Pullout capacity of earth anchors

Capacité de résistance à l'arrachement d'ancrages souterrains

R.S.Merifield, A.V.Lyamin, A.Pearce, H.S.Yu & S.W.Sloan – *University of Newcastle, NSW, Australia*

ABSTRACT: The design of many engineering structures require foundation systems to resist vertical uplift or horizontal pullout forces. These types of structures, which may include transmission towers or earth retaining structures, are commonly supported directly by soil anchors. This paper presents results from laboratory testing of circular anchors in sand, and three dimensional numerical limit analyses of circular anchors in sand. The results presented are part of a large ongoing research project at the University of Newcastle on the stability of earth anchors.

RÉSUMÉ: La conception d'un bon nombre de structures de génie civil nécessite des systèmes de fondations capable de résister à des efforts d'arrachement horizontaux et verticaux. Ces types de structures, qui incluent entre autres les tours élancées maintenues par câbles et les structures de soutènement, sont directement supportées par des ancrages dans le sol. Les résultats présentés ici sont issus d'un grand projet de recherche, actuellement mené à l'Université de Newcastle, qui porte sur la stabilité des ancrages.

1. INTRODUCTION

The University of Newcastle is currently undertaking a large research project entitled "Theoretical and Experimental Investigation of Earth Anchors". The aim of this research project is to develop rigorous stability solutions for earth anchors and to verify these solutions with high quality data obtained from laboratory pullout tests.

Soil anchors can be square, circular or rectangular in shape and are commonly used as foundation systems for structures requiring uplift resistance, such as transmission towers, or for structures requiring lateral resistance, such as sheet pile walls. As the range of applications for anchors expands to include the support of more elaborate and substantially larger structures, a greater understanding of their behaviour is required.

The general layout of the problem to be analysed is shown in Figure 1.

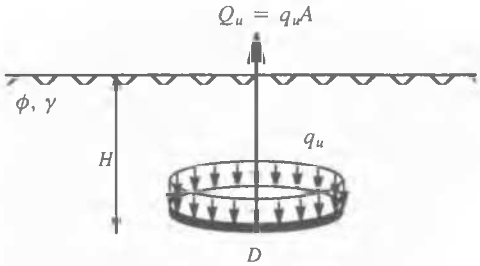

Figure 1. Problem Definition

Laboratory and numerical studies have been performed for problems where the embedment ratio (H/D) varies from 1 to 15. It is anticipated that this will cover most problems of practical interest.

During the last thirty years various researchers have proposed approximate techniques to estimate the ultimate uplift capacity of anchors in sand. The majority of past research has been experimentally based and, as a result, current design practices are largely based on empiricism (Meyerhof and Adams, 1968; Baker and Kondner, 1965; Murray and Geddes, 1987; Sakai and Tanaka, 1998; Dickin, 1988, Tagaya *et al.* ,1988).

Very few rigorous numerical analyses have been performed to determine the pullout loads of circular anchors in sand. To date, the majority of numerical analyses have been limited to plane strain problems whereby the anchor is idealised as a continuous strip. A condition of plane strain is typically assumed for numerical convenience. However, in reality anchors come in various shapes and sizes and therefore it is unlikely that the assumption of plane strain will be valid for all problems. This paper applies numerical limit analysis to evaluate the effect of anchor shape on the pullout capacity of horizontal anchors in sand.

Numerical estimations for the pullout capacity of circular anchors in sand can be found in the works of Tagaya *et al.* (1988), Sakai and Tanaka (1998), Saeedy (1987), Koutsabeloulis and Griffiths (1989), Ghaly and Hanna (1994) and Murray and Geddes (1987).

2. NUMERICAL ANALYSIS OF ANCHORS

The upper and lower bound methods constitute what are known as the limit theorems of classical plasticity, and were developed by Drucker *et al.*(1952). These theorems are applicable for perfectly plastic materials that obey an associated flow rule. Since their proof, the bounding theorems have provided a powerful tool for analysing stability problems in soil mechanics. Numerical upper and lower bound techniques have recently been used to study numerous problems including the undrained stability of a trapdoor (Sloan *et al.* 1990), the stability of plate anchors in undrained clay (Merifield *et al.* 1999a,b) and the bearing capacity of foundations (Merifield *et al.* 1999).

