BEHAVIOUR OF A MODEL MV PILE IN SAND

BEARING CAPACITY IMPLICATIONS

JOHN EVELYN GUY

A dissertation submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in fulfilment of the requirements for the Degree of Master of Science in Engineering.

JOHANNESBURG, 1987

DECLARATION

I declare that this dissertation is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

(Signature of Candidate)

26TH day of NOVEMBER 198[illegible].

ABSTRACT

This report presents the results of a model MV pile test conducted in the laboratory and provides data on measurements of the displacement of sand around the base of the pile, as well as on the load distribution down the pile during testing and installation. The experiments were conducted in an attempt to clarify some of the bearing capacity characteristics of the MV pile. This is a driven displacement grouted in situ pile formed by using a mandrel to drive an over-sized pile shoe, the void behind the shoe being continually supplied with grout.

During the experiment, a 60 mm diameter semi-circular half pile was driven into a steel box filled with an approximately homogenous dense sand placed by a "raining" technique. The pile was installed with the sand submerged in water and with the flat face of the semi-circular pile shoe lying against a glass panel which formed one side of the sand box. Loads in the pile shaft were measured during pile installation and subsequent static load testing by strain gauges located in the drive mandrel. Photographs of the soil around the base of the pile were obtained at small penetration

increments and the displacement components of individual grains measured using a Wild A7 stereoplotter. Detailed contour diagrams of the soil displacements and the associated strain components are presented and discussed.

It was found that the lateral earth pressure co-efficients applicable to the shaft friction capacity approached those of the passive lateral earth pressure co-efficients. The possible slip-line field around the pile toe was found to differ significantly from that conventionally assumed in the theoretical determination of the bearing capacity factory, Nq. It was found instead to resemble that indicated by an analysis of the expansion of a cavity in an elastic solid. A possible explanation for the reduction of shaft friction capacity near a pile toe, referred to in the literature as an "arching" effect, may be due to slip lines originating at the pile toe interfering with those originating at the pile shaft.

A preliminary proposal for a design method for MV piles is presented on the basis of the model test results and selected field test results.

ACKNOWLEDGEMENTS

Many people have been involved in the completion of this report. To all of them, I wish to record my heartfelt gratitude for their help, support, encouragement and advice. Special mention should be made of the following:

The workshop and technical staff of the Department of Civil Engineering for their never failing good humour and companionship. In particular, Vince Newey and Bernie de Bernier - craftsmen both.

Professor J M Ridley and Dr G S Wells of the Department of Mathematics for their co-ordinate transformation technique.

My colleagues at Grinaker Dura Piling who have uncomplainingly tolerated the additional workload they have had to carry during my absences. Special thanks to Nico Maas, Brian McCartney and Chris Long.

Michel Antelme and his colleagues who graciously and patiently allowed me free rein in the use of their Wild A7 stereoplotter.

Dr Irvin Luker, who taught me a great deal, assisted me in all stages of this work, and who always made time for discussions.

Professor G E Blight for his support and patience.

Gunther Müller whose enthusiasm for this project never waned.

The Directors of Grinaker Dura Piling (Pty) Ltd for their continued support and for permission to publish this report.

The Directors of Grinaker Holdings (Pty) Ltd and, in particular Mr Wouter de Villiers and Mr Jack Saulez, for their considerable patience and support. Thank you.

Jean Campbell who cheerfully typed the text many times over.

Finally, I would like to thank my wife Lynda, and children Michelle and Carol, for their encouragement and stoic support.

Financial assistance was provided by the Grinaker Group of Companies.

TABLE OF CONTENTS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PHOTOGRAPHS

1. INTRODUCTION

1.1 Historical

Piles perform their load carrying function by transmitting the load applied at the pile head to the underlying soils through skin friction on the pile shaft and end bearing on the pile toe. Some piles generate most of their load carrying capacity through shaft friction - for example MV piles - and are therefore suitable as "floating" or "friction" piles whereas others, such as cased oscillator piles (eg Benoto piles), are more suited for use as end bearing piles.

Generally, each piling system is sufficiently different in both installation technique and load performance for different pile types of similar dimensions in the same soil to develop significantly wide ranging load capacities. Braadtvedt (1980) highlighted this point when he stated that, depending solely on installation technique, the capacity of a pile of given dimensions may vary by as much as a factor of six.

