha}{6} ( 2 + B_z/B) + \frac{\pi}{2} K\gamma B_z \tan\varphi ( D - D_z )^2$$

### 6.4.3 Choice of K

The ultimate resistance of the anchor as given in Section 6.4.2 depends very much on the frictional resistance along the vertical failure surface. This is because for a particular value of $B_z/B$, $D_z$ is constant and the weight of the soil in the truncated cone will be constant. The weight of the cylindrical soil section does not contribute much to the total ultimate uplift resistance of the anchor. Therefore neglecting the terms other than the one involving K the total ultimate resistance can be given as,

$$P = \frac{\pi}{2} K \gamma B_z \tan\varphi \, ( D - D_z )^2$$

But for a particular $B_z/B$ value, $D_z$ is constant.

$$P = cK \tan\varphi ( D - D_z )^2 \quad \text{where } c = \frac{\pi}{2} B_z \gamma$$

Thus the load P on the anchor depends on K, $\varphi$ and $( D - D_z )^2$. But for particular values of B, $B_z$ and D $( D - D_z )^2$ could be evaluated.

The deciding factors for the ultimate resistance P of the anchor are thus K and $\varphi$. For an anchor embedded in a disturbed zone the lower strength of either the infill ( disturbed zone ) of the host soil ( homogeneous sand bed ) governs the anchor pullout load as discussed earlier. So the values of K and $\varphi$ for the disturbed zone should be used in the calculations. The value of $\varphi$ for the disturbed zone was $36.3^\circ$ ( R.D. = 49% ) leaving only the appropriate value of K to be adopted for the ultimate load. If the tentative guidelines suggested by Kulhawy ( 1985 ) were adopted then the appropriate values of K are as follows;