The most commonly used numerical implementation of the lower bound theorem is based on a finite element discretization of the soil mass. This type of approach has been widely used and is described in detail, for example, in Bottero *et al.* (1980) and Sloan (1988). Despite being quite successful over the last two decades, the linear programming approach is limited to dealing with two–dimensional problems. Indeed, the optimisation problem resulting from any discrete limit analysis formulation in three–dimensions cannot be easily reduced to a linear programming problem, and must be solved using the power of non–linear programming methods, such as those developed by Zouain *et al.* 1993.

Estimates of the ultimate pullout load within this paper were obtained using a recently developed three dimensional numerical procedure developed by Lyamin(2000), based on a finite element formulation of the lower bound theorem of limit analysis. Full details of the formulation can be found in Lyamin (2000) and Lyamin and Sloan (1997) and will not be repeated here.

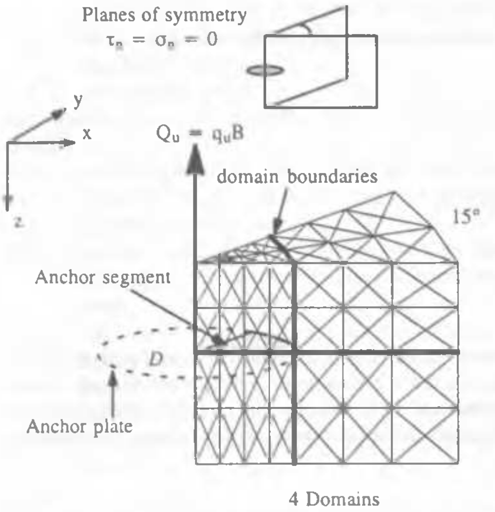

Figure 2 Circular anchor mesh arrangement

A simplified representation of the lower bound finite element mesh arrangement used to analyse circular anchors is illustrated in Figure 2. The soil mass is first discretized into a number of domains where the boundaries between adjacent domains may be specified as a stress discontinuity or rigid joint. Each domain is then subdivided in three dimensional space to form a number of tetrahedral elements within each domain.

By taking symmetry into account, the overall problem size can be reduced. For circular anchors anchors, symmetry can be used so that only a small 15 degree slice of the anchor needs to be analysed (Figure 2). The boundaries of domains lying on the planes of symmetry are subject to certain stress boundary conditions.

2.1. *Problem Definition*

The uplift capacity of anchors is typically expressed in terms of a bearing capacity/break-out factor which is a function of the anchor shape, embedment depth, overburden pressure and the soil properties.

For numerical convenience, the anchor capacity q_u will be presented in a form analogous to Terzaghi's equation which is used to analyse surface footings, namely:

$$q_u = \frac{Q_u}{A} = \gamma H N_\gamma \qquad (1)$$

where N_γ is referred to as the break-out factor, and A is the anchor area. Past experimental research has typically used equation (1) to back calculate the break-out factor after determining the ultimate anchor load from model anchor pullout tests.

Previous laboratory studies have suggested anchor behaviour can be divided into what is known as shallow or deep anchor behaviour based on the mode of failure. This is best illustrated by referring to Figure 3. An anchor is classified as shallow if, at ultimate collapse, the observed failure mechanism reaches the surface. In contrast, a deep anchor is one which is not affected by the location of the soil surface and failure takes place by localised shear contained around the anchor.

While it is generally agreed that shallow and deep anchor behaviour exists, the actual failure mechanisms associated with shallow and deep failures are unclear. Some authors suggest a transitional zone exists where failure modes gradually transform from shallow to deep (Ghaly and Hanna, 1994; Andreadis, Harvey and Burley, 1981). Other authors indicate no transitional zone or propose a critical depth below which anchors are 'deep' and above which anchors are 'shallow' (Meyerhof and Adams, 1968; Baker

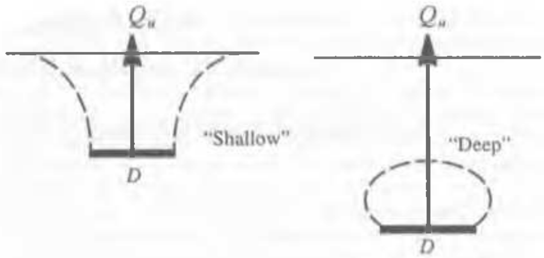

Figure 3. Shallow and deep anchor behaviour.

and Kondner, 1965; Murray and Geddes, 1987; Sakai and Tanaka, 1998; Dickin, 1988). This suggests an instantaneous transition from shallow to deep.While the overall geometry of each type of failure mode is distinct, the exact shape of the failure surface within each mode is an area of significant disagreement. As a result, significant variations exist between most approximate numerical solutions whereby an assumption is made regarding the shape of the failure surface.