Piled foundations have been a recognised construction technique for many centuries. "Certainly, as far back as the time of Vitruvius we find piles being recommended when a suitable foundation cannot be found by excavating; the various lake dwellings in Europe provide examples of still earlier use. In Arrian's Life of Alexander the Great (Book II) a description is given of the use of piles for the construction of the mole used for the reduction of the city of Tyre (20th Aug., 332 B C)" (Little, [1961, p.128]). Caesar, in De Bello Gallico IV (translated by Handford [1951]) presents a detailed description of the use of piled wooden piers for a bridge constructed across the Rhine river.

Piles were not uncommon in the foundations of medieval structures. Little (1961, p.125), for example, mentions the use of piles in the construction of Winchester Cathedral (completed A D 1093) and Filarete (translated by Spencer [1965]) describes how piles would be used for construction of a bridge (Spencer, 1965 p.164 and 290). The cover panels of Chellis' book "Pile Foundations" (1961) present some excellent illustrations of medieval and primitive pile driving equipment.

"Until some time in the nineteenth century, wood piles were the only common type; since then, steel, concrete and composite piles of many kind have come into wide use" (Taylor, 1948, p.640). It is not altogether clear from the early literature what the exact sequence was in either the emergence of new pile types or the development of new installation and driving techniques. Poulos and Davis (1980 p.1) suggest that, "Modern literature on piles can be said to date from the publication of 'Piles and Pile Driving' edited by Wellington of the Engineering News ..... in 1893 .....". It is apparent, however, that the period 1850 to 1950 saw a proliferation of pile equipment and design techniques of large magnitude which reflected the rapid industrial expansion of the period. The industrialization of South Africa lagged somewhat behind that of the northern hemisphere countries and it was not until about 1938 that South Africa's first specialist piling contractor, McLaren and Eger, came into existence.

1.2 Pile Design

Increasing cost and frequency of use of pile foundations during the last century have obliged the foundation engineer to develop methods to predict more accurately and reliably the performance of the piles specified in his design. "..... until the late nineteenth century, the design of pile foundations was based entirely on experience, or even divine providence." (Poulos and Davis, op cit). Despite the considerable volume of published literature on the subject, both practical and theoretical research into pile performance parameters have advanced hesitantly with the result that today pile design still tends to be largely dependent on empirical data and thus on the experience of the designer.

Nevertheless, the design of a single pile foundation for ultimate axial capacity is conveniently divisible into three major approaches which may be applied singly or in combination. These include use of static bearing capacity formulae, dynamic bearing capacity formulae and empirical approaches based on past experience and/or on in situ test methods. Chapter 2 presents a brief review of these topics.

This report presents the results of an attempt to obtain data on both the installation characteristics and loading behaviour of a laboratory scale model pile installed in sand. The pile type used for this investigation was a driven displacement grouted in situ pile known as the MV pile. Features of the MV pile are described in Chapter 3.

2. PILE DESIGN

2.1 Static Bearing Capacity Design

Static bearing capacity design methods attempt to determine ultimate pile capacity by use of in situ soil strength parameters. Typically, ultimate capacity, Qu, is expressed as the sum of the ultimate shaft friction, Qs, and the ultimate toe resistance, Qb, (eq Bowies 1977 p.523),

$$Qu = Qs + Qb \qquad (2\text{-}1)$$

where:

$$Qs = As\ (K.\ \overline{pv}.\tan \partial) \qquad (2\text{-}2)$$

and:

$$Qb = Ab\ (\gamma.b.N_\gamma.S_\gamma + c.Nc.Sc. + pv.Nq.Sq) \qquad (2\text{-}3)$$

for which As = surface area of the pile shaft; K = a lateral earth pressure coefficient; $\overline{pv}$ = average effective overburden pressure along the shaft; ∂ = friction angle between pile and soil; Ab = area of the pile toe; γ = effective unit weight of the soil at the pile point; b = toe diameter; c = cohesion of the soil; pv = effective overburden pressure at the pile toe; and N_γ, Nc, Nq = theoretical bearing capacity factors related to the friction angle of the soil and on assumed failure pattern; and S_γ, Sc, and Sq are shape factors.

No single equation has been found to be adequate for all situations.

Most of the standard foundation texts describe the use of these equations in pile design. It is not certain where or when the static bearing capacity design approach originated, but it is apparent (Terzahi [1943], p.137) that it had been proposed prior to 1910.

