Characterisation of Model Uncertainty for Reliability-Based Design of Pile Foundations

by

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Declaration

I, the undersigned, hereby declare that the work contained in this dissertation is my own original work and that I have not previously in its entirety or in part submitted it at any university for a degree.

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Abstract

To keep pace with international trends, the introduction of geotechnical limit state design in South Africa is inevitable. To pave the way for implementation of limit state pile design in the country, the study quantifies model uncertainty in the classic static pile design formula under the Southern African geologic environment. The generated model uncertainty statistics are used to calibrate partial resistance factors in a reliability-based design framework.

A series of pile performance predictions by the static formula are compared with measured performances. To capture the distinct soil types for the geologic region of Southern Africa as well as the local pile design and construction experience base, pile load tests and associated geotechnical data from the Southern African geologic environment are used. The methodology of collecting, compiling, and analyzing the pile load tests to derive the measured ultimate pile capacities is described. To facilitate the computation of the theoretical capacities, the site specific geotechnical data in the database are transformed to the desired engineering soil properties through well established empirical correlations.

For a given pile test case, model uncertainty is presented in terms of a model factor computed as the ratio of the measured to the theoretical capacity, leading to ***n*** realisations of the model factor. To facilitate further interpretation and generalisation of the model factor realisation data, statistical analysis is carried out. The statistical analysis comprises of graphical representation by histograms, outliers detection and correction of erroneous values, and using the corrected data to compute the sample moments (mean, standard deviations, skewness and kurtosis) needed in reliability analysis. The analyses demonstrate that driven piles depict higher variability compared to bored piles irrespective of materials type. Furthermore, for a given pile installation method (driven or bored) the variability in non-cohesive materials is higher than that in cohesive materials.

In addition to the above statistics, reliability analysis requires the theoretical probability distribution for the random variable under consideration. Accordingly it is demonstrated that the lognormal distribution is the most appropriate theoretical model for the model factor. Another key basis for reliability theory is the notion of randomness of the basic variables. To verify that the variation in the model factor is not explainable by deterministic variations in the database, an investigation of correlation of the model factor with underlying pile design parameters is carried out. It is shown that such correlation is generally weak.

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Correlation can have a significant impact on the calculated reliability index if not accounted for. Accordingly, the effects of the exhibited correlation is investigated through an approach based on regression theory in which systematic effects of design parameters are taken into account (generalised model factor). The model factor statistics from the conventional approach and those from the generalised model factor approach are used to determine reliability indexes implied by the current design practice. It is demonstrated that no significant improvement in values of the reliability indexes is gained by taking into account the effects of the weak correlation.

The model factor statistics derived on the basis of the standard model factor approach are used to calibrate resistance factors. Four first order reliability methods are employed for the calibration of resistance factors. These include; the Mean Value First-Order Second Moment approach, an Approximate Mean Value First-Order Second Moment approach, the Advanced First-Order Second Moment approach using Excel spreadsheet, and the Advanced First-Order Second Moment approach (design point method). The resistance factors from the various calibration methods are presented for the target reliability index values of 2.0, 2.5, and 3.0. The analyses of the results demonstrate that for a given target reliability index, the resistance factors from the different methods are comparable. Furthermore, it is shown that for a given material type, the resistance factors are quite close irrespective of the pile installation method, suggesting differentiation of partial factors in terms of materials types only. Finally, resistance factors for use in probabilistic limit state pile design in South Africa are recommended.

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Samevatting

Ten einde in pas te bly met internasionale neigings, is dit onafwendbaar dat geotegniese limietstaat-ontwerp in Suid Afrika ingevoer word. Ter voorbereiding vir die plaaslike toepassing van limietstaatontwerp op heipale, kwantifiseer hierdie ondersoek onsekerheid rondom die model vir klassieke statiese heipaalontwerpformules in die Suid Afrikaanse geologiese omgewing. Die statistiek van modelonsekerheid wat gegenereer is, word gebruik om parsiële weerstandsfaktore in 'n betoubaarheid-gebasseerde ontwerpraamwerk te kalibreer.

'n Reeks voorspellings van die gedrag van heipale volgens die statiese formules word vergelyk met die gemete gedrag. Om die kenmerkende grond-tipes in die geologiese gebied van Suidelike Afrika sowel as die plaaslike ondervinding met heipaalontwerp en -konstruksie vas te lê, word heipaaltoetse en die gassosieerde geotegniese data vanuit hierdie geologiese omgewing gebruik. Die metodiek vir die versameling, saamstelling en analise van heipaaltoetse om uiterste kapasiteite daarvan te bepaal, word beskryf. Terreinspesifieke geotegniese data in die databasis word getransformeer na die vereisde ingenieurseienskappe volgens gevestigde empiriese korrelasies.

Vir 'n gegewe heipaaltoets word modelonsekerheid weergegee in terme van 'n modelfaktor wat bereken word as die verhouding van die gemete tot die teoretiese kapasiteit waaruit ***n*** uitkomstes van die modelfaktor dus gegenereer word. Om verdere interpretasie en veralgemening van die modelfaktordata te vergemaklik, word 'n statistiese analise daarop uitgevoer. Die statistiese analise bestaan uit grafiese voorstellings deur middel van histogramme, uitkenning van uitskieters en verbetering van foutiewe waardes, waarna die statistiese momente (gemiddeld, standaardafwyking, skeefheid en kurtose) vir gebruik in betroubaarheidsanalise bereken word. Volgens die analises toon ingedrewe heipale 'n groter veranderlikheid as geboorde pale, ongeag die grondtipe. Verder is die veranderlikheid van heipale in kohesielose materiale hoër as in kohesiewe materiale, ongeag die installasiemetode (ingedrewe of geboor).

Bykomend tot bogemelde statistiek, vereis betroubaarheidsanalise die teoretiese waarskynlikheidsdistribusie van die ewekansige veranderlike onder beskouing. Ooreenkomstig word illustreer dat die log-normale verspreiding die mees toepaslike verspreiding vir die modelfaktor is. 'n Verdere sleutelvereiste vir betroubaarheidsteorie is

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die mate van ewekansigheid van die basiese veranderlikes. Om te bepaal of die variasie in die modelfaktor nie deur deterministiese veranderlikes in die databasis verduidelik kan word nie, word ’n ondersoek na die korrelasie van die modelfaktor met onderliggende heipaal-ontwerpfaktore uitgevoer. Sodanige korrelasie is in die algemeen as laag bevind.

Korrelasie kan ’n belangrike invloed op die berekende betroubaarheidsindeks hê indien dit nie in ag geneem word nie. Dienooreenkomstig word die effek van die getoonde korrelasie ondersoek met behulp van die metode van regressie-analise waarin sistematiese effekte van ontwerpparameters in berekening gebring word (veralgemeende modelfaktor). Die modelfaktorstatistiek wat volg uit die konvensionele benadering en dié van die veralgemeende benadering word gebruik om betroubaarheidsindekse te bepaal wat deur die bestaande ontwerppraktyk geïmpliseer word. Die bevinding is dat daar nie ’n noemenswaardige verbetering in die waardes van die betroubaarheidsindekse is wanneer die effek van die swak korrelasie in berekening gebring word nie.

Die statistiek van die modelfaktor wat afgelei is volgens die standaardbenadering word gebruik om die weerstandsfaktore te kalibreer. Vier eerste-orde betroubaarheidsmetodes word gebruik om die weerstandsfaktore te kalibreer, naamlik die Gemiddelde Waarde Eerste-Orde Tweede Moment benadering, die Benaderde Gemiddelde Waarde Eerste-Orde Tweede Moment benadering, die Gevorderde Eerste-Orde Tweede Moment benadering waarin ’n Excel sigblad gebruik word en die Gevorderde Eerste-Orde Tweede Moment benadering (die ontwerppuntmetode). Die weerstandsfaktore vanaf die verskillende kalibrasiemetodes word weergegee vir waardes van 2.0, 2.5 en 3.0 van die teiken-betroubaarheidsindeks. ’n Ontleding van die resultate toon dat vir ’n gegewe teiken betroubaarheidsindeks die weerstandsfaktore vanaf die verskillende metodes vergelykbaar is. Verder word getoon dat vir ’n gegewe grondsoort, die weerstandsfaktore vir verskillende metodes van installasie van die heipaal nie veel verskil nie. Dit wil dus voorkom asof parsiële faktore in terme van die grondsoort uitgedruk kan word. Ten slotte word weerstandsfaktore vir gebruik in plastiese limietstaatontwerp van heipale in Suid Afrika aanbeveel.

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Chapter 1

INTRODUCTION

1.1 BACKGROUND

The presence of uncertainties and their significance in relation to performance of the designed systems has long been acknowledged by the engineering profession. Traditionally, a single global factor of safety is adopted to account for all the associated uncertainties. However, in today's increasingly safety conscious civil engineering industry, it is becoming not only necessary to determine if the proposed design is safe, but also to determine how safe or reliable it actually is (Cameron, 2002). The need to quantiy the level of safety explicitely calls for identification, explicit quantification and systematic incorporation of the key uncertainties in the design process. Currently only the reliability-based design approach is capable of treating uncertainties in a rational and explicit manner. Accordingly reliability-based design is on the forefront of design philosophies in civil engineering worldwide (Harr, 1977).

1.1.1 Uncertainties in geotechnical design

In common with other engineering designs, geotechnical design is performed under a considerable degree of inherent uncertainty. Sources of uncertainties in civil engineering designs have been identified by a number of researchers (e.g. Phoon, 1995; Whitman, 1984; Jaksa, 1982; Griffiths et al 2002) and may incorporate one or more of the following:

- Variability of material properties;
- Uncertainties associated with the measurement and conversion of design parameters;
- Inaccuracies that arise from the models which are used to predict the performance of the design;
- Inconsistencies associated with the magnitude and distribution of design loads;
- Anomalies that occur as a result of construction variability;
- Human gross errors.

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For most geotechnical designs, two predominant sources of uncertainties can be distinguished: (i) uncertainties associated with the evaluation of design soil properties and (ii) calculation model uncertainties.

Soil parameter uncertainty arises from the variability exhibited by properties of geotechnical materials from one location to the other, even within seemingly homogeneous profiles. Tang et al (1984), Whitman (2000), Baecher (1986), Tabba et al (1981), Christian et al (1994) Phoon and Kulhawy (1999a) and many other researchers have identified the following as the key sources of geotechnical parameter prediction uncertainties: inherent spatial variability; measurement noise/random errors, systematic measurement errors, and statistical uncertainties. Conversely, model uncertainty emanates from imperfections of analytical models for predicting engineering behaviour. Mathematical modelling of any physical process generally requires simplifications to create a useable model. Inevitably, the resulting models are simplifications of complex real world phenomena. Consequently there is uncertainty in the model prediction even if the model inputs are known with certainty.

For a rational design, both soil parameter and model uncertainty need to be quantified. With regard to geotechnical properties uncertainties, significant research work has been carried out to generate statistics on individual components of soil parameter uncertainty. Accordingly first-order estimates of inherent variability, measurement errors and correlation uncertainties have been reported (e.g. Kulhawy et al, 1991; Phoon, 1995; Phoon and Kulhawy, 1996; Kulhawy and Trautmann, 1996; Lacasse and Nadim, 1996; DeGroot, 1996; Phoon and Kulhawy, 1999a; Phoon and Kulhawy, 1999b; Jones et al, 2002). Relevant tables from some of the published works are presented in Appendix A. Now statistics on soil parameter uncertainties are available for more rigorous calibration of geotechnical reliability based design equation. Conversely model uncertainty statistics are generally scarce. In fact, the lack of model statistics is a key impediment to the development of geotechnical reliability based design (Phoon, 2005). Therefore there is a need for concerted research effort to develop model uncertainty statistics worldwide.

In civil engineering, limit state design represents the-state-of-the-art design philosophy in which ultimate failure and serviceability conditions can be evaluated considering the uncertainties in load and materials resistances. In structural design, limit state design has developed to a level where it has been merged with reliability theory leading to incorporation of probabilistic concepts in structural codes. Prompted by developments in

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structural design, in recent years much work has also been devoted to developing limit state methodologies for geotechnical engineering based on probabilistic techniques (Christian, 2003). Accordingly probabilistic limit state design (level 1 reliability methods) has been internationally accepted as the standard basis on which the new generation of geotechnical codes are being developed today.

1.1.2 Status of probabilistic geotechnical limit state design

As already alluded to, initiatives to bring geotechnical design within the reliability based design framework as is the case in structural engineering are in progress in most parts of the world. Already there are some geotechnical codes with the same design format as structural codes. Examples include Eurocode 7, the Canadian Foundation Engineering Manual, the AASHTO Standard Specification for Highway Bridges, Geoguide 1, etc.

Although the Load and Resistance Factor Design (LFRD) or its equivalent in Canada and Europe have the same design format as that for structural codes, several researchers (e.g. Phoon, 2004a; DiMagio et al, 1999; Paikowsky and Stenersen, 2000) have expressed concern that in general there is a lack of analytical calibration and verification. Mostly the partial factors have been developed on the basis of judgement by rearranging the existing global safety factors (Phoon et al 2003, Phoon, 2004a). Therefore, in essence geotechnical limit state design is being implemented in a deterministic framework. Phoon et al (2003), assert that implementation of limit state design within a deterministic framework does not address the drawbacks associated with the traditional working stress design approach. For a probabilistic limit state design, the partial factors need to be derived using reliability theory. In this regard, the key long-term objective of the probabilistic limit state initiative is to determine all the load and resistance factors using reliability theory. Accordingly advancement of reliability based methodologies on determination of partial factors is among the terms of references for the Technical Committee (TC 23) of the International Society of Soil Mechanics and Geotechnical Engineering on Limit State Design in Geotechnical Engineering.

1.1.3 State of limit state design in SA

Currently in South Africa, foundation design codes of practice (SABS 088- Pile foundations and SABS 0161: 1980 – Design of foundations for buildings) are still based on the working

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stress design philosophy. However, in order to keep pace with international developments, the Geotechnical Division of South African Institute of Civil Engineers embarked on some initiatives to convert to limit state design as far back as 1993. A chronological account of these initiatives is as follows:

- A few years before 1993, the SABS canvassed the opinion of the industry as to whether the existing piling code was to be updated or re-written. Most respondents were in favour of re-writing the code.
- In 1993, a committee to undertake the task of re-writing the new piling code was formed.
- The committee advised that, in order to keep pace with recent developments overseas, the new code should preferably be written in terms of limit state principles.
- The committee also observed that it would not be possible to introduce limit state design into geotechnical engineering on an ad hoc basis and recommended that the profession should look into the merits of adopting the Eurocodes approach for geotechnical design as a whole.
- As an initial step, the Geotechnical Division of SAICE invited Dr Ovesen and Dr. Simpson, chairmen of Eurocode 7 drafting committee to address a seminar held in Pretoria in October 1995. At the seminar it was agreed that:
    - a number of geotechnical designers would apply the latest version of Eurocode 7 (ENV-1997-1) in parallel with design methods currently in use for a trial period of two or three years.
    - At the end of the trial period, another seminar will be held and then a final decision on adopting Eurocode 7 in South Africa would be made.
- Following the seminar, a meeting between the Geotechnical division and the SABS Building Codes section resolved that should the former recommend the adoption of Eurocode 7, it would be acceptable to the latter. It was also agreed that with permission of the European Committee for Standardisation, the code would then be published by the SABS as joint Eurocode and SABS code.

Despite the above resolutions, the implementation of Eurocode 7 was not entirely successful (Day, P. 1997). The main stumbling block to the adoption of the Eurocodes in general is the difference between the load factors in the Eurocodes in those contained in the South African Loading Code (Day, P. 1997, Day et al, 2000). In view of this problem, the initiatives were held back pending the revision of the South African Loading Code (Day et al, 2000).

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The revision of the South African loading code is at an advanced stage. The code is being revised along the lines of the Eurocodes with a strong reliability basis. A provision for geotechnical design has been made. The code forms the foundation for the materials codes including geotechnical design codes. In a way, the revised South African loading code indirectly forces materials codes including geotechnical design to be based on reliability principles. It is evident from these latest developments that future geotechnical design codes in South Africa will be a probabilistic limit state design code.

The impending transition from the current design practice to reliability based geotechnical design requires some reference materials in the form of detailed studies into various aspects of limit state design under the local conditions. Such studies will form the basis on which code writing committees will base their decisions.

1.2 RESEARCH STATEMENT

The South African piling industry expressed the desire to rewrite the current South African Code of Practice for Pile Foundation design (SABS 088-1972: Pile foundations) on the basis of limit state design principles in 1993. However, geotechnical design as a whole in South Africa and most African countries is still based on the traditional working stress design approach. Recent international and local developments have now added impetus to the introduction of probabilistic limit state design in South Africa. These include; (i) the international acceptance of probabilistic limit state as the standard basis on which the new generation of geotechnical codes are being developed today (ii) the formulation of the draft South African Loading Code (SANS 10160) on the basis of reliability framework, which implies that the subsequent materials codes including geotechnical design code will be based on the same framework (iii) increased interest in harmonisation of technical rules for design of building and civil engineering works across disciplines and national borders.

To keep pace with all these developments, geotechnical design in South Africa has no choice but change to limit state design as a matter of urgency. Given that uncertainties are the hallmark of geotechnical engineering, it is only logical to adopt limit state design in a probabilistic framework. Probabilistic limit state is currently implemented in the form of level 1 reliability design approach in which a prescribed level of reliability is imparted to the designed element through the use of predetermined partial factors rather than performing

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reliability analysis. Central to this approach is the determination of load and resistance factors.

Calibration of partial factors is dependent on local design practice, experience and environment such as local geology, soil type and conditions, site investigation practices (extend, methods, standards, equipment advances). Since these factors generally differ from one country to another, values developed for a specific country can not be simply adopted by another country. The need to calibrate geotechnical resistance factors for different applications utilising local databases was further emphasised in a study to review developments of limit state design or LRFD methods in Canada, Germany, France, Denmark, Norway and Sweden (DiMaggio et al, 1999). For the same reason, Eurocode 7 leaves the calibration of partial factors to individual countries. Therefore to introduce probabilistic geotechnical limit state design to the design of pile foundations in South Africa, calibration studies in which the South African design experience is captured are required.

A prerequisite to such reliability calibration of partial factors is the identification and quantification of uncertainties associated with pile design models. In common with other geotechnical design, the major uncertainties can be broadly classified as those relating to determination of soil design parameters and those pertaining to calculation models. As already alluded to, statistics on parameter uncertainties as well as first order second moment models for combining such uncertainties are available. Conversely statistics on model uncertainties are scarce worldwide.

In spite of scarcity of model uncertainty statistics, prediction symposiums and research studies have indicated that model uncertainty is normally the predominant source of uncertainty for pile design. A typical example is the results of an international competition to test the ability of experienced practising engineers to predict the behaviour of plain driven and jet grout enhanced piles under static and cyclic loading (Jardine et.al, 2001). The results were such that even for the conventional driven piles, the ultimate capacity predictions were not within 50% of the measured capacities while the settlement was overestimated by about 100%. Further more, Ronold and Bjerager (1992) carried out a first order reliability analysis which incorporated model uncertainty on tension piles. Through sensitivity factors of variables for the performance function, the study showed that the model uncertainty exclusively is the most important source of uncertainty by a contribution to the total

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uncertainty by close to 100%. It concluded that other sources of uncertainties such as evaluation of soil parameters could just as well be neglected. The high contribution of model uncertainty to the total uncertainty was attributed to the limited physical understanding of the pile-soil interaction problem.

In recognition of the significance of model uncertainty, EN 1997-1 recommends that if the ultimate pile capacity is determined on the basis of a theoretical model, a model factor may be introduced to ensure that the predicted compressive resistance is sufficiently safe. However, EN 1997-1 does not provide reference values of model factors and instead recommends that such values be set by national annexes. This is a further demonstration of the scarcity of model uncertainty or model factor statistics.

Motivated by the need for the introduction of probabilistic limit state pile design in South Africa, this study quantifies model uncertainty in the classic static pile design formula. The static formula is the main theoretical pile design method; hence model uncertainty associated with this approach is of international interest. In fact, due to lack of model uncertainty statistics, characterisation of model uncertainty in any geotechnical design model is a major contribution to the development of probabilistic geotechnical limit state design internationally. As a contribution to the South African geotechnical practice, particularly the piling industry, the developed model uncertainty statistics are used to calibrate resistance partial factors for pile foundations.

1.3 AIMS AND SCOPE OF THE STUDY

The main aim of the study is to characterise model uncertainty in analytical pile design methods and to use the generated model factor statistics to calibrate partial resistance factors in a reliability based framework. Specific research objectives are:

- To quantify model uncertainty in the classic static pile design formula in terms of model factor statistics.
- To provide insight into the degree of conservatism in the static formula.
- To compare model factor statistics developed on the basis of the conventional approach (standard model factor) to that based on regression theory where systematic effects of design parameters are taken into account (generalised model factor).

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- To use the developed model factor statistics as input in reliability calibration of partial resistance factors.
- To recommend a suitable target reliability index for design of pile foundations in South Africa.
- To recommend suitable resistance factors for design of piles in South Africa.
- Since the Eurocodes are the reference codes in South Africa, the study will make recommendations as to which of the three EN 1997-1 design approaches will be more appropriate for South Africa.
- To contribute to the general introduction of probabilistic geotechnical limit state design in South Africa.

The study is limited to pile foundations for structures within the scope of the South Afican Loading Code (SANS 10160-Draft). These include buildings and industrial structures. Both driven and bored piles in cohesive and non-cohesive soil are considered. Even though pile foundations can resist various types of loading such as lateral loads, tension loads and compression loads, the study focuses on piles subjected to axial load only.

Although limit state design requires the structure to satisfy two principal criteria of ultimate limit state and serviceability limit state, this study focuses on ultimate limit state.

1.4 LAYOUT OF THE DISSERTATION

The dissertation details the current research undertaken to quantify model uncertainty in the static formula for computing the ultimate capacity of pile foundations. The generated model factor statistics were used to calibrate partial resistance factors in reliability based design framework.

In Chapter 2 the principles and basis of geotechnical limit state design are set. Given that the future South African geotechnical design code will be based on the Eurocodes format, the basis of design will be in accordance with Eurocodes approach. Among the issues considered include selection of characteristic geotechnical design parameters, design approaches, actions and action combinations, verification of ultimate and serviceability limit states.

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Chapter 3 presents the fundamental reliability background required for reliability-based geotechnical design. This entails an overview of reliability theory and reliability calibration principles.

Chapter 4 describes the compilation of a database of local static pile load tests along with the associated geotechnical data (soil profiles, field and laboratory test results). The Chapter also describes the processing of the pile tests records to evaluate the interpreted capacities (measured capacities).

In Chapter 5, the specific site measurements presented in Chapter 4 are transformed to the desired engineering soil properties by means of empirical correlations. Furthermore, evaluated soil properties are used to derive other required pile design parameters (e.g. N_q, K_s, N_c, δ, and α). Finally the soil parameters and the associated pile design parameters are used to calculate the theoretical or predicted capacities.

In Chapter 6, the results obtained from Chapter 4 and Chapter 5 are used to determine the model factors. The generated model factors are analysed statistically to facilitate their further interpretation and generalisation beyond the database.

In Chapter 7, the statistical dependencies between the model factor and the predicted capacity observed in Chapter 6 are either removed or taken into account. The effects of further treatment of the correlation on the model factor statistics and the calculated reliability indexes are investigated.

Chapter 8 applies the results of model uncertainty quantification in reliability calibration of partial resistance factors.

A summary and conclusions of the study, as well as areas for further research are then presented in Chapter 9.

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Chapter 2

GENERAL INTRODUCTION TO GEOTECHNICAL LIMIT STATE DESIGN

2.1 Introduction

As mentioned in Chapter 1, uncertainties are an inherent part of all civil engineering works. In spite of the existence of uncertainties, society expects the designed structures to be safe for the people who use them or who are in their vicinity (Schneider, 1997). Accordingly, the civil engineering profession has risen to the challenge and has been continuously refining design methodologies to ensure that structures are designed to fulfil certain performance criteria. One such design approach is limit state design.

The basis of the limit state method is the acknowledgement that the structure may fail to meet its design requirements through a number of possible shortcomings (Day, R. 1997). In this context, limit state design is a formal and methodical way of ensuring each of the design criteria will be properly considered. The performance of a structure or structural element is described with reference to a set of limit states. Limit states are states beyond which the structure no longer satisfies the design performance requirements (EN 1990). Each such limit state is considered separately, and its occurrence is shown to satisfy the design criteria.

The impetus in the development of limit state design comes from the structural engineering profession. However in recent years much work has been devoted to development of geotechnical limit state design. A noticeable initiative is the establishment of a technical committee (TC 23) on Limit State Design in Geotechnical Engineering under the auspices of the International Society of Soil Mechanics and Geotechnical Engineering in 1990. The committee was mandated with promoting and enhancing professional activities in the limit state design in geotechnical engineering practice. Accordingly the committee has been organising international symposiums on limit state design in geotechnical engineering practice. Examples of such include; Copenhagen Symposium on Limit State Design in Geotechnical Engineering (1993), seminar in London entitled Eurocode 7 – Towards Implementation (1996), LSD 2000 in Melbourne Australia, LSD 2002 in Kamakura Japan,

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LSD 2003 in Boston USA and an International Symposium on New Generation Design Codes for Geotechnical Engineering Practice (Taipei, Taiwan, 2006) and many others.

Historically the limit state design philosophy has developed in parallel with the application of statistics, probability theory and partial factors to the design of structures (Borden, 1981; Day, R. 1997). The three concepts (limit state design, partial factors, and probabilistic considerations) are often wrongly thought of as being inseparable. In essence, there is no fundamental connection between them (Simpson, 2001).The original concept of limit state design does not specify a particular way in which non-ascendance of the relevant limit states are ensured (Borden, 1981; Simpson et al, 1981; Becker, 1996a). The level of safety may be provided through partial safety factors, global factor of safety or any other means. Therefore limit state design should not only be associated with partial factor method and a probabilistic design as it is commonly the case.

Two forms of geotechnical limit state design have emerged. One can be termed deterministic limit state design and the other termed probabilistic limit state design. The fundamental difference is in the derivation of the associated partial factors. With the deterministic limit state design, partial factors are determined by engineering judgement and by fitting to the existing design practice. Conversely partial factors for the probabilistic limit state are based on reliability calibration. Given that coping with uncertainty is the hallmark of geotechnical practice, it is only logical to develop geotechnical limit state in a probabilistic framework. This Chapter reviews the evolution of geotechnical design methodologies and fundamental concepts of the modern limit state design.

2.2 Evolution of geotechnical design

Over the years, geotechnical design procedures have evolved to ensure that designs meet performance requirements in the face of uncertainties. Generally, civil engineering design philosophies evolved from experience based design, working stress design, limit state design and the current probabilistic limit states design (level 1 reliability based design).

2.2.1 Experience based design approach

In the old days, the experience of the ancient builders guaranteed the safety of structures. The Gothic cathedral of Amiens in France built during the period 1220-1280 is generally

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viewed as the best example of the knowledge of the medieval builders (Van Straalen, 1999). These experience based design approaches were used for many centuries. A turning point in the guarantee of safety of designs occurred during the industrial revolution in the early 19th century (Becker, 1996a). The industrial revolution brought with it new materials and proliferation of new technology which forced modern builders to develop more rational design procedures. This new design approach came to be known as working stress design or allowable stress design.

2.2.2 Working stress design

Since its introduction in the early 1800's, the working stress design has been the traditional design basis in geotechnical engineering world wide. The working stress design philosophy attempts to ensure that the applied service load or the stresses induced in the soil mass do not exceed some allowable limit, often taken to be the limit of elastic resistance. In other words, safety is achieved by restricting the applied loads to values less than the ultimate geotechnical resistance divided by a factor of safety using the mathematical relationship of the general form:

$$\frac{R}{FS} \geq \sum Q \qquad [2.1]$$

in which, R = ultimate geotechnical resistance; FS = factor of safety; ∑Q = summation of loads effects (dead and live loads).

In this context, the factor of safety provides reserve strength in the event that an unusually high load occurs or in the event the resistance is less than expected. Therefore it is an empirical, but arbitrary number greater than unity used to reduce the potential for adverse performance. However in practice different engineers use different approaches of selecting the ultimate strength and load. Some may use mean values while others use nominal or characteristic values. Consequently two alternative definitions of the factor of safety emerge and are defined as follows:

a) Mean factor of safety $= \frac{\overline{R}}{\overline{Q}}$ [2.2]

b) Nominal factor of safety $= \frac{R_n}{Q_n}$ [2.3]

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The numerical values of FS for the two cases are not equal. The mean FS is higher than the nominal FS. Therefore for the same numerical value of safety factor, the margin of safety can be very different.

It is evident from the forgoing that all uncertainty in the variation of the applied load and the ultimate capacity of the soil are lumped in a single factor. In geotechnical design, the factor of safety is generally applied to the resistance side on account that uncertainties in soil parameters are the largest uncertainties affecting geotechnical design. The values of the global factors of safety for various geotechnical structures have been developed from previous experience with similar structures under similar conditions (Becker, 1996a). They reflect past experience and the consequences of failure.

In geotechnical design the concept of a factor of safety in stability estimation was introduced in the 18th century by Belidor and Coulomb who both suggested placing a value of 1.23 on the width of retaining walls determined from earth pressure theory and later Kery introduced a factor of about 1.5 for stability of slopes and retaining walls and recommended a range of 2-3 on ultimate bearing capacity of foundations (Meyerhof, 1995). Similar global factors of safety became customary for geotechnical design in Europe, North America and other parts of the world. Ranges of global factor of safety commonly used in geotechnical design throughout the world were compiled by Terzaghi and Peck (1967) and are presented in table 2.1.

Table 2-1: Ranges of global factor of safety for geotechnical design
(After Terzaghi and Peck, 1948, 1967)

Failure type	Item	Factor of safety
Shearing	Earthworks	1.3-1.5
	Earth retaining structures	1.5-2
	Foundations	2 – 3
Seepage	Uplift heave	1.5-2
	Exit gradient, piping	2-3
Ultimate pile loads	Load tests	1.5-2
	Dynamic formulae	3

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As illustrated by table 2.1, similar values of factor of safety have become customary for geotechnical design through out the world regardless of the different soil conditions encountered. Obviously the use of the same numerical value of safety factor for conditions that involve widely varying degrees of uncertainty, results in different levels of safety.
Working stress design is a simple and straight forward approach that is still in use in some countries including South Africa. However, when viewed from the perspective of current advances in engineering design, the working stress design approach has several limitations. These limitations are discussed below:

a) As already alluded to, with this approach, all uncertainties are lumped under a single factor of safety and therefore there is no distinction between model and soil parameter uncertainty. This makes it difficult to justify any reduction in safety level if there is additional information or advances in the state-of-the-art (Phoon 1995);
b) The fact that values of safety factor for various geotechnical applications have been developed subjectively on the basis of experience suggests that the approach can not be extrapolated rationally and consistently to accommodate new design situations;
c) The level of safety associated with the value of FS depends on its definition (mean or nominal model factor). Therefore the use of a global safety factor does not lead to consistent level of safety;
d) Earth structures have failed even though the computed FS was greater than one (Becker, 1996a). This indicates that a value of FS greater than one does not necessarily ensure safety. Similarly failure does not necessarily occur when the computed FS is less than 1. Therefore the actual level of safety implied by the FS is not known to the engineer;
e) In foundation design, the safety factor applied to the ultimate bearing capacity is deemed to limit settlement to acceptable limit without computing the actual settlement. Even though this is a common practice, Becker (1996a) argues that the specific values of FS were not derived for the separate consideration of soil rupture or collapse under bearing capacity considerations.

2.2.3 Limit state design philosophy

Given the drawbacks of the working stress design approach, the quest for a better design methodology continued, leading to the introduction of the limit state design approach. As already pointed out, with limit state design, the performance of a structure or part of a structure is described with reference to a set of limit states beyond which the structures is

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deemed to have failed to satisfy the fundamental requirements. Therefore essentially limit state design entails:

- Identification of all potential limit states and related failure modes;
- Checking of the occurrence of each limit state;
- Demonstration that the occurrence of the limit states is improbable or within acceptable risk.

In general, two main categories of limit states are normally considered. These include the ultimate limit state and serviceability limit state. Ultimate limit states are associated with the total or partial collapse of the structure (e.g., strength, ultimate bearing capacity, overturning, sliding, etc). It concerns the safety of the structure and its contents including people. Typical examples include (i) loss of equilibrium of a part or all of a structure as a rigid body leading to overturning, sliding, etc (ii) rupture of critical components, causing partial or complete collapse.

Conversely, serviceability limit states correspond to those conditions beyond which specific requirements of the structure or structural element are no longer met. Serviceability limit states concern the functionality of the structure, the comfort of people and aesthetic appearance of the structure. Examples include deformations, settlement, vibrations, cracks, and local damage of the structure in normal use under working loads such that it ceases to function as intended.

The distinction between ultimate and serviceability limit states is also applicable to geotechnical practice. In this regard, Geotechnical ultimate limit states have been defined as failure or excessive deformation of the ground where the strengths of the soil or rock are significant in providing resistance (EN 1997-1). With respect to foundation structures, ultimate limit state emanates from soil bearing capacity failure. Bearing capacity failure usually occurs as a shear failure of the soil. The three principal modes of shear failures (general shear failure, punching shear and local shear failure) lead to an occurrence of an ultimate limit state. However, the development of the ultimate limit state associated with the three modes of shear failure differs.

General shear failure is charecterised by the development of a well defined failure pattern, consisting of a continuous slip surface form the edge of the footing up to the ground level.

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Ground failure is sudden and catastrophic accompanied by substantial rotation of the foundation. For structures for which by virtue of their configuration, do not prevent the rotation of the foundation, general shear failure lead to total collapse of the structure. Examples of such structures include silos, tanks, and towers. In conventional structures, rotation of the entire foundation system is prevented by the configuration of the structure. Although rotation may be prevented, the distress caused by the ground movement may lead to rupture of critical elements of the structure, resulting in partial or total collapse.