$$K = K_a = \tan^2( 45 - 36.3^\circ/2 ) = 0.256$$

$$K = K_p = \tan^2( 45 + 36.3^\circ/2 ) = 3.906$$

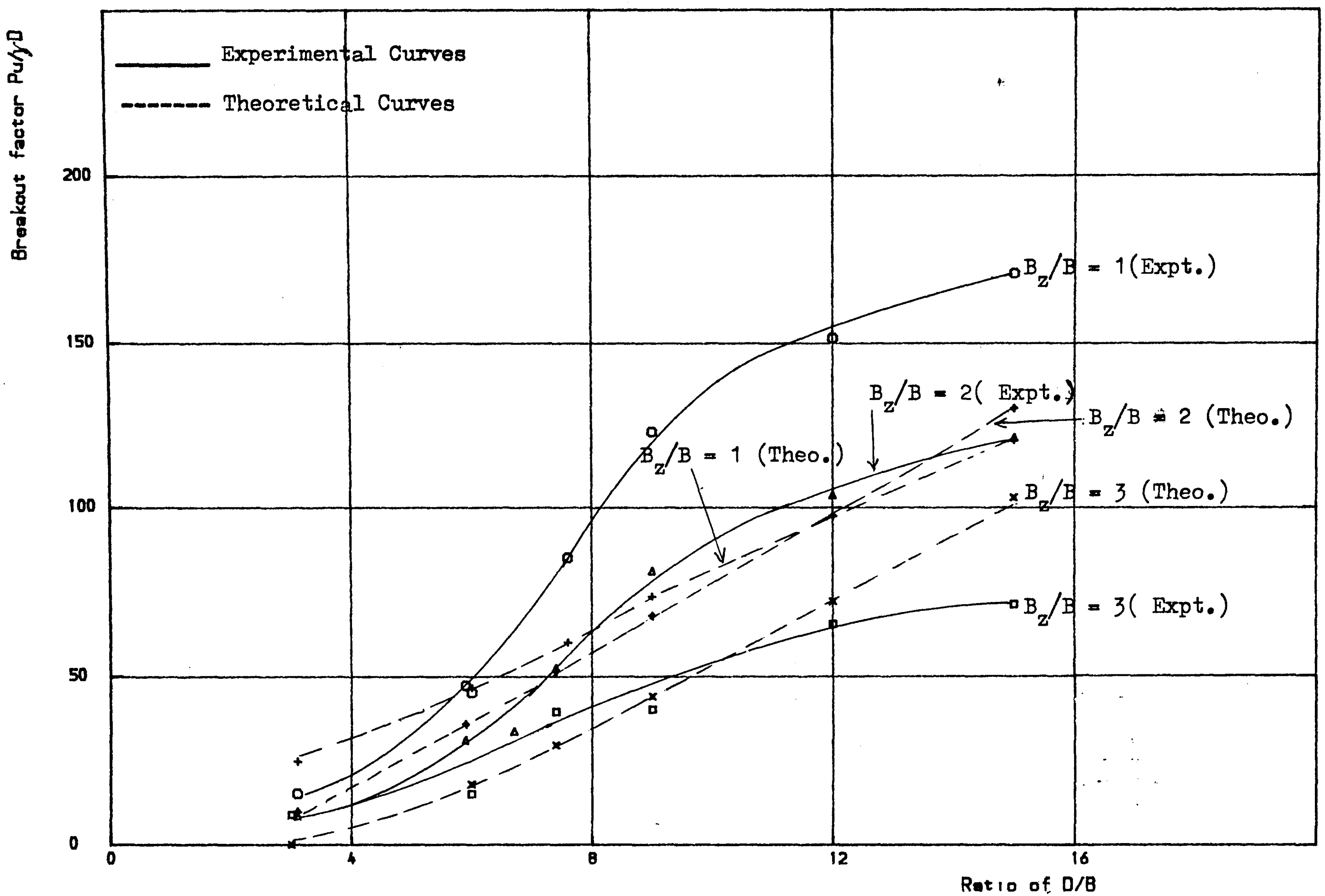



Fig.6.4 Comparison between the Generalised Method and Experimental Results ( Basic Dense Bed)

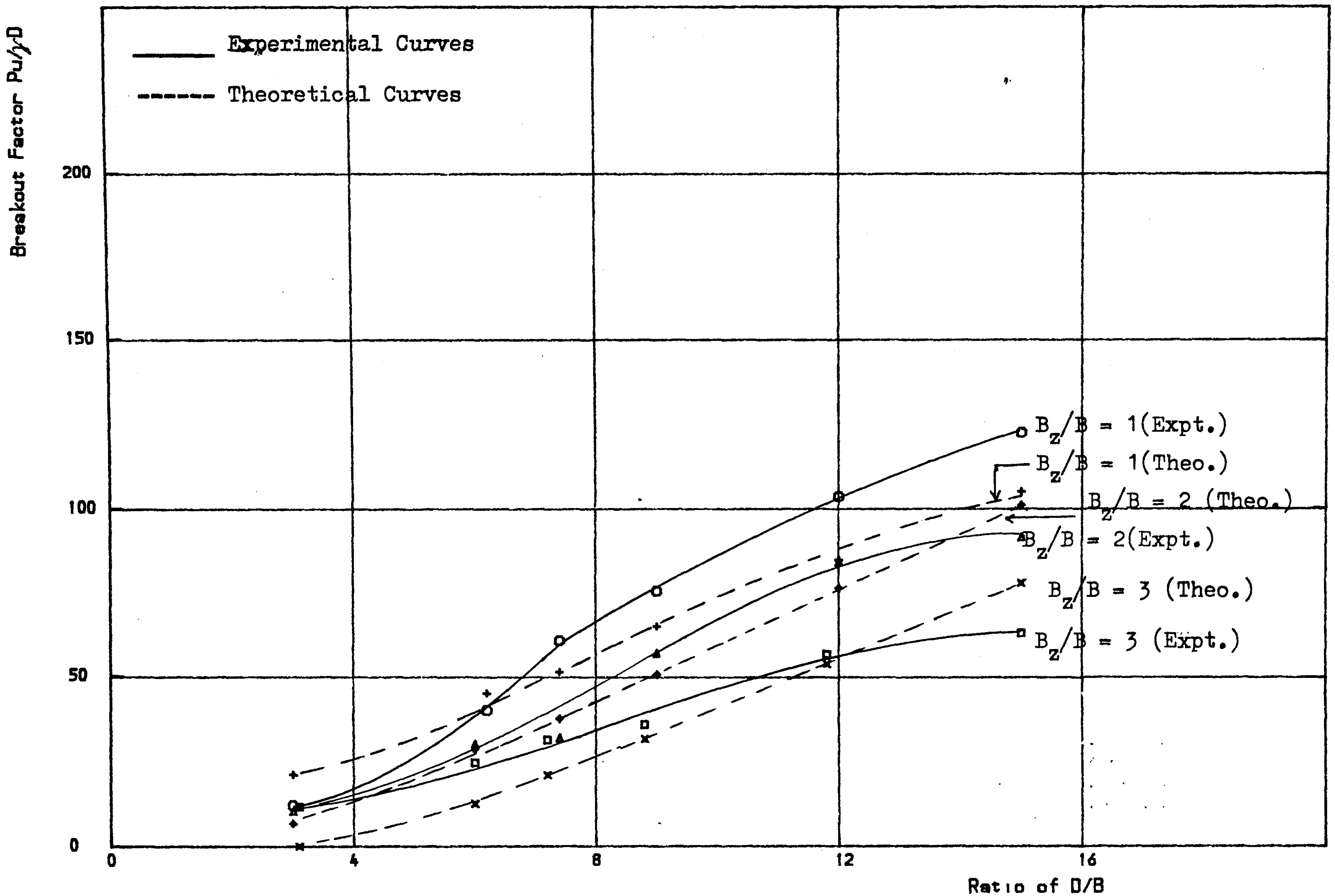



Fig. 6.5 Comparison between the Generalised Method and Experimental Results (Basic Medium Dense Bed)