2.2 Dynamic Bearing Capacity Design

Estimation of driven pile capacity by consideration of the dynamics of the pile/hammer system, has lead to two different approaches. The first, and historically the oldest involves a large family of "dynamic formulae". These are based on impulse-momentum principles using classical mechanics for which an energy balance equation is established. The equation is usually expressed in a form in which the pile "set" (penetration per hammer blow) required to ensure a particular pile capacity, can be calculated. Bowles (1977, p.561) presents an in depth review of how the detailed version of such an equation termed the "rational pile formula", is derived. The second approach to the determination of driven pile capacity by dynamic methods, is based on a mathematical model using the one dimensional wave equation for stress transmission in elastic rods.

Simplifications and assumptions applied to the "rational pile formula" by various authors have led to the publication of a large family of related dynamic formulae, some of which date from the early 1800's. Chellis (1961 Appendix 1) lists about forty different versions, some of which are also discussed by Bowles (1977) and Taylor (1948). According to Smith (1960), "... the editors of Engineering News Record have on file four hundred and fifty such formulae." Terzaghi (1943 p.141-142), basing his comments on a report by Cummings (1940), expressed reservations about both the theoretical basis of the impact equations and their applicability to the pile/soil system. He comments as follows, "On account of their inherent defects all the existing pile formulas are utterly misleading as to the influence of vital conditions, such as the ratio between the weight of the pile and the hammer, on the result of the pile driving operations." Despite the undeniable objections to the theoretical basis of the dynamic formulae, they have remained in popular use and have been incorporated into various standards (eg SABS 088 [1972]).

A number of statistical studies into the reliability of dynamic pile driving formulae have been undertaken. Some of these include the Michigan State highway Commission (1965), Olsen and Flaate (1967) and ASCE (1946). Although a large amount of data was considered in these analyses, no single preferred dynamic formula could be identified. The better performers were found to include the Hiley, Janbu and Gates formulae. Commenting on the ASCE (1941) report on their investigation into pile driving formulae, Peck (1942) was prompted to observe that a bearing capacity formula of Qu = 91 tons was, for the recorded data, statistically as accurate as the dynamic formulae used to predict their capacity.

It appears that the first to recognise the applicability of the "wave equation" to the analysis of pile driving was Isaacs in 1931. Fox (in Glanville et al, 1938) also published a driven pile solution to the wave equation. The design method given by these two authors, however, was not widely adopted, mainly because of the tedious calculation involved. Edward A. Smith of the Raymond Concrete Pile Company provided the first tractable application of the wave equation to driven piles.

Smith formalized his ideas in publications dated 1955 and 1960. A considerable amount of practical and theoretical research into the wave equation and related methods have since been carried out, particularly in America. The first international conference on the application of stress wave theory on piles was held at Stockholm in June 1980 (Bredenberg [1980]). A state of the art review of the wave equation and a list of the major research publications was presented by Goble et al (1980).

Some excellent correlations between pile capacity predicted by the wave equation and measured failure loads, have been reported (eg Goble et al [1975], Rausche et al [1972], Forehand and Reese [1964]). Goble et al (1975) for example indicate an error range of ± 10% from the predicted pile capacity. Other authors (eg Lowery et al [1968], Ramey and Hudgins [1977]), found that the error ranged up to as high as ± 40% while Tavenas and Audibert (1977) concluded that there was no correlation at all between wave equation predictions and measured pile performance. It must be accepted therefore that the wave equation, in common with other pile design techniques, has its limitations.

2.3 Empirical Pile Design

Empirical design by the installation and testing of trial piles is relatively expensive. Consequently, only contracts of reasonable size can justify the experimental costs. Further, such experiments are time consuming and require between three and five weeks to conduct.

Because empirical design by use of full scale field tests is not generally justifiable, much effort has been expended in developing alternative in situ test techniques. Most of these involve the use of penetrometers, the argument for their use being that the resistance to penetration experienced by the penetrometer can be correlated to pile bearing capacity. Also in favour of using penetrometers as a pile design tool, is that they are frequently used in site investigation to obtain both soil properties and an indication of the subsurface profile.