In contrast to the general shear failure, the punching shear failure mode is charecterised by a failure pattern that is not easily observed. Slip lines do not develop and little or no bulging occurs at the ground surface. However, there is a large vertical movement of the foundation due to the soil compressibility. The soil outside the loaded area remains uninvolved and there are no movements of the soil on the sides of the foundation (Vesic, 1973). Similarly local shear failure is characterised by a failure pattern that is not well defined except immediately beneath the foundation. The slip surfaces end somewhere in the soil mass. Failure is not catastrophic and tilting is insignificant. However, the vertical movement of the foundation is significant.

From the description of the punching and local shear failure, it can be concluded that the two modes of failure do not lead to catastrophic collapse of the ground which may result in the total collapse of the structure and hence the attainment of ultimate limit state condition. However, ultimate limit states in the structure can be reached as a result of excessive deformation of the ground associated with both the punching and local shear failure.

From the geotechnical engineering perspective, serviceability limit state is settlement. Due to the presence of voids, soils always settle under loading. Soil settlement when related to the structure can be classified as uniform or differential settlement. Uniform ground settlement is not really detrimental to the structure. However, it leads to distress in service pipes and cables connected from external mains to the structure. Such distress may result in the fracturing of the service pipes. It is the differential settlement that may affect the overall efficiency of the structure. Serviceability limit states are reached when the angular distortion is 1/300 (Burland et al, 1978). The most common symptom is the development of cracks in elements such as walls, floors, beams, etc. Cracks lead to water penetration, resulting in corrosion of reinforcement and gradual distortion of the structure. Cracks also result in reduced weather- tightness, dampness; heat loss and reduced sound insulation. Another

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symptom of violation of serviceability limit state is deflection of elements. Deflection affects operation of lifts and other precision machinery. Both cracks and deflection of elements render a structure visually unacceptable.

Due to this inter-relationship between the ground and the structure, limit states are generally defined in terms of damage to the structure as damage to the ground is rarely of significance in itself (Simpson, 1981). Actions from the superstructure lead to accidence of the above limit states in the ground. The violation of the ground limit states in turn lead to distress in the structure or its elements which may lead to failure or serviceability problems.

It is evident from the foregoing discussion that limit state approach to design requires that the designer should check the adequacy of the structure against collapse and serviceability. Many geotechnical engineers (e.g Boden, 1981; Simpson, 1981; Becker, 1996a) are of the opinion that this approach has always been used in one form or the other in geotechnical design and therefore limit state design is not a radically new method compared to earlier design practice. First limit states concept in geotechnical engineering include work by Coulomb and Rankine (Meyerhof, 1995). Coulomb derived the critical height of a vertical embankment in cohesive soil based on limit states considerations while Rankine established limit states of active and passive earth pressures. However, the classical geotechnical limit state approach became well established when Terzaghi introduced the modern approach to soil mechanics. In 1934, Terzaghi classified geotechnical problems into two categories, namely stability problems and elasticity problems. The stability problems deals with conditions immediately before ultimate failure by plastic flow without consideration of strain effects while elasticity problems deal with soil deformation either under self-weight or external forces without consideration of stress condition for failure. The above two classes of geotechnical problems coincide with ultimate limit state and serviceability limit state respectively in the current limit state design philosophy.

The first limit state code of practice was the 1956 Danish Standard for foundations. This resulted from work by Taylor and Brinch Hansen. Taylor (1948) introduced separate factors of safety on the cohesive and frictional components of the shear strength parameters in the analysis of stability of slopes. The approach was generalised by Brinch Hansen (1965) when he proposed partial factors on different type of loads, shear strength parameters and pile capacities for ultimate limit state design of earth retaining structures and foundations. From the forgoing, it appears that in earlier days, geotechnical engineering was ahead of structural engineering in the knowledge and application of limit state design philosophy.

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The current format of limit state design represents a clearer formulation of some widely accepted principles. From this premise, limit state design is regarded as a calculation tool in the design process where satisfactory performance is the primary objective. In fact the design philosophy is similar to that for working stress design approach with the emphasis shifted from elastic theory and materials strength to failure of the structure to perform its intended function (Becker 1996a).

2.2.3.1 Verification of non-exceedance of limit states

In principle any method (e.g. global factor, partial factors) can be used to give confidence that the limit states are satisfied. However, during the development of the approach the partial factors format became the routine method for ensuring non-exceedance of the limit states. Originally the partial factors were determined by subjective means such as engineering judgement and fitting to the working stress design approach. However the advent of the application of probability theory to design provided a direct link between the partial factors and probability of failure. Accordingly a second method of deriving partial factors based on probabilistic considerations emerged. Both approaches are currently in use, leading to two distinct types of limit state design (i.e. deterministic and probabilistic).

2.2.3.2 Deterministic limit state design

Limit state design in the partial factors format became the general design approach in structural practice in the 1970s. Limit state design in this format was also permeating into geotechnical design. Christian (2003) asserts that the interest was driven by the desire to apply the same mathematical insights to geotechnical practice that have proved successful for structural engineering. The move was also motivated by the desire to achieve compatibility between geotechnical and structural engineering. However in geotechnical engineering, the use of the partial factors has generated some controversy. The proponents of the approach argue that if the use of the approach is felt to represent an improvement in structural design, why should its use not be equally valid in geotechnical design? Some of the reservations expressed by the geotechnical profession (e.g. Boden, 1981; Semple, 1981; Simpson et al, 1981; Semple, 1981; Ovesen , 1981; Fleming, 1989) include:

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- Not too many foundations and substructures are failing at the moment or appear to have excessively conservative designs, so why the fuss?
- The method is cumbersome to use, with a multiplicity of coefficients and increases the chances for computational errors;
- Results of analysis using partial factors must fit experience and therefore do not produce substantial differences in overall safety factors;
- Splitting the safety margin into components associated with loads and resistances introduces uncertainty as to whether any function of the original safety factor has been omitted;
- When failures occur, it is due to serious errors in unforeseen conditions. Failures due to excessive variation of recognised parameters are rare and should not be given undue emphasis by focusing attention on partial factors;
- Prescribed factors applied to soil properties might define soils which could not possibly exist. Under such circumstances the design could hardly be considered to be realistic;
- The method of applying a safety factor in the working stress design approach have been developed to enable the best use to be made of decades of full-scale and model evidence. The factors are applied to the ground properties or loads or to some derived quantity depending upon which approach has shown to be the most appropriate. Therefore there is considerable reluctance to depart from this practice unless there are convincing reasons of overriding importance;
- The majority of geotechnical design procedures used throughout the world are rooted in engineering judgement and empiricism. However when used in the traditional way which draws on many years of experience, satisfactory designs have been produced. Under these circumstances, the replacement of the traditional method of factoring by a range of new prescribed partial factors has obvious dangers;
- The partial factor approach integrates safety factors into analysis, tending to distort perception of parameter and behaviour. Attention is likely to focus on satisfying the code requirements than on the truly important aspect of understanding soil distributions, properties and behaviour. The conventional separation of behavioural assessment from safety considerations enhances geotechnical design practice;
- Partial factors can lead to probability theory and encourage statistical assessment of measured data. In geotechnical engineering, these provide little insight and divert attention from reality. Statistical analysis may cause major errors in selection of soil parameters, there being no comparable problem with structural design in manufactured materials;

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- The true measure of safety is how easily lost is the excess of resistance over load. This can be studied by sensitivity study analyses which are compatible with the global safety factor approach. Partial factors specified in codes and incorporated into analyses may discourage proper in-depth study in complicated problems just as they tend to complicate routine design without apparent benefit.

Phoon (1995) adds the following reservations:

- Partial factors for soil parameters are not dependent on influential factors such as the design equation and the procedure for determining the soil strength;
- Definition of nominal values to be factored is not clearly given, resulting in different engineers adopting different values. For an example, one might use average soil parameters while another may use conservative soil parameters as the nominal value;
- Partial factors do not account for the variation in uncertainty of soil parameters from site to site;
- It is not clear how partial factors can assist in extrapolating the experience of safe practice to new conditions or can permit full advantage to be taken of improvements in the knowledge base.

Closely associated with the partial factors is the use of statistics and probability. The following reservations against the application of statistics and probability in geotechnical engineering have been expressed:

- Man-made steels and concretes are fundamentally different from naturally occurring geotechnical materials. The former are ideally suited to definitions by statistics of variations; it is the essence of their quality control during construction. No such control was applied during the formation of soil or rock and the subsequent modification of the earth crust. In view of this situation statistics have no place in geotechnics;
- There is danger in the use of statistics as a less experienced engineer might place too great an emphasis on the role of statistics to the detriment of the geotechnics;
- The properties and the three-dimensional geometry of the ground are often typified by extreme values (e.g. a soft layer under one corner of the structure) rather than by randomly distributed values having means and standard deviations which are valid for design purposes. Under such conditions, the use of statistics might not help much. The use of statistics might encourage an unjustified feel of security in the designer;

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- The probability theories are even more complex than the geotechnical theories themselves. In these circumstances there would be a danger that the attention of the designer could be diverted from understanding the main problem (i.e. the geotechnics) to attempting to understand the probability theory which is just an aid to the design process.

Despite all the controversy, geotechnical limit state design continued to develop to its current status. The motivation to continue applying limit state design to geotechnical practice stems from three main factors:

- The need to achieve compatibility between structural and geotechnical design. Structural design codes of practice have progressed to the new limit state approach while geotechnical codes have not. Therefore different design procedures are being used on either side of the interface between the structure and the ground. Although the methods and analysis used on both sides produce reasonable designs within their own current-state-of-the-art, it is desirable for the two design processes to be compatible (Borden, 1981);
- There has been an intensive limit state design awareness campaign by the Technical committee (TC23) of the International Society of Soil Mechanics and Geotechnical Engineering on Limit State Design in Geotechnical Engineering. As already mentioned the committee has organised many conferences on limit state design in geotechnical engineering practice;
- The application of statistics, probability and reliability theory to analysis and design of structures made it possible for probabilistic assessment of level of safety in terms of probability of failure. The load effects and resistance can now be expressed in terms of their distributions and the overlap of the two distribution curves signifies a condition where the resistance is less than the load effect and hence a probability of failure. This lead to a transparent and rational treatment of uncertainties. Even the partial factors could now be derived on the basis of reliability theory.

2.2.3.3 Probabilistic limit state design

The controversy and severe criticism on the application of limit state design with partial factors to geotechnical practice appear to stem from the lack of a theoretical basis on which the partial factors were developed. Just like the global factors, the partial factors were developed purely on basis of intuition and judgement. Further more the new approach was

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required to produce designs similar to the existing working stress design approach. If the new design approach yields the same results as the old approach then what is the need for the new approach? Clearly there is no noticeable benefit from the transition from the working stress design to the limit state design in a non-probabilistic framework. Without reasons of overriding importance, it is understandable that there was a considerable reluctance to depart from the familiar factoring method. Therefore an important step in justifying the use of limit state design is to ensure that the partial factors are derived on the basis of reliability theory and not through simply scaling to achieve the same design as the methods it would replace.

The non-probabilistic limit state design just like the working stress design does not explicitly quantify the level of safety achieved. Therefore in recent years, probability and reliability theory have been added to the original limit state design leading to the semi-probabilistic limit state design. A further motivation is the explicit and rational treatment of uncertainties. Essentially, semi-probabilistic limit state is a level 1 reliability based design method. Level 1 reliability methods are design methods in which appropriate levels of reliability are provided on the structural component by the use of prescribed partial factors. The derivation of the partial factors explicitly and systematically incorporates the major sources of uncertainties. Ideally the partial factors are calibrated on the basis of reliability theory. The use of such partial factors ensures consistent level of reliability over a range of structures.

Although the approach is set within a probabilistic framework, it does not require explicit use of the probabilistic description of the variables. This is an advantage in that even engineers with no knowledge of probability and reliability theory can produce designs at a prescribed level of reliability. Due to their convenience and simplicity, level 1 reliability methods form the basis of current reliability based design codes. Accordingly new generation design codes for geotechnical engineering are now developed on the basis of level 1 reliability methods. The many advantages and benefits of using reliability based design methods include the following (Ayyub et al, 2000):

- They provide the means for the management of uncertainty in loading, strength, and degradation mechanism;
- They provide consistency in reliability;
- They result in efficient and possibly economical use of materials;

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- They allow for future changes as a result of gained information in prediction models and material and load characterisation;
- They allow for performing system reliability analysis.

In addition to the above advantages, reliability analysis provides a unifying framework for risk assessment across disciplines (especially structural and geotechnical design) and national boundaries. Traditionally geotechnical engineering has been a localised practice based on the excuse that geotechnical materials differ from one region to the other. However, globalisation dictates harmonisation of technical rules for design of buildings and civil engineering works. The harmonisation of design rules for all structures dictates that geotechnical design being based on the same limit state design as for structural design involving other materials. This need has led to what is termed as basis of design in the Eurocodes (EN 1990, 2002) and the revised South African Loading code (SANS 10160-Draft).

2.3 Basis of geotechnical limit state design to SANS 10160

Basis of design provides a common basis and general principle for the design of building and civil engineering works within the limit state design framework. In general, common rules are required for performance requirements, specification of the limit states, design situations to be checked, reliability requirements, and treatment of basic variables (actions, materials properties, and geometric data). This section presents the basis of design pertaining to geotechnical limit state design in the context of EN 1990, EN 1997-1, JCSS model code, ISO 2394 and SANS 10160-Draft.

2.3.1 Fundamental requirements

The fundamental requirements for all structures are outlined in EN 1990, JCSS model code, ISO 2394 and SANS 10160-Draft. In accordance with these references, a structure is required to fulfil the following requirements:

- Remain fit for the use for which they are required (serviceability limit state requirements);
- Withstand actions and influences occurring during their construction and anticipated use (ultimate limit state requirements);

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- Shall not be damaged by accidental events like fire, explosions, impact, or consequences of errors to an extent disproportionate to the original cause (robustness requirements).

These requirements can be achieved through the choice of suitable geotechnical parameters, use of appropriate calculation models and by specifying control procedures for design, production, execution, and use relevant to the particular project.

2.3.2 Design situations

Variations of material properties, environmental influences and actions with time can be accommodated by the selection of an appropriate design situation. The design situations are generally classified as (JCSS model code, 2001):

- Persistent situations, which refer to conditions of normal use of the structure and are generally related to the working life of the structure;
- Transient situations, which refer to temporally conditions of the structure, in terms of its use or exposure;
- Accidental situations, which refer to exceptional conditions of the structure or its exposure.

In establishing the appropriate design situation for geotechnical design, EN 1997-1 gives a comprehensive list of factors to be taken into account as follows:

- the actions, their combinations and load cases;
- the general suitability of the ground on which the structure is located with respect to overall stability and ground movements;
- the disposition and classification of the various zones of soil, rock and elements of construction, which are involved in any calculation model;
- dipping bedding planes;
- mine workings, caves or other underground structures;

In the case of structures resting on or near rock:

- interbedded hard and soft strata;
- faults, joints and fissures;
- possible instability of rock blocks;

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- solution cavities, such as swallow holes or fissures filled with soft material, and continuing solution processes;

The environment within which the design is set, including the following;

- effects of scour, erosion and excavation, leading to changes in the geometry of the ground surface;
- effects of chemical corrosion;
- effects of weathering;
- effects of freezing;
- effects of long duration droughts;
- variations in ground-water levels, including, e.g. the effects of dewatering, possible flooding, failure of drainage systems, water exploitation;
- the presence of gases emerging from the ground;
- other effects of time and environment on the strength and other properties of materials: e.g. the effect of holes created by animal activities;
- earthquakes;
- ground movements caused by subsidence due to mining or other activities;
- the sensitivity of the structure to deformations;
- the effect of the new structure on existing structures, services and the local environment.

To establish the above factors, detailed soil exploration is required. In order to establish minimum requirements for the extent and content of geotechnical investigations, calculations and construction control checks, the complexity of each geotechnical design need to be identified together with the associated risks. In this regard EN 1997 specifies three categories of complexity as follows:

- Geotechnical Category 1, which includes small and relatively simple structures for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations. Generally for this category, risk is considered negligible;
- Geotechnical Category 2, which include conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions. Design of

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structures within this category requires quantitative geotechnical data and analysis to ensure that the fundamental requirements are satisfied;

- Geotechnical Category 3, which includes structures or parts of structures, which fall outside the limits of Geotechnical Categories 1 and 2. Geotechnical Category 3 structures require alternative provisions and rules.

2.3.3 Limit states

Generally limit states are classified into ultimate limit states and serviceability limit states. Depending on the nature of the geotechnical structure at hand five ultimate limit states are recognised in SANS 10160-Draft and EN 1997-1. These include:

a) Loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing the resistance (EQU);
b) Internal failure or excessive deformation of the structure or structural element including footings, piles and basement walls in which the strength of the structural materials is significant in providing resistance (STR);
c) Failure or excessive deformation of the ground, in which the strength of the soil is significant in providing resistance (GEO);
d) Loss of equilibrium of the structure or the ground due to uplift by water pressure or other vertical actions (UPL);
e) Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD).

In practice, experience will often show which type of limit state will govern the design and the avoidance of other limit states may be verified by a control check.

2.3.4 Design methods

Consistent with the practice world wide, four fundamental ways of carrying out geotechnical design are recognised in EN 1997.

- Using calculations based on analytical , semi-empirical or numerical models;

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- Adopting prescriptive measures involving conventional and generally conservative procedures;
- Using experimental models and load tests carried out on a sample of construction;
- Using the observational methods, in which the design is continuously reviewed during construction.

For category 2 structures (i.e. commonly encountered structures) general designs are carried out by the calculations approach. Further more, design by calculation is amenable to harmonisation of basic rules with that for design of structures involving other materials, hence it is further explored in the subsequent subsections.

2.3.5 Design by calculations

Consistent with structural design, geotechnical limit state design calculation involves:

- Considerations of actions (i.e. load types, load combinations and load factors);
- Geotechnical materials properties;
- Geometric data;
- Calculation models.

These components are generally referred to as the basic variables. Uncertainties in the basic variables propagate through the rest of the calculation and eventually affect the performance. In the spirit of harmonisation of design rules, a standardised approach to characterisation of the basic variables in terms of characteristic values and design values have been adopted in ISO 2394, Eurocodes and SANS 10160-Draft.

2.3.5.1 Load types, load combinations and load factors

The design of any structure requires the understanding of the types and magnitude of the loads that are expected to act on the structure during its life span. Although a major input in the design, loads (except geotechnical loads) required for geotechnical design are usually provided by the structural engineer. Although it is not within the scope of geotechnical design to develop loads and load combinations required, it is useful to understand the principles underlying the development of load combination schemes and the associated load

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factors. It is for this reason that this section is dedicated to description of load types, load factors and load combinations.

2.3.5.1.1 Load types

Eurocode 7 provides a comprehensive list of sources of loads or actions to be considered for geotechnical design. The list is as follows:

- the weight of soil, rock and water;
- stresses in the ground;
- earth pressures and ground-water pressure;
- free water pressures, including wave pressures;
- ground-water pressures;
- seepage forces;
- dead and imposed loads from structures;
- surcharges;
- mooring forces;
- removal of load or excavation of ground;
- traffic loads;
- movements caused by mining or other caving or tunneling activities;
- swelling and shrinkage caused by vegetation, climate or moisture changes;
- movements due to creeping or sliding or settling ground masses;
- movements due to degradation, dispersion, decomposition, self-compaction and solution;
- movements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads;
- temperature effects, including frost action;
- ice loading;
- imposed pre-stress in ground anchors or struts;

A close scrutiny of the above list of possible actions reveals that they can be classified into two broad classes of:

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a) External load applied to the soil or to the geotechnical structure. This comprises of loads from the superstructure such as column load on a foundation;
b) Loads originating from the soil itself. These are actions generated by the soil itself such as earth and water pressure and are generally known as geotechnical loads.

2.3.5.1.2 External actions

In recognition that external actions are random variables in time, they are normally classified by their variation in time (EN 1990, SANS 10160) as follows:

(i) **Permanent actions**: These are gravitational loads caused by the weight of structural materials such as concrete, steel, etc. In general, the magnitude of this class of actions usually does not change during the life of the geotechnical system. Also it can be estimated relatively accurately. Tang (1981) identified the following as possible sources of uncertainties associated with permanent loads:

- Variability in materials density due to inhomogeneity and tolerance of manufacturers;
- Discrepancy in dimensions from design values;
- Uncertainty in the final choice of building materials;
- Variability in the non-structural components such as decorative architectural forms.

To account for these uncertainties, a coefficient of variation (COV) of 10% is usually adopted for the variation of permanent actions in design of superstructures. In foundation design, the total dead load is composed of the sum of contributions from many components of dead loads and therefore in accordance with the law of large numbers, the variability in the total dead load will be less than the 10% values suggested for the superstructure (Tang, 1981).

(ii) **Variable actions:** These are loads that vary with time. They can be further classified into sustained live loads and transient life load. Sustained life loads represents weight of people and their possessions, furniture, movable partitions and other portable fixtures and equipment (Nowak and Collins, 2000). These are basically imposed loads on building floors, beams and roofs. The magnitude depends on the

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type of occupancy. The occupancy classes and their respective load magnitudes are given in most national loading codes including SANS 10160.

Surveys to obtain statistical data on the sustained live load have been carried out by many researchers (e.g. Corotis and Doshi, 1977; Ellingwood et.al 1980). The results of the surveys indicate that sustained live load can be modelled as a gamma distributed random variable.

Transient live loads or extraordinary loads on the other hand include forces due to wind, snow, storm waves, and anticipated rare events, such as concentrations of persons or of furniture, or the moving or stacking of objects which may occur during reorganisation or redecoration (Tang 1981). The major characteristics of this type of load are:

- For most of the time they do not occur
- When they occur, the magnitude and the duration of the load are uncertain.

For design purposes both the sustained and the transient loads need to be taken into account. Ellingwood et.al (1980) suggests that the total variable load is modeled by an Extreme value Type I distribution.

(iii) **Accidental actions:** These haven been defined in EN 1990 as actions that are usually of short duration but of significant magnitude and are unlikely to occur on a given structure during the design working life. Typical examples include fire, explosions or impact loads. Further more, EN 1990 and SANS 10160 classify actions due to earthquakes as accidental actions.

Although other classification schemes are available, the classification based on variation in time is more appropriate for the establishment of action combination schemes.

2.3.5.1.3 Characteristic and design values of external actions

In accordance with SANS 10160 and the Eurocodes approach, all actions are introduced in design calculations through their characteristic values. Depending on the available data and experience, characteristic values may be specified as a mean, an upper or lower value of a

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nominal value. Table 2.7 presents the description and statistical meaning of actions in accordance with SANS 10160. Further more, for variable actions, several independent variable actions may act on the structure. However it is unlikely that all the actions will reach their maximum values in a given reference period simultaneously. To take account of this, a combination factor is applied to the accompanying variable actions. In SANS 10160, a single combination value is specified for each variable action, as opposed to the Eurocode scheme of specifying the three values of combination, frequent and quasi-permanent values. For accidental actions, SANS 10160 and EN 1990 state that the design value A_d should be specified for individual projects.

Table 2-2: Definitions of characteristic values

TYPE OF LOAD	CHARACTERISTIC VALUE	
	DESCRIPTION	STATISTICAL DEFINITION
PERMANENT	Small variability (cov < 0.1) High variability (cov > 0.1)	Mean value Two values G_k = 5% fractile G_k = 95% fractile
VARIABLE	Upper value with intended probability not to be exceeded Nominal	98% fractile based on a reference one year period (i.e. expected maximum over 50 years design life) No reference to statistical distribution

2.3.5.1.4 Geotechnical actions

In addition to the permanent and variable actions described above, substructures such as foundations and retaining walls are subjected to geotechnical loads. Depending of the nature of the geotechnical structure, geotechnical loads may consist of some of the following action types:

- Vertical earth loading
- Earth pressure
- Water pressure (groundwater and free water)
- Actions due to ground displacement
- Earth surcharge loading

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Vertical earth loading: Vertical earth loading comprises the weight of backfilling material (i.e. overburden pressure). Overburden pressure is classified as permanent load. It is calculated on the basis of nominal dimensions shown on the drawing and the mean unit weight.

Lateral earth pressure: The force effects of lateral earth pressure due to partial or full embedment into soil must be considered in the design of substructures. The magnitude of the lateral earth pressure on the structure is a function of:

- Type of structure (e.g. gravity, cantilever anchored or mechanically stabilized earth wall or flexible or rigid buried structure);
- Type of retained materials (unit weight and strength properties);
- Magnitude and direction of lateral movement of the structure relative to the retained materials;
- Compaction effort used during placement of soil fill;
- Location of ground water table and the seepage forces within the retained materials;
- Presence of surcharge loads on the retained soil mass.

Nonetheless, the stiffness of the structure and the type of the retained materials are the most significant factors governing the distribution of the developed earth pressure. Structures that can move laterally away from the retained earth mobilise an active state stress in the retained soil mass. Consequently such structures should be designed using an active earth pressure distribution. Conversely structures which for one reason or the other are forced to move laterally towards the retained soil mass mobilise passive earth pressure and should therefore designed to resist the passive earth pressure. When no movement of the structure relative to retained soil mass is anticipated, the pressure is calculated from the at-rest state of stress.

Surcharge load: A surcharge load is any load imposed upon the surface of a retained soil mass. Consequently it causes a lateral pressure to act on the retaining structure in addition to the basic earth pressure. Examples of surcharge loads include materials stockpiles or construction machinery, traffic loads on adjacent streets, spoil embankments adjacent to the structure, loads from adjacent structures, and rail traffic. Given the wide range of sources of surcharge loads, some will fall under permanent while others under variable action.

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Generally the loads are transmitted through the soil to the substructure. The magnitude of the additional lateral earth pressure due to surcharge load is governed by the type (i.e. point or uniformly distributed), the shear strength of the retained soil mass, the stiffness of the retaining structure relative to lateral displacement, and the magnitude and proximity of the surcharge load to the structure. The distribution of lateral pressure due to surcharge loads is usually estimated using Boussinesq's theory.

Water pressure: Forces on structures due to water action include static pressure, buoyancy and stream pressure. Static water pressure needs to be considered when differential water loads develop on a structure. Consideration for buoyancy is critical when the structure is constructed below the water table (e.g. a spread footing or pile cap located below the water table). Effects of stream pressure including floating debris, waves and stream currents and scour.

The main parameters in the calculation of the water pressure are the unit weight of water and the level of the water (water table or free water level). While the unit weight is well known, the water level poses the greatest uncertainties in determining the characteristic or design water pressure. This is due to the fact that the water level is dictated by:

- Seasonal changes
- Rainstorms
- Dewatering activities in the neighbourhoods

If piezometric measurements are available, long-term predictions could be made from the recorded seasonal cycles and short-term fluctuations caused by rainstorms. Eurocode 7 concurs with this approach and states that the geometric data which determine the regime of free water or ground water shall be made on the basis of locally available data for the hydraulic and hydrogeological conditions at the specific site. Bauduin (1998) further elaborates this clause by suggesting the following approach to the selection of the design water level when reliable measurements of water levels are available:

- select the characteristic water level on the base of a high water level or low water level fractile based on a reference period of one year,
- establish the design value by adding or subtracting a certain value on the characteristic levels.

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If piezometric data are not available, Tang (1981) suggests the use of hydrological models considering precipitations and infiltration rates to predict water levels. Bauduin (1998) on the other hand suggests that, when only a few measurements are available, the design value may be found by adding or subtracting a certain value to the most unfavourable known or measured water level.

Actions due to ground displacement: The ground may be subjected to displacements as a result of a number of natural processes such as consolidation, swelling, subsidence of adjacent deep excavations or mines, creeping of the soil, land slides and earthquakes. Structures which were initially in equilibrium with the supporting strata are required to react to the imposition of additional actions due to ground movement. In the process of reacting to the additional forces, internal stresses develop in structural components leading to tilting and rotation of the entire structure. Burland et al (1978) identified four possible consequences resulting from ground movement. These are:

(i) Impair visual appearance of the structure as a whole: This is usually in the form of tilting of walls, floors and the whole building. A structure in such a condition could be unpleasant or even alarming for the occupants and visitors;

(ii) Impair visual appearance of architectural materials: Visible damage such as cracks and deflection of cladding is unsightly;

(iii) Affect serviceability or function of the structure: Structural damage from ground displacement can affect the overall efficiency of the building such as reduced weather-tightness, rain penetration, dampness, draughts, heat loss, reduced sound installation, windows and doors sticking. Ground movement can also affect the basic function of the structure such as the operation of lifts or precision machinery, access ramps and fracturing of service pipes;

(iv) Stability problems: In addition to the violation of serviceability limit states presented above, large ground displacement can lead to violation of ultimate limit state. This manifest itself in severe damage to the cladding and fittings and eventually to the collapse of the structure.

Depending on the nature of the structure and the soil type, principal actions imposed on the structure by ground movements include: down-drag, heave, and transverse loading. These

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forces exclude forces and associated damages stemming from landslides, earthquakes and mining subsidence as they are generally regarded as accidental actions.

Down-drag acts as a permanent additional axial load on a pile and if the force is of sufficient magnitude, bearing capacity failure at the base is possible. This is a downward drag force on the pile by the soil surrounding (Das, 2003). Down-drag generally occurs when a hard layer is overlain by a consolidating stratum. Typical conditions include:

- When a fill of clay soil is placed over granular soil layer into which a pile is driven. Under this condition the consolidation process of the fill will produce a downward drag force on the pile;
- When a fill of granular material is placed over a layer of soft clay. In this case the fill will induce the process of consolidation in the clay layer, thereby producing a down-drag force on the pile;
- Lowering of ground water table. This increases vertical effective stress on the soil and thereby causing consolidation in the soil with the resultant downward drag forces being developed on the pile.

In contrast to down-drag, heave is an upward force which can be generated beneath shallow foundations and along the pile shaft. It is essentially caused by the expansion of the ground. Typical conditions that may result in the expansion of the ground include:

- When clay is able to absorb more water than it had hitherto. Under these circumstances, the clay expands and thereby producing an upward force on affected structures. The increase in water content may result from the removal of trees, cessation of abstraction from aquifers, prevention of evaporation and from accidents (EN 1997);
- Reduced pressure on over-consolidated clay. As a result of reduced pressure the clay tries to expand vertically to be restored back to what it was prior to being compressed in the past. Reduced pressure is associated with excavation and unloading activities;
- Heave may also be caused by frost action and driving of adjacent piles.

Both down-drag and heave are a result of vertical ground movement. Horizontal ground movements are also possible in practice. Structures that are likely to be subjected to horizontal loads due to ground displacement are those constructed in areas of mining subsidence, above tunnels, adjacent to deep excavation and in hillsides. With respect to pile foundations, Eurocode 7 lists the following design situations under which horizontal loads need to be considered:

- Different amounts of surcharges on either side of a pile foundation;
- Different levels of excavation on either side of the pile foundation;
- The pile foundation is located at the edge of an embankment;
- The pile foundation is constructed in a creeping slope;
- Inclined or battered piles in settling ground;
- Piles in seismic regions.

2.3.5.1.5 Characteristic values of geotechnical actions

No specific information on geotechnical actions is provided in EN 1990 and reference is made to EN 1997 for guidelines. However, EN 1997 does not provide general guidelines for treatment of geotechnical actions. In fact EN 1990 states the characteristic values of actions (all actions including geotechnical actions) should be derived in accordance with EN 1990. In SANS 10160-Draft, there is provision for a separate treatment of geotechnical actions based on EN 1997. Table 2.3 summarises the treatment of characteristic values of geotechnical actions.

Table 2-3: Characteristic values of geotechnical actions

Action	Type	Characteristic value description
Vertical earth loading	Permanent	Mean value calculated on the basis of nominal dimensions and mean unit masses
Lateral earth pressure	Permanent	Resultant (mean)
Water pressure	- Permanent (ground water pressure) - Variable (free water above ground) Accidental (floods)	Resultant/mean N/A
Surchage load	-Permanent -Variable	Resultant force
Ground displacement - Down-drag -Heave - Transverse loading - Subsidence - Earthquake	 Permanent Permanent Permanent Accidental Accidental	 Mean Mean Mean N/A N/A

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2.3.5.1.6 Load statistics

The calibration of load and resistance factors requires statistical data (bias factors and coefficient of variations) for load and resistance. The collection of this data is a significant effort in the calibration process. In principle, the bias factors and associated coefficients of variation for load can be established from physical measurements or data provided by professional publications and engineering reports. In this study, work by Kemp et al (1987) serves as the main reference on load statistics assumed in the South African loading code Table 2.4 presents the load statistics compiled by Kemp et al (1987). It is evident from table 2.4 that the load statistics assumed by the South African loading code (SABS 0160) are comparable to load statistics in ANSI A58 and Australian loading codes. Accordingly the load statistics suggested by SABS 0160 were adopted for this study.

Table 2-4: Load statistics (Kemp et al, 1987)

Type of load	Code	Mean load / Nominal load	Coefficient of variation	Type of distribution
Dead (permanent) load	ANSI A58 Australian SABS 0160	1.05 1.05 1.05	0.10 0.10 0.10	Normal Lognormal lognormal
Live (office): Lifetime max.	ANSI A58 Australian SABS 0160	1.0 0.7 0.96	0.25 0.26 0.25	Type 1 Type 1 Type 1
Live (office): Point in time	ANSI A58 Australian SABS 0160	0.25 0.19 0.25	0.71 0.79 0.60	Gamma Weibull Gamma
Wind: Lifetime max.	ANSI A58 Australian SABS 0160	0.78 0.30 0.41	0.37 0.43 0.52	Type 1 lognormal Type 1
Wind: Point in time	ANSI A58 Australian SABS 0160	0.0097 0.022 0.05	6.95 0.94 1.08	Type 1 Weibull Weibull

2.3.5.1.7 Fundamental load combinations and load factors

In accordance with EN 1990, the fundamental combinations for ultimate limit states verification are as follows:

- Persistent and transient design situations

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$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j} \quad \text{"+"} \quad \gamma_{Q,1}Q_{k,1} \quad \text{"+"} \quad \sum_{i\succ 1}\gamma_{Q,i}\psi_{0i}Q_{k,i} \qquad [2.4]$$

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j} \quad \text{"+"} \quad \gamma_{Q,1}\psi_{0,1}Q_{k,1} \quad \text{"+"} \quad \sum_{i\succ 1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad [2.5]$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j} \quad \text{"+"} \quad \gamma_{Q,1}Q_{k,1} \quad \text{"+"} \quad \sum_{i\succ 1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad [2.6]$$

Where:

"+"	implies "to be combined with"
$\sum$	implies "the combined effect of"
$\gamma_{G,j}$	is the partial factor for the permanent action *j*;
$G_{k,j}$	is the characteristic value of permanent action *j*;
$\gamma_{Q,1}$	is the partial factor for the leading variable action;
$Q_{k,1}$	is the characteristic value of the leading variable action;
$\gamma_{Q,i}$	is the partial factor for the accompanying variable action *i*;
$Q_{k,i}$	is the characteristic value of the accompanying variable action *i*;
ψ_i	is the action combination factor corresponding to the accompanying variable action *i*;
ξ	is a reduction factor for unfavourable permanent actions *G*.