2.2. *Results*

Finite element limit analyses were performed to obtain lower bound estimates of the anchor break-out factor N_γ for the range of embedment ratios previously mentioned. These results are discussed in the following sections. Where possible, past numerical results are compared to results obtained from the current study.

The computed lower bound estimates of the anchor break-out factor N_γ are shown graphically in Figure 4 for dense ($\phi = 40°$) and loose($\phi = 30°$) sands. Also shown in Figure 4 are the results obtained from existing numerical studies performed on circular anchors by various authors.

When compared to the lower bound finite element predictions, the limiting equilibrium solutions of Murray *et al.* plot slightly above the lower bound solutions for loose sand ($\phi = 30°$). For denser sands where $\phi = 40°$ however, the limit equilibrium results tend to be more conservative and plot between 2% and 20% below the lower bound results. In general the results of Murray *et al.* compare favourably with the numerical lower bounds solutions.

The solutions of Ghaly and Hanna are also based on the limit equilibrium method and appear to be only slightly unconservative when compared to the finite element results for denser sands ($\phi = 40°$). For looser sands ($\phi = 30°$), the results of Ghaly and Hanna (1994) tend to become more unconservative and plot between $20 - 30\%$ above the lower bound solutions.

The numerical estimates of Meyerhof *et al* (1968) and Saeedy(1987) plot well below the numerical lower bound estimates for looser sands ($\phi = 30°$), and compare only slightly more favourably for denser sands ($\phi = 40°$).

A comparison of the finite element lower bound solutions and the laboratory test results presented by several authors for dense sands is shown in Figure 5. All of these laboratory tests were performed on small scale model anchors between 38mm and 90mm in size. The lower bound results show reasonable agreement with the laboratory results. It should be remembered that inherent to the limit theorems is the principle of an associated flow rule, and as such the soil dilation ψ is assumed equal to the internal friction angle. In reality, as Rowe and Davis (1982) explained, the soil will exhibit friction and dilatancy for peak values of friction angle, yet may reduce to the ultimate value corresponding to zero volume change $\psi = 0°$ at large strains. As a result, it is anticipated that the lower bound solutions may over-predict the collapse load slightly, particularly for soils with high friction angles. However, the influence of soil dilation is yet to be fully verified in the literature.