Sanglerat (1972, 1979) has described in some depth, the theory and practice related to many of the large variety of both static and dynamic penetrometers in use around the world. He also describes an equally large variety of pile/penetrometer design correlations. As a method of design, it would appear that the subject is little better off than alternative methods. Amongst the many penetrometer pile design techniques those of Van der Veen and Boersma (1957), de Beer (1965) and Meyerhof (1956, 1976) may be considered the most popular.

There is considerable potential in the use of penetrometer data for the design of piles. Until penetrometer/soil interaction is properly understood, and until some technique is developed to extrapolate this information in a rational manner to pile behaviour however, the design correlations must remain empirical and subject to uncertainty.

2.4 Comment on Pile Design

Of the three main approaches to pile design, pile bearing capacity determination by the use of "static" formulae is considered the most important method. The static design approach is usually used in conjunction with, or in support of the other design methods, and will therefore be routinely used in most situations where the need for piles is being considered. Considerable reliance is therefore often placed on this method. Static pile design, however, remains a largely empirical science. In a succinct analysis of existing design methods, Tomlinson (1977) has drawn attention to some of the weaknesses of the design formula. In his analysis Tomlinson makes the following comments:

"... the effects of the various methods of pile installation on the carrying capacity and deformation characteristics (of piles) cannot be calculated by strict application of soil or rock mechanics theory". (Op cit, p.11.) Consequently, calculation of the load carrying capacity of piles, "... is based partly on theoretical concepts ..., but mainly on empirical methods based on experience". (Op cit, p.10.) Despite the apparently scientific nature of pile design, therefore "The overriding influence of experience and the limitations of theory make piling practice an art rather than a science ...". (Op cit, p.11.) In an attempt to reduce this dependence on experience as a necessary component in pile design, various theoretical and practical (laboratory and full scale) studies have been carried out to investigate the parameters thought to affect pile behaviour. Summaries of the findings of the major full scale investigations can be found in the standard texts. Theoretical studies into the behaviour of piles are limited by the applicability of the soil models used in the analyses and by the availability of suitable experimental data with which to compare the theoretical findings. Some of the laboratory scale studies which have been carried out, are discussed in Chapter 4. As Tomlinson notes (op cit, p.11), however, "Attempts to study the behaviour of piles in the laboratory have not produced any worthwhile design rules because of the impossibility of reproducing installation effects in model piles. At best, they have given an insight into the fundamental matters of the transfer of load from the pile to the soil ...". Whilst reproduction of pile installation effects using model piles may be of limited value when extrapolating the findings to full scale piles, it is suggested that model pile tests constitute the most reliable way of investigating the mechanism of pile/soil interaction and through this, of obtaining improved design methods.

3. THE MV PILE

3.1 Description

The MV pile may be described as a driven displacement grouted in situ pile. The equipment generally comprises an oversize pile shoe, a drive mandrel which may or may not be a permanent fixture and a grouted annulus which fills the cavity formed by the shoe between the soil and the mandrel. Pile installation is achieved by dynamic methods and use may be made of any of the pile driving hammer techniques including a vibro-hammer. Usually grout is pumped continuously to the pile toe during driving. Figure 3.1 shows the general arrangement of some of the MV pile types which have been used in past contracts.

Within this general arrangement for the MV piling system a considerable variety of piles can be manufactured for particular applications. Shaft diameter and shape are determined by the diameter and shape of the pile shoe. Complex shapes have been used to ensure panel interlocking for contiguous pile walling. Typically the MV pile shoes are made of manufactured plate steel box structures or precast reinforced concrete elements.

The drive mandrel may be made from any dimensionally and structurally suitable steel section (including hollow