In addition, EN 1990 allows for the modification of Eq. 2.5 to cater for permanent actions only, also applied in the present South African loading code (SABS 0160). The resulting expression is given by:

$$E_d = \gamma_G G_k \qquad [2.6]$$

Eq. 2.4 is the single expression combination scheme presented in EN 1990 as Expression 6.10 while Eq. 2.5 and Eq. 2.6 constitute the alternative dual expression combination scheme (EN 1990 Expressions 6.10 a & b). For geotechnical design, expression 6.10 (Eq. 2.4) has been adopted. Accordingly EN 1997-1 only provides recommended values of partial factors corresponding to equation 6.10 (table A.3 of Annex A). This derives from the fact that the recommended geotechnical values come from a few calibration studies performed using values of expression 6.10, while on the other hand there is no experience

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on the use of expressions 6.10a and 6.10b in geotechnical engineering (Frank, 2005). Likewise, SANS 10160 has adopted EN 1990's expression 6.10 for geotechnical design.

Partial factors to be used with expression 6.10 vary with the specific ultimate limit state. Generally three set of partial factors to be applied to the characteristic values of actions are introduced in SANS 10160 (Set A, Set B and Set C):

- Set A is used to verify EQU limit state
- Set B is for verification of STR limit state
- Set C is for verification of GEO limit state

Table 2.5 presents the recommended values of partial factors for Sets A, B and C extracted from SANS 10160.

Table 2-5: Values of action partial factors for Set A, B and C

Action type	Value of partial factors		
	Set A (EQU)	Set B (STR)	Set C (GEO)
Permanent			
- unfavourable	1.2	1.2	1.0
- favourable	0.9	0.9	1.0
Variable			
- unfavourable	1.6	1.6	1.3
- favourable	0.0	0.0	0.0

For the GEO limit state, the values of the partial factors are the same as those prescribed in EN 1997-1.

2.3.5.2 Geotechnical materials properties

Properties of materials including soils are described by measurable physical quantities corresponding to the properties considered in the calculation model (ISO 2394). Further more, EN 1990 requires that properties of materials including geotechnical materials should be represented by characteristic values. Unlike the properties of other structural materials, soil properties are not specified but determined by testing on a site specific basis. With all the uncertainties discussed earlier, determination of characteristic values of geotechnical parameters from available measurements is extremely difficult. In fact, the assessment of subsoil properties and conditions from available laboratory and field measurements is an old

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problem that has continuously plagued the geotechnical profession (Tabba and Young, 1981). In acknowledging this problem Kulhawy (2004) states:

The geotechnical parallel to the mythological quest for the Holy Grail is the search for a means by which geotechnical properties of a soil or rock may be determined straightforwardly and reliably from relatively simple in-situ tests.

2.3.5.2.1 Determination of characteristic values of geotechnical parameters

Sources of geotechnical parameter prediction uncertainties have already been mentioned as inherent spatial variability, measurement noise/random errors, measurements/model bias, and statistical uncertainties. In recognition of these uncertainties, the concept of characteristic/nominal values has been introduced into the geotechnical design process. Uncertainties in the prediction of in-situ soil parameters are accounted for implicitly at the characteristic value selection stage. Within the current format of limit state design approach, partial safety factors are applied to the characteristic values of the design variables (loads and material properties) to obtain the respective design values. With this approach, the safety level achieved depends not only on the partial safety factor values specified by the code but also on the way the characteristic values are obtained (Orr, 2002). Therefore a clear definition and rational methodology for its selection are essential. However, geotechnical design codes give little guidance as to how such values should be determined (Borden, 1981; Hicks and Samy, 2002). In the absence of well defined guidelines, the procedure for the determination of geotechnical characteristic values has become an empirical undertaking left to the designer's discretion. Cardoso and Fernandes (2001) argue that it is not logical to apply partial factors of safety to poorly defined characteristic values. But what exactly is this characteristic value?

The characteristic value has been defined as the value assigned to a basic variable associated with a prescribed probability of not being violated by unfavourable values during some reference period (EN 1990). In essence, this definition implies a fractile value of the measured data, usually a 5% fractile. This is applicable to structural materials such as concrete and steel as in these materials failure is governed by weaker localised portions. However in geotechnical design, failure is governed by average soil strength along the failure plain and therefore defining the characteristic value as a 5% fractile is generally not acceptable in geotechnical engineering.

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A definition of the characteristic value that seems to be more representative of the geotechnical design environment is provided in Eurocode 7. Eurocode 7 defines the characteristic value is a "cautious estimate of the value of the parameter governing the limit state". Depending on the extent of the zone of ground governing the behaviour of the geotechnical structure at the limit state being considered, the governing parameter can either be (i) the mean value over the affected surface or volume or (ii) the lower bound value.

A value close to the true mean governs the limit state when:

- A large volume within the homogenous layers is involved, allowing for compensation of weaker areas by stronger areas,
- The structure carried by the soil allows transfer of forces from weaker foundation points to stronger points.

Under these circumstances, the characteristic value of a geotechnical parameter is to be regarded as a cautious estimate of the mean value.

A lower value close to the lower bound values of the soil parameter governs the limit state when:

- A small soil volume is involved, which does not allow for compensation,
- The structure carried by the soil can not resist local failure.

Under these circumstances, the characteristic value of a geotechnical parameter is to be regarded as an estimate of the lower bound value.

In most geotechnical design situations, the zone of ground governing the behaviour of a geotechnical structure is usually much larger and therefore geotechnical performance is often governed by the spatial average of soil properties, such as average compressibility of a volume of soil beneath the footing, or the average shear strength along a potential failure surface. Hence the characteristic value of geotechnical parameters should be the cautious or conservative estimate of the mean value. The use of adjectives "cautious or conservative" is to reflect the fact that the true mean is not known and therefore it can only be estimated on the basis of the available test data with unavoidable uncertainties.

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Even though the definition seems to be suitable, detailed procedure on how to determine the characteristic value has not been provided. Eurocode 7 mentions in general terms various methods for deriving characteristic values. The methods suggested by Eurocode 7 are shown in figure 2.1. Even though the sources of the uncertainties are known, they are accounted for in an implicit manner. Since each of the various methods employed to derive the characteristic value accounts for uncertainties differently, it is not surprising that each method yields different results. A number of research works have been reported in the geotechnical literature where experienced geotechnical engineers were given the same raw data to evaluate the characteristic value (e.g. Fellin, 2004; Ovesen, 1995). In all the reported cases, totally different answers were given to the same problem. For example, Ovesen (1995) reports of an experiment where twenty five Eurocode 7 committee members were asked to determine the characteristic value for the ultimate state for the following ten test results of a certain soil; 138,140,170,171,179,179,182,232,258,272 kN/m^2. The determined characteristic values ranged from 145 to 200 kN/m^2. This indicates that given the same tests results, different engineers with varying background and experience would come to completely different results.

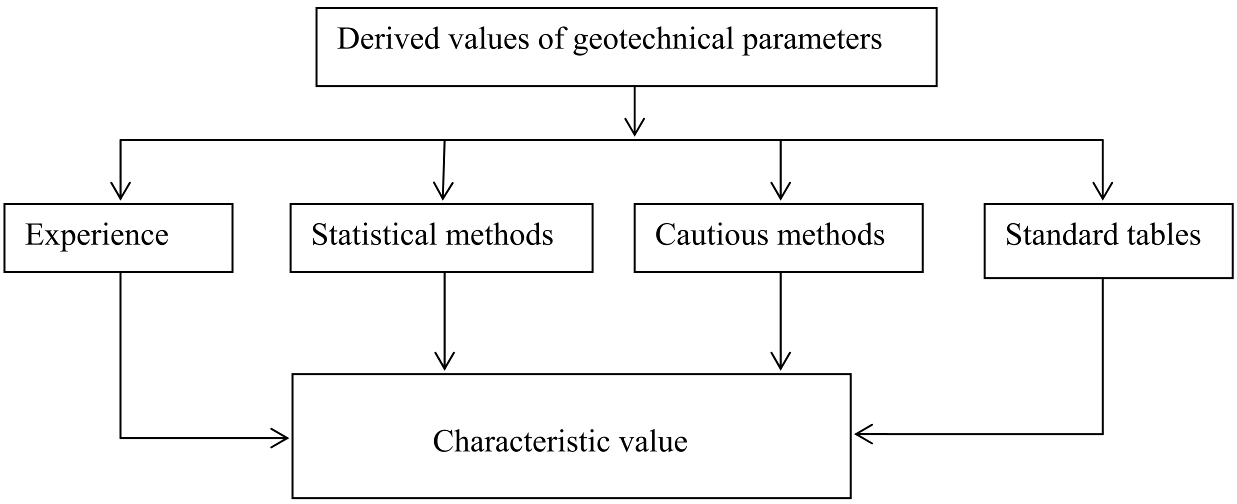

Figure 2.1: Various methods for determining the characteristic value

2.3.5.2.2 Statistical approach to characteristic geotechnical properties

Despite the initial criticism to the application of statistics to the geotechnical practice, in recent years its use has gained popularity. A recent survey conducted by Orr et al (2002) regarding the usefulness of statistical approach, it was found out that 64% of the

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respondents accept the effectiveness of statistical approach in determining characteristic values. The outcome of a similar questionnaire administered in Japan by Shiroto et al (2002) also indicated that over 50% of the respondents were in favour of the use of statistical approach. Those in favour of the use of statistical approach mentioned objectiveness and transparency as the key advantages. Further comments in favour of statistical approach were as follows;

- Personal differences in determining the parameter values can be eliminated;
- It is possible to avoid biased judgement;
- It is an excellent tool in terms of accountability and objectivity to explain to a third party the basis for selecting a particular value.

In any case, using statistical methods as the basis for determining characteristic values structures the decision making process and makes it clearer and more exchangeable between different engineers (Fellin, 2004). Lump (1974) adds that:

"Statistical methods can be of great value to the designer since it is possible to express many of the decision uncertainties in terms of numerical probabilities, thus allowing quantification of judgement to some extent and clarification of the problems."

As shown in figure 2.1, statistical approach is one of the methods for determining characteristic values permitted by Eurocode 7. In this regard Eurocode 7 further states that if statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of a limit state is not greater than 5%. The statement is further interpreted to mean that:

- In the case that the characteristic value is a cautious estimate of the mean value, then the calculated characteristic value is an estimate of the true mean such that there is a probability of 95% that the true mean value is higher than the calculated value. In this respect the characteristic value is the lower bound of the 95% confidence interval of the mean value;
- In the case that the characteristic value is a cautious estimate of the lower bound value, it should be taken as the 5% fractile of the distribution of the parameters i.e. the value for which there is only 5% chance that a lower value may be found.

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2.3.5.2.3 Characteristic value as mean value at 95 % confidence level

In statistical terms, a mean value at a given confidence level is an interval estimate for the mean. Interval estimate for the mean can be defined as the process of utilising the available observations to calculate two numbers (the lower and upper limits) that define an interval that will enclose the true mean with a high degree of confidence. The resulting interval is termed a confidence interval and the probability that it contains the true mean is the confidence level or confidence coefficient. The confidence level is expressed by; (1-α) 100%, where α is level of significance. For the 95% confidence level, (1-α) 100% = 95%. Although the choice of confidence level is arbitrary, Eurocode 7 specifies a 95 % confidence interval.

The theoretical interpretation of the 95% confidence level is that, if a sample size n were to be repeatedly collected from the population and a 95% confidence interval computed for each sample, then 95 % of the intervals will enclose the true parameter value. In practice, only one sample of size n is collected from the population and therefore a 95 % mean confidence interval is assumed to imply that the calculated interval will enclose the true mean with a probability of 95%. Eurocode 7 applies this practical interpretation of 95% confidence interval to the statistical determination of characteristic values of geotechnical properties. Accordingly the characteristic value is the lower limit of the 95% confidence interval. This lower limit value has a 95 % probability that the true mean is equal or higher than it.

The basic equation for the characteristic value as mean value at 95% confidence level is given by (Bauduin, 1998):

$$X_k = \overline{X}\left(1 - Z_{\alpha/2} V \sqrt{\frac{1}{n}}\right) \qquad [2.7]$$

Where; X_k is the characteristic value, $\overline{X}$ is the arithmetic mean of the test results, V is the coefficient of variation of the desired property, n is the number of test results, and $Z_{\alpha/2}$ is the standard normal variate that locates an area of α/2 to its left.

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Equation 2.7 is based on the central limit theorem, which states that for a large sample size (n > 30), the mean is approximately normal irrespective of the distribution assumed. However, since the central limit theorem applies to large samples only, for small sample size the assumption that the sampling distribution of the mean is approximately normal no longer holds. Therefore the sample standard deviation may not be a satisfactory approximation to population standard deviation if the sample size is small. For small sample size as is the case in the geotechnical practice, the student distribution is recommended. The general expression for the lower limit (characteristic value) of a confidence interval for the mean of a small sample, based on the student's t-distribution is given by:

$$X_k = \overline{X}\left(1 - t_{n-1}^{0.95} V \sqrt{\frac{1}{n}}\right) \qquad [2.8]$$

Where; X_k is the characteristic value, $\overline{X}$ is the arithmetic mean of the test results, V is the coefficient of variation of the desired property, n is the number of test results, and t is the value of the student distribution corresponding to a confidence level of 95% and a degree of freedom of n-1.

Equation 2.8 is the basic equation for determining characteristic values for geotechnical properties. When prior information about the coefficient of variation of the desired property is available, V is regarded as known and a limiting value of $t = 1.645$ is used.

2.3.5.2.4 Characteristic value as a 5% fractile

In the case that the characteristic value is a cautious estimate of the lower bound value, a 5% fractile is considered as the characteristic value as it is the case with structural materials. In general, the p fractile x_p is the value of the variable X for which the probability that the variable X is less than or equal to x_p is equal to p. Mathematically, p-fractile is expressed as;

$$P(X \le x_p) = P(x_p) = p \qquad [2.9]$$

For characteristic properties of materials, EN 1990, set the probability p to 5%. This means there is only 5% chance that a lower value may be found. Several methods have been

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developed to estimate the p-fractile. The prediction method is amongst the common methods and it has been specifically mentioned in the geotechnical literature. With this method, the lower p-fractile is assed by the prediction limit $x_{p,\ \mathrm{pred}}$, determined in such a way that an additional value x_{n+1} randomly taken from the population would be expected to occur below $x_{p,\ \mathrm{pred}}$ with the probability p. Thus,

$$P\left(x_{n+1} \langle\ x_{p,\,pred}\right) = p \qquad [2.10]$$

For the case of small sample size the prediction methods lead to the following expression for determining characteristic value as a 5% fractile.

$$X_k = \overline{X}\left(1 - t_{n-1}^{0.95} V \sqrt{\left(\frac{1}{n} + 1\right)}\right) \qquad [2.11]$$

Where; X is the characteristic value, $\overline{X}$ is the arithmetic mean of the test results, V is the coefficient of variation of the desired property, n is the number of test results, and t is the value of the student distribution corresponding to a confidence level of 95% and a degree of freedom of n-1.

From equations 2.8 and 2.11, it can be concluded that the characteristic value is a scaled mean value. For geotechnical properties, the value of the scaling factor depends on whether the characteristic values is considered as a 5% fractile or an estimate of the mean value at 95% confidence level. For the purposes of this study, the scaling factors are designated α_1 and α_2 for the 95% confidence level and 5% fractile respectively. These scaling factors are given by the following expressions;

$$\alpha_1 = 1 - t_{n-1}^{0.95}\, V \sqrt{\frac{1}{n}} \qquad [2.12]$$

$$\alpha_2 = 1 - t_{n-1}^{0.95}\, V \sqrt{\left(\frac{1}{n} + 1\right)} \qquad [2.13]$$

From the expressions for the scaling factors, it is apparent that for a given value of V and n, α_2 gives a smaller factor compared to α_1. Therefore α_1 lead to a characteristic value that is

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closer to the mean compared as to α_2. To further illustrate this point, figure 2.2 shows the relative positions of the characteristic value as a 95% confidence level and the characteristic value as a 5% fractile on a probability density function. It is evident from figure 2.2 that the characteristic value as a 95% confidence level yields values that are close to the mean compared to the characteristic value as a 5% fractile.

Despite the theoretical basis underlying the determination of characteristic values statistically, there is a danger of averaging data from different populations. This can be avoided by rationally dividing the profile into sub-regions, each of which is treated as an independent population prior to the application of statistical techniques. This is consistent with the general feeling in the geotechnical practice that statistics should only be used to help in the assessment of the soil or rock properties after the geological and geotechnical aspects of the problem have been examined. In other words statistics can not replace the importance of understanding the true underlying geology.

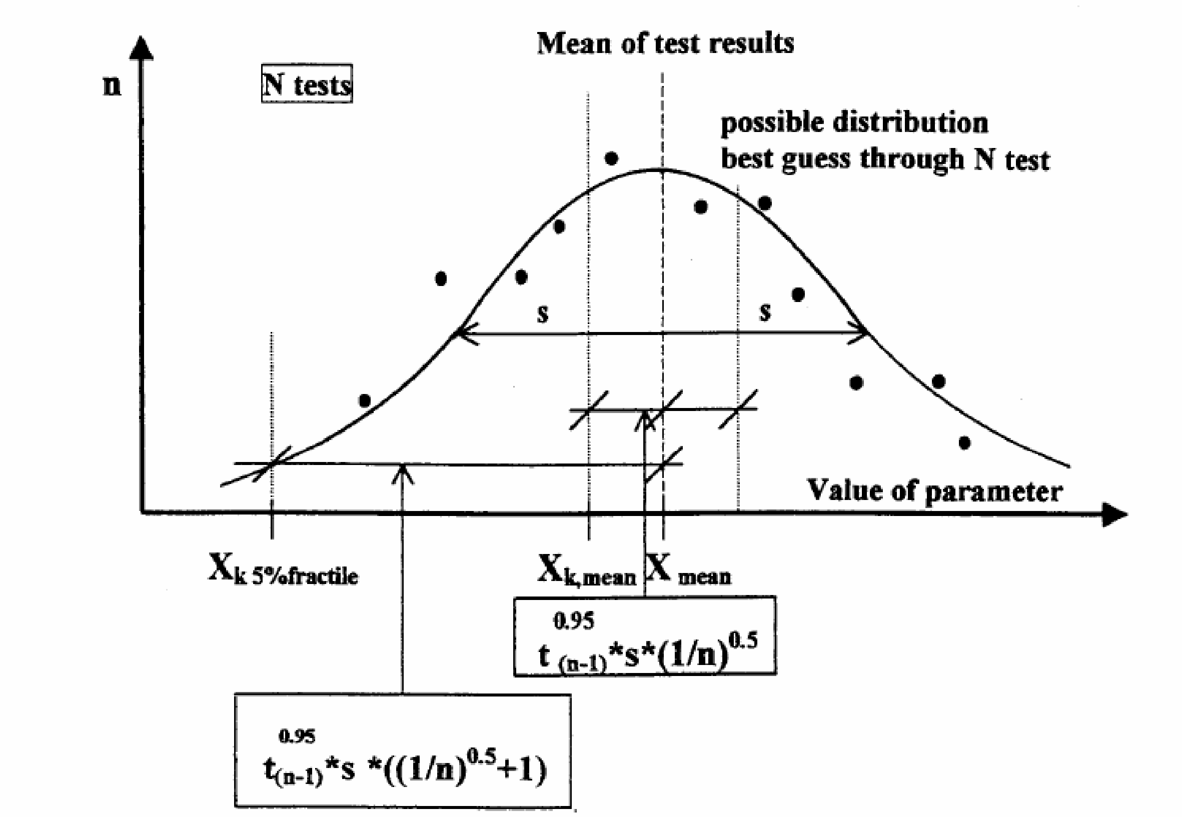

Figure 2.2: Difference between characteristic values as a 5% fractile and mean estimate at 95% confidence level (After Bauduin, 1998)

2.3.5.3 Geometric data

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Geometric quantities describe the shape, size and overall arrangement of structures, structural elements and cross-sections (ISO 2394). In geotechnical engineering this translates to the level and slope of the ground surface, water levels, levels of interfaces between strata, excavation levels and the dimensions of the geotechnical structure. The variability of geometric quantities is generally small or negligible compared to that of materials properties and actions, hence characteristic values of geometric quantities are generally taken as specified on drawings or measured on site.

2.3.6 Design approaches for STR/GEO limit state

There has not been a consensus worldwide regarding where in the calculation process partial factors should be applied (Driscoll and Simpson, 2001, Simpson 2000). The main alternatives include:

- Factors be applied to the primary variables (actions and material properties);
- Factors be applied to action effects and resistances;
- Factors be applied to action and resistance models.

To accommodate the divergent opinions and practices, Eurocode 7 provides three design approaches for design of substructures or structural members involving geotechnical actions. These are: (a) Design Approach 1, (b) Design Approach 2, and (c) Design Approach 3. The attributes of the three design approaches are detailed in Eurocode 7 and have been discussed by many commentators (e.g. Driscoll and Simpson, 2001; Orr, 2002, 2006; Frank, 2002, Schuppener and Frank, 2006). These three design approaches differ in the way they distribute the partial factors on actions and ground resistances/properties. With regard to actions, there are two sets of partial factors:

- Set A1 in which the factors are greater than one for both unfavourable permanent and variable actions.
- Set A2 in which the factors are equal to unity except for unfavourable variable actions.

Material factors also comprises of two sets; M1 and M2.

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- Set M1 in which geotechnical properties are unfactored (i.e. partial factors are equal to one).
- Set M2 in which the partial factors on geotechnical properties are greater than one except for unit weight.

There are also 4 sets of resistance factors as follows: R1 applicable to Design Approach 1, R2 applicable to Design Approach 2, R3 applicable to Design Approach 3 and R4 applicable to Design Approach 1 for design of piles and anchors.

The numerical values of the various sets of partial factors are given in table 2.6: The main features of the three design approaches are outlined below.

Design Approach 1: Except for the design of piles and anchors, two combinations of partial factors need to be investigated. Combination 1 aims to provide safe designs against unfavourable deviation of the actions from their characteristic values (Schuppener and Frank, 2006). Hence, partial factors greater than one (Set A1) are applied to the permanent and variable actions from the structure and the ground. With regard to ground resistance, calculations are performed with characteristic values of soil properties (i.e. unfactored parameters). Accordingly partial materials factors, γ_{φ}, γ_c, and γ_{cu} are all set to unity (Set M1). Further more, the calculated resistance is also not factored (i.e. Set R1).

Conversely, Combination 2 aims to provide safe design against unfavourable deviations of the ground strength properties from their characteristic values and against uncertainties in the calculation model (Schuppener and Frank, 2006). Hence, geotechnical parameters are factored for the calculation of geotechnical actions and for the calculation of resistances. Regarding action from the structure, permanent actions are assumed to be at their characteristic values while the variable actions are assumed to deviate only slightly from their characteristic values and hence a relatively smaller factor is applied. Partial factors Sets A2, M2 and R1 are applicable for this combination.

When it is obvious that one of the combinations governs the design, it is not necessary to carry out calculations for the other combination. Accordingly it has become customary that: Combination 1 governs structural design of the elements while Combination 2 governs the sizing of the elements.

For the design of axially loaded piles and anchors, again two combinations need to be considered:

Combination 1: A1 + M1 + R1
Combination 2: A2 + (M1* or M2**) + R4

in which * applies when calculating the resistance of piles and anchors, ** applies when calculating unfavourable actions on piles owing to negative skin friction or lateral loading.

Design Approach 2: In this design approach, partial factors of greater than unity are applied to actions and resistances while the partial factors on soil parameters are set to unity. Thus design approach provides essentially a resistance factor approach. The partial factors applied to the geotechnical actions and effects of actions are the same as those applied to the action on or from the structure. In this regard, partial factors for Sets A1, M1 and R2 are used with this design approach.

Design Approach 3: Applies partial factors for Set A2 to geotechnical actions and simultaneously applying partial factors for Set A1 to the other actions on/from the structure. In calculating the ground resistance partial factors are applied to the ground strength parameters and not to the obtained resistance (i.e. Sets M2 and R3).

2.3.7 Selection of design approach for the study

The selection of a particular design approach is a matter for national determination. Accordingly South Africa will have to select one of the three design approaches. As part of the current study, an investigation was carried out as to which of the three design approaches was suitable for the South African environment. The basis for deciding which of the three design approaches will be more suited to the South African design environment were as follows:

(a) Design approach that can accommodate a wide spectrum of geotechnical design; situations (e.g. spread foundations, pile foundations, retaining walls, slope stability, etc);
(b) Design procedure that is close to the current design;
(c) Design approach that leads to safe but economic designs.

Table 2-6: Numerical values of various sets of partial factors

1. Partial factors for actions	**Design Approach 1**				**Design Approach 2**	**Design Approach 3**	
	Any other structures		Piles and Anchors				
	Set		Set		Set	Set	
	A1	A2	A1	A2	A1	A1	A2
Permanent (unfavourable)	1.35	1.00	1.35	1.00	1.35	1.35	1.00
Variable (unfavourable)	1.50	1.30	1.50	1.30	1.50	1.50	1.30
Pamanent (favourable)	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Variable (fafourable)	0	0	0	0	0	0	0
2. Partial material factors	Set		Set		Set	Set	
	M1	M2	M1	M1* or M2**	M1	M2	
Shearing resistence	1.0	1.25	1.0		1.0	1.25	
Effective cohesion	1.0	1.25	1.0		1.0	1.25	
Undrained strength	1.0	1.40	1.0		1.0	1.40	
Unconfined strength	1.0	1.40	1.0		1.0	1.40	
Weight density	1.0	1.00	1.0		1.0	1.00	
3. Partial resistance factors	Set		Set		Set	Set	
	R 1	R 1	R 1	R 4	R 2	R 3	
Spread foundations							
Bearing resistance	1.00	1.00			1.40	1.00	
Sliding resistance	1.00	1.00			1.10	1.00	
Driven piles							
Base			1.00	1.30	1.10	1.00	
Shaft (compression)			1.00	1.30	1.10	1.00	
Total/combined (compression)			1.00	1.30	1.10	1.00	
Shaft (tension)			1.25	1.60	1.15	1.10	
Bored piles							
Base			1.25	1.60	1.10	1.00	
Shaft (compression)			1.00	1.30	1.10	1.00	
Total/combined (compression)			1.15	1.50	1.10	1.00	
Shaft (tension)			1.25	1.60	1.15	1.10	
CFA piles							
Base			1.25	1.45	1.10	1.00	
Shaft (compression)			1.00	1.30	1.10	1.00	
Total/combined (compression)			1.15	1.40	1.10	1.00	
Shaft (tension)			1.25	1.60	1.15	1.10	
Retaining structures							
Bearing capacity	1.00	1.00			1.40	1.00	
Sliding resistance	1.00	1.00			1.10	1.00	
Earth resistance	1.00	1.00			1.40	1.00	

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The three design approaches were compared on the basis of the above criteria.

(a) Accommodation of a wide spectrum of geotechnical design situations

Uncertainties are accounted for by the use of partial factors. During the process of drafting the Eurocode 7, it emerged that a partial factor should be applied to basic variables (material properties and actions), or to quantities derived later in the calculation (resistances and action effects) according to where its effect will be most severe. This leads to two broad geotechnical design situations. The first situation is where the greatest uncertainties are with the geotechnical parameters. Examples include slope stability problem, spread foundations, earth retaining structures, etc. Under this condition, it is logical to apply partials factors to the geotechnical properties. The second design situation is where the greatest uncertainty is with the calculation model. Specific examples for this category are pile foundations and anchors. When the predominant uncertainty is associated with the model, it is logical to apply the partial factor to the resistance.

Only Design Approach 1 accommodates both material factor and resistance factor approaches depending on the nature of the geotechnical structure being designed. Design Approach 2 is a resistance factor approach and therefore does not represent uncertainties in material properties adequately. For example, bearing capacity increases more, in proportion, than the angle of shearing resistance from which it is calculated, so it is appropriate to apply the factor to the material property rather than the bearing resistance. From this perspective, Design Approach 2 is suited to design of pile foundations and anchors only. Design Approach 3 is a material property factor approach and therefore does not allow for uncertainty in the calculation model. Based on the above considerations, Design Approach 1 ranks higher than the other two design approaches.

(b) Design procedure that is similar to the current design

The choice between factored materials and factored strength has become a matter of convenience and familiarity. In concept, the resistance factor approach is similar to the global factor of safety used in the working stress design and therefore the method is simple and familiar to the South African geotechnical engineers. On this basis, Design Approach 1 and 2 are preferable to Design Approach 3. For pile foundations, the main difference

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between design approach 1 and Design Approach 2 is on the action side. For the geotechnical ultimate limit state, Design Approach 1 factor only variable actions while permanent actions are at their characteristic values. On the other hand, both variable and permanent actions are factored in Design Approach 2. Therefore Design Approach 1 is more representative of the geotechnical design tradition whereby unfactored load are used in the design. On this account, design approach 1 ranks above design approach 2.

(c) Design approach that lead to safe but economic designs

A preliminary assessment was carried on the basis of a few published examples and design examples carried out by the author. The design examples reviewed are for geotechnical structures within the scope of SANS 10160-Draft (foundations and earth retaining structures).

(i) Spread foundations

Orr and Farrell (2000), present an example of a pad foundation with a vertical load on cohesive soil. For this loading and ground conditions, the results were as follows;

- With design approach 1, combination 2 controls the design.
- Of the three design approaches, design approach 1 gives the smallest foundation size while design approach 3 gives the largest foundation size.

In a presentation to the South African Loading committee, Holicky (2005) presented two examples of a footing on cohesive soil and another footing on non-cohesive soil. The results were as follows:

- For the footing on cohesive soil, the trend is similar to that reported by Orr and Farrell above,
- For the footing on non-cohesive soil, design approach 2 gives the smallest footing size while design approach 3 still gives the largest size. For design approach 1, still combination 2 controls the design.

As part of an in-house report, the author carried out a design example of an axially load pad foundation on cohesionless soil. The results showed a similar trend to that reported by Holicky.

Based on the above design examples, it can be concluded that, in cohesionless soil, Design Approach 2 gives the smallest footing size while Design Approach 3 leads to the largest size. However in cohesive soil, Design Approach 1 produces the smallest foundation and Design Approach 3 still produces the largest foundation size. In terms of safety, Design Approach 3 is the safest but the also the most uneconomic. Therefore Design Approaches 1 and 2 rank higher than Design Approach 3.

(ii) Pile foundations

Driscoll (2005) presented an example of a bored pile in clay. The design was carried out in accordance with the semi-empirical alternative for designing pile foundations on the basis of ground test results. For the design situation considered, it was observed that;

- Combination 1 and 2 of Design Approach 1, gave very similar results for pile length.
- Similar pile lengths were obtained in all the design approaches.

As part of the present study, the design of a bored pile in cohesionless soil was carried out. The final results are presented in table 2.7.

Table 2-7: Final results of a pile design example

Example	Parameter	Design approach			
		DA1.1	DA1.2	DA.2	DA.3
Pile foundation from soil parameter values	L	24.0	24.7	22.5	26.7

A study of table 2.7 lead to the following observations:

- As expected, for Design Approach 1, combination 2 controls the design length. However the two combinations produce close results as reported by Driscoll (2005);
- For this design situation, design approach 2 gives the shortest pile length while design approach 3 gives the longest pile length;

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- The pile length for Design Approach 1 is equal to the average length for Design Approaches 2 and 3. In other words, Design Approach 1 falls midway between Design Approaches 2 and 3. Therefore in terms of safety and economy, it is the optimum design.

The results seem to follow the trend depicted by shallow foundations (i.e. Design Approach 2 produces the smallest foundation size, Design Approach 3 yields the largest foundation size while Design Approach 1 produces an optimum design. Therefore Design Approach 1 ranks higher than the other two Design Approaches.

(iii) Retaining walls

Examples provided by Orr and Farrell (2000) indicated that the three Design Approaches produced approximately the same design (wall size). Therefore from safety and economic perspective, the three Design Approaches are comparable.

From the comparison of the three Design Approaches under various criteria, Design Approach 1 ranked higher than the other two Design Approaches in two out of the three selection criteria. On the third criteria (safety and economic considerations), there was no strong evidence to suggest the one Design Approach was superior to others. However, a design example on pile foundation in cohesionless soil indicated that Design Approach 1 produces an optimum design.

The results of the recent international workshop on the evaluation of Eurocode 7 confirm the foregoing observations. Prior to the workshop, ten geotechnical design examples involving five different areas were distributed. Orr (2005) provided model solutions to the various design examples. A summary of the results of the model solutions are presented in table 2.8. On the basis of the model solution, it was concluded that Design Approach 3 gave the most conservative designs, Design Approach 2 the least conservatism designs, and Design Approach 1 generally gave designs between the other two approaches. These results agree with the findings of the preceding preliminary investigation.

Therefore for this study, Design Approach 1 was selected as the basis for calibrating the partial factors. Based on the same line of reasoning, SANS 10160 has also opted for Design Approach 1.