$K = K_{onc} = 1 - \sin 36.3^{o} = 0.408$

According to Kulhawy for neat excavation ( $B_z/B = 1$ ) the overall value of K that should be used is given as,

$K = K_b K_h$ where

$K_b$ = coefficient of lateral earth pressure of host soil which varied from $K_a$ to $K_{onc}$

$K_h$ = coefficient of lateral earth pressure of infill

= 2/3 $K_o$ where $K_o$ is the in-situ horizontal stress coefficient

If the value of $K_h$ for the host soil was used then the theoretical uplift load on the anchor in the disturbed zone would be very small compared to its experimental load. So it followed that the choice did not give a satisfactory solution to the problem. The most appropriate value of K was thus $K_p$( = 3.906 ) but this choice would have violated the tentative guidelines as given in Table 2.2. Nevertheless this choice seemed to show reasonable agreement with the experimental results.

Similarly in the case of $B_z/B > 1$ the tentative guidelines did not apply to the present investigation. For instance, the surrounding sand bed did not influence the anchor uplift load in the disturbed zone as reported by Kulhawy. So the value of K that should be used for the infill or disturbed zone varied from $K_a$ to $K_{onc}$ .

From the arguments above it followed that the tentative guidelines did not apply at all to the present problem. It was found that in all cases the values of K that should be used in the theoretical uplift load were in the range from 1 to $K_p$ where $K_p$ was the coefficient of passive lateral earth pressure of the disturbed zone. The following K values were adopted for plotting the curves shown in Fig.6.4 and Fig.6.5.

| $B_z/B$ | Value of K |
|---|---|
| 1 | 3.906 ( = $K_p$ ) |
| 2 | 2.510 |
| 3 | 1.670 |

## 6.5 Comparison with Experimental Results

From Fig.6.4 it is seen that for $B_z/B = 1$, the experimental results overestimated the theoretical load for the range of D/B ratios from 3 to 15. However agrement was obtained to a certain extent for $B_z/B = 2$ and 3 upto D/B ratios of about 15 and 12 respectively.

In the case of medium dense bed as shown in Fig. 6.5 the theoretical curve for $B_z/B = 1$ overestimated the anchor pullout capacity in the dense homogeneous bed upto D/B ratio of about 6.5 beyond which it underestimated the experimental results. For $B_z/B = 2$ reasonable agreement was obtained between the theoretical curve and the test results while for $B_z/B = 3$ the theoretical curve underestimated the test results upto a D/B ratio of about 12. Beyond this ratio the theoretical curve seemed to show higher $P_u/\gamma D$ values compared to the experimental results.

Generally it could be concluded that the theoretical analysis gave a reasonable solution to the anchor capacity in a disturbed zone for $B_z/B = 2$ and 3.

# CHAPTER 7

## SUGGESTIONS FOR FURTHER WORK

From the experiments it was shown that the installation disturbance greatly reduced the anchor pullout capacity. Also it was shown that there were some anomalies in the tests results themselves which gave rise to the consideration in the method of forming the disturbed sand zone in the test bed. In this connexion further work is still needed to find a better technique of simulating the effect of installation disturbance on the anchor.

1). Different types of sand should be used to generalise the effect of disturbance on the anchor capacity quantitatively.

2). It might be interesting to evaluate the anchor uplift resistance in a disturbed zone by means of a finite element technique. So far the technique has been employed for a plate anchor in a homogeneous sand bed. The boundary effects from the sand container were normally ignored because they were taken some distances away from the anchor axis. But for a plate anchor embedded in a disturbed zone within a certain range of $B_z$ /B ratios, there exists a distinct separation boundary between the loose sand zone and the dense sand bed. Thus the boundary effects cannot be neglected in this case.

3). A photographic technique can be used to investigate the actual failure surface for the anchor that occurs within the disturbed zone. This method was used by many investigators previously to study the failure surface of an anchor in a homogeneous sand bed.

4). Anchors are sometimes required to withstand cyclic loads as found in the onshore and offshore operations. Thus a research can be conducted in the laboratory as a preliminary investigation into the behaviour of an anchor in a disturbed zone under cyclic loading conditions.

The behaviour of an anchor in a disturbed zone is not well understood. Upto to now there has been no comprehensive study being undertaken to investigate this particular problem although from a practical point of view this zone of disturbance does exist to a certain extent. Perhaps the research can be extended to investigating similar effects in cohesive soils.

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