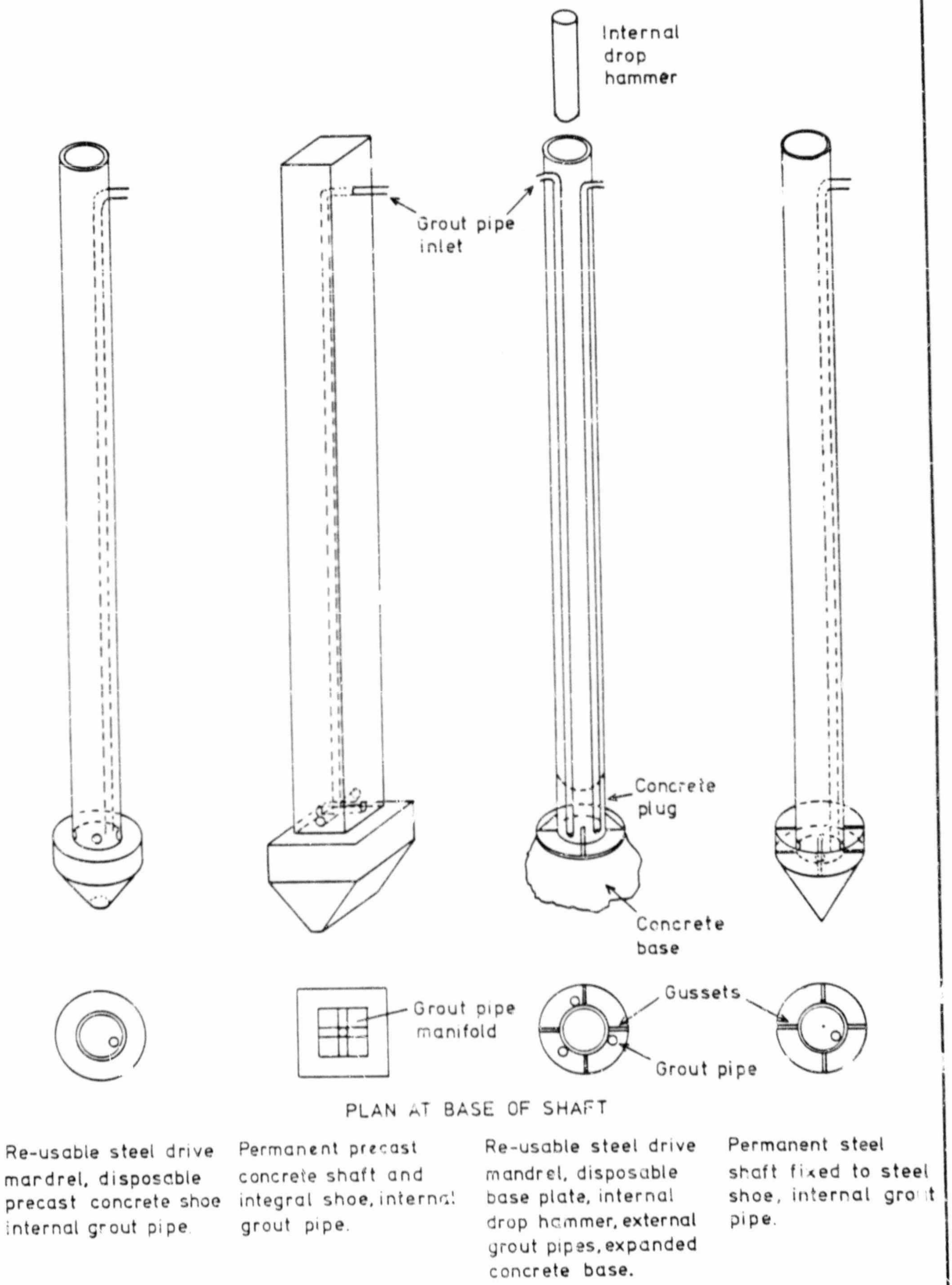

FIGURE 3.1 : General arrangement of some MV pile types

sections) or it may be made from a reinforced precast concrete column. Hollow steel section mandrels may also be filled with concrete on completion of driving, or a bulbous shape may be formed by driving out a concrete plug with an internal drop hammer.

MV piles may be installed at rakes ranging from vertical to as little as 20 degrees inclination from the horizontal. Techniques have also been developed to install MV piles under water. Shaft friction capacity of MV piles in a given soil is high in comparison to, for example, preformed driven displacement piles such as precast piles or steel joists, and is of a similar order of magnitude to the shaft capacity of similar sized driven displacement cast in situ pile types such as Franki piles. For this reason MV piles are generally identified as "friction piles" and consequently their most frequent application is in deep soft soils including loose sands, and soft silty and clayey soils. Their low driving resistance also enables MV piles to be installed into gravels and dense sands.