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Table 2-8: Summary of model solutions (After Orr, 2005)

Example	Parameter	Design approach			
		DA1.1	DA1.2	DA.2	DA.3
1. Spread foundation vertical central load	B	1.62	**2.08**	**1.87**	**2.29**
2. Spread foundation inclined eccentric load	B	3.46	**3.98**	**3.77**	**4.23**
3. Pile foundation from soil parameter values	L	**14.9**	14.6	**14**	**16.7**
4. Pile foundation from load test results	N	9	9	10	-
5. Gravity retaining walls	B	3.85	**5.03**	**4.21**	**5.03**
6. Embedded retaining wall	D	3.14	**4.73**	**4.68**	**4.73**
7. Anchored retaining wall	D	2.6	**3.64**	**3.67**	**3.64**
8. Uplift	T	**0.60**			
9. Heave	H	**6.84**			
10. Embankment	H	**2.90**	2.40	**2.15**	**2.40**

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Chapter 3

RELIABILITY BASIS FOR GEOTECHNICAL LIMIT STATE DESIGN

3.1 Introduction

Uncertainties associated with the geotechnical design process were pointed out in Chapter 1. To deal with such uncertainties, the geotechnical fraternity has developed several strategies. Traditionally a global factor of safety is applied to the resistance side of the equation to cater for all uncertainties. As pointed out earlier, this approach has several drawbacks including the fact that the level of safety achieved is not known. Because it is not known as to what uncertainties are accounted for, Seidel (2002) rightly referred to it as the factor of ignorance. Christian (2004) in the thirty-ninth Terzaghi lecture outlined the following additional approaches for dealing with geotechnical uncertainties:

- Ignoring it
- Being conservative
- Using the observational method
- Quantifying it

Ignoring the uncertainties lead to baseless decisions with catastrophic consequences while being conservative, although guarantees safety, is usually uneconomical. The two design extremes are illustrated in figure 3.1 for the case of rock-bolting. Clearly these two approaches do not meet the fundamental design requirements of simultaneously achieving safety and economy. The observational method or “learn-as you go” is suitable for large projects with complex ground conditions necessitating contract documents tailored for the specific project. The approach is closely related to the Bayesian updating technique. However, under the normal design setting where a complete design is required to facilitate tendering of the construction stage of the project by various contractors, the approach is not feasible. Concerning quantifying the uncertainties, there is consensus that the approach is the most rational and transparent. Christian (2004) asserts that quantifying the uncertainties is consistent with the philosophy of the observational method and should therefore be considered as a logical extension of the approach that accommodates modern developments in probabilistic methods.

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Communication of risk within a transparent and rational framework is further motivated by the increasing pressure in code harmonisation as results of greater economic cooperation and integration brought by the advent of the World Trade Organisation, public involvement in defining acceptable risk levels, and risk-sharing among client, consultant, insurer and financier. The need for a framework that can treat uncertainties in transparent and rational manner can not be over emphasised. The critical question now is what framework is capable of dealing with uncertainties in this desired manner?

Historically, probability theory has been the primary tool for modelling uncertainties. Therefore the framework should ideally be based on probability theory. If the framework is not reliability analysis, then what alternative is available (Phoon, et al, 2003; Phoon, 2004)? Certainly with the current state of knowledge, only reliability analysis and design can provide a consistent method for propagating uncertainties throughout the design process. In addition to dealing with uncertainties, the reliability based design framework provides a unifying framework for risk assessment across disciplines and national boundaries. This is important for achieving compatibility between structural design and geotechnical design so as to avoid the current scenario whereby different approaches are applied to two sides of the soil-structure interface.

Consistent with the robustness of reliability theory in dealing with uncertainties, geotechnical limit state design is now generally based on level 1 reliability methods. Therefore this Chapter provides the reliability background for geotechnical limit states in general. This entails an overview of reliability theory and reliability calibration principles. Although probability theory is the basis for reliability analysis, it is of peripheral nature to this study and therefore it will not be reviewed. Besides, several text books have been devoted to probability theory. Text books on basic probability concepts specifically written for engineers are also available (e.g. Benjamin and Cornel, 1970; Ang and Tang, 1975; Harr, 1987; Smith, 1986; Baecher and Christian, 2003; etc).

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(a) Ignoring uncertainties

(b) Being conservative

Figure 3.1: Rock-bolting alternatives (After a carton in a brochure on rock falls published by the Department of Mines of Western Australia)

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3.2 Reliability Analysis

Reliability analysis falls within the broad subject of probabilistic methods. Accordingly the U.S. Army Corps of Engineers defines probabilistic methods as techniques that may be called or include reliability analysis, risk analysis, risk based analysis, life-data analysis and other similar terms (ETL 1110-2-556). Based on the dependency of reliability on probability theory, Phoon (1995) defines reliability analysis as simply the consistent evaluation of design risk using probability theory. But what does the term reliability mean? The reliability of an engineering system is defined as its ability to fulfil its design purpose for a specific time period (ISO 2394, 1998; Harr, 1987; Ayyub and Popescu, 1998). Probability theory provides the basis for measuring this ability. In a probabilistic setting, the reliability of an engineered system is the probability of its satisfactory performance for a specific period under specific service conditions (i.e. the probability of survival). This definition recognises the fact that design variables (loads, materials properties, etc) are uncertain and therefore success of failure of a designed engineering system can only be assessed in probabilistic terms. It is also consistent with the earlier observation that probability and reliability analysis are interrelated. The complement of probability of survival or reliability is the probability of failure, hence reliability and probability of failure sum to unity. Mathematically the relationship between probability of failure (P_f) and reliability (R) is expressed as follows:

$$R + P_f = 1 \qquad [3.1a]$$

$$R = 1 - P_f \qquad [3.1b]$$

$$P_f = 1 - R \qquad [3.1c]$$

The reliability of a structure is generally expressed in terms of either the probability of failure or reliability index.

3.2.1 Probability of failure

The basic measure of reliability is the probability of failure. In evaluating the probability of failure, the behaviour of the system is described by a set of basic variables $X = (x_1, x_2, \ldots x_n)$ characterising actions, material properties, geometric data and model

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uncertainty. Moreover, the limit state (ULS or SLS) is defined by the performance function given by:

$$g(X) = 0 \tag{3.2}$$

in which $g(X)$ is the performance function, the vector X is a set of n random variables.

The performance function defines three different regions:

- $g(X) = 0$, limit state
- $g(X) > 0$, safe region
- $g(X) < 0$, unsafe region

The probability of failure is equal to the probability of limit state violation which is mathematically expressed as:

$$P_f = P[g(X) < 0] \tag{3.3}$$

The probability of failure can be assessed if basic variables are described by their probabilistic models. For the time-invariant reliability problem described by a time independent joint probability density function, the probability of failure can be determined using the following integral:

$$P_f = \int \ldots \int_{g(x) \le 0} f_x(X) dx \tag{3.4}$$

in which $f_x(X)$ is the joint probability distribution function of the n-dimensional vector X.

The domain of integration is illustrated by the shaded region in the left panel of figure 3.2 while the probability density function is represented by a 2-D surface in the right panel. Eq. 3.4 constitutes the basic reliability analysis equation and the approach is termed the full probabilistic approach. However, to directly evaluate the above n-fold integral using standard methods of integration is a formidable task and closed form solutions do not exist except for very simple cases (Hadj-Hamou et al, 1995). Numerical integration becomes

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extremely complex because the limits of the integrals are not constants but functions of the system variables. For a large number of variables, the numerical integration becomes very time consuming or even intractable. Furthermore, the joint probability density function of random variables is practically not possible to obtain, and the PDF of the individual random variables may not always be available in explicit form (Christensen and Baker, 1982). Therefore for practical purposes, analytical approximation of the integral are employed to simplify the computations of the probability of failure.

The most common methods of approximation are the First Order Second Moment reliability methods (FOSM) also known as level II reliability methods. In the FOSM approximation, the probability of failure is expressed in terms of the reliability index (β).

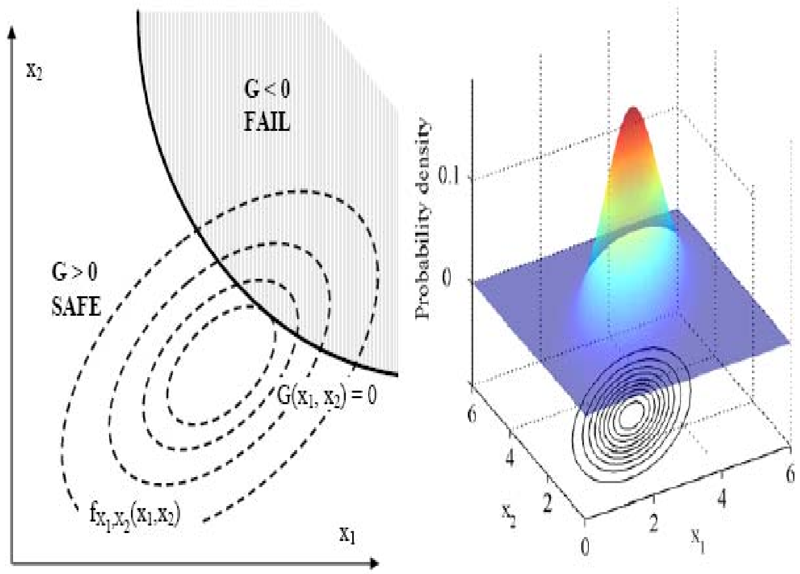

Figure 3.2: Domain of integration and probability density function (After Phoon, 2004b)

3.2.2 Reliability index

In the computation of the reliability index, all what is required are the statistics of the random variables (mean and coefficient of variation). FOSM reliability methods can be further classified into Mean Value First-Order Second Moment (MVFOSM) method and Advanced First-Order Second Moment method (AFOSM).

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3.2.2.1 Mean value first order second moment (MVFOSM) analysis

For the approximate procedures, the concept of reliability index (β) is used to quantify the reliability of the engineering system. This method derives its name from the fact that it is based on a first order Taylor series approximation of the limit state function linearised at the mean values of the random variables and it uses only second moment statistics (mean and variance) of the random variables. When represented by its Taylor expansion, the performance function becomes:

$$g = g(\mu_{X_1}, \mu_{X_2}, \ldots \mu_{X_n}) + \sum \left(\frac{\partial g}{\partial X_i} \right)_{\mu_{X_i}} [X_i - \mu_i] \qquad [3.5]$$

By applying second moment techniques to equation Eq. 3.4, the first-order approximation for the mean and variance are obtained. The respective expressions are as follows:

$$\mu_g = g(\mu_1, \mu_{2,} \ldots \mu_n) \qquad [3.6]$$

$$\sigma_g^2 = \sum_i \left(\frac{\partial g}{\partial X_i} \right)^2 \sigma_i^2 + \sum_i \sum_j \left(\frac{\partial g}{\partial X_i} \right) \left(\frac{\partial g}{\partial X_j} \right) \sigma_{ij} \qquad [3.7]$$

If the random variables are independent the covariance disappears and the resulting approximated variance is:

$$\sigma_g^2 = \sum_i \left(\frac{\partial g}{\partial X_i} \right)^2 \sigma_i^2 \qquad [3.8]$$

Where;

μ = mean of the random variable

μ_g = mean of the performance function g

σ_g^2 = Variance of the performance function

$\frac{\partial g}{\partial X_i}$ = Partial derivative of g, evaluated at the mean of the random variables

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For linear and normally distributed random variables, the performance function is also normally distributed. If the performance function is linear, the expressions for g, μ, μ_g and σ_g are as follows:

$$g(x) = a_0 + \sum_{i=1}^{n} a_i . x_i \qquad [3.9]$$

$$\mu_g = a_0 + \sum_{i=1}^{n} a_i . \mu_{x_i} \qquad [3.10]$$

$$\sigma_g = \sqrt{a_1^2 \sigma_{x_1}^2 + ...a_n^2 \sigma_{x_n}^2} \qquad [3.11]$$

in which a is a constant in a given performance function value.

For a simple performance function with only two variable R and Q representing the resistance and load effect respectively, the above expressions becomes;

$$g(x) = R\text{-}Q;\ \mu_g = \mu_R - \mu_L \text{ and } \sigma_g = \sqrt{\sigma_R^2 + \sigma_L^2} \qquad [3.12]$$

From these expressions, reliability index (β) is given by;

$$\beta = \frac{\mu_g}{\sigma_g} = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad [3.13]$$

For several random variables;

$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \mu_{xi}}{\sqrt{\sum_{i=1}^{n} \left(a_i \sigma_{x_i}\right)^2}} \qquad [3.14]$$

The geometric definition of β is illustrated in figure 3.3. From figure 3.3 it can be seen that β is a measure of safety margin in terms of the number of standard deviations separating the

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mean and the failure boundary. The distance from the mean (μ_g) to the failure boundary (g(x)=0) equals $\beta\sigma_g$. Therefore $\mu_g = \beta\sigma_g$ and

$$\beta = \frac{\mu_g}{\sigma_g} \qquad [3.15]$$

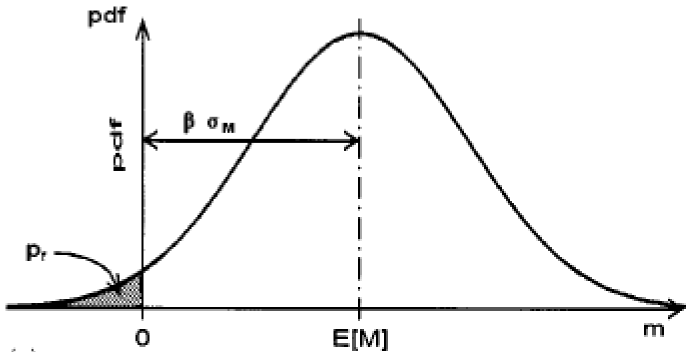

Figure 3.3: Geometrical definition of reliability index (After Christian, 2004)

If the two random variables R and Q are lognormally distributed, then the performance function is expressed as:

$$g = \ln(R) - \ln(Q) = \ln(R/Q) \qquad [3.16]$$

If R and Q are assumed to be mutually independent as is usually the case, the mean and standard deviation of g are expressed as follows:

$$\bar{g} = \ln\left[\frac{\bar{R}}{\bar{Q}}\sqrt{\frac{1+COV_Q^2}{1+COV_R^2}}\right] \qquad [3.17]$$

$$\sigma_g = \sqrt{\ln\left[\left(1+COV_Q^2\right)\left(1+COV_R^2\right)\right]} \qquad [3.18]$$

Where: $\bar{R}, \bar{Q}$ = mean values of R and Q, COV_R, COV_Q = coefficients of variation of R and Q.

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From the definition of reliability index as the ratio of mean of the performance function to its standard deviation;

$$\beta = \frac{\ln\left[\left(\overline{R}/\overline{Q}\right)\sqrt{1+COV_Q^2/\left(1+COV_R^2\right)}\right]}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right]}} \qquad [3.19]$$

$\overline{R}$ and $\overline{Q}$ can be expressed in terms of predicted resistance and load and their respective bias factors as follows:

$$\overline{R} = M_R R_n \text{ and } \overline{Q} = M_Q Q_n \qquad [3.20]$$

in which R_n, Q_n = nominal (predicted) resistance and load, M_R, M_Q = bias or model factors for the resistance and load respectively.

Subsisting Eq. 3.20 into Eq. 3.19 yields:

$$\beta = \frac{\ln\left[\left(M_R R_n / M_Q Q_n\right)\sqrt{1+COV_Q^2/\left(1+COV_R^2\right)}\right]}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right]}} \qquad [3.21]$$

R_n and Q_n can be related through the factor of safety (i.e. FS = R_n/Q_n). Then R_n = FS Q_n. Taking into account that Q_n comprises of dead load (Q_D) and live load (Q_L), $M_Q Q_n = M_{QD} Q_D + M_{QL} Q_L$ in which M_{QD} and M_{QL} are the bias factors for dead load and live load respectively. Also $R_n = FS\,(Q_D + Q_L)$ and Q_D and Q_L are assumed to be independent leading to:

$$COV_Q^2 = COV_{QD}^2 + COV_{QL}^2 \qquad [3.22]$$

As a result Eq. 3.21 can be written as follows:

$$\beta = \frac{\ln\left[\dfrac{M_R FS\left(Q_D + Q_L\right)}{M_{QD} Q_D + M_{QL} Q_L}\sqrt{\left(1+COV_{QD}^2 + COV_{QL}^2\right)/\left(1+COV_R^2\right)}\right]}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_{QD}^2 + COV_{QL}^2\right)\right]}} \qquad [3.23]$$

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Further more, the load combination can be expressed in terms of the dead load such that;

$$\left(Q_D + Q_L\right) = Q_D\left(1 + Q_L / Q_D\right) \qquad [3.24]$$

Eq. 3.24 implies dividing the dead load and the live load by the dead load. Applying this to Eq. 23, lead to:

$$\beta = \frac{\ln\left[\frac{M_R FS\left(1 + Q_L / Q_D\right)}{M_{QD} + M_{QL} Q_L / Q_D}\sqrt{\left(1 + COV_{QD}^2 + COV_{QL}^2\right) / \left(1 + COV_R^2\right)}\right]}{\sqrt{\ln\left[\left(1 + COV_R^2\right)\left(1 + COV_{QD}^2 + COV_{QL}^2\right)\right]}} \qquad [3.25]$$

Eq. 3.25 constitutes a closed form solution for reliability index. It is apparent from Eq. 3.25 the reliability index is a function of the safety factor FS, the live load to dead load ratio (Q_L/Q_D), the resistance statistics (M_R, COV_R), and the load statistics (M_{QD}, M_{QL}, COV_{QD}, COV_{QL}). This expression is particularly useful for computing reliability index of a given domain of structures whose load and resistance statistics have been gathered. In this study the expression was used to compute the reliability indices for pile foundations implied by the current practice.

Although the MVFOSM analysis provides a closed form solution for reliability index, it has been found that when the performance function is complex (non-linear), the calculated reliability index is subject to an invariance problem (i.e. the value of the reliability index depends on the specific form of the performance function. For instance, the method regards an expression such as g = 2x(x+y) as different to the expression g = $2x^2$ +2xy and would yield a different value of β for each expression (Smith, 1986). Also a significant error may be introduced by neglecting higher order terms. To overcome the deficiencies of this approach, the Advanced First-Order Second Moment (AFOSM) analysis is normally used to determine the reliability index.

3.2.2.2 Advanced first order second moment (AFOSM) analysis

The invariant problem was solved by Hasofer and Lind (1974) leading to a general procedure that can handle both linear and non-linear performance functions and any kind of variable distribution. The Hasofer and Lind approach has become the state of the art method

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in reliability analysis (Schneider, 1979). It is this approach that is generally referred to as the Advanced First-Order Second Moment method. With this approach, the Taylor series expansion of the performance function is evaluated at some point on the failure surface referred to as the design point instead of the mean values. The design point is regarded as the most likely failure point. However, the design point is generally not known in advance and therefore an iteration technique is used to solve for the reliability index.

The Hasofer-Lind approach requires the reliability problem to be transformed from the x-space of physical variables to a standard normal space (u-space). For a random variable X_1 with a mean μ_1 and standard deviation σ_1, then the corresponding reduced variable Z_1 is given by the expression:

$$Z_1 = \frac{X_1 - \mu_1}{\sigma_1} \qquad [3.26]$$

It is more convenient to work in terms of the standardised variables because properties of the standardised normal distribution are well documented and readily available in the form of tables. The tabulated properties of the standardised normal distribution can then be used to determine the probability that the reduced variable will lie between the desired limits. Generally, the standardised variables are dimensionless with a mean of zero and a standard deviation of one.

The failure surface is approximated by a linear surface which is a tangent to the initial failure surface at the point with the shortest distance from the origin. This point is termed the design point. The schematic illustration of the limit state function in both the x-space and u-space is shown in figure 3.4. In the reduced coordinates system, β represents the shortest distance from the origin of the reduced variable to the failure surface (Fig. 3.4).

For a linear limit state function with independent and normally distributed random variables, the AFOSM method gives an identical reliability index as the MVFOSM method. Considering a case with two random variables R and Q denoting the resistance and load respectively, the performance function G = R-Q in the reduced form becomes;

$$\sigma_R Z_R + \mu_R - \sigma_L Z_L - \mu_L = 0 \quad \text{or}$$

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$$\sigma_R Z_R - \sigma_L Z_L + (\mu_R - \mu_L) = 0 \qquad [3.27]$$

On close observation, Eq. 3.27 is similar to the Hessian form for representing equation of a straight line given by;

$$Ax + By + C = 0 \qquad [3.28]$$

where A = σ_R; x = Z_R; B = σ_L; y =Z_L; C = (μ_R- μ_L)

From the geometry of a straight line, the distance from the origin to the line Ax + By + C = 0, is given by;

$$d = \beta = \frac{C}{\sqrt{A^2 + B^2}} = \left(\frac{\mu_R - \mu_L}{\sqrt{\sigma_R^2 + \sigma_L^2}} \right) = \frac{\mu_g}{\sigma_g} \qquad [3.29]$$

This expression for β is the same as that obtained by using MVFOSM method. It follows that for several variables the expression for reliability index is the same as that given by Eq. 3.14.

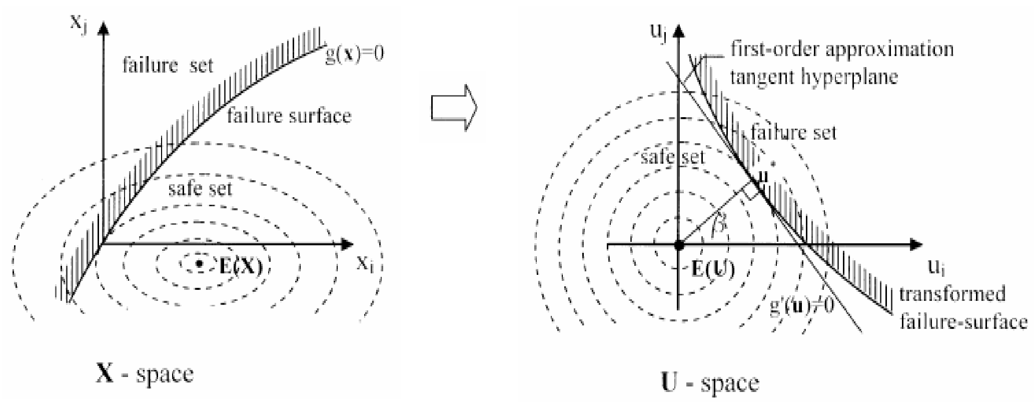

Figure 3.4: Performance function in x-space and u-space (After ISSC committee VI.1, 2006)

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3.2.2.2.1 Extension to non-linear performance functions and non-normal random variables

The Hasofer-Lind approach presented in the preceding section is valid for linear limit state functions with independent and normally distributed random variables (Schneider, 1996; Baecher and Christian, 2003). For other cases (non-linear performance function, non-normal random variables, and correlated variables), β can only be determined through an iteration process. In structural reliability analysis, the iteration process is carried out using the Rackwitz-Fiessler (1978) algorithm. The procedure has also been adopted for geotechnical reliability analysis and has been reported in numerous geotechnical reliability studies (e.g. Phoon, 1995; Paikowsky et al, 2004; Rahman et al, 2002, Kim et al, 2002, Smith 1986).

In practice, a reliability analysis problem may be composed of both normal variables and non-normal variables. Under such circumstances, the non-normal random variables must be transformed to equivalent normally distributed random variables. The parameters of the equivalent normal distribution are $\mu_{X_i}^N$ *and* $\sigma_{X_i}^N$. These parameters are estimated by imposing two conditions;

i. The cumulative density distribution of the actual function is equal to cumulative density function of the equivalent normal distribution at the design point.

$$F_i\left(X_i^*\right) = \Phi\left(\frac{X_i - \mu_{X_i}^N}{\sigma_{X_i}^N}\right) \qquad [3.30]$$

ii. The probability density function of the actual distribution is equal to the probability density function of the equivalent normal distribution at the design point.

$$f_i\left(X_i^*\right) = \phi\left(\frac{X_i - \mu_{X_i}^N}{\sigma_{X_i}^N}\right) \qquad [3.31]$$

By manipulating the above equations, expressions for parameters of the equivalent normal distribution ($\mu_{X_i}^N$ *and* $\sigma_{X_i}^N$) are obtained.

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$$\sigma_{X_i}^N = \frac{\phi\left(\Phi^{-1}\left[F_i\left(X_i^*\right)\right]\right)}{f_i\left(X_i^*\right)} \qquad [3.32]$$

$$\mu_{X_i}^N = X_i^* - \Phi^{-1}\left[F_i\left(X_i^*\right)\right]\sigma_{X_i}^N \qquad [3.33]$$

Where; F_i = the actual (non-normal) cumulative distribution function, f_i = the actual (non-normal) probability density function, Φ = cumulative distribution function of the standard normal variate, ϕ = probability density function of the standard normal variate and X_i^* = the design value of the random variable of interest.

The computational steps in determining β in conjunction with the Rackwitz-Fiessler procedure are as follows:

(a) Transform the original basic random variables to standardised normal variables using Eq. 3.26,
(b) Re-formulate the performance function in terms of the reduced variables,
(c) For non-normal random variables, determine the equivalent normal mean ($\mu_{X_i}^N$) and standard deviation ($\sigma_{X_i}^N$) from Eq. 3.33 and 3.32 respectively.
(d) Select the initial design point estimate. The mean values of the basic random variables are often used for the purpose.
(e) Compute the directional cosine α for each random variable as follows:

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial Z_i}\right)_*}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial g}{\partial Z_i}\right)_*^2}} \quad \text{but} \quad \frac{\partial g}{\partial Z_i} = \frac{\partial g}{\partial X_i}\frac{\partial X_i}{\partial Z_i} = \frac{\partial g}{\partial X_i}\sigma_{X_i}$$

Therefore;

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial X_i}\right)_* \sigma_{X_i}}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial g}{\partial X_i}\sigma_{X_i}\right)_*^2}} \qquad [3.34]$$

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(f) Form an expression for the new design point estimate:

$$X_i^* = \mu_{X_i} - \alpha_i \beta \sigma_{X_i} \tag{3.35}$$

(g) Substitute Eq. 3.35 into the performance function given by $g\left(X_1^*, X_2^*, \ldots\ldots, X_n^*\right) = 0$ to yield $g\left[\left(\mu_{X_i} - \alpha_i \beta \sigma_{X_i}\right) \ldots\ldots \left(\mu_{X_n} - \alpha_n \beta \sigma_{X_n}\right)\right] = 0$ and solve for β using a root searching method or by trial and error.

(h) Using the β obtained in step (g), a new design point is obtained using Eq. 3.35,

(i) Repeat steps (a) to (g) until convergence of β is achieved.

3.2.2.2.2 Extension to correlated random variables

In many practical applications, some of the random variables may be correlated. The presence of correlation between some variables can have a significant impact on the calculated reliability index (Nowak and Collins 2000). Therefore it is important to take the correlation into account. The classical approach is to transform the original variables to a set of uncorrelated variables. For this purpose, transformation schemes such as the Rosenblatt transformation if the joint probability distribution can be completely described or the Nataf transformation if only marginal probability distributions and correlation data are available. Both these transformations have been discussed in details in many structural reliability text books including Melchers (1999). However, these transformation approaches make reliability computations to become cumbersome. To improve the computational efficiency a General First Order Second Moment (GFOSM) method was developed by Ker-Fox.(2002). The approach starts by establishing a general direction cosine for correlated basic variables given by:

$$\alpha_i = \frac{\sum_{j=1}^{n} \left(\frac{\partial g}{dX_j}\right)_* \rho_{i_j} \sigma_{x_j}}{\sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} \left(\frac{\partial g}{\partial X_i}\right)_* \rho_{i_j} \sigma_{xi} \sigma_{xj}}} = \frac{K_i}{\sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial X_i}\right)_* K_i}} \tag{3.36}$$

The full iterative algorithm is as follows:

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(i) For the first iteration, the failure point is assumed to be equal to the mean value for the particular variable ($X_i^* = \mu_{X_i}$)

(ii) Determine the partial derivatives evaluated at the failure point in the standard normal space:

$$\left(\frac{\partial g}{\partial X_i^{'}}\right)_* = \left(\frac{\partial g}{\partial X_i}\right)_* \sigma_{X_i} \qquad [3.38]$$

(iii) The factor *K*, which improves computational efficiency is introduced:

$$K_i = \sum_{j=i}^{n} \left(\frac{\partial g}{\partial X_i^{'}}\right)_* \rho_{ij} \qquad [3.39]$$

(iv) Determine the variance of *g* as function of K_i factors:

$$Var[g(X)] = \sum K_i \left(\frac{\partial g}{\partial X_i^{'}}\right)_* \qquad [3.40]$$

(v) Calculate the direction cosine as a function of K_i

$$\alpha_i = \frac{K_i}{\sqrt{\mathrm{var}[g(X)]}} \qquad [3.41]$$

(vi) The failure point coordinate is found by substituting $X_i^* = \mu_{X_i} - \alpha_i \beta \sigma_{X_i}$ into the limit state function $g(X_i^*) = 0$ and solving for the unknown β.

(vii) Setting the assumed failure point in (i), equal to the failure point calculated in (vi), repeat steps until convergence is achieved for β.

3.2.2.3 Computation of *β* using Excel

It is evident that in using existing algorithms the computations become cumbersome unless a computer programme is employed to facilitate the iterations. Accordingly simple and practical computational procedures have been developed by exploiting the nonlinear

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optimisation function in spreadsheets such as EXCEL or QUATTRO-PRO and other mathematical software such as MATHLAB or MATHCAD. In the geotechnical literature, the computation of β using the optimisation feature of Excel (i.e. Solver add-in) has been reported by Low (1996, 1997), Low and Tang (1997a, 1997b); Low and Phoon (2002); etc. Also in the current study, reliability indices implied by the current design practice were determined using Excel (Chapter 7, Chapter 8 and Chapter 9).

The spreadsheet approach is based on the definition of reliability as the shortest distance from the origin to a point on the failure surface. In the multidimensional space, the distance from the origin to the failure surface in the reduced coordinates is given by (Baecher and Christian, 2003):

$$d = \sqrt{z_1^2 + z_2^2 .....z_n^2} \qquad [3.42]$$

in which z is the standardised normal variable.

The problem reduces to the minimisation of Eq. 3.42 subject to the constraint that the performance function is equal to zero (i.e. g(z)=0). Accordingly Eq. 3.42 can be written as:

$$d_{\min} = \beta = \min \sqrt{z_1^2 + z_2^2 .....z_n^2} \qquad [3.43]$$

In matrix notation Eq. 3.4 3 can be written as:

$$\beta = \min \left(\mathbf{z}^{\mathrm{T}}\, \mathbf{z}\right)^{1/2} \qquad [3.44]$$

where $\mathbf{z}$ =($z_1, z_2, ...z_n$) and $\mathbf{z}^{\mathrm{T}}$ = transpose of the line matrix **z**

Eq. 3.44 can be set on a spreadsheet as the target cell together with other relevant information such as the limit state function (constraint cell) and the random variables which constitute the changing cells. Once all the relevant information have been set, the solver is invoked by minimising the target cell by adjusting the changing cells subject to the constraint that the limit state function is equal to zero. Typical spreadsheets for specific reliability problems are presented in Chapters 7 through Chapter 9 of this study.

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3.2.3 Relationship between P_f and β and interpretation

If the basic variables (e.g. R and Q) are normally distributed, then β is related to the probability of failure by the following expression:

$$P_f = \Phi(-\beta) \qquad [3.45]$$

where Φ = cumulative distribution function (CDF) of the standard normal distribution.

For lognormally distributed R and Q Rosenblueth and Esteva (1972) suggested the following approximate relationship:

$$P_f = 460e^{-4.3\beta} \quad \text{for} \quad 2 < \beta < 6 \qquad [3.46]$$

Making β the subject gives:

$$\beta = \frac{\ln(460/P_f)}{4.3} \; for \; 10^{-1} < P_f < 10^{-9} \qquad [3.47]$$

Values of P_f and β based on Eq. 3.47 are given in table 3.1.

Table 3-1: Relationship between probability of failure and reliability index for Lognormal distribution

Probability of failure (P_f)	Reliability index (β)
1 x 10^{-1}	1.96
1 x 10^{-2}	2.50
1 x 10^{-3}	3.03
1 x 10^{-4}	3.57
1 x 10^{-5}	4.10
1 x 10^{-6}	4.64
1 x 10^{-7}	5.17
1 x 10^{-8}	5.71
1 x 10^{-9}	6.24

After the reliability analysis, the end results (P_f or β) need to be interpreted. Some of the critical questions include: What is the meaning of the calculated probability of limit state violation? What does probability of failure mean? Can it be related to observed rates of

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failure for real situations? How can the knowledge of probability of failure help in achieving safer and more economic designs? Melchers (1999), asserts that these are important questions and ones about which a degree of controversy and disagreement still exists.

The difficulty surrounding the interpretation of the probability of failure stems from the controversy of the meaning of the term probability. Currently the debate over the interpretation of probability seems to have narrowed down to two contrasting alternative views (Baecher and Christian, 2003). One school of thought is that probability is the frequency of occurrence of some event in a long series of similar trials or observations. This is known as the frequentist definition and it implies that there is some underlying frequency with which things happen and that repeated trials or experiments will reveal it (Christian, 2004). This interpretation further suggests that probability is a property of the world and operates outside human manipulation (Baecher and Christian, 2003). An alternative meaning is that probability expresses a rational degree of belief. This alternative is known as the degree of belief or subjective probability interpretation. In contrast with the frequentist interpretation, this interpretation suggests that probability is in the mind of the individual and the role of the analyst is to elicit it.

In the context of engineering systems, the frequentist interpretation suggests that the calculated probability of failure is a long run failure frequency and is therefore the percentage of a large number of statistically similar items in the same conditions that would fail within the duration of the reference period. This interpretation seems to suite electronic and mechanical elements such as fuses, bulbs, transistors, etc as such components deteriorate during use, hence failure within the reference period is inevitable. Also such items are usually produced in larger numbers leading to an existence of an underlying population of nominally identical components, making it possible to interpret failure probabilities in terms of relative frequencies (Thoft-Christensen and Backer, 1982). In civil engineering the concept of repeated trials is meaningless as it is not feasible to produce infinite identical structures under similar conditions in order to establish what fraction would fail. On this basis, a frequentist interpretation is not appropriate for civil engineering structures. To further demonstrate the inappropriateness of frequentist approach, Thoft-Christensen and Backer (1982) assert that once a particular structure has been designed and constructed, the probability of failure becomes the probability of the fixed but unknown resistance will be exceeded by the as yet un-sampled reference period extreme load effect. Based on this assertion, they concluded that:

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… the calculated probability of failure for a particular structure is not a unique property of the structure but a function of the analyst's lack of knowledge of the properties of the structure and the uncertain nature of the loading to which it will be subjected in the feature.