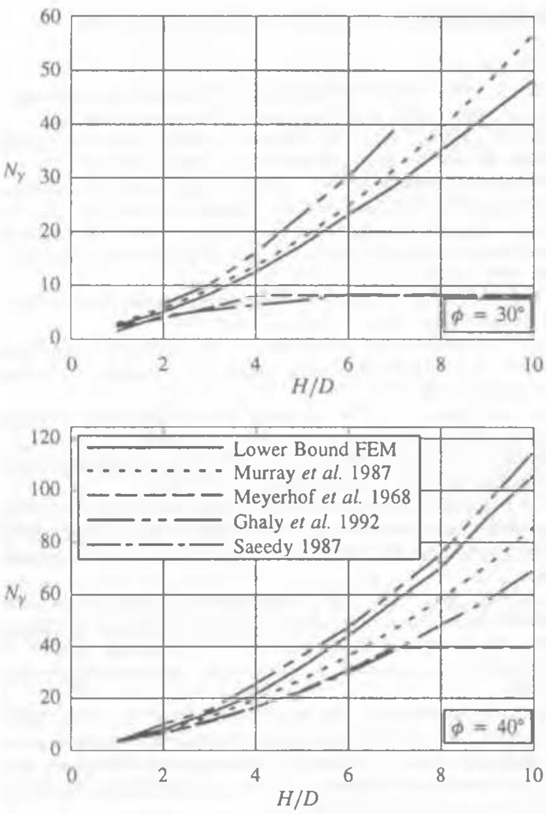

Figure 4 Numerical results and comparison

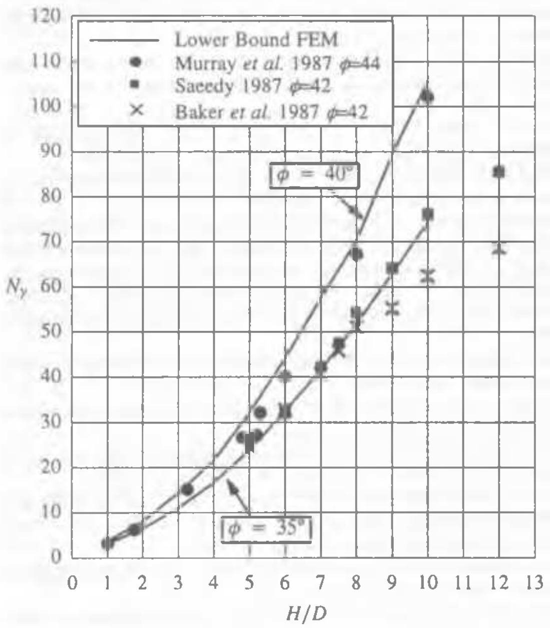

Figure 5 Comparison of numerical and experimental results

3. LABORATORY STUDY OF ANCHORS

3.1. *Background*

The aim of the laboratory investigation was to undertake an experimental study into the load capacity and deformation behaviour of plate anchors under controlled conditions. In order to do this a series of constant displacement rate tests were carried out in a rigid boundary calibration chamber system. A summary of the equipment and experimental results are presented in the following sections.

3.2. *Experimental Setup*

The calibration chamber used in this study was built at the University of Newcastle, Australia, by Ajalloeian (1996).The chamber is cylindrical in shape having dimensions of approximately 1 m in diameter and 1 m in height.

The model anchors used for pullout tests varied in diameter (D) from 50 – 125mm and were constructed from 8mm(t) mild steel. Anchors used in this study are generally larger in diameter than those used by previous researchers (Rowe and Davis 1982; Othman and Edil 1993; Andreadis, Harvey and Burley 1981). Anchor sizes were generally selected so as to minimise scale and boundary effects. Findings by Andreadis *et al.* (1981) suggested that an anchor with a diameter (D) of 50 mm or less would be influenced significantly by scale effects. Anchor sizes up to D=125mm were used to assess the likelihood of chamber boundary effects.

According to the unified Soil Classification System for engineering purposes, the sand (Stockton Beach Sand) is classified as a silty sand. The effective particle size D_{10}, uniformity coefficient C_u and coefficient of gradation C_c were estimated as 0.24mm, 1.71 and 1.32 respectively. Extensive testing by Ajalloeian (1996) indicates that $\phi_{cv} \approx 31°$.

3.3. *Experimental Procedure*

One hundred and three (103) constant displacement rate pullout tests were conducted on circular plate anchors buried in sand. The majority of pullout tests were carried out using a displacement rate of 3 mm/min. Four soil densities; ρ = 1.749, 1.698, 1.649 and 1.515 t/m^3, referred to as very dense, dense, medium dense and loose respectively, were investigated with anchors being buried between relative depths of H/D= 2 to 15.

3.4. *Results and Discussion*

For each test the load displacement response was constantly recorded allowing anchor behaviour to be observed. Figure 6 displays the typical load (in terms of the non-dimensional break-out factor)against normalised displacement response curves for 75 mm anchors buried in very dense sand.

Shallow anchor behaviour was observed to occur at H/D < 5, deep anchor behaviour at $H/D \geq 6$ and transitional behaviour at $H/D \approx 5$.

The load displacement response for an anchor buried at a relative depth of H/D = 2 can be seen to reflect a typical 'shallow anchor' response (Figure 6). Once the peak load is attained, a rapid decrease in load occurs, indicating relatively rapid failure of the soil. In comparison, for H/D = 6 and 7.5, the curves in Figure 6 reflect a typical deep anchor response. For these cases, no peak load hump is observed and the load tends to even out to an approximately constant value over a wide range of displacements.