Due to the high shaft friction capacity a frequent application of MV piles is in tensile load applications such as high capacity retaining wall anchors and dry dock and weir foundations. Transmission line tower masts and chimney foundations have also been supported on MV piles, which, in such cases, are used to provide reaction to both tensile and compressive loads. In terms of numbers of piles installed the most frequent use of MV piles has been as anchor piles, particularly in harbour and canal construction where they have been used as a cost effective alternative to conventional tendon type ground anchors. The harbour authorities of Duisborg, West Germany for example consider MV pile anchors a preferred anchor technique, and the EAU (1975) recommend MV piles for retaining wall support and other tensile load applications.

3.2 Historical development

The MV piling system was invented and developed by Dr. -Ing Ludwig Müller in the early 1950s in West Germany. The Company, Dr. -Ing Ludwig Müller und Sohne Gesellschaft für Neuzeitliche Bautechnik K.G. (later to become Dr. -Ing Ludwig Müller und Söhne Gesellschaft für Bautechnik MBH & Co K.G.), was subsequently established in Marburg an der Lahn, West Germany, to market the MV piling system under a licensee arrangement, with Dr. -Ing Müller remaining as patent holder and licensor. The first licensee of the MV piling system was Dortmund Hoeder Hüttenunion, a West German sheet pile manufacturer, in 1955. The initials "MV" in MV pile are an abbreviation of the German - Müller Verpress pfahl -, which roughly translated, means Müller grouted pile.

Fundamental to the successful operation of the MV piling system is the use of a grout with suitable stability, flow and strength properties. Early MV grouts contained materials such as trass, bentonite and asbestos fibre in addition to a fluid sand-cement-water mix. Tricosal MV, an additive to MV grout which now finds wide acceptance and success, was subsequently developed specifically for Müller und Söhne by Chemische Fabrik Grünau GmbH, also of West Germany. The original intention with early MV piles was that the hardened grout need be only as strong as was required to safely transfer the applied axial load in shear to the surrounding soil and that the drive mandrel would be designed to support applied axial and bending loads.

Grout made with Tricosal MV, however, possesses sufficient strength for it to be used to both shed load to the soil through shaft friction as well as to function as the main axial load bearing member. The drive mandrel thus becomes a recoverable item, and its original function is now usually supplanted by the grout column, reinforced as required.

To date, over 600 000m of MV piles ranging in length from 3,0 to rarely more than 30m (typically about 8m) and with diameter ranging from 0,1m up to 0,9m have been installed throughout the world, but mostly in West Germany and bordering European countries. Despite their considerable success in Europe very little appears to be known about the MV piling system elsewhere. Begemann (1965) briefly refers to the MV pile in presenting a tensile load capacity design approach using the "adhesion jacket cone". Hammond (1967 pp. 58-61) describes MV piles as "a driven steel pile in which pressure grouting is ingeniously employed", and Bazant (1979 p. 363) simply observes that MV piles are useful in resisting tensile loads. A number of European publications describe site conditions and contracts, often with test load data, in which MV piles were used. Some of these include, Müller (1961, 1965), Kiessling (1960), Förster (1956), Hecht (1961), Jeske and Haman (1962) and Köhling (1962). The "Grundbau Taschenbuch" (1966, p. 621) refers briefly to MV piles as a general piling technique, while MV piles are recommended as a preferred anchoring technique in embankment construction in EAU (1975). MV piles had not been incorporated in EAU recommendations prior to 1975. Their inclusion in the 1975 edition signifies important recognition of the technique by construction authorities in West Germany. The Larssen sheet piling handbook (Larssen-Handbuch [1960]) frequently refers to MV piles, again as an alternative to conventional ground anchors.

Jelinek (1962) submitted the first proposal for a general design approach to MV pile bearing capacity and Jelinek and Ostermayer (1964) discussed the tensile load capacity of MV piles.

The Larssen-Handbuch (1960) describes the design of MV piles when used as retaining wall anchors. Their approach is similar to that of EAU (1975, S. 238 fl).

3.3 South African Experience

South African experience with the MV piling technique has been limited to a single contract of significance, though a number of small contracts have been performed. During 1977 Grinaker Piled Foundations (Pty) Limited installed some 1800m of MV piles, rated at working loads of between 350kN and 1200kN for the Wentworth Hospital redevelopment. The site is located in the Bluff area of Durban, Natal, where extensive and deep deposits of Berea Red Sand occur. Pile lengths ranged from 4m to 9m. The piling contract was won in competition to vibroflotation.

An MV pile field testing programme has also been initiated by the South African Licensees and the present laboratory investigation is an extension of this work.