The above statement means that the calculated failure probability is not an inherent state of nature but a measure of the analyst's confidence in an uncertain outcome. Therefore the calculated failure probability reflects the evaluating engineer's degree of belief. In geotechnical design, soil parameters constitute the key input values of the reliability model. However, the selection of the design values of soil parameters is based on the analyst's personal experience and judgement. Thus the input values become of the personalistic nature and as it is the custom in geotechnical design, will vary from one engineer to the other. It follows that the failure probability obtained on the basis of personalistic input quantities reflects the analyst's rational degree of belief. Therefore in geotechnical engineering, the calculated probability should be interpreted as a degree of belief. In agreement Christian (2004) writes:

....in geotechnical engineering, the most important issues involve the engineer's degree of belief, especially when engineering judgement is employed.

Another issue pertaining to the calculated probability of failure is the fact that it does not take into account gross and other errors on design as well as construction variabilities. Also the information of the models on which reliability analyses are based on are not complete. In view of these limitations, the calculated failure probability should be considered as a nominal measure of failure and not an absolute failure (Melchers, 1999). Nonetheless, Melchers (1999) suggests that the calculated probability of failure can be accepted as a measure of safety of a structure if interpreted in the same sense that the factor of safety has been used (i.e. as a purely nominal measure and it does not consider gross errors). Therefore it is rational to use the calculated probability of failure for design to obtain structural sizes.

3.2.4 Choice between probability of failure and reliability index

The choice between using probability of failure or reliability index as a measure of design risk is a matter of preference. From the discussions in the previous section, the probability of failure appears to be more physically meaningful. However, in most cases it involves

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very small values which are generally difficult to handle and it also carries a negative connotation of ”failure” (Phoon, 1995). Therefore the reliability index is becoming more popular especially in code calibration.

3.3 Simplified reliability based design

To carry out reliability based design in accordance with the level II methods presented in the previous section requires that the calculated probability of failure or the reliability index is less than the target value.

$$P(R < Q) = P_T \qquad [3.48]$$

This entails the repeated use of reliability analysis to evaluate the probability of failure of the design situation at hand until the computed probability of failure is close to the target value. Given the complexity of the computations for failure probability or reliability index, this approach although rigorous might not be suitable for routine designs. Further more the approach requires formal training in reliability theory as well as probability theory. Therefore most geotechnical engineers would feel uncomfortable with performing reliability computations because of their lack of proficiency in probability theory (Whitman, 1984; Phoon, 1995). As mitigation to this problem, design methods satisfying reliability requirements without performing reliability computations have been developed. Such design methods are referred to as level 1 reliability design methods.

With the level 1 design approach, the appropriate degree of reliability is provided through the use of partial factors. Such partial factors are derived using level II reliability methods and are associated with the major sources of uncertainties in the basics variables. The process of assigning partial factors to resistances or loads is generally termed calibration. In this study only calibration for resistance factors was performed since predetermined load factors in the South African loading code were used.

3.3.1 Calibration of resistance factors

Calibration of partial factors can be carried out by any of the following approaches:

- Engineering judgement and experience

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- Fitting to existing design practice
- Reliability theory

Calibration by engineering judgment has been the main methods until 10-20 years ago (Faber and Sorenson 2002). The approach requires experience. For instance, poor past performance of foundations may necessitate the adjustment of the existing code until satisfactory results are achieved. In the process, code parameters for structures that perform satisfactorily are accepted, even if they are excessively conservative (FHWA HI-98-032, 2001). The main draw-back of this approach is that it results in non-uniform level of conservatism.

Calibration by fitting to an existing practice entails using partial factors that would result in the same minimum permissible dimensions of the element of interest (e.g. foundation) as that obtained by the current design method. Calibration by fitting is usually done after there has been a fundamental change in either design philosophy of design specification format (FHWA-NHI-05-052, 2005). A typical example of change in design philosophy is the conversion from the Working Stress Design (WSD) to Limit State Design (LSD) approach as it is currently the case in geotechnical engineering. One of the fundamental reasons for performing calibration by fitting as opposed to the formal reliability calibration is that detailed statistical data on loads and resistances would have yet to be gathered. The rationale behind this approach is that if the factors of safety used in the past or current practice have resulted in consistently successful designs, one will at least maintain that degree of success at the same cost as that required to meet previous practice.

The general equation for fitting to WSD is derived from the design equations used in working stress design and limit state design. The basic equation for the working stress design is given by:

$$\frac{R}{FS} = \sum Q \qquad [3.49]$$

Where; R= nominal resistance, FS= factor of safety, and Q =load.

From Eq. 3.49:

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$$R = FS\sum Q \quad [3.50]$$

For the limit state design approach, the basic design equation is given by:

$$\phi R = \sum \gamma_i Q \quad [3.51]$$

Where; ϕ = resistance factor, R = nominal resistance, γ_i = load factor, and Q =load.

From Eq. 3.51,

$$R = \frac{\sum \gamma_i Q}{\phi} \quad [3.52]$$

Setting the expressions for R equal (i.e. Eq. 3.50 and Eq. 3.52) yield:

$$FS\sum Q = \frac{\sum \gamma_i Q}{\phi} \quad [3.53]$$

Making ϕ the subject in Eq. 3.53 gives:

$$\phi = \frac{\sum \gamma_i Q}{FS\sum Q} \quad [3.54]$$

Considering the load (Q) to consist of dead (D_n) and live (L_n) loads, Eq. 3.54 becomes:

$$\phi = \frac{\gamma_D D_n + \gamma_L L_n}{FS(D_n + L_n)} \quad [3.55]$$

Where; D_n = nominal dead load, γ_D = partial factor for dead load, L_n = nominal live load, and γ_L = partial factor for live load.

Dividing the numerator and denominator on the right side of Eq. 3.55 by D_n gives:

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$$\phi = \frac{\gamma_D + \gamma_L (L_n / D_n)}{FS(1 + L_n / D_n)} \qquad [3.56]$$

This calibration method is still very much practiced world wide. It should be noted that the value of the partial factor obtained by Eq. 3.56 represent the resistance factor that need to be used in the limit state equation to obtain a factor of safety equal to that of the working stress design approach.

Partial factors derived by calibration by fitting and engineering judgment constitute non-probabilistic limit state design. The use of such partial factors in geotechnical engineering have been heavily criticised and the main criticisms were presented in Chapter 2. One of the criticisms is that since the application of such partial factors should result in approximately the same design dimensions as that from the traditional practice, these partial factors are just a rearrangement of the global factor of safety used in the working stress design (i.e. splitting the safety margin into component associated with loads and resistance). This has led to the question that if the resulting overall safety factors must be about the same as that at present so where is the improvement in economy and safety? Therefore the approach does not achieve more uniform margins of safety than the WSD approach it replaces.

In spite of the several limitations of the non-probabilistic limit state design, it is still the basis for most geotechnical codes. As an example figure 3.5 presents a diagrammatic overview of the various methods considered for calibration of partial factors in Eurocodes. However clause C4 (4) of EN 1990 states that the Eurocodes have been primarily based on method a (i.e. deterministic methods) while method c (level II reliability methods) have been used for further development of the code. In this regard Eurocode 7 is a deterministic limit state design code.

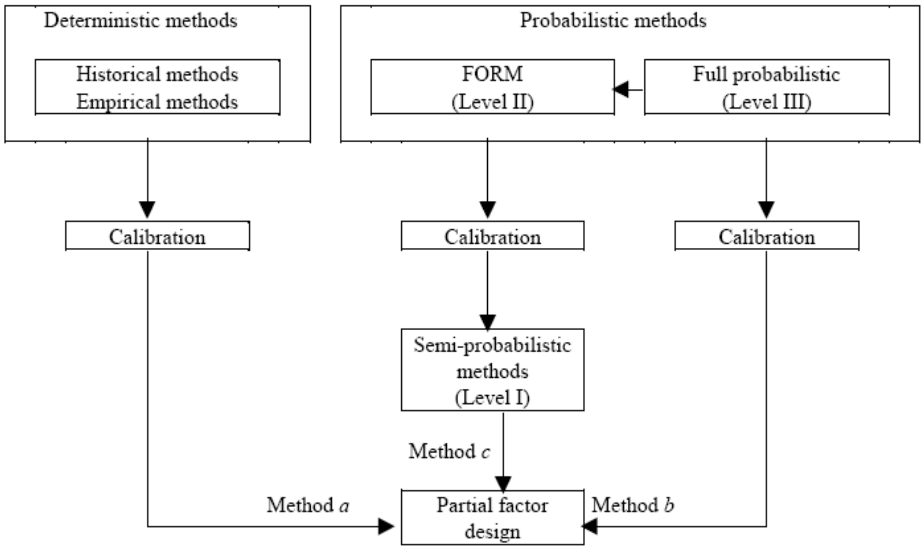

Figure 3.5: Overview of calibration methods in Eurocodes (After EN 1990, figure C1)

Calibration using reliability theory is the state-of-the-art approach that results in partial factors that achieve designs with a prescribed level of reliability. The approach involves the lengthy process of collecting the statistical data (mean, COV and distributions) for both the load effect and the resistances. This was the calibration approach employed in this study and will be discussed further in the subsequent section. If the partial factors were derived using reliability theory, the limit state design is then termed probabilistic limit state design or reliability based design. The design formats for both deterministic limit state design and probabilistic limit state design are identical:

$$\frac{R_k}{\gamma_R} = \sum \gamma_i Q_i \qquad [3.57]$$

in which R_k = characteristic resistance, γ_R = partial resistance factor, $\sum \gamma_i Q_i$ = total factored load.

The similarities of the design formats have led to the confusion in which some people associate limit state design with probabilistic design only. Accordingly some geotechnical codes have been wrongly classified as level I reliability based codes even though the partial factors were derived by engineering judgment and fitting. Eurocode 7 is a typical example of such codes. Kulhawy and Phoon (2002), argue that such codes are fundamentally

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incompatible with reliability based structural codes. The basis of their argument is that the geotechnical codes lack one or more of the following:

- The primary objective of reliability based- design is to achieve a minimum target reliability index across a specific domain. Therefore it requires deliberate and explicit choices to be made on target reliability index, scope of calibration domains, and representative design populating each domain. This is philosophically different from the objective of achieving designs comparable to WSD.
- The secondary object of RBD is to increase uniformity of reliability across the domain of interest, which is rarely emphasised and verified in geotechnical limit state design.
- Although soil variability is the predominate source of uncertainties, it is not quantified in a robust way and incorporated explicitly in the calibration process.
- Probabilistic load models compatible with the relevant structural codes are not spelled out clearly.
- Rigorous analysis using FORM is not used as the main tool to integrate loads, soil parameters, and calculation models in a realistic and self consistent way, both physically and probabilistically.
- No guidelines on the selection of characteristic soil parameters are usually provided. Consequently it is not clear how partial resistance factors will be affected by measurement techniques and correlation models used to derive the soil properties from laboratory tests or field measurements.

3.3.2 Reliability calibration

The limitations of deterministic calibration procedures serve as a motivation to consider reliability analysis as a necessary theoretical basis for calibration of resistance factors. As already alluded to, the partial factors in the reliability framework are determined on the basis of a prescribed target reliability index. Accordingly reliability calibration is considered to be a two step procedure: (i) to set target reliability index and (ii) to derive partial factors that ensures that the prescribed reliability level is attained.

3.3.2.1 Target reliability index

The selection of the target reliability index (β_T) is the first step in the calibration process. Several approaches for setting the target reliability index are available. The methods include:

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- Cost benefit analysis;
- Failure rates estimated from actual case histories;
- Value set by regulatory authorities for a given limit state;
- Range of beta values implied in the current design practice.

The most rational approach for establishing the target beta value is through cost-benefit analysis. Cost-benefit analysis entails study of the variation of the initial cost, maintenance costs, and the costs of expected failure. While the initial costs and maintenance costs can be easily and accurately determined, it is generally difficult to quantify the consequences of failure. Consequences of failure include the consideration of: (i) loss of human life (ii) environmental and social consequences (iii) economic consequences and (iv) the value of the loss of human life. Quantification of some of the factors listed above is not within the competency of engineers. Due to the difficulty of evaluation of cost of failure (e.g the value of the loss of human life), the approach is not practical.

The target probability of failure for a given structure can be established on the basis of failure rates estimated from actual case histories. Figure 3.6 presents a common guidance on empirical rates of failure for civil engineering facilities. For the case of foundations it can be seen that probability of failure range from 0.1% to 1%. However many authors (e.g. Phoon, 1995; Baecher and Christian, 2003; Christian, 2004) have cautioned that the probability of failure for constructed facilities is not solely a function of the design process uncertainties as is the case for the calculated failure probabilities. Therefore for comparison with calculated failure probabilities, the rate of failure in figure 3.6 should be adjusted by one order of magnitude downward (Phoon, 1995). If the suggested adjustments are effected, the probability of failure for foundations becomes 0.01 to 0.1% which correspond to reliability index values of between 3.1 and 3.7.

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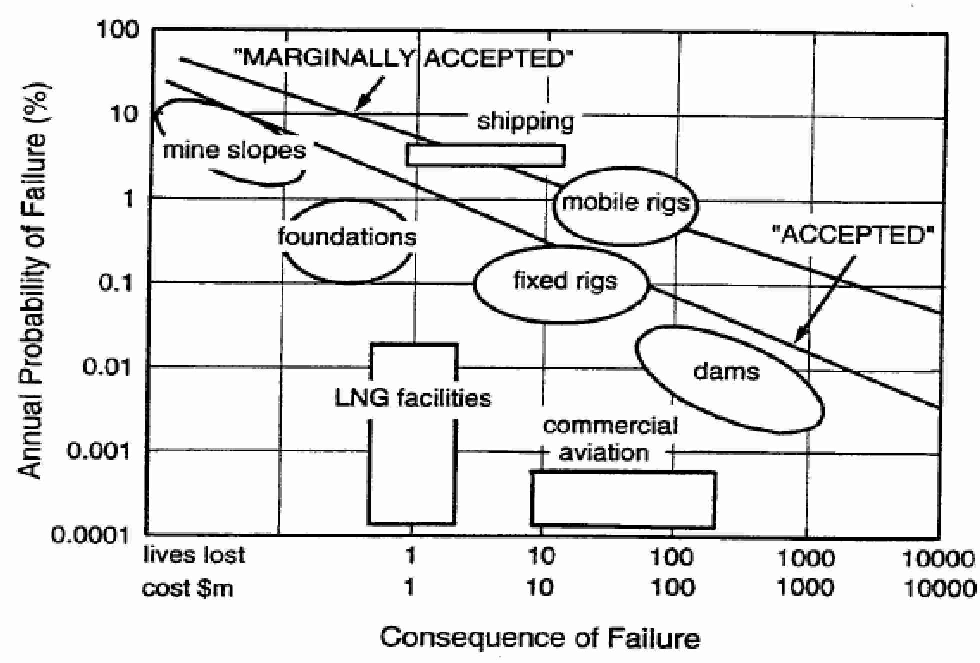

Figure 3.6: Empirical rates of failure of civil engineering facilities

(After FHWA HI-98-032, 2001)

Currently the most widely used selection criteria is the range of beta values implied in the past or current practice. The value of beta implied by the safety factor prescribed in past or current working stress design is used as the starting point to establish the target beta value. The main reason behind this is that if the safety factor from the current working stress design approach has been proven from experience to consistently produce safe designs, the safety level can be assumed to be adequate or even higher than what is needed. Furthermore, keeping the design methodology compatible with the existing experience base is consistent with the evolutionary nature of codes and standards that require changes to be made cautiously and deliberately (Phoon, 1995).

Although the margin of safety implied in the current practice is the leading factor in the selection of the target beta value, consideration is also given to the need for consistency with target beta value set by regulatory authorities for a given limit state. In South Africa, a standardised formulation for preparing limit states codes for different materials has been recommended by SABS 0160-1989 (under revision). This is to be achieved through a two-phase process:

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- Development of a loading code prescribing a set of partial load factors and a uniform system for defining load combinations which would be applicable to all materials;
- Subsequent calibration of partial materials and resistance factors appropriate to each limit state in each materials code.

The two-phase framework ensures that the materials and resistance factors are compatible with the load factors derived in the loading code. The compatibility between the materials/resistance factors and the load factors ensures that the minimum level of reliability of a certain class of structures and type of failure mechanism is independent of the type of material and the loading conditions. It is at the first phase (loading code) that the target reliability indexes for various classes of structures have been set. The draft revised loading code (SANS 10160-Draft) specifies three reliability classes as a function of consequences of failure. The link between consequence of failure, reliability class and respective values for the reliability index is presented in table 3.2. The reliability indexes presented in table 3.2 are for a 50 year design working life.

From table 3.2, class *RC* 2 represents building structures and other common structures (reference class). In accordance with SANS 10160, such class of structures have a 50-year design life. For this class of structures, the target reliability index has been set to 3.0. It follows from the principle of the two-phase calibration framework that geotechnical design should also be based on the same target reliability index (β_T =3). However overseas experiences have shown that basing geotechnical design on similar reliability index as the superstructure does not always yield satisfactory results. A typical example was the second edition of the Ontario Highway Bridge Design code (1993). The code was developed based on a reliability index of 3.5 for superstructure elements. The results of using a similar reliability index in geotechnical engineering were not encouraging since the foundation elements generally became larger and thus leading to more conservative designs. Therefore to avoid a similar situation, the target beta of 3 set by the loading code need to be compared with the theoretical value implicit in the current working stress design. For pile foundations, which is the focus of this study, reliability indices implicit in the current practice are presented in Chapter 7.

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Table 3-2: Reliability classification as a function of consequences of failure (from draft SANS 10160)

Class	Consequences	Examples	β
RC3	**High** for loss of human life, OR **Very great** for economic, social or environmental consequences	Grandstands, public buildings where consequences of failure are high (e.g. concert hall)	3.5
RC2	**Medium** for loss of human life, economic, social OR **considerable** for environmental consequences	Residential and office buildings, public buildings where consequences of failure are medium (e.g. office building)	3.0
RC1	**Low** for loss of human life, economic, social OR **small or negligible** for environmental consequences	Agricultural buildings where people do not normally enter (e.g. storage buildings) greenhouses	2,5

Another consideration in the selection of a target beta value for a given limit state and structure component being designed is the redundancy inherent in the system. This begs the question: if the component fails, would failure of the system result, or would load redistribution to adjacent components occur? If load redistribution is possible, then the probability of failure of the entire structural system is reduced. The effects of redundancy are explored further in Chapter 8 in relation to the selection of target reliability index for pile foundation in South Africa.

3.3.2.2 Reliability Calibration methods

Conceptually, the determination of partial factors in the reliability framework is the reverse of the process for computing the reliability index. Therefore in principle the various methods used to compute β can also be used to derive partial factors. As was the case for the β values, the partial factors from the various approaches will differ. The methods general employed in the reliability calibration include:

a) Advanced first-order second moment approach (AFOSM)
b) Mean value first-order second moment approach (MVFOSM)
c) Approximate to mean value first-order second moment approach

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3.3.2.2.1 Advanced first-order second moment method

As already stated, the Advanced First-Order Second Moment entails linearising the performance function at the design point. In the context of partial factors calibration, this means finding the design point of design values of the basic random variables corresponding to the target β. Since the design point must be on the failure boundary, the performance function for a given target β is given by:

$$g\left(X_1^*, X_2^* \ldots\ldots X_n^*\right) = 0 \qquad [3.58]$$

in which X_i^* represents the design value of a given basic random variable.

Figure 3.7 shows the relationship between the design value, mean value and characteristic value of load and resistance variable. On the basis of this relationship, a partial factor is a scaling factor that converts the mean or characteristic value of a given random variable to the design value for the target β. In this regard the failure surface is as follows:

With respect to mean value:

$$g\left(\gamma_1 \mu_{X_1}, \gamma_2 \mu_{X_2}, \ldots., \gamma_n \mu_{X_n}\right) = 0 \qquad [3.59]$$

in which γ_i, μ_{X_i} = partial factor and mean value of a given basic random variable.

From Eq. 3.59,

$$X_i^* = \gamma_i \mu_{X_i} \text{ and } \gamma_i = \frac{X_i^*}{\mu_{X_i}} \qquad [3.60]$$

With respect to the characteristic value:

$$g\left(\gamma_1 X_{k_1}, \gamma_2 X_{k_1}, \ldots., \gamma_n X_{k_n}\right) = 0 \qquad [3.61]$$

in which γ_i, X_{k_I} = partial factor and characteristic value of a given basic random variable

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$$X_i^* = \gamma_i X_{k,i} \text{ and } \gamma_i = \frac{X_i^*}{X_{k_i}} \qquad [3.62]$$

Eq. 3.61 or Eq. 3.62 are generally referred to as the design point approach and constitutes the basis for partial factors in some of the structural design Eurocodes.

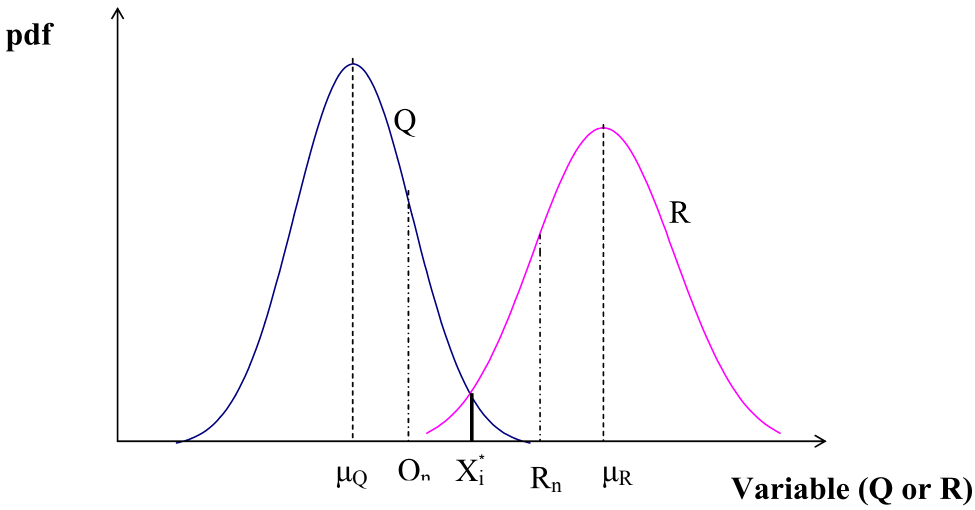

Figure 3.7: Relationship between design value, characteristic value and mean value

Alternatively the nonlinear optimisation function in spreadsheets described earlier in relation to computation of β can be used to determine the partial factors corresponding to the target β. This is the principal method adopted in this study. The details of such a spread sheet are presented in Chapter 8.

3.3.2.2.2 Mean value first-order second moment method (MVFOSM)

As previously noted the MVFOSM provides a closed form solution for reliability index. Like wise, there exists a closed form solution for derivation of partial resistance factors. The principle design equation for level 1 code is given by:

$$\phi R_n = \sum \gamma_i Q_i \qquad [3.63]$$

from which;

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$$\phi = \frac{\sum \gamma_i Q_i}{R_n} \qquad [3.64]$$

but from $\overline{R} = \lambda R_n$, $R_n = \frac{\overline{R}}{\lambda}$ [3.65]

Substituting equation 3.65 into equation 3.64 gives;

$$\phi = \frac{\lambda_R \sum \gamma_i Q_i}{\overline{R}} \qquad [3.66]$$

For a lognormal resistance distribution, a closed form solution for reliability index was given by Eq. 3.19 as:

$$\beta = \frac{\ln\left[(\overline{R}/\overline{Q})\sqrt{(1+COV_Q^2)/(1+COV_R^2}\right]}{\sqrt{\ln\left[(1+COV_R^2)(1+COV_Q^2\right]}}$$

From equation 3.19:

$$\overline{R} = \frac{\overline{Q}\exp\left\{\beta\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right]}\right\}}{\sqrt{\left(1+COV_Q^2\right)/\left(1+COV_R^2\right)}} \qquad [3.67]$$

Substituting $\overline{R}$ given by equation 3.67 into equation 3.66 and replacing beta by the target value gives:

$$\phi = \frac{\lambda_R\left(\sum \gamma_i Q_i\right)\sqrt{(1+COV_Q^2)/(1+COV_R^2)}}{\overline{Q}\exp\{\beta_T\sqrt{\ln\left[(1+COV_R^2)(1+COV_Q^2\right]}\}} \qquad [3.68]$$

When permanent and variable loads are considered separately equation 3.68 can be written as:

$$\phi = \frac{\lambda_R\left(\gamma_{QD}Q_D + \gamma_{QL}Q_L\right)\sqrt{(1+COV_{QD}^2+COV_{QL}^2)/(1+COV_R^2)}}{\overline{Q}\exp\{\beta_T\sqrt{\ln\left[(1+COV_R^2)(1+COV_Q^2\right]}\}} \qquad [3.69]$$

Diving the numerator and denominator by Q_D, equation 3.69 becomes:

$$\phi = \frac{\lambda_R \left(\gamma_{QD} + \gamma_{QL} \frac{Q_L}{Q_D} \right) \sqrt{\frac{1 + COV_{QD}^2 + COV_{QL}^2}{1 + COV_R^2}}}{(\lambda_{QD} + \lambda_{QL} \frac{Q_L}{Q_D}) \exp\left[\beta_T \sqrt{\ln(1 + COV_R^2)(1 + COV_{QD}^2 + COV_{QL}^2)} \right]} \quad [3.70]$$

in which γ_{QD} and γ_{QL} are the load factors for permanent and variable loads, λ_{QD} and λ_{QL} are the model factors for permanent and variable loads respectively.

From equation 3.70, it can be seen that the parameters needed to derive the partial factors are:

- λ_R and COV_R = Resistance statistics to be generated in Chapter 6 and 7 of this study
- λ_{QD} and COV_{QD} = Permanent action statistics deduced from current SA loading code
- λ_{QL} and COV_{QL}= Variable load statistics estimated from the literature and SA loading code
- γ_{QD} and γ_{QL} = Partial factors for permanent and variable loads respectively from the SA loading code.
- Q_L/Q_D = ratio of variable load to permanent load

3.3.2.2.3 Approximation to the MVFOSM approach

For lognormally distributed and statistically independent R and Q, the reliability index is again as given by equation 3.19:

$$\beta = \frac{\ln\left[(\overline{R}/\overline{Q}) \sqrt{(1 + COV_Q^2)/(1 + COV_R^2)} \right]}{\sqrt{\ln\left[(1 + COV_R^2)(1 + COV_Q^2 \right]}}$$

For small COV (less than 0.6) the above equation can be greatly simplified (Scott et.al, 2003) as follows:

- The quotient under the radical in the numerator will be close to unity,
- The function under the radical in the denominator can be approximated as follows:

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$$\ln\left[\left(1+COV_Q^2\right)\left(1+COV_R^2\right)\right] \approx COV_Q^2 + COV_R^2 \qquad [3.71]$$

According to MacGregor (1976), the error in equation 3.71 is less than 2% for COV of 0.3 and increasing to about 10% for COV of 0.6. Coefficients of variation for various geotechnical properties and resistances fall within this range and therefore this simplification is applicable to geotechnical applications. Based on the above simplifications, equation 3.19 can be rewritten as;

$$\beta = \frac{\ln\left(\overline{R}/\overline{Q}\right)}{\sqrt{COV_Q^2 + COV_R^2}} \qquad [3.72]$$

From which:

$$\ln\left(\overline{R}/\overline{Q}\right) = \beta\sqrt{COV_Q^2 + COV_R^2} \qquad [3.73]$$

Lind (1971) suggested a further linear approximation to the square root term as follows:

$$\sqrt{COV_Q^2 + COV_R^2} \approx \alpha\left(COV_R + COV_Q\right) \qquad [3.74]$$

where α is a separation coefficient or fitting factor having values between 0.707 and 1 depending on the value of the ratio COV_R/COV_Q (Scott et.al, 2003; Becker, 1996b).

Applying the simplifications, equation 3.72 can be expressed as:

$$\ln\left(\overline{R}/\overline{Q}\right) = \alpha\beta\left(COV_R + COV_Q\right) \qquad [3.74]$$

Taking antilog on both sides of equation 3.74 gives:

$$\overline{R}/\overline{Q} = \exp\left[\alpha\beta\left(COV_R + COV_Q\right)\right] \qquad [3.75]$$

Rearranging Eq. 3.75 gives:

$$\overline{R}\exp\left(-\alpha\beta\, COV_R\right) = \overline{Q}\exp\left(\alpha\beta\, COV_Q\right) \qquad [3.76]$$

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Introducing the bias factor and setting β to β_T, Eq. 3.76 becomes:

$$\lambda_R R_n \exp(-\alpha\beta_T \, COV_R) = \lambda_Q Q_n \exp(\alpha\beta_T \, COV_Q) \qquad [3.77]$$

Equation 3.77 is similar to the basic design equation:

$$\phi R_n = \gamma Q \qquad [3.78]$$

From the comparison of the two equations:

$$\phi = \lambda_R \exp(-\alpha\beta_T \, COV_R) \qquad [3.79]$$

$$\gamma = \lambda_Q \exp(\alpha\beta_T \, COV_Q) \qquad [3.80]$$

Eq. 3.79 and 3.80 allow for separate determination of resistance and load factors. This is an advantage given that for a particular geotechnical application, resistance and load statistics are not readily available and it would be a lengthy process to collect the necessary data.

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Chapter 4

PILE LOAD TESTS DATA COLLECTION AND PROCESSING

4.1 Introduction

The current state of the art in developing model uncertainty statistics in various fields including geotechnical engineering, involves comparing predicted performance with measured performance. Accordingly in this study, model uncertainty statistics were developed on the basis of comparing predicted pile capacity with measured pile capacities. The approach requires a substantial amount of local pile load tests results with the associated geotechnical data. Accordingly a database of static pile load tests along with the associated geotechnical data (soil profiles, field and laboratory test results) was compiled. The use of the local load tests data captures: (i) the distinct soil types for the geologic region of southern Africa and (ii) the local pile design and construction experience base. This chapter describes the methodology of collecting, compiling, and analyzing the pile load tests to derive the measured ultimate pile capacities. Although the collected geotechnical data is also presented, the derivation of geotechnical properties from such data is treated in Chapter 5.

4.2 Data collection

The load testing of piles is a well established practice and hence on medium and large piling contracts it is a contractual requirement. The main purpose of such testing is to verify the pile capacity assumed in the design. Such pile load tests reports were collected from piling companies. More than half of the data was collected from Franki Africa. Other companies that contributed pile test data include: Dura Piling, Gauteng piling, Stefanutti and Bressan, and Jones and Wagener. Although most of the projects on which the pile tests were carried out were within South Africa, there were a significant number of projects from other Southern African countries such as Botswana, Lesotho, Mozambique, Zambia and Swaziland.

The pile load test data collected from the various companies were carefully studied in order to evaluate their suitability for inclusion in the current study. For each load test, emphasis was placed on the completeness of the required information which includes: test pile size (length and diameter), proper record of the load-deflection data, and availability of

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subsurface exploration data for the site. Pile load tests with missing information were discarded.

4.3 Compilation of the data

The data on the selected pile load tests were further processed and presented in the form of tables. The information extracted from the reports included: project description, test pile characteristics, soil types and soil properties. The pile test data were divided into two broad groups of piles in cohesionless materials and piles in cohesive materials. The piles in each group were further subdivided into driven piles and bored piles. This resulted in the test piles data and the accompanying geotechnical data to be sorted into four pile classes. These classes are:

- Driven piles in non-cohesive materials (DNC)
- Bored piles in non-cohesive materials (BNC)
- Driven piles in cohesive materials (DC)
- Bored piles in cohesive materials (BC)

4.3.1 Test pile characteristics and projects description

Tables 4.1 through 4.4 present the test piles characteristics (pile type and pile size) and projects description. Close inspection of the table led to the following observations:

- Majority of the piles are made of concrete and only a few steel piles, suggesting that the principal pile material in South Africa is concrete.
- The pile lengths range from 3 to 27 m for DNC, 6 to 16.5 for BNC, 3.5 to 29.3m for DC and 4.5 to 24 m for BC. Generally the extremely long piles (e.g. 27 and 29 m) are steel piles.
- The concrete pile diameters vary from 330 to 610 mm for DNC, 360 to 520 for BNC, 250 to 750 mm for DC and 300 to 910 mm for BC. Except for one case, all the steel piles comprises of H-piles of 305 x 305 mm in size. The exception is a steel tube pile of 560 mm diameter.
- The pile types include Franki (expanded base) piles, Auger piles, Continuous Flight Auger (CFA) piles and steel piles. For driven piles the Franki pile is the most popular while for bored piles the Auger and CFA are more prevalent. Installation details of the

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various pile types are described the Guide to Practical Geotechnical Engineering in Southern Africa (i.e. Byrne et al, 1995)

- As already alluded to, there are a significant number of projects from other Southern African countries.