In Figure 6 the curve for H/D = 5 represents a typical 'transitional anchor' response and displays characteristics of both shallow and deep anchor behaviour.

Load oscillation is a distinct feature for all load displacement curves and is significantly larger in magnitude for deeper anchors. For shallower anchors, oscillation commences at smaller displacements. The oscillation is due to the failure and flow of sand from just above the plate perimeter into the void that forms below the plate as displacement continues. As the flow of sand into the developing void increases so does the oscillation magnitude.

For shallow anchors (H/D < 5), Q_u was taken as the maximum load attained while for deep and transitional anchors ($H/D \geq 5$), Q_u was selected as the largest load attained prior to sudden change in oscillation behaviour. This zone where sudden oscillation changes occur in the load displacement curves are indicated in Figure 6.

3.4.1 *Break-out Factor*

From each load displacement response curve, the ultimate load, Q_u, was obtained and the break-out factor N_γ was back calculated using equation (1). Figure 7 presents a plot of N_γ against H/D for anchors buried in dense to very dense sand and medium dense sand respectively.

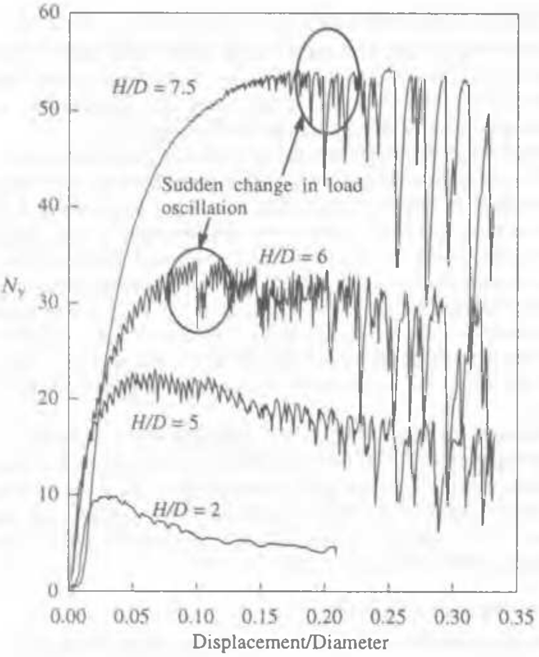

Figure 6 Load displacement response for a 75 mm anchor buried in very dense sand.

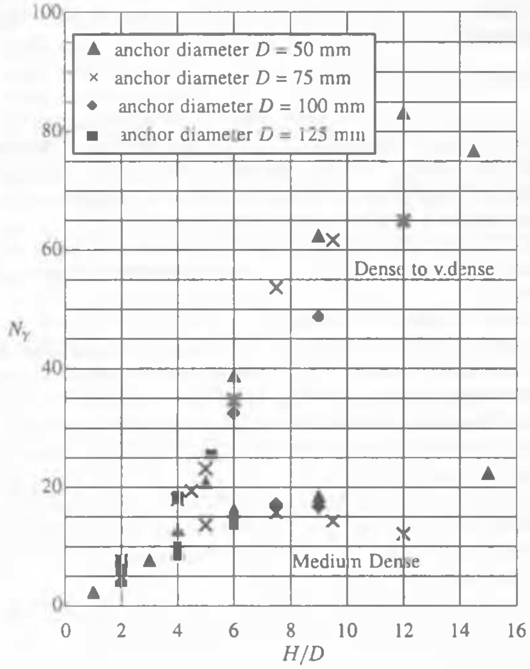

Figure 7 Laboratory test results

For dense to very dense sand, N_γ increases in a nonlinear fashion with *H/D* over the range of embedment depths analysed. For anchors buried in medium dense sand N_γ increases in a similar manner until reaching an approximate peak value at embedment depths greater than 8.

4. CONCLUSIONS

The pullout capacity of circular horizontal anchors in sand has been analysed using a recently developed 3 dimensional numerical procedure based on a finite element formulation of the lower bound theorem of limit analysis. These results compare favourably with laboratory pullout tests presented previously in the literature. A more comprehensive comparison of all laboratory and numerical results is currently being undertaken.

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