Table 4-1: Test pile characteristics and projects description for DNC

Case No.	Project description	Pile type	Shaft dia. (mm)	Base dia.(mm)	Length (m)
1	Dar es Salaam (Pile	Franki	520	760	8.0
2	Dar es Salaam (Pile	Franki	520	760	6.0
3	Dar es Salaam (Pile	Franki	520	760	11.0
4	Dar es Salaam (Pile	Franki	520	760	4.8
5	Dar es Salaam (Pile	Franki	520	760	6.0
6	Dar es Salaam (Pile	Franki	520	760	6.0
7	Sua Pan, Botswana (pile	Franki	330	750	6.0
8	Sasolburg (pile no.55)	Franki	410	650	5.8
9	Uni. of Botswana (Pile	Franki	410	650	3.0
10	University of Botswana	Franki	610	800	3.0
11	University of Botswana	Franki	610	800	3.0
12	Southern Freeway (Pile	Franki	520	800	15.6
13	Southern Freeway (Pile	Franki	611	760	6.4
14	Bank of Tanzania	Franki	520	840	6.0
15	Experimental piles in	Franki	406	470	6.5
16	Experimental piles in	SLUMP CAST	406	406	6.5
17	Experimental piles in	SLUMP CAST	406	405	7.8
18	J.C 1004 (Police station)	Franki	520	800	9.5
19	Jwaneng mine (pile	Franki	520	800	6.0
20	Jwaneng mine (pile	Franki	520	800	5.0
21	H. Smelter site 1;DCIP 1	Franki	410	600	7.0
22	H. Smelter site 1;DCIP 2	Franki	520	800	7.8
23	H. Smelter site 2;DCIP 1	Franki	410	600	9.2
24	H. Smelter site 2;DCIP 2	Franki	520	800	9.8
25	H. Smelter site 3;DCIP 1	Franki	520	800	12.5
26	H. Smelter site 3;DCIP 2	Franki	520	800	15.0
27	Saldanha steel project	Steel	305	305	25.0
28	Saldanha steel project	Steel	305	305	25.3
29	Saldanha steel project	Steel	305	305	27.0

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Table 4-2: Test pile characteristics and projects description for BNC

Case	Project description	Pile type	Shaft dia.	Base dia.	Length
30	Balfour Park (Pile No. C127)	Auger	430	430	8.0
31	Balfour Park (Pile No. C159)	Auger	600	750	9.0
32	Balfour Park (Pile No. 26)	Auger	750	600	11.0
33	Experimental piles in Durban	CFA	360	350	7.8
34	Malgate (pile A)	CFA	400	400	9.5
35	Malgate (pile B)	CFA	400	400	9.5
36	SAPREF converyor (test A)	CFA	400	400	8.0
37	SAPREF converyor (test B)	CFA	400	400	9.5
38	Durban West	CFA	400	400	9.0
39	Marine Parade	Auger	520	520	16.5
40	Consol Glass Clayville	Auger	430	430	11.5
41	Lobatse High Court	Auger	450	450	9.0
42	H. Smelter site 1;CFA 1	CFA	400	400	10.0
43	H. Smelter site 1;CFA 2	CFA	400	400	7.0
44	H. Smelter site 1;CFA 3	CFA	500	500	7.8
45	H. Smelter site 1;CFA 4	CFA	500	500	10.0
46	H. Smelter site 2;CFA 1	CFA	400	400	11.0
47	H. Smelter site 2;CFA 2	CFA	400	400	9.2
48	H. Smelter site 2;CFA 3	CFA	500	500	9.5
49	H. Smelter site 2;CFA 4	CFA	500	500	11.8
50	H. Smelter site 3;CFA 1	CFA	500	500	12.3
51	H. Smelter site 3;CFA 2	CFA	500	500	14.5
52	Saldanha steel project	Steel	305	305	13.0
53	Saldanha steel project	Steel	305	305	13.0
54	Saldanha steel project	Steel	305	305	13.0
55	Saldanha steel project	Steel	305	305	13.0
56	Mozal Smelter (Pile No. RS1	Franki	520	760	12.2
57	Mozal Smelter (Pile No. RS2	Franki	520	760	12.2
58	Mozal Smelter (Pile No. AN2	Franki	520	760	12.0
59	Mozal Smelter (Pile No. AS3	Franki	520	760	12.0
60	Mozal Smelter (Pile No.	Franki	520	760	15.0
61	Mozal Smelter (Pile No.	Franki	520	760	6.0
62	Mozal Smelter (Pile No.	Franki	520	760	8.0

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Table 4-3: Test pile characteristics and projects description for DC

Case No.	Project description	Pile type	Shaft dia. (mm)	Base dia.(mm)	Length (m)
63	Proclare – Claremont	Franki	520	760	13.0
64	Pumbing Staion, Vereeniging	Franki	300	300	8.5
65	Pumbing Staion, Vereeniging	Franki	450	450	8.5
66	Sua Pan, Botswana (pile no. 1)	Franki	330	750	21.5
67	Sua Pan, Botswana (pile no. 2)	Franki	330	750	21.5
68	Sua Pan, Botswana (pile no. 3)	Franki	450	900	21.5
69	Bank City	Franki	600	880	12.0
70	Sand Bypass (Pile No. 17)	Franki	410	880	3.5
71	Mopani mine, Zambia (Pile no.51)	Franki	610	880	10.8
72	Tembisa (pile No. P3-12)	Franki	610	800	9.5
73	Tembisa (pile No. A1-7)	Franki	610	800	9.5
74	Tembisa (pile No. A2-1)	Franki	610	800	9.5
75	Tembisa (pile No. RW-3)	Franki	610	800	9.5
76	Nedbank (Pile No. 36)	Steel	558.8	560	29.3
77	Sasol Pension (JC 1714)	Franki	520	620	15.0
78	Ons Tuis (Pile No.2C)	Franki	530	620	15.5
79	Tsoaing	Franki	610	800	7.7
80	CIMANGOLA MILL BASE (Pile No.	Franki	520	760	9.0
81	Pretoria college-JC1593 (trial	Franki	520	760	10.0
82	Pretoria college-JC1594 (trial	Franki	430	500	15.0
83	Pretoria college-JC1595 (Pile no.7)	Franki	530	760	10.0
84	Pretoria college-JC1595 (Pile	Franki	530	450	10.0
85	Pretoria college-JC1595 (Pile	Franki	430	760	10.0
86	Pretoria college-JC1595 (Pile	Franki	430	500	10.0
87	Pretoria college-JC1595 (Pile	Franki	430	500	10.0
88	Pretoria college-JC1595 (Pile	Franki	430	500	10.0
89	Pretoria college-JC1595 (Pile	Franki	430	500	10.2
90	Pretoria college-JC1595 (Pile	Franki	430	500	10.0
91	Krugersdorp Civic Centre	Franki	430	500	7.0
92	UB hostels - JC 28839 (Pile test	Franki	360	400	4.5
93	UB hostels - JC 28839 (Pile test	Franki	410	500	6.3
94	UB hostels - JC 28839 (Pile test	Franki	520	760	6.4
95	Tabong Church	Franki	360	400	4.5
96	New refactory - Botswana (Pile	Franki	360	400	5.5
97	New refactory - Botswana (Pile	Franki	260	300	5.4
98	`NGAMAKWE PUPUMA (RHS)	Franki	250	300	9.0
99	`NGAMAKWE PUPUMA (LHS)	Franki	250	300	9.0
100	Nkana Cobalt Plant (pile no.11)	Franki	520	760	6.4
101	Sasol Secunda (pile No.G39)	Franki	450	450	9.5
102	Sasol Secunda (pile No.J25)	Franki	750	750	11.5
103	Sasol Secunda (pile No.C7)	Franki	450	450	9.5
104	Naspers (pile No.32)	Franki	600	600	13.5
105	Mwanza	Franki	605	900	11.0
106	Durban (case Y)	Franki	615	1230	12.0
107	OK Bazaar (13 A)	Franki	520	760	7.5
108	OK Bazaar (13 B)	Franki	520	760	7.5
109	Misgund (Pile No.57)	Franki	520	760	6.5
110	Misgund (Pile No.116A)	Franki	520	760	7.0
111	Misgund (Pile No.92)	Franki	520	760	7.0
112	Misgund (Pile No.149)	Franki	520	760	7.0
113	Witbank (N0.2/3782)	Franki	520	760	8.0
114	Witbank (N0.3/3782)	Franki	520	760	8.0
115	Motloutse	Franki	430	600	12.0
116	ATTS	Franki	300	500	4.0
117	Hoog & Droog (test 1/223)	Franki	530	760	5.0
118	Hoog & Droog (test 2/223)	Franki	530	760	5.0
119	Hoog & Droog (test 1/224)	Franki	530	760	6.0
120	Hoog & Droog (test 2/224)	Franki	530	760	6.0
121	Univ. Bots.(B.C 1002)	Franki	520	760	5.5

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Table 4-4: Test pile characteristics and projects description for BC

Case No.	Project description	Pile type	Shaft dia. (mm)	Base dia.(mm)	Length (m)
122	Engen Skydeck	Auger	600	600	9.0
123	Sasol Alcohol (Pile No. 6246)	Auger	600	600	11.5
124	Sasol Alcohol (Pile No. 7066)	Auger	750	750	21.8
125	Serowe sports facility	Auger	350	350	17.3
126	Mwanza-Tanzania (pile test 4)	Auger	610	610	6.5
127	Richards Bay Heavy minerals (CI8)	Auger	600	600	6.5
128	Richards Bay Heavy minerals (PI8)	Franki	600	800	24.0
129	Goodwood flats (trial pile)	Auger	610	610	9.0
130	Goodwood flats (working pile)	Auger	610	610	7.0
131	Bank City block E	CFA	750	750	13.0
132	Jwaneng - JC1677	Auger	450	450	9.0
133	PortD'Afriqe	CFA	350	350	5.0
134	PortD'Afriqe	CFA	500	500	6.0
135	Chiselhuston (test 1)	CFA	600	600	6.0
136	Chiselhuston (test 2)	CFA	450	450	6.0
137	Chiselhuston (test 3)	CFA	300	300	6.0
138	Greenstone (Pile No. 138)	CFA	600	600	9.6
139	Greenstone (Pile No. 3033)	CFA	400	400	8.7
140	Greenstone (Pile No. 3299)	CFA	350	350	8.7
141	Moremi pipe Bridge (pile no.6)	CFA	410	410	11.0
142	Durban (case X)	Auger	615	615	12.0
143	Durban (case Z)	Auger	615	615	12.0
144	Goodwood flats	Auger	610	610	7.0
145	Goodwood flats	Auger	610	610	1.5
146	Northgate	Auger	500	500	7.8
147	Africana Museum (J/burg)	Auger	430	430	6.5
148	Mbabane GVT Offices	Franki	450	600	15.5
149	Rosebank	Auger	750	750	10.2
150	Witbank (Pile A/3782)	Auger	450	450	8.0
151	Witbank (Pile B/3782)	Auger	450	450	8.0
152	Witbank (Pile No.3A/3103)	Auger	450	450	8.0
153	Witbank (Pile No.3B/3103)	Auger	450	450	8.0
154	Witbank (Pile No.148A/3103)	Auger	450	450	8.0
155	Witbank (Pile No.35B/3103)	Auger	450	450	8.0
156	Witbank (Pile No.79B/3103)	Auger	450	450	8.0
157	Witbank (Pile No.34/3785)	Auger	450	450	8.0
158	Univ. Bots.(social science)	Auger	450	750	4.5
159	MOH Maseru	Auger	430	430	7.0
160	Old Mutual	Auger	550	550	6.0
161	Hoog & Droog (test 1/222)	Auger	910	910	12.0
162	Hoog & Droog (test 2/222)	Auger	910	910	9.0
163	Hoog & Droog (test 1/226)	Auger	910	910	9.0
164	Hoog & Droog (test 2/226)	Auger	910	910	9.0
165	Shashe Bridge	Auger	530	430	8.0
166	Shaft 10 (pile 61)	Auger	600	600	14.9
167	Shaft 10 (pile 149)	Auger	600	600	14.6
168	Shaft 10 (pile 155)	Auger	750	750	15.4
169	Shaft 10 (pile 229)	Auger	600	600	14.7
170	Shaft 10 (pile 236)	Auger	750	750	15.7
171	PAS 2005: Coega (pile no. T1a)	CFA	500	500	7.2
172	PAS 2005: Coega (pile no. T1b)	CFA	750	750	7.2
173	PAS 2005: Coega (pile no. T2a)	CFA	500	500	7.2
174	PAS 2005: Coega (pile no. T2b)	CFA	750	750	7.2

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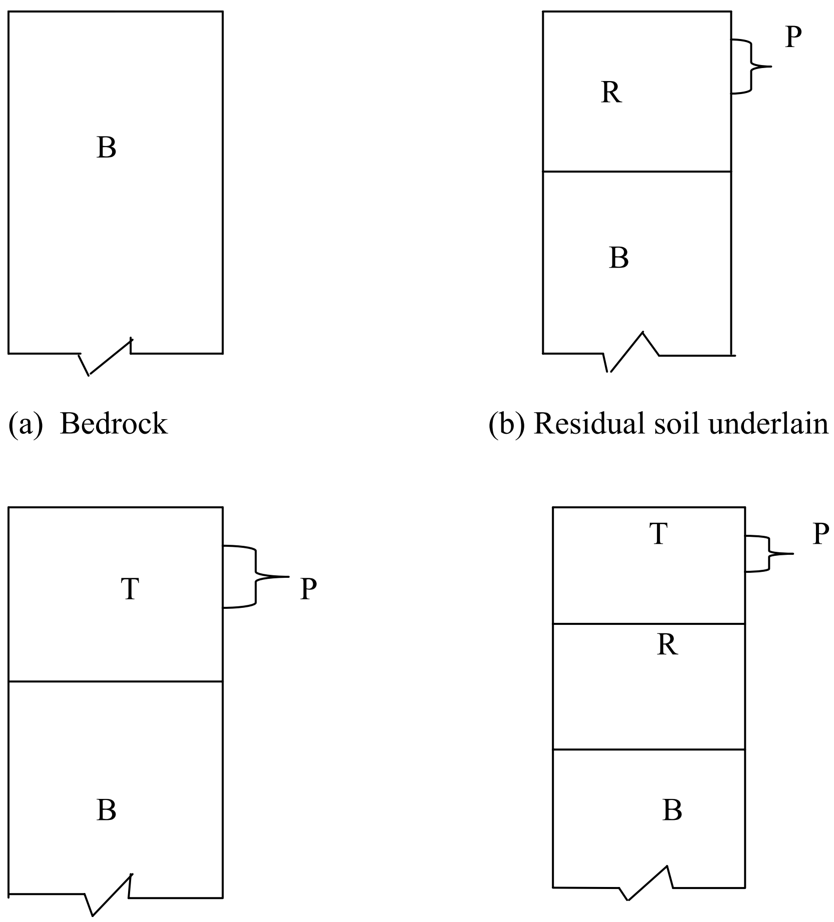
4.3.2 Associated geotechnical data

For each test pile (case 1 – 174), there was some accompanying soil data. A sample pile test record and geotechnical data are presented in appendix B. The soil data was mainly in the form of borehole log descriptions and standard penetration test (SPT) results. Table 4.5 through table 4.8 presents the associated geotechnical data. The following conclusions were drawn from the examination of these tables:

- The soil types conform to materials occurring in a typical Southern African soil profile as described by Jennings et al (1973). They fall into one of the following four natural categories: transported soil, residual soil, pedogenic material, and rock. Within a specific soil profile, a number of possible combinations of these materials can exist. Figure 4.1 illustrates the possible combinations.



(a) Bedrock

(b) Residual soil underlain by bedrock

(c) Transported materials underlain by bedrock

(d) Residual soil underlain by bedrock and overlain by transported materials

Figure 4.1: Possible combinations of materials in a typical profile (After Jennings et al 1973) where B = bedrock; R = residual soil; T = transported materials; P = Pedogenic material (may be present, absent or weakly developed)

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The specific soil type within the transported materials horizon varies in accordance with the transportation agency. The materials identified as sand, gravel, silt and clay in table 4.5 to 4.8 fall within this horizon. Where the depth of the transported materials horizon is too big, piles are founded within these materials.

- This was the case for test piles represented by tables 4.5 and 4.6. Within the transported materials horizon, Pedogenic materials can develop. In the current database, such materials have been described as calcrete (cases 52 -55) or silcrete (cases 66 – 68).
- In cohesive materials the piles are generally founded in the residual materials horizon. In South Africa residual soil is defined as a soil-like material derived from the in-situ weathering (both physical and/or chemical weathering) and decomposition of rock which has not been transported from its original location (Blight, 1994). The degree of weathering in a typical residual soil tends to decrease with depth (Blight and Brummer, 1980). The materials are described in terms of the rock from which they have been derived. Typical descriptions that have been used in tables 4.7 to 4.8 include: residual granite, weathered diabase, residual andesite, residual sandstone etc. It is evident from descriptions such as soft rock for base materials in some cases that weathered materials gradually merge into the unweathered rock. The internationally accepted typical weathering profile of residual soils is shown in figure 4.2.

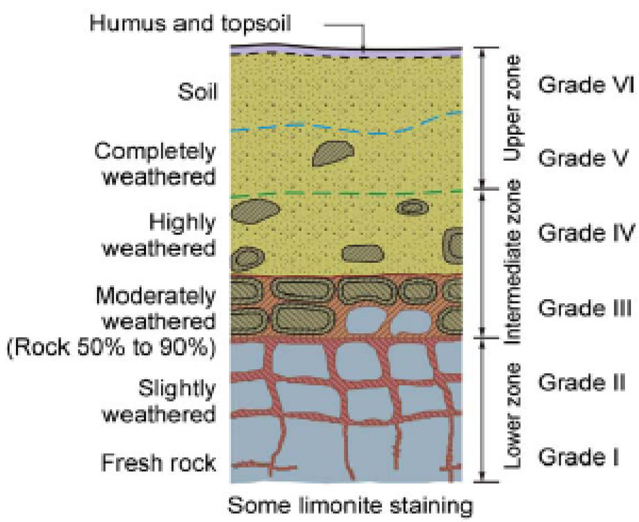

Figure 4.2: Typical weathering profile of residual soils (After Little 1969)

- The soil test results comprises mainly of SPT measurements and a few cone penetration test (CPT) results. This is an indication that the SPT is the most popular field soil test in Southern Africa.

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- In general, for a given case the SPT N-value for the base materials is higher than that for the shaft materials. This trend is marked in cohesive materials (tables 4.7 and 4.8), indicating that in residual materials, the pile passes through soil strata of increasing consistency and is founded on a rock consistency stratum (Lloyd and Gowan, 1975).

Table 4-5: Geotechnical data for DNC

Case No.	Soil type		SPT N-value		CPT	
	base	shaft	base	shaft	base	shaft
1	Sand	sand	32	19		
2	Sand	sand	12	10		
3	Sand	sand	33	23		
4	Sand	sand	30	16		
5	Sand	sand	33	16		
6	Sand	sand	33	16		
7	silty sand	Lined	27			
8	Sand	sand	27	21		
9	Sand	sand	40	25		
10	Sand	sand	40	25		
11	Sand	sand	40	25		
12	Sand	sand	12	15		
13	silty sand	sand	15	15		
14	Sand	Sand	25	22		
15	Sand	Sand			10	6
16	Sand	Sand			11	7.2
17	Sand	Sand			12.4	8
18	Sand	sand	24	16		
19	sand	sand	20	24		
20	Sand	sand	21	24		
21	Sand	Sand	21	16		
22	Sand	Sand	27	16		
23	Sand	Sand	21	13		
24	Sand	Sand	21	14		
25	Sand	Sand	21	14		
26	Sand	Sand	10	13		
27	very dense sand	sand	50	49		
28	very dense sand	sand	50	49		
29	very dense sand	sand	50	49		

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Table 4-6: Geotechnical data for BNC

Case No.	Soil type		SPT N-value		CPT	
	base	shaft	base	shaft	base	shaft
30	M. desnse sandy gravel	M. desnse sandy gravel	30	20		
31	gravel	gravel	30	20		
32	gravel	gravel	30	20		
33	Sand	Sand			11.2	5.4
34	Sand	Sand			9	5
35	Sand	Sand			9	5
36	Sand	Sand			9	5
37	Sand	Sand			9	5
38	Sand	Sand			9	5
39	Sand	Sand			15	11
40	Sand	Sand	25	17		
41	Dense sand	sand	25	15		
42	Sand	Sand	22	17		
43	Sand	Sand	19	16		
44	Sand	Sand	19	16		
45	Sand	Sand	22	17		
46	Sand	Sand	25	16		
47	Sand	Sand	24	13		
48	Sand	Sand	25	14		
49	Sand	Sand	26	17		
50	Sand	Sand	13	14		
51	Sand	Sand	13	13		
52	Calcareous medium sand	Calcareous medium sand	50	50		
53	Calcareous medium sand	Calcareous medium sand	100	100		
54	Calcareous medium sand	Calcareous medium sand	50	50		
55	Calcareous medium sand	Calcareous medium sand	100	100		
56	very dense sand	sand	11	18		
57	very dense sand	sand	12	17		
58	Residual sandstone	Residual sandstone	12	9		
59	very dense sand	sand	17	9		
60	Dense gravel	medium sand	17	14		
61	dense sand	medium sand	36	32		
62	very dense sand	medium sand	30	20		

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Table 4-7: Geotechnical data for DC

Case No.	Soi type		SPT N-value		CPT	
	base	shaft	base	shaft	base	shaft
63	weathered granite	Clay	35	17		
64	Clay	clay	30	15		
65	Clay	clay	30	15		
66	Silcrete	Lined	50	10		
67	Silcrete	Lined	60	10		
68	Silcrete	Lined	60	10		
69	Residual andesite	Residual andesite	60	15		
70	Residual andesite	Clayey sand	60	15		
71	Stiff Clay	Lined	40	0		
72	Residual syenite	Silty Clay	50	12		
73	Residual syenite	Silty Clay	50	12		
74	Residual syenite	Silty Clay	50	12		
75	Residual syenite	Silty Clay	50	12		
76	Mudstone	Clay	Ref.*	25		
77	Stiff silty Clay	sand alluvium	30	15		
78	Residual material	Clay	50	10		
79	Residual material	Residual material	70	20		
80	Residual material	clay	100	10		
81	V.soft rock	Residual material	100	35		
82	V.soft rock	Residual material	Ref.	35		
83	Residual material	Residual material	100	35		
84	Residual material	Residual material	100	35		
85	V.soft rock	Residual material	Ref.	35		
86	V.soft rock	Residual material	Ref.	35		
87	Residual material	Residual material	60	20		
88	Residual material	Residual material	70	20		
89	Residual material	Residual material	60	20		
90	Residual material	Residual material	60	20		
91	Weathered diabase	clay	60	20		
92	Residual granite	sandy clay alluvium	50	15		
93	Residual granite	sandy clay alluvium	50	15		
94	Residual granite	sandy clay alluvium	50	15		
95	Residual material	stifff clay	60	40		
96	Residual material	Clay alluvium	60	21		
97	Residual material	Clay alluvium	60	21		
98	Residual material	clay	100	25		
99	Residual material	clay	100	25		
100	Residual material	Residual material (socket)	60	40		
101	Residual material	Clay	60	15		
102	Residual material	Clay	100	15		
103	Residula material	Clay	60	15		
104	Residual material	Residual material	100	20		
105	V.stiff silty clay	clay	35	14		
106	Residual diabase	Clay	32	28		
107	Residual shale	Clay alluvium	90	35		
108	Residual Andesite	Clayey sand	80	15		
109	Residual Andesite	Clayey sand	80	15		
110	Residual Andesite	Clayey sand	80	15		
111	Residual Andesite	Clayey sand	80	15		
112	Residual Andesite	Clayey sand	80	15		
113	Residual dorelite	Clayey sand	100	17		
114	Residual dorelite	Clayey sand	100	17		
115	Medium hard rock	silty sand	100	20		
116	weathered granite	Clayey sand	40	15		
117	Residual diabase	Clay	100	23		
118	Residual diabase	Clay	100	23		
119	Residual diabase	silty clay	100	27		
120	Residual diabase	silty clay	100	27		
121	Residual granite	Clayey silty sand	35	21		

* SPT refusal (N > 100)

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Table 4-8: Geotechnical data for BC

Case No.	Soil type		SPT N-value		CPT	
	base	shaft	base	shaft	base	shaft
122	V. Soft rock	Residual material	Ref.*	20		
123	Very soft rock	Residual material	Ref.	80		
124	Very soft rock	Residual material	Ref.	80		
125	Residual material	Residual standstone	Ref.	60		
126	Residual material	sandy clay alluvium	100	15		
127	Very soft rock	sandy clay alluvium	Ref.	15		
128	Residual material	sandy clay alluvium	60	15		
129	decomposed shale	decomposed shale (socket)	100	20		
130	Weathered shale	Weathered shale (socket)	100	20		
131	Weathered rock	Residual material	100	20		
132	Weak rock, Shale	Weak rock, Shale	Ref.	Ref.		
133	Very soft rock	Very soft rock (socket)	Ref.	20		
134	Very soft rock	Very soft rock (socket)	Ref.	20		
135	Weathered diabase	Clay alluvium	20	10		
136	weathered granite	Residual granite	100	20		
137	V.soft rock	Residual materials	Ref.	20		
138	V.soft rock	silty clay	Ref.	20		
139	Residual granite	silty clay	100	20		
140	Residual granite	silty clay	60	20		
141	Residual material	sand alluvium	100	20		
142	Residual diabase	Clay	32	28		
143	Residual diabase	Clay	32	28		
144	Residual shale	Residual shale	90	70		
145	Residual shale	Residual shale	90	70		
146	Residual granite	silty sand	Ref.	17		
147	Residual Andesite	Clay alluvium	38	25		
148	weathered granite	Clay alluvium	40	13		
149	Residual granite	Clayey sand	Ref.	35		
150	Residual dorelite	Clayey sand	100	17		
151	Residual dorelite	Clayey sand	100	17		
152	Residual sandstone	Silty sand	Ref.	12		
153	Residual sandstone	Silty sand	Ref.	12		
154	Residual sandstone	Silty sand	Ref.	12		
155	Residual sandstone	Silty sand	Ref.	12		
156	Residual sandstone	Silty sand	Ref.	12		
157	Soft rock sandstone	Clayey sand	100	19		
158	Soft rock granite	Clayey granite	Ref.	20		
159	Very soft rock, mudrock	Clay silty sand	90	15		
160	Residual Andesite	Clayey sand	Ref.	15		
161	Soft rock lava	Clayey silt	100	28		
162	Soft rock lava	Clayey silt	100	28		
163	Soft rock lava	Clayey silt	Ref.	31		
164	Soft rock lava	Clayey silt	100	31		
165	Gneiss soft rock	silty sand	Ref.	12		
166	Soft rock, mudstone	silty clay	Ref.	20		
167	Soft rock, mudstone	silty clay	Ref.	20		
168	Soft rock, mudstone	silty clay	Ref.	20		
169	Soft rock, mudstone	silty clay	Ref.	20		
170	Soft rock, mudstone	silty clay	Ref.	20		
171	Soft rock sandstone	calcrete	Ref.	70		
172	Soft rock sandstone	calcrete	Ref.	77		
173	Very soft rock, siltstone	calcareous clay	100	60		
174	Very soft rock, siltstone	calcareous clay	100	60		

* SPT Refusal (N > 100)

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4.3.3 Pile load test database summary

After discarding the load test data with insufficient information, 172 cases were selected for the study. Out of this, 60 cases comprise of piles in cohesionless materials while 112 cases consist of piles in cohesive materials. The 60 cases of piles in cohesionless materials are further subdivided as follows:

- 29 cases of driven piles
- 31 cases of bored piles

Conversely the 112 cases of piles in cohesive materials are further subdivided into:

- 53 cases of driven piles
- 59 cases of bored piles

4.4 Evaluation of ultimate pile capacity from available load tests

Once the pile load test data and the associated geotechnical data have been processed as described in the previous sections, the next step was to evaluate the ultimate capacities of all the selected test piles. In accordance with worldwide practice, the ultimate capacities were interpreted from the respective load-settlement curves.

4.4.1 The South African pile load test procedure

The most common form of load test in South Africa is the static compression test in which a load is gradually applied to the pile head while the deflection is monitored. Although in principle piles can also be tested using dynamic or semi-dynamic testing procedures, such procedures are not currently in use in Southern Africa (Byrne et al, 1995). Static load testing of piles can be performed on working piles or trial piles specifically installed for the purpose. With working piles, the maximum test load is limited to avoid damaging the pile. In South Africa, the maximum test load is normally limited to one and half the design load. The main reason for performing load test on a working pile is to check the actual performance of the pile against that specified in the contract documents. Secondary benefits include confirmation of pile design parameters and a check on the structural integrity of the piles. Conversely, trial piles are loaded to failure. Testing piles to failure provides more

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accurate and meaningful design data. A trial pile programme enables the installation of more than one pile type whose performance for a given site are desired. This assists in deciding which pile type provides the most economic solution. More over, the trial piles can be installed to different depths so as to establish the optimum founding level. However, the prohibitive cost of installing additional piles just for testing purposes outweighs their advantages. Therefore, except for very large contracts, most pile load testing is carried out on working piles. On very large contracts the substantial cost of trial piles testing can often be compensated for many times over through achieving economic designs (Byrne et al, 1995).

SABS 1200 F-1983: Standardised Specification for Civil Engineering Construction Part F-Piling constitutes the pile testing standard in South Africa. SABS 1200F outlines two compressive load test procedures. Theses are termed the British and the Danish procedure. Both procedures entail a series of test load cycles in which the pile is loaded in gradual increments and then unloaded in a similar manner. Nonetheless, Byrne et al, 1995, suggest that in South Africa most specifications call for the use of the British method or a variation thereof as the Danish procedure is very time consuming. The common procedure reflected in the pile test records is as follows:

- A first cycle of loading is started and it entails increasing the load in 25 percent increments up to the design load.
- The first cycle is then unloaded in 25 percent increments back to zero.
- The load kept at zero for a period of one hour after which the second cycle commences.
- The second cycle is also loaded in 25 percent increments up to a maximum load of one and half times the design load.
- The second cycle is then unloaded in 25 percent increments back to zero.

The intermediate load increments are maintained until two successive readings 30 minutes apart show that the head deflection has not changed by more 0.1 mm. Appendix B.2 shows a typical load-deflection curve pile in which the procedure described above is captured.

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4.4.2 Analysis and interpretation of load test results

For each test pile there was a load test results in the form of Appendix B 1. The load test results were further analysed by plotting the load versus the head deflection to produce load-deflection curves. Having plotted the load deflection curve, the major task was to determine the ultimate capacity. However, establishing the ultimate pile capacity from a load test is not a straight forward matter. The problem arises as to what constitutes failure or ultimate capacity of the test pile. Ideally, ultimate capacity of a pile would be defined as the point along the load-deflection curve where the test experiences continuous deflection at no increase in load (i.e plunging in case of an axial compression test). However, large movements are required for a pile to reach a plunging state. In most cases, a distinct plunging failure is not obtained in the test and therefore the ultimate capacity has to be estimated by some other methods.

Fellenius (2000) warns that without a proper definition, ultimate pile capacity interpretation becomes meaningless for cases where obvious plunging has not occurred. Ideally an ultimate capacity definition should be based on some mathematical rules and should result in repeatable values that are independent of scale effects and individuals' personal opinion (Fellenius, 1990; Prakash & Sharma, 1990). Research to develop proper failure criteria has been ongoing for decades. Consequently a considerable number of different failure criteria have been proposed in the geotechnical literature. Some of the methods developed are actually used in specifications and codes of practice around the world. Some of the failure criteria reported in the geotechnical literature are listed below:

- Terzaghi's 10% criterion (Terzaghi, 1942)
- Brinch-Hansen's 80% criterion (Hansen, 1963)
- Brinch-Hansen's 90% criterion (Hansen, 1963)
- Brinch-Hansen's parabolic construction (Hansen, 1963)
- Vander Veen (1953)
- DeBeer (1970)
- Fuller and Holly (1970)
- Buttler and Holly (1977)
- O'Neill and Reese (1999)
- Mazurkiewicz (1972)
- Davison's offset limit criterion (Davison, 1972)

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- Kyfor et al (1992)
- Texas A & M Method for drilled shafts (Barker et al. 1993)
- Chin 's extrapolation method (Chin, 1970, 1971)
- Fleming's method (1992)
- Decourt extrapolation (Decourt, 1999)

Most of these failure criteria are achieved after a substantial pile head movement. Therefore such failure criteria are applicable to cases where the test pile is loaded near to failure. As noted earlier, testing piles to failure is only done on very big piling contracts and such contracts are generally very few. For the common medium to large contracts, only working piles are tested. Consequently the majority of the test piles in the database are working piles tested to a maximum load of one and half times the design load. This limits the movement to which the pile head is subjected and requires an extension to the load-settlement curve to determine the ultimate capacity.

Currently the estimation of ultimate capacity from proof tests (test on working piles) is only achieved through extrapolation procedures. Only a few extrapolation techniques have been reported in the literature. These include:

- Brinch-Hansen's 80% criterion (1963)
- Brinch-Hansen's polynomial construction (1963)
- Chin's extrapolation method (1970, 1971)
- Decourt extrapolation (1999)
- Fleming's method (1992)

Out of these extrapolation techniques, Chin's method is most popular worldwide. Paikowsky and Tolosko (1999) investigated extrapolation techniques for obtaining the ultimate capacity from proof tests. The extrapolation techniques investigated are the Chin's method and Brinch-Hansen's 80% criterion. These two approaches allow for fitting of a theoretical curve to the test data according to some mathematical relations. The techniques were examined through a database of 63 driven piles loaded to failure. The load-settlement data were truncated to 25%, 33%, 50%, 75% and 100 % of the actual failure load. The truncated data were extrapolated using the different techniques and the obtained ultimate capacity compared to the actual measured capacities. However, it was observed that the use of mathematical equations alone to extrapolate proof tests results in higher values of

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ultimate capacity than the measured values. Other researchers including Prakash and Sharma (1990) and Fellenius 1990, and Fleming 1992 have drawn attention to the fact that the Chin method appears to over-predict the ultimate capacity. Chin and Vail (1973) attributes the over-prediction to the fact that in the pile load test, it is the settlement at the pile head and not the displacement of the supporting soil that is measured. Therefore the measured displacement not only reflects the soil settlement but also the elastic shortening of pile. Further more, the measured displacement may include deformation of the pile due to any eccentricity.

To obtain more conservative estimates Paikowsky and Tolosko combined the use of Davisson's criterion and the extrapolation methods. Therefore the pile capacity estimation methods studied include: Chin's method, Brinch-Hansen's 80% criterion, Chin/Davisson's method, and Brinch-Hansen's 80% criterion/Davisson's method. That study showed that the ultimate capacity from Chin/Davisson method was the most conservative.

Building on the work by Paikowsky and Tolosko, Ooi et.al (2005) conducted a similar study on a database of bored shafts. In addition to the extrapolation methods used by Paikowsky and Tolosko, two more methods were investigated. The additional methods were: Brinch-Hansen's polynomial construction and Brinch-Hansen's polynomial construction/Davisson method. The study concluded that among the three extrapolation/ Davison methods, the Chin/Davisson procedure appears to be the most reliable with least scatter, confirming Paikowsky and Tolosko's findings

Based on the results of the above studies, it can be concluded that the Chin/Davisson procedure is superior to other methods in terms of consistency and conservatism. Therefore the Chin/Davisson procedure was adopted for the current study to determine ultimate capacity of proof tests results. The key steps of the procedure are as follows:

- Use Chin's method to fit a hyperbolic curve to the available load-settlement points of a proof test and extrapolate.
- Use Davisson's offset criterion to determine the pile capacity at the intersection of the extrapolated load-settlement curve and Davisson's offset line.

4.4.2.1 Fitting a hyperbolic curve using Chin's method

The method evolved from work by Kondner (1963) on hyperbolic stress-strain response of cohesive soils in a compressive triaxial test. Kondner demonstrated that the load-deformation behaviour of soils follow hyperbolic laws and therefore the nonlinear stress-strain curves for soils may be approximated by hyperbolae with a high degree of accuracy. The hyperbolic equation proposed by Kondner was of the form:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{a + b\varepsilon} \qquad [4.1]$$

Where σ_1 and σ_3 = the major and minor principal stresses respectively; $(\sigma_1 - \sigma_3)$ = the deviator stress; ε = axial strain; a and b = constants (hyperbolic parameters) whose values may be determined experimentally.

The resulting hyperbola is shown in figure 4.3. From figure 4.3, it can be seen that the constants a and b have physical meanings as follows: a is the reciprocal of the initial tangent modulus EI and b is the reciprocal of the asymptotic value of the stress, i.e. ultimate value of $(\sigma_1 - \sigma_3)$.

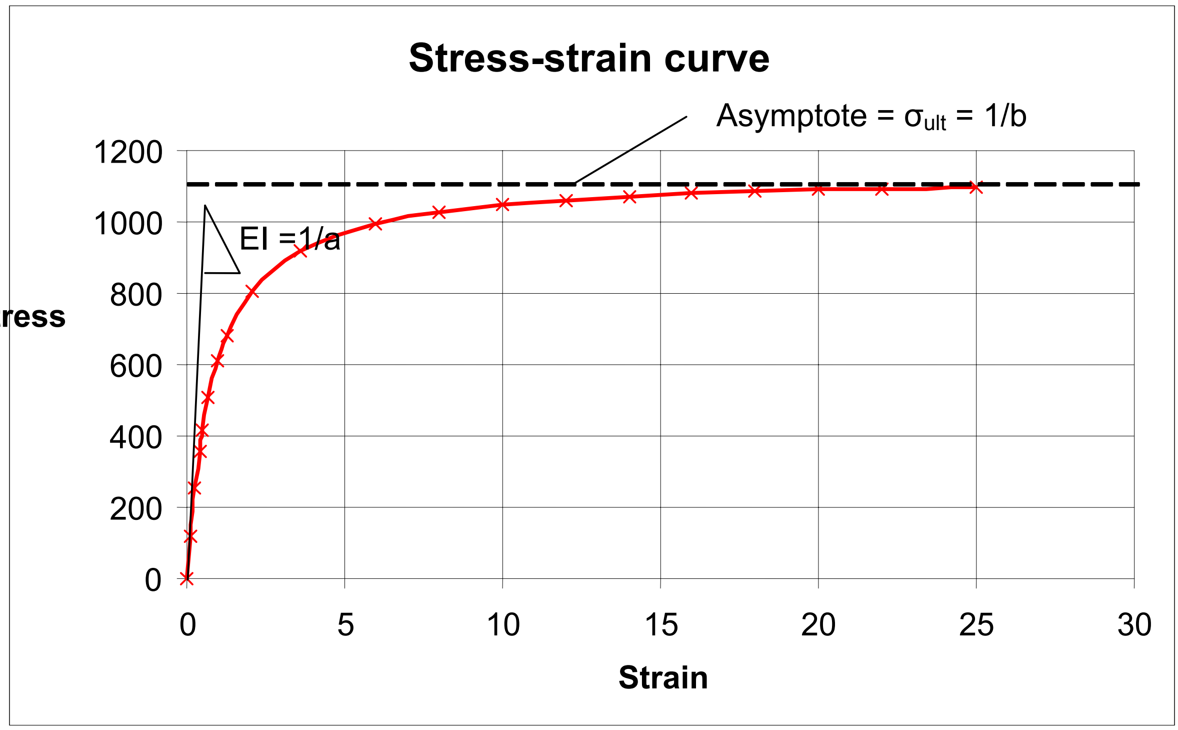

Figure 4.3: Hyperbolic representation of stress-strain relationship

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Kondner also showed that the values of the constants a and b may be determined most readily if Eq. 4.1 is transformed to represent a linear relationship as follows:

$$\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = a + b\varepsilon \qquad [4.2]$$

Figure 4.4 shows the transformed plot of $\frac{\varepsilon}{\sigma_1 - \sigma_2}$ as a function of ε. The hyperbolic parameters a and b now represent the intercept on ordinate $\varepsilon / (\sigma_1 - \sigma_3)$ and the slope of the straight line respectively. The inverse slope of the straight line is a measure of the position of the horizontal asymptote and corresponds to the ultimate stress.

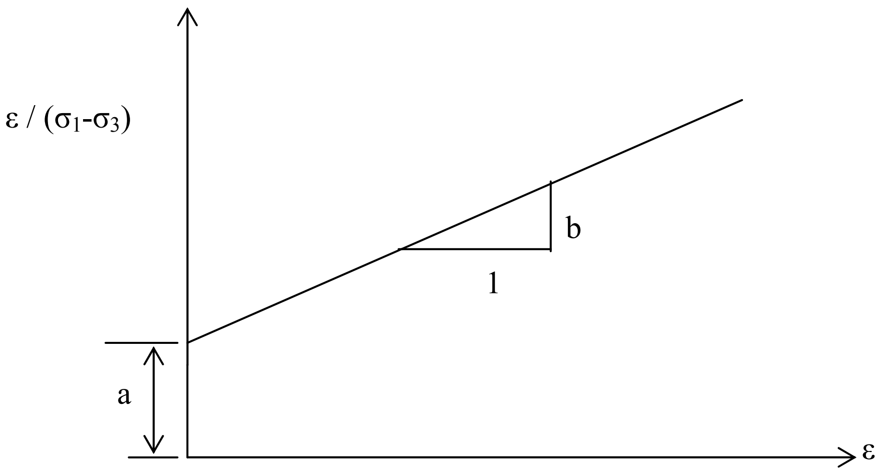

Figure 4.4: Transformed linear plot

Following the work by Kondner, Chin (1970, 1971) extended the hyperbolic model to load-settlement behaviour of pile foundations. In accordance with Chin's formulation, equations 4.1 and 4.2 become equations 4.3 and 4.4 respectively.

$$Q = \frac{S}{a + bS} \qquad [4.3]$$

$$\frac{S}{Q} = a + bS \qquad (4.\ 4]$$

Where Q is the applied load, S is the pile head movement, a and b are constant parameters as described in Kondner's formulation.

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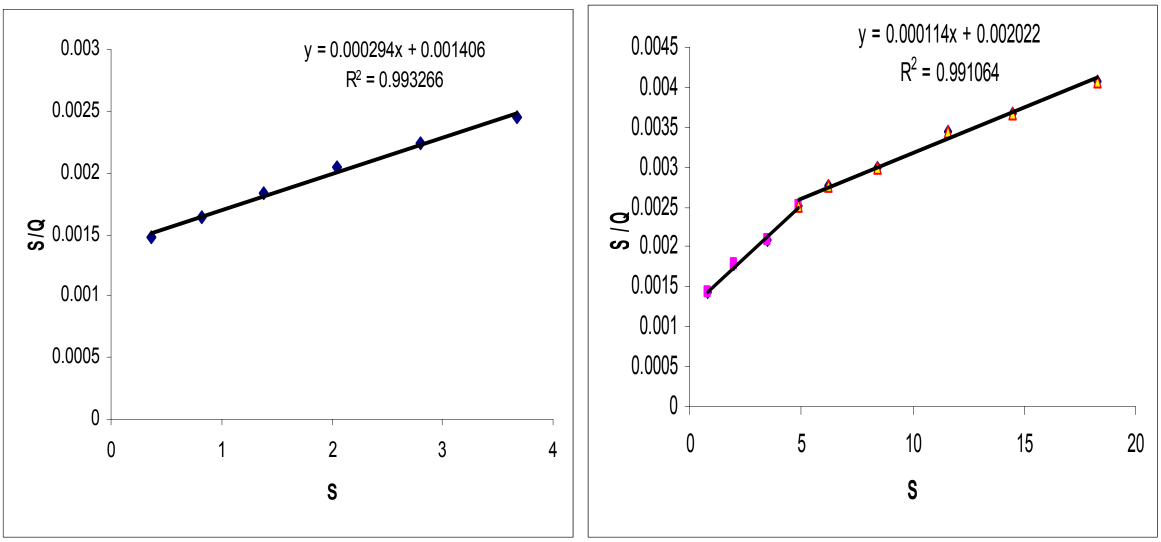
The validity of equation 4.4 with regard to pile load-settlement data was investigated through full scale pile tests and published load test results. It was found that the plot of S/Q versus S is linear and therefore the load –settlement behaviour of piles is hyperbolic and the inverse slope of this linear relationship is then the ultimate value of Q (i.e. the ultimate capacity of the test pile). This method of determining the ultimate capacity of a test pile is termed Chin's method.

For a given test pile data the step by step extrapolation procedure was as follows:

- First the hyperbolic parameters a and b were obtained as follows:
  - The transformed linear plot of S/Q versus S was plotted using the load-deflection data.
  - Best fit line was fitted to the plotted data.
  - From the equation of the best fit line, the hyperbolic parameters a and b were obtained (i.e. a = y-intercept and b= the slope of the line). Typical transformed plots are shown in figure 4.5. These are the two possible forms of the transformed plots and are interpreted in accordance with Chin and Vail (1973). The first case is a single straight line, denoting a situation in which the resistance is wholly provided by either the shaft materials only (friction piles) or the base materials only. The second case comprises a bilinear plot, denoting a situation in which the resistance is provided by both the shaft and base materials. In this case the first line represents shaft resistance while the second represent the ultimate capacity. In this case the hyperbolic parameters are obtained from the equation of the second line.
- The load-deflection curve (Q Vs S) was plotted. In the load-defection curve, the unload curves were left out.
- Using Eq. 4.4 in collaboration with the determined hyperbolic parameters, a theoretical load-deflection curve was generated and superimposed on the load deflection curve generated from the actual test data. The theoretical curve can be extrapolated to the desired magnitude of settlement to reach a plunging state. A typical theoretical load-deflection curve superimposed on the curve for actual test data is shown in figure 4.6.



(a) (b)

Figure 4.5: Transformed plots (a) shaft or base resistance and (b) Both shaft and base resistances.

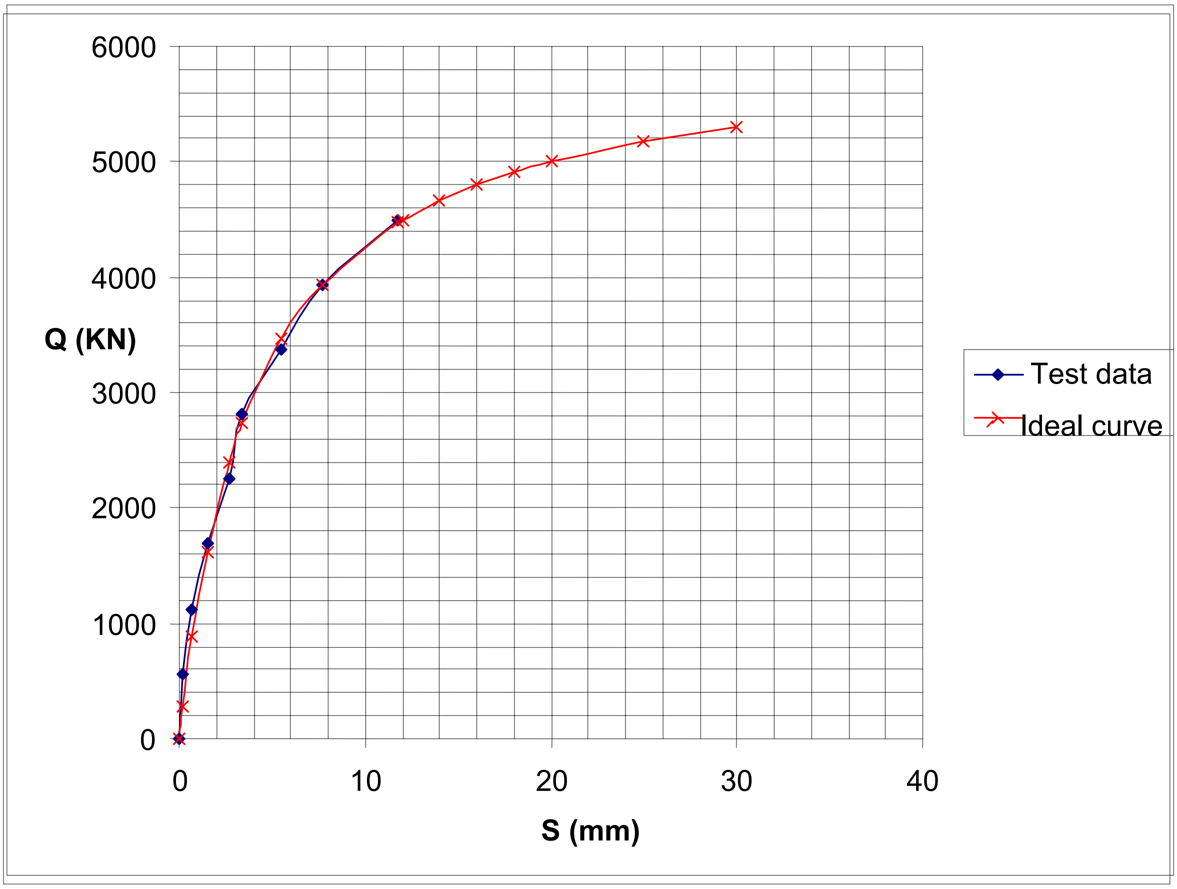


Figure 4.6: Extrapolated curve

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4.4.2.2 Davisson's failure criterion

With the Davisson failure criterion, the ultimate load is defined as the load corresponding to the intersection between the elastic deformation curve of the pile, shifted along the settlement axis by a value equal to 4mm plus the diameter of the pile (in millimetres) divided by 120, and the load-deflection curve from the load test data (Robert, 1997). The general equation for the Davisson's offset line is given by:

$$S = \frac{QL}{AE} + \frac{D}{120} + 4 \qquad [4.5]$$

Where S is the pile head deflection in mm, A is the cross-sectional area of the pile shaft, E is the Young's modulus of the pile material, L is the length of the pile, Q is the maximum applied load and D is the pile diameter or width in mm.

The step by step procedure is as follows:

a) Calculate the elastic compression of the pile ($\Delta = QL/AE$);
b) Draw the elastic line on the extrapolated load-settlement curve (the initial straight line portion of the curve);
c) Draw the Davisson's offset limit line parallel to the elastic line at a distance of :

$$\frac{QL}{AE} + \frac{D}{120} + 4$$

d) The ultimate capacity is then at the intersection of the Davisson's offset limit line with the extrapolated load- deflection curve (fig. 4.7).

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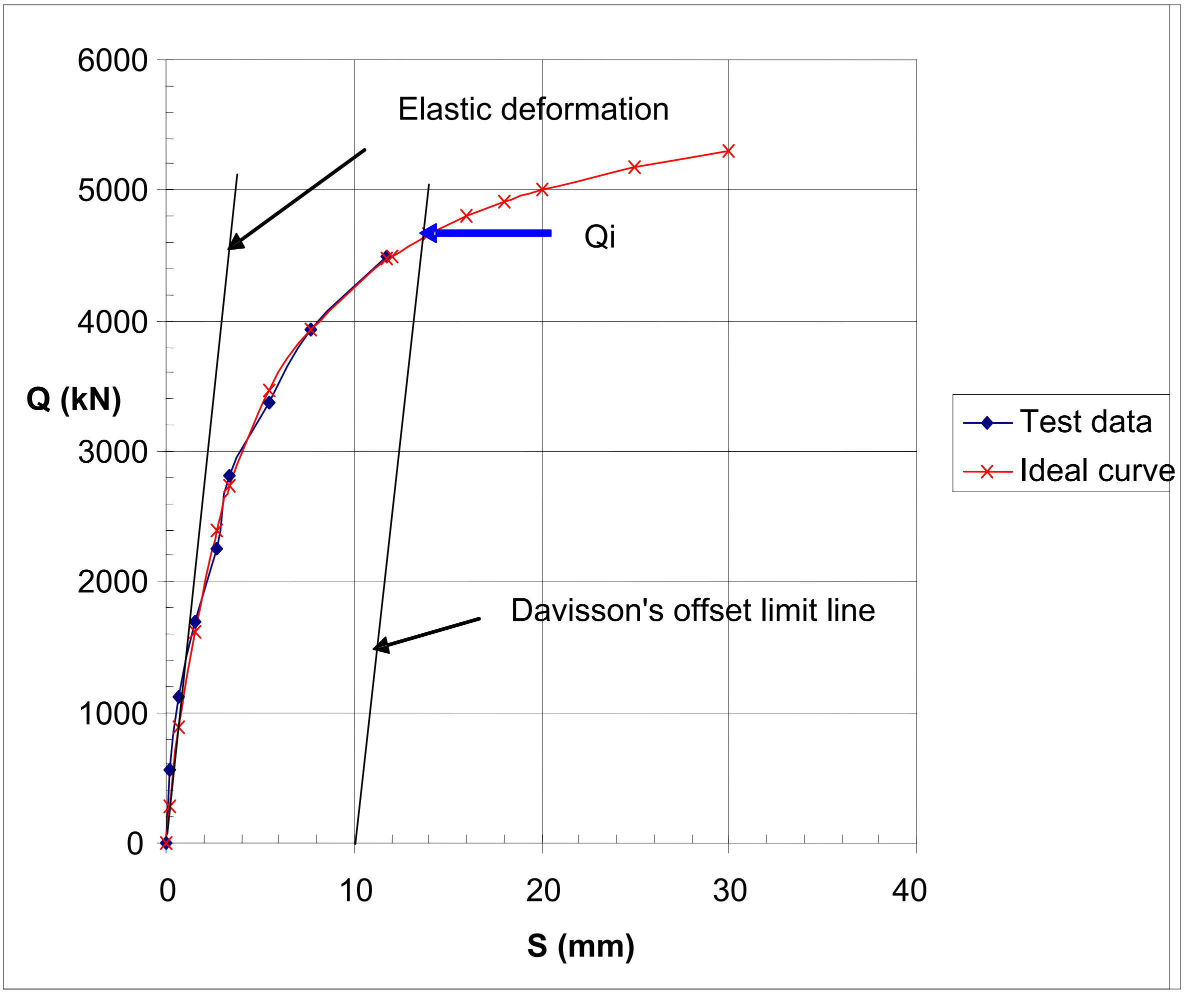

Figure 4.7: Davisson's failure criterion

4.4.2.3 Interpreted capacities and hyperbolic parameters obtained

The interpreted capacities and the associated hyperpolic parameters are presented in table 4.9 through table 4.12. For trial piles (total of 8 cases) only the Davisson failure criterion was applied while for working piles (total of 164 cases) the combied Chin-Davison approach was used to obtain the interpreted capacities. The hyperbolic parameters can be used to re-draw the respective load-deflection curves if required. The Q_i values range from 1310 – 6000 kN for DNC, 440 – 6000 kN for BNC, 490 – 7000 kN for DC and 510 – 13000 kN for DC. The Q_i values are influenced by both the pile size and the soil properties. The average values for Q_i are 2785 kN for DNC, 1992 kN for BNC, 2613 kN for DC and 2600 kN for BC. The average Q_i values for piles in cohesive materials are quite close. Conversely average Q_i values for piles in non-cohesive materials are appreciably different. The difference is attributed to the coincidental absence of smaller pile capacities (Q_i <1000 kN) from the database for BNC.

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Table 4-9: Interpreted capacities and hyperbolic parameters for DNC

Case No.	Q_i (kN)	a (mm/kN)	b
1	2850	3.81E-03	2.32E-04
2	2550	1.51E-03	3.00E-04
3	2700	1.06E-03	3.02E-04
4	1900	2.60E-03	3.60E-04
5	3650	2.43E-03	1.68E-04
6	2080	1.45E-03	3.86E-04
7	1350	1.91E-03	5.84E-04
8	2280	2.00E-03	3.00E-04
9	2200	2.10E-03	3.00E-04
10	4100	2.10E-03	1.40E-04
11	3880	1.54E-03	1.58E-04
12	1725	3.87E-04	5.47E-04
13	3100	1.50E-03	2.00E-04
14	3400	1.70E-03	2.00E-04
15	3075	2.28E-03	2.29E-04
16	1800	3.81E-03	4.16E-04
17	2400	4.62E-03	2.91E-04
18	2200	9.56E-04	3.86E-04
19	2175	8.85E-04	3.84E-04
20	2600	9.08E-04	3.03E-04
21	1480	3.00E-03	4.50E-04
22	2200	2.52E-03	3.22E-04
23	1310	3.20E-03	5.00E-04
24	3400	3.78E-03	1.47E-04
25	2800	2.59E-03	2.41E-04
26	1550	5.20E-03	4.00E-04
27	6000	4.66E-03	6.00E-05
28	4800	6.77E-04	1.66E-04
29	5220	1.26E-03	1.23E-04

Table 4-10: Interpreted capacities and hyperbolic parameters for BNC

Case No.	Q_i (kN)	a (mm/kN)	b
30	1375	4.00E-05	7.20E-04
31	4600	1.60E-04	2.04E-04
32	3000	1.35E-04	3.24E-04
33	1050	7.60E-03	5.78E-04
34	1357	1.03E-03	6.54E-04
35	1380	1.05E-03	6.69E-04
36	1050	7.64E-04	8.79E-04
37	1225	1.61E-03	6.83E-04
38	840	4.41E-03	8.63E-04
39	5600	6.99E-04	1.54E-04
40	1250	5.97E-04	7.52E-04
41	1500	1.39E-03	5.36E-04
42	970	4.00E-03	7.58E-04
43	540	5.35E-03	1.35E-03
44	600	1.82E-02	7.60E-04
45	1175	4.50E-03	5.00E-04
46	800	7.40E-03	9.00E-04
47	480	1.64E-02	1.20E-03
48	625	1.30E-02	7.00E-04
49	1025	1.02E-02	5.00E-04
50	1160	3.70E-03	6.00E-04
51	1500	2.60E-03	5.00E-04
52	2010	4.20E-03	2.00E-04
53	6000	6.18E-04	1.27E-04
54	1200	4.00E-03	5.00E-04
55	4650	2.02E-03	1.14E-04
56	1690	4.63E-04	1.22E-04
57	1620	7.00E-04	6.00E-04
58	1650	1.80E-03	5.00E-04
59	2100	1.60E-03	4.00E-04
60	2680	2.00E-03	3.00E-04
61	3650	1.26E-03	1.92E-04
62	5600	2.07E-03	1.40E-04

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Table 4-11: Interpreted capacities and hyperbolic parameters for DC

Case No.	Q_i (kN)	a (mm/Kn)	b
63	3290	2.41E-03	1.04E-04
64	680	2.03E-03	1.28E-03
65	1160	1.66E-03	7.39E-04
66	1800	1.50E-03	4.90E-04
67	2800	1.20E-03	3.10E-04
68	3400	1.00E-03	2.40E-04
69	3600	1.15E-03	2.13E-04
70	2050	2.90E-03	3.00E-04
71	2330	1.59E-03	3.09E-04
72	3150	2.10E-03	2.00E-04
73	2490	1.27E-03	3.19E-04
74	2300	1.20E-03	3.50E-04
75	2140	1.73E-03	3.56E-04
76	6080	9.00E-04	1.00E-04
77	2750	9.00E-04	3.00E-04
78	2180	9.00E-04	4.00E-04
79	4050	8.00E-04	2.00E-04
80	5200	8.02E-04	1.45E-04
81	5600	1.24E-03	1.19E-04
82	2780	1.40E-03	3.00E-04
83	4600	1.40E-03	1.50E-04
84	2380	2.06E-03	3.09E-04
85	7000	1.34E-03	7.51E-05
86	3300	1.60E-03	2.00E-04
87	1750	1.91E-03	4.34E-04
88	2300	4.05E-03	2.42E-04
89	1860	3.20E-03	3.80E-04
90	1500	1.75E-03	5.29E-04
91	1650	3.43E-04	5.73E-04
92	980	2.40E-03	8.00E-04
93	1320	3.40E-03	5.30E-04
94	3140	1.90E-03	2.00E-04
95	960	1.40E-03	9.00E-04
96	850	1.90E-03	1.00E-03
97	550	2.90E-03	1.50E-03
98	1280	2.80E-03	6.00E-04
99	1050	1.20E-03	8.44E-04
100	1800	9.00E-04	4.80E-04
101	1180	2.48E-03	6.30E-04
102	5100	3.60E-04	1.72E-04
103	1340	5.00E-04	7.00E-04
104	2880	2.41E-03	2.15E-04
105	2600	2.24E-03	2.67E-04
106	3700	1.31E-03	2.06E-04
107	3150	1.70E-03	2.19E-04
108	1140	1.37E-03	7.67E-04
109	3600	1.54E-03	1.88E-04
110	2550	1.41E-03	3.23E-04
111	4400	5.12E-04	1.92E-04
112	3600	4.47E-04	2.42E-04
113	2500	8.95E-04	3.39E-04
114	3090	7.26E-04	2.70E-04
115	1560	1.57E-03	5.25E-04
116	490	4.96E-03	1.56E-03
117	3470	1.54E-03	1.91E-04
118	2700	7.96E-04	3.09E-04
119	2200	8.06E-04	3.86E-04
120	2470	9.72E-04	3.31E-04
121	2120	2.72E-04	4.46E-04

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Table 4-12: Interpreted capacities and hyperbolic parameters for BC

Case No.	Q_i (kN)	a (mm/kN)	b
122	4800	1.00E-04	2.00E-04
123	4100	7.00E-04	2.00E-04
124	7100	5.00E-04	1.00E-04
125	760	7.00E-05	1.30E-03
126	2300	1.48E-03	3.47E-04
127	3100	1.60E-03	2.70E-04
128	3360	3.70E-03	1.90E-04
129	2650	2.00E-03	2.70E-04
130	1800	6.00E-04	5.00E-04
131	3700	8.25E-04	2.17E-04
132	2930	6.00E-04	3.00E-04
133	1700	1.50E-03	4.50E-04
134	1900	1.50E-03	4.00E-04
135	520	5.10E-03	1.20E-03
136	1175	7.00E-04	8.00E-04
137	1080	1.10E-03	8.00E-04
138	3500	8.35E-04	2.28E-04
139	1240	6.57E-04	7.50E-04
140	825	2.13E-03	1.02E-03
141	1450	1.00E-03	6.00E-04
142	3000	1.61E-03	2.50E-04
143	2450	1.60E-03	3.00E-04
144	1400	1.70E-03	5.00E-04
145	510	1.97E-03	1.82E-03
146	3600	7.42E-04	2.32E-04
147	1150	2.92E-03	6.42E-04
148	2310	1.29E-03	3.55E-04
149	8500	4.64E-04	9.30E-05
150	1230	6.83E-03	4.03E-04
151	1820	9.10E-04	4.65E-04
152	2580	5.88E-04	3.46E-04
153	2670	8.01E-04	3.21E-04
154	2790	8.57E-04	3.00E-04
155	2900	7.58E-04	2.92E-04
156	4200	9.19E-04	1.80E-04
157	2800	6.21E-04	3.11E-04
158	9600	4.40E-04	7.90E-05
159	1660	1.01E-03	5.15E-04
160	4800	3.04E-04	1.87E-04
161	7050	1.52E-04	1.31E-04
162	5900	1.67E-04	1.58E-04
163	9700	4.72E-04	7.39E-05
164	7000	2.54E-04	1.23E-04
165	1880	1.93E-03	4.01E-04
166	5430	6.40E-04	1.46E-04
167	3000	4.35E-04	3.06E-04
168	6250	3.08E-04	1.42E-04
169	4450	4.37E-04	1.96E-04
170	13000	4.13E-04	5.80E-05
171	3300	4.98E-04	2.62E-04
172	4810	1.83E-04	1.93E-04
173	2200	1.27E-03	3.70E-04
174	5300	7.61E-04	1.46E-04

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4.4.2.3 Accuracy of the Chin/Davisson approach

The Davison failure criterion is well known and widely used to estimate ultimate pile capacity for trials piles. However, besides the few research studies mentioned earlier, there is not much information in the literature on the use Chin/Davisson approach. Therefore the applicability of the trends observed by the few studies to different databases is questionable. To double check the reported results, the load-settlement data for the cases that were tested to failure were truncated to a maximum load of one and half times the design load. The truncated data were extrapolated using Eq. 4.4 and the ultimate capacities obtained on the basis of Davisson's failure criterion. The capacities were then compared to the ones obtained from full test data load-settlement curve using Davisson's failure criterion. Pile load test data from two trial piles at different sites were used in this exercise. These are pile cases 63 and 56. The particulars of these test piles and the associated geotechnical data are given in the relevant tables.

Pile case 63

The load-settlement data are presented in table 4.13. The last column of table 4.13 contains the results of S/Q required for drawing the transformed linear plot. The data was truncated at Q = 1500 KN. This is the load corresponding to the maximum load of one and half times the design load. The transformed plot for the truncated data is shown in figure 4.8 and the hyperbolic parameters a and b were obtained from the equation of the best fit line.

Using the hyperbolic parameters in conjunction with Eq. 4.4, a theoretical curve was fitted to the truncated test data. Figure 4.9 shows the resulting theoretical curve superimposed on the truncated test data curve. The value of the ultimate capacity interpreted from the extrapolated curve is 3290 kN. Using the full test data, the actual load-deflection curve was plotted (figure 4.10). From figure 4.10, the interpreted capacity is 3490 KN. The two values of the interpreted capacities are quite close. Further more, the extrapolated curve has been superimposed on the full test data curve in figure 4.11. It is evident from figure 4.11 that the extrapolated curve reasonably fits the actual full test data curve.

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Table 4-13: Load deflection data for case 63

Q	S	S/Q
0	0.0	
250	0.2	0.000800
500	0.5	0.001000
750	1.0	0.001333
1000	1.3	0.001300
1250	1.8	0.001440
1500	2.5	0.001667
1750	3.5	0.002000
2000	4.8	0.002400
2250	5.8	0.002578
2500	7.2	0.002880
2750	9.0	0.003273
3000	12.0	0.004000
3250	15.5	0.004769
3500	21.8	0.006229
3750	31.0	0.008267

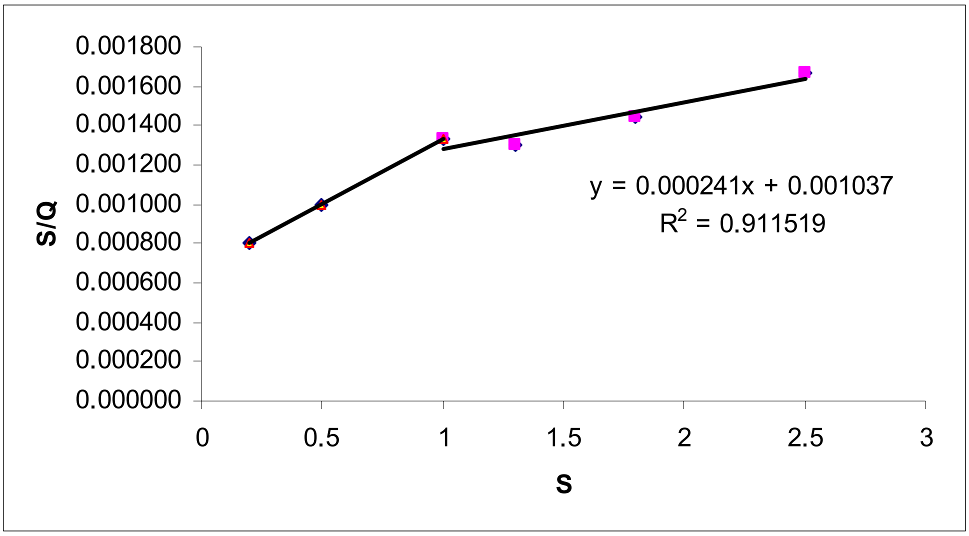

Figure 4.8: **Transformed plot for case 63**

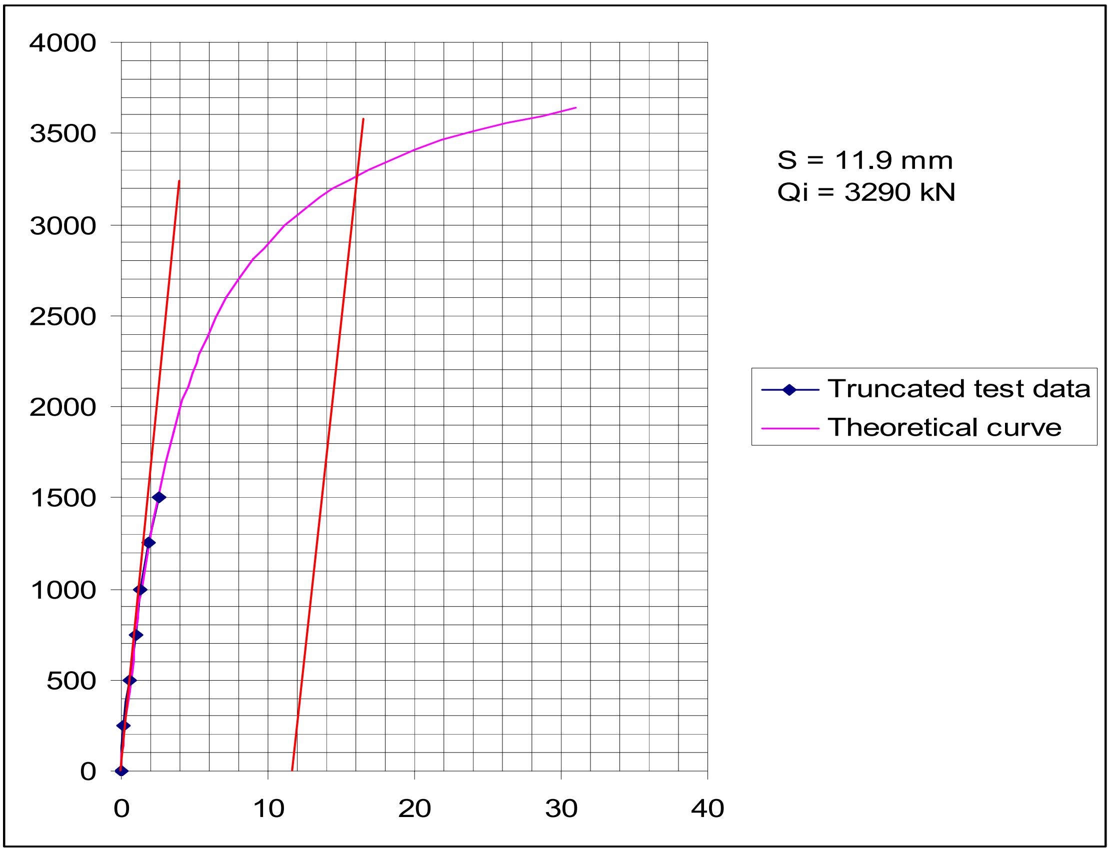

Figure 4.9: Extrapolated curve for case 63

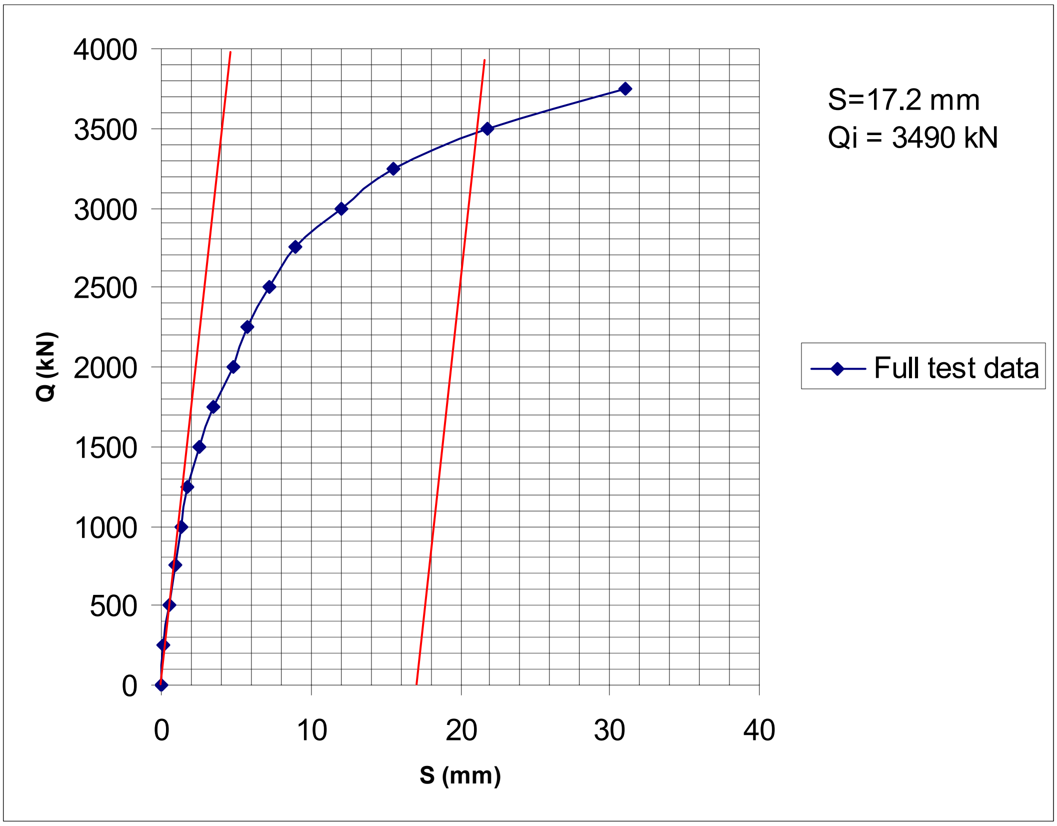

Figure 4.10: **Full test data curve for case 63**

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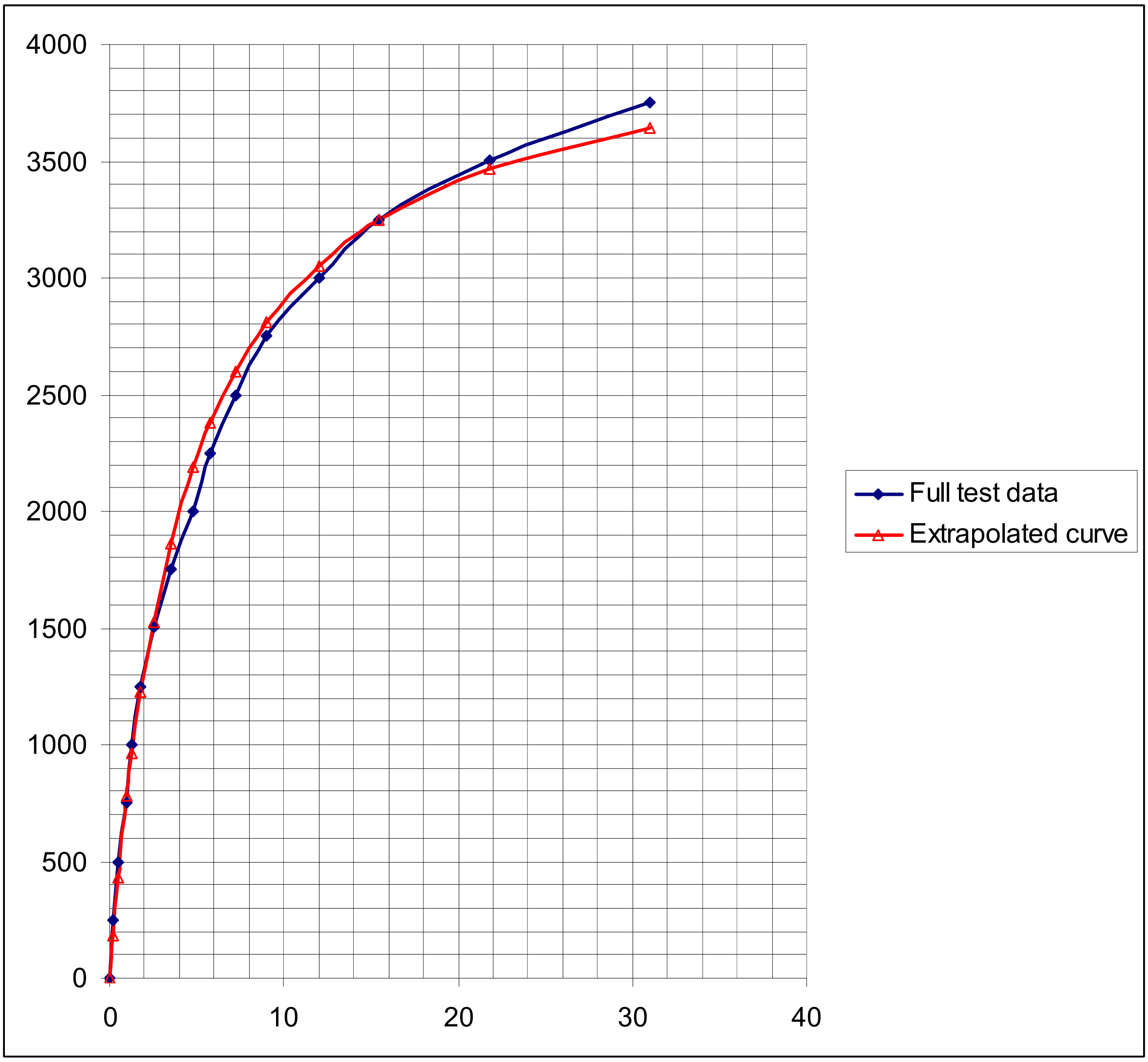

Figure 4.11: Extrapolated curve superimposed on full test data curve for case 63

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Pile case 56

To double check that the results obtained for case 63 have not occurred by chance, load test results from a trial pile at another site were analysed in a similar way. For this case the load-deflection data are presented in table 4.14. The load of 1.5 times the design load corresponds to 600 kN and therefore the data were truncated at this value.

Table 4-14: Load-settlement data for pile case 56

Q (kN)	S (mm)	S/Q
0	0	
100	0.09	0.0009
200	0.23	0.00115
300	0.42	0.0014
400	0.6	0.0015
500	0.8	0.0016
600	1	0.001667
700	1.1	0.001571
800	1.3	0.001625
900	1.6	0.001778
1000	1.8	0.0018
1100	2	0.001818
1200	2.4	0.002
1300	2.8	0.002154
1400	3.5	0.0025
1500	5	0.003333
1625	8	0.004923
1700	10	0.005882
1850	15	0.008108
1900	20	0.010526
1950	30	0.015385
1950	40	0.020513
1950	50	0.025641

The ensuing transformed plot, extrapolated curve and full test data curve are presented in figures 4.12, 4.13 and 4.14 respectively. The ultimate pile capacity from extrapolated curve is 1800 kN while that from full test data is also 1800 kN. Figure 4.15 shows the extrapolated curve superimposed on the full test data curve and it can be seen that the fit between the two cases is good.

The results of the two cases confirm that the extrapolated curves using Eq. 4.3 reasonably simulate load-deflection curves for piles tested to failure. Also the interpreted capacities from extrapolated curves are quite close to those from full test data. Therefore the interpreted capacities derived in this study can be used with confidence.

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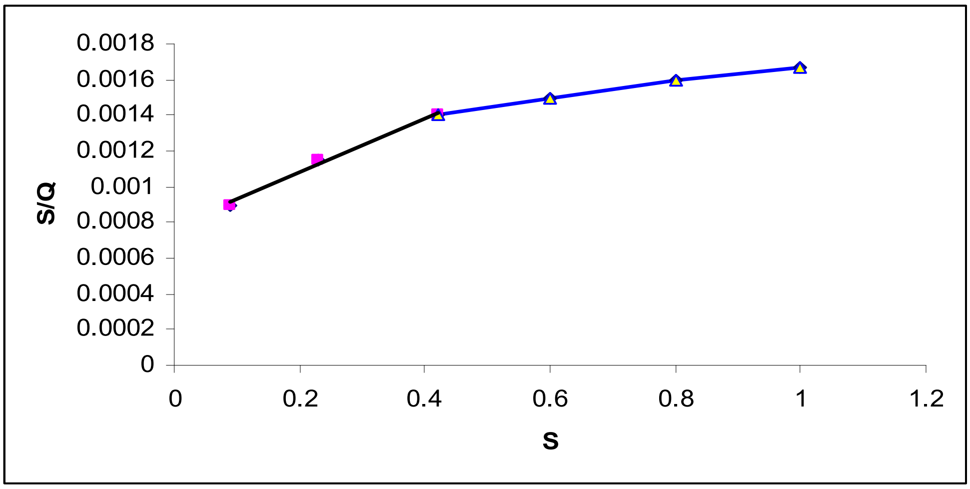

Figure 4.12: Transformed plot for pile case 56

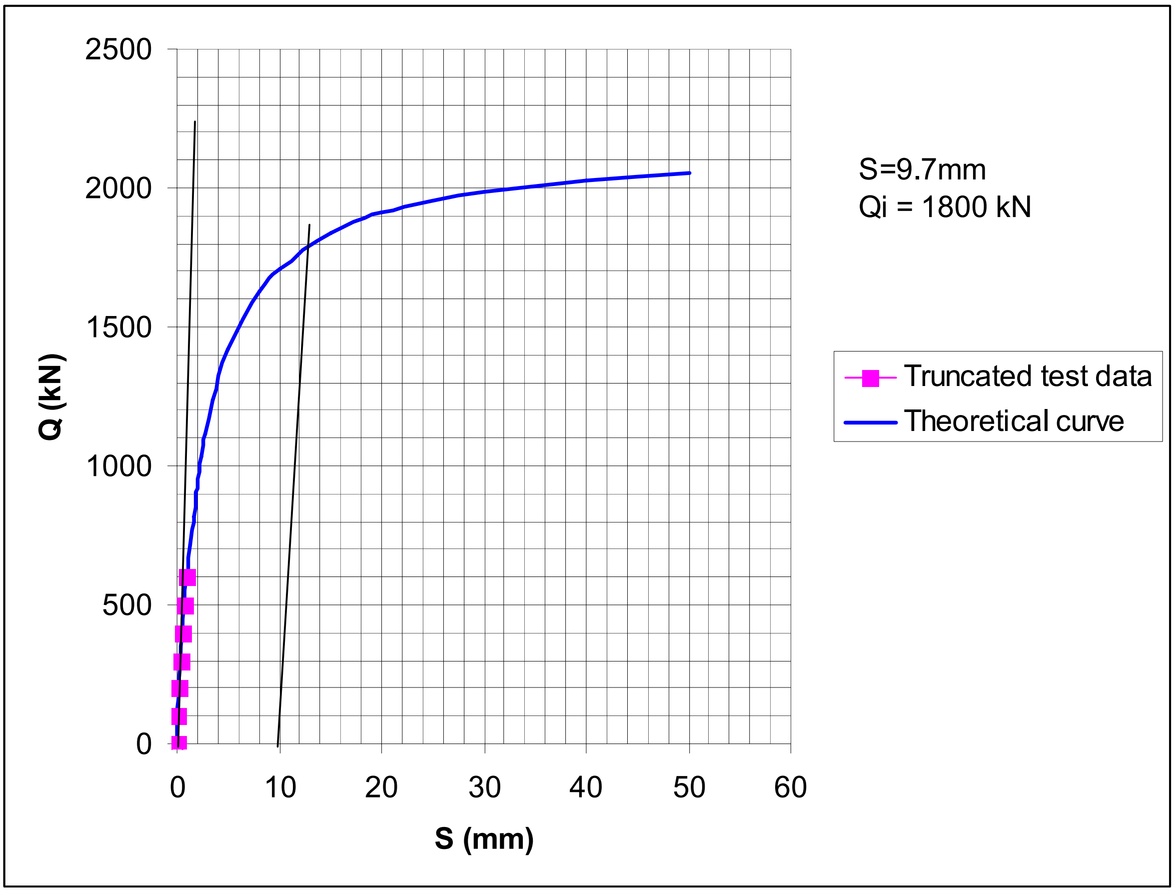

Figure 4.13: Extrapolated curve for case 56

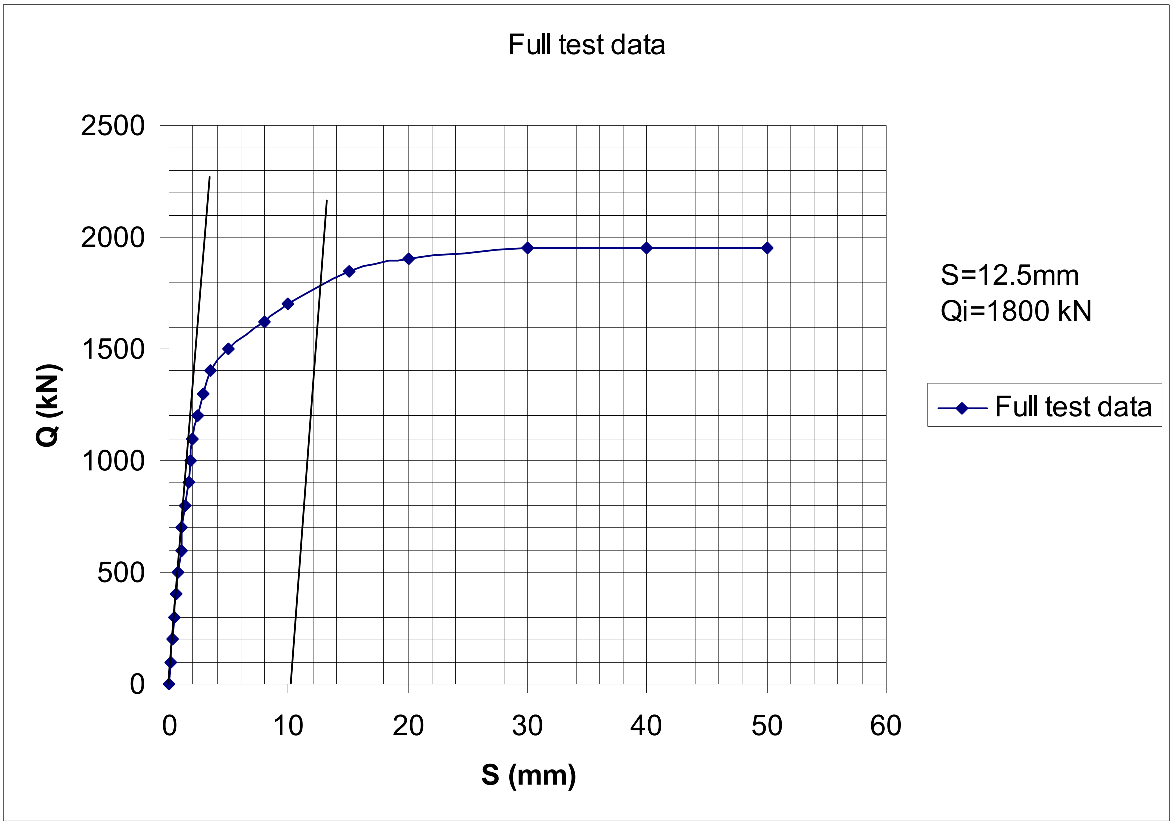

Figure 4.14: Full test data curve for case 56

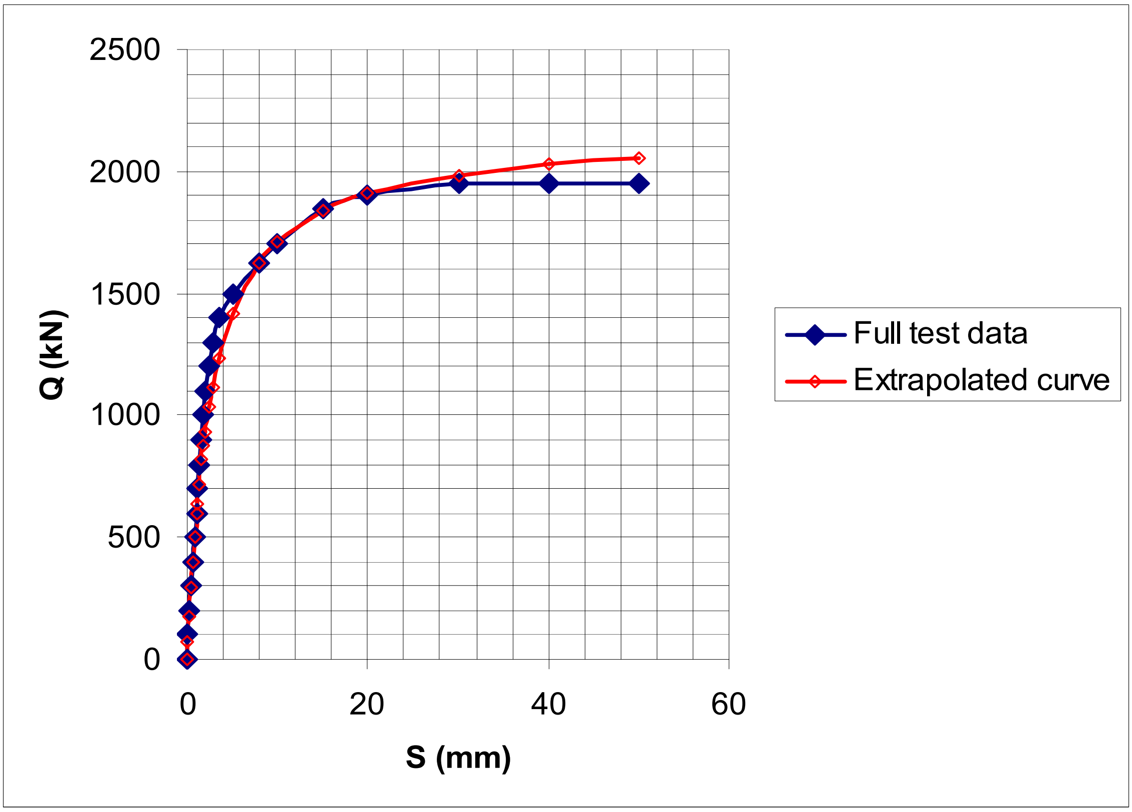

Figure 4.15: Extrapolated curve superimposed on full test data curve for case 56

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Chapter 5

EVALUATION OF GEOTECHNICAL DESIGN PARAMETERS AND COMPUTATIONS OF PREDICTED CAPACITIES

5.1 Introduction

The site specific geotechnical data presented in Chapter 4 were in the form of soil profile descriptions and penetration tests results. However, the theoretical computations of the bearing capacity of geotechnical materials require the engineering properties of the materials as input in the calculation model. Therefore the available measurements (in-situ or laboratory measurements) need to be transformed to the desired soil engineering properties through empirical correlation. In general the process of mapping the available measurements to engineering properties is not well defined. The procedure is still arbitrary in the sense that upon repetition it does not yields the same results whether by the same engineer of by several engineers (Tabba and Young, 1981). Accordingly this Chapter presents the basis for the selection of the various geotechnical parameters used in this study. The selected parameters are then used to compute the theoretical pile capacities.

5.2 Methods for predicting ultimate pile capacity in South Africa

In South Africa there are two codes of practice which specifically relate to geotechnical aspects of pile foundations. These are: (1) SABS 1200 F-1983: Standardised Specification for Civil Engineering Construction, Part F- piling and (2) SABS 088-1972: South African Standard Code of Practice for Pile Foundations. SABS 1200 F-1983 covers the construction of piles and does not cover any design criteria while SABS 088-1972 provides a general description of the types of piling at present in use, together with precautions recommended for observance in their design and application.

In addition to the two above codes, a Guide to Practical Geotechnical Engineering in Southern Africa was published by Frankipile South Africa in 1976. The main purpose of the guide was to create a practical reference on all aspects of soil investigation and piling as carried out by the company in Southern Africa. The guide is now in its third edition and has become a standard text for all those in the piling industry in Southern Africa (Knight, 1995).

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Both SABS 088-1972 and the Guide to Practical Geotechnical Engineering in Southern Africa state that the load that a pile can support, in relation to the strength of the soil, can be determined by:

(i) Static analysis using engineering properties of the soil as determined from laboratory or in-situ field testing.

(ii) Empirical analysis by directly using standard field tests results
  - CPT
  - SPT

(iii) Dynamic driving resistance
  - Pile driving formulae
  - Wave equation

(iv) Full scale pile load tests

These four methods constitute the main categories of pile capacity prediction world wide. However for each category, many variations of the basic approach have evolved. SABS 088-1972 recognises the variability of preference for formulae for static, empirical and dynamic analysis by refraining from prescribing specific pile capacity prediction equations. The code recommends the use of standard text books on foundation and soil mechanics and data from specialised piling firms familiar with the pile type and soil conditions as reference for the detailed design of any given pile foundation. Therefore, there is no distinctively South African method for determining pile capacity. However, from published local case studies and the Guide to Practical Geotechnical Engineering in Southern Africa it is evident that the static analysis using engineering properties of the soil is the predominant pile design approach in South Africa. For this reason, the static analysis approach was selected for quantification of model uncertainty in this study. The analytical model based on static analysis is generally referred to as the "Static Formula".

5.3 The static formula

The static analysis for the ultimate axial compressive capacity is based on the assumption that the capacity comes from two distinct physical mechanisms:

(i) Shaft resistance due to the development of shear stresses along the pile shaft as a result of sliding between the ground and the pile material. Mathematically the side shear is modelled using Mohr-Coulomb failure criterion. With this failure criterion,

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shearing comprises of an adhesion component as well as a stress-dependent friction component. The summation of the shear stress along the pile shaft produces a resultant force that provides resistance to the applied load.

(ii) Bearing capacity at the base of the pile foundation that is conceptually identical to that of shallow foundations.

Based on the above mechanisms, the basic formula as given by SABS 088-1972, the guide to piling and foundation systems, and several foundation engineering text books is expressed as;

$$Q_{ult} = Q_b + Q_s - W \qquad [5.1]$$

Where Q_{ult} is the ultimate pile capacity, Q_b is the end-bearing capacity and Q_s is the frictional capacity along the pile perimeter and W is the weight of the pile.

The terms in equation 5.1 are generally further expanded as follows;

For base capacity:

$$Q_b = A_b\left(cN_c + 0.5\gamma BN_\gamma + \gamma D_f N_q\right) \qquad [5.2]$$

The term ($0.5\gamma BN_\gamma$) is very small compared to the other two and is therefore generally ignored in practical designs. When this term is left out, equation 5.2 reduces to:

$$Q_b = A_b\left(c_u N_c\right) \text{ for piles in cohesive materials} \qquad [5.3]$$

$$Q_b = A_b\left(\sigma'_v N_q\right) \text{ for pile in non-cohesive materials} \qquad [5.4]$$

For shaft capacity:

$$Q_s = p\sum_{L=0}^{L=L} f_s \Delta L \qquad [5.5]$$

Also the specific equation for shaft capacity is dependent on the soil type as follows:

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Cohesive materials: $Q_s = \alpha\, c_u A_s$ [5.6]

Non-cohesive materials: $Q_s = (K_s \sigma'_v \tan\delta) A_s$ [5.7]

The complete theoretical equations for axial compression capacity of a pile in non-cohesive and cohesive materials respectively are as follows:

$$Q_{ult} = A_b\left(N_q \sigma'_v\right) + A_s\left(K_s \sigma'_v \tan\delta\right) \quad [5.8]$$

$$Q_{ult} = A_b\left(C_u N_c\right) + \alpha C_u A_s \quad [5.9]$$

Where:

A_b = base area,

A_s = shaft area

$N_c; N_\gamma;\ and\ N_q$ = bearing capacity factors

c_u = undrained shear strength

γ = unit weight,

B = width or diameter of the pile

D_f = depth to the pile tip

σ'_v =effective vertical stress

P = pile perimeter

f_s = unit shaft friction

L = length of the pile

δ = soil-pile interface friction

α = adhesion factor

The pile weight is routinely neglected in practice. The basis for this is that the weight of pile is approximately equal to weight of soil removed or displaced to make way for the pile. Since in the calculation of the net bearing capacity of the soil, the overburden pressure at the base of the pile is not taken into account, it follows that the weight of the pile should not be included.

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For piles in non-cohesive materials, the effective vertical stress in Eq. 5.4 and Eq. 5.7 suggests that the pile resistance increases with depth. However field pile tests have shown that both the base capacity and the shaft capacity do not increase continually with depth. In fact they increase with depth up to a maximum limit at a depth of about 15-20 times the pile diameter and remain constant thereafter (Das, 2003). The depth at which the resistance stops to increase with depth is termed the critical depth. Although the concept of the critical depth is a contentious issue, it was adopted in this study.

Conceptually equations 5.7 and 5.8 are straight forward to use. The validity of the equations has been confirmed repeatedly throughout the years by research (Horvath, 2002). The difficulty in applying the equations in practice has always been to correctly quantify the various variables that appear in the equations. The determination of the variables is further complicated by the fact that some of the variables are dependent on both the geostatic stress state in the ground prior to pile installation and the specifics of the installation process (i.e. bored or driven). Consequently various approaches for the determination of the soil input parameters have been suggested by different researchers. Generally the determination of the soil design parameters is a two-step process that encompasses (i) derivation of soil properties from the available laboratory or field measurements and (ii) correlating design parameters such as bearing capacity factors, cohesion factor and earth pressure coefficient from the derived soil properties.

5.4 Derivation of soil properties from available measurements

As already pointed out, the available geotechnical data was limited to soil profile descriptions and penetration tests results. The penetration tests results included Standard Penetration Test (SPT) blow-counts, Dynamic Probe Super Heavy (DPSH) results and Cone Penetration Test (CPT) results. However, measurements from SPT were the most common for most of the sites. This suggests that SPT is the most popular subsurface investigation method in Southern Africa. For derivation of soil properties, the DPSH and CPT measurements were converted to equivalent SPT N-values. The SPT N-values were then used to estimate the desired geotechnical properties.

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5.4.1 Background and interpretation of SPT results

5.4.1.1 Historical development

The standard penetration test owes its origin from the extensive work geared toward developing an effective pipe sampler in the 1920s. The samplers evolved from the simple Gow sampler in 1902 followed by the split sampler in 1920s. The split sampler was further modified to the Raymond sampler. During the same period, investigation towards measuring the resistance of the material in terms of the number of blows required to penetrate a specific depth got under way. In 1954, the conventional procedure whereby blows are recorded for each of three 6 inch (15.25 cm) increments was introduced. The procedure was such that the value for the first round of advance is discarded because of fall-in and contamination in the borehole. The second pair of numbers are then combined and reported as a single value for the last 12 in. (30.5 cm). Terzaghi furthered the research on the Raymond sampler and realised that the penetration resistance of the split spoon sampler could provide useful in-situ test data which might be correlated to the consistency and density of the soil encountered. During the writing of the text on Soil Mechanics in Engineering Practice, Terzaghi in collaboration with Mohr developed the correlations between the number of blow-counts and various salient soil properties including: relative density of sands, consistency and unconfined strength of clays, and allowable bearing pressure on both sands and clays. In 1947, Terzaghi named the Raymond sample procedure as the "Standard Penetration Test" in a presentation titled "Recent Trends in Subsoil Exploration". The presentation was derived at 7^{th} Conference on Soil Mechanics and Foundation Engineering.

5.4.1.2 Test procedure

National standards outlining the procedure for carrying out the test are available in many countries. Some of the most commonly followed standards are the British Standard (BS 1375: Part 9, 1990), the American Standard (ASTM D1586, 1984), and the Japanese Standard (JIS-A219, 1976). Further more, CIRIA report CP/7 (Clayton, 1995) gives the procedures adopted around the world as well as describing its strength and weakness and its use for geotechnical design. The test procedure is illustrated in plate 5.1 and briefly described below.

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- The boring is advanced to the desired sampling depth and the split sampling apparatus is lowered to the bottom of the borehole.
- The sampler is than driven into the soil by a 63.5 kg hammer raised 0.76m above the upper face of the drivehead assembly. Common methods of raising and dropping the hammer include trip hammer, semi-automatic and automatic drop systems (plate 5.2).
- The numbers of blows for each of the three 150 mm increments of penetration are recorded.
- The penetration resistance (N-value) is taken as the sum of the blows required for the second and third 150 mm increments of penetration. The first 150 mm penetration is considered to be a seating drive.

5.4.1.3 Factors Affecting SPT N -values

Several research studies on factors influencing SPT results have been carried out (e.g. Schmertmann, 1979; Decourt, 1989; Skempton, 1986; Kulhawy and Tautmann, 1986). Table 5.1 presents a summary of the various factors that have been found to influence the SPT results. A closer look at table 5.1 reveals that most influences are either attributed to equipment effects or procedural/operator effects. Therefore with good equipment care and site supervision, most of the factors can be minimised. Given that SPT tests are carried out by professional and experienced drilling companies, it can be assumed that quality control is good and therefore the SPT measurements gathered for this study are reliable.

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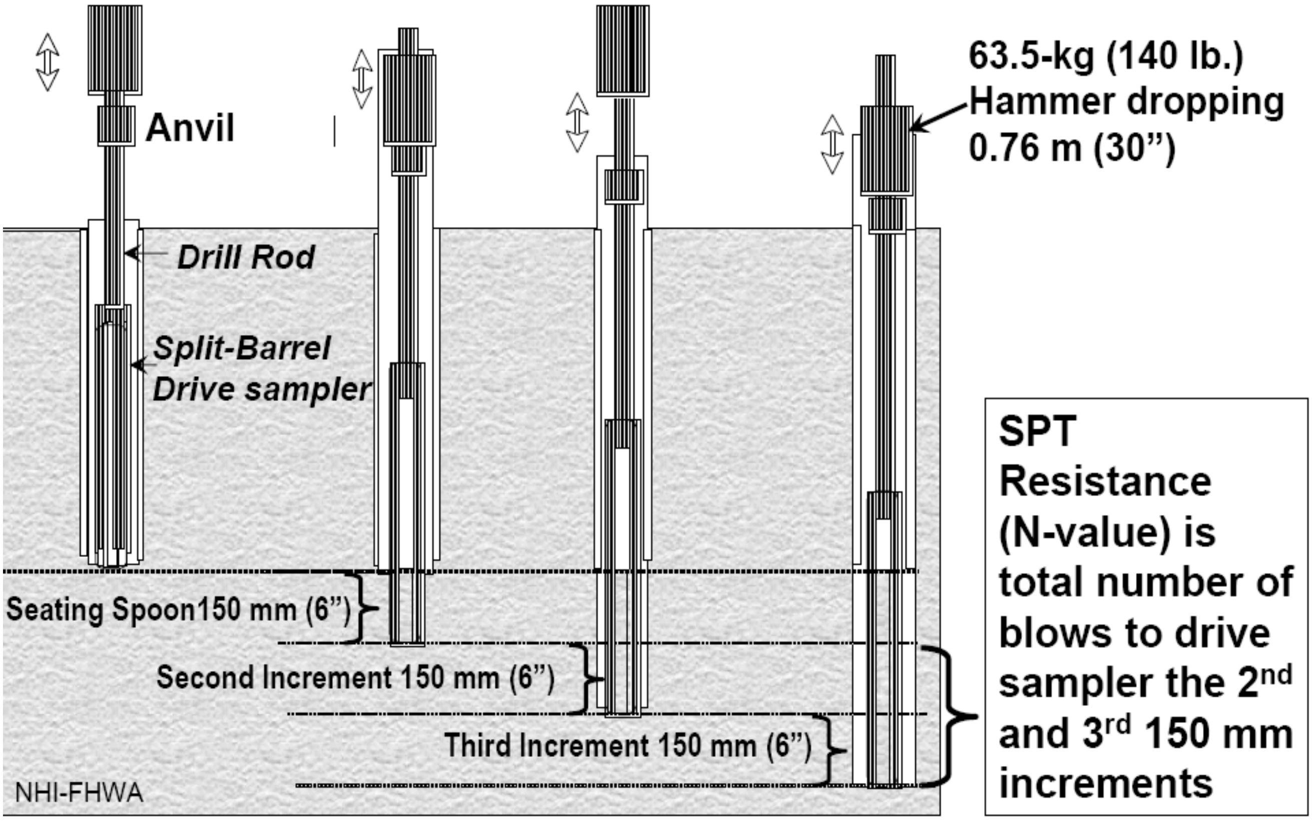

Figure 5.1: SPT test procedure (After FHWA-HI-97-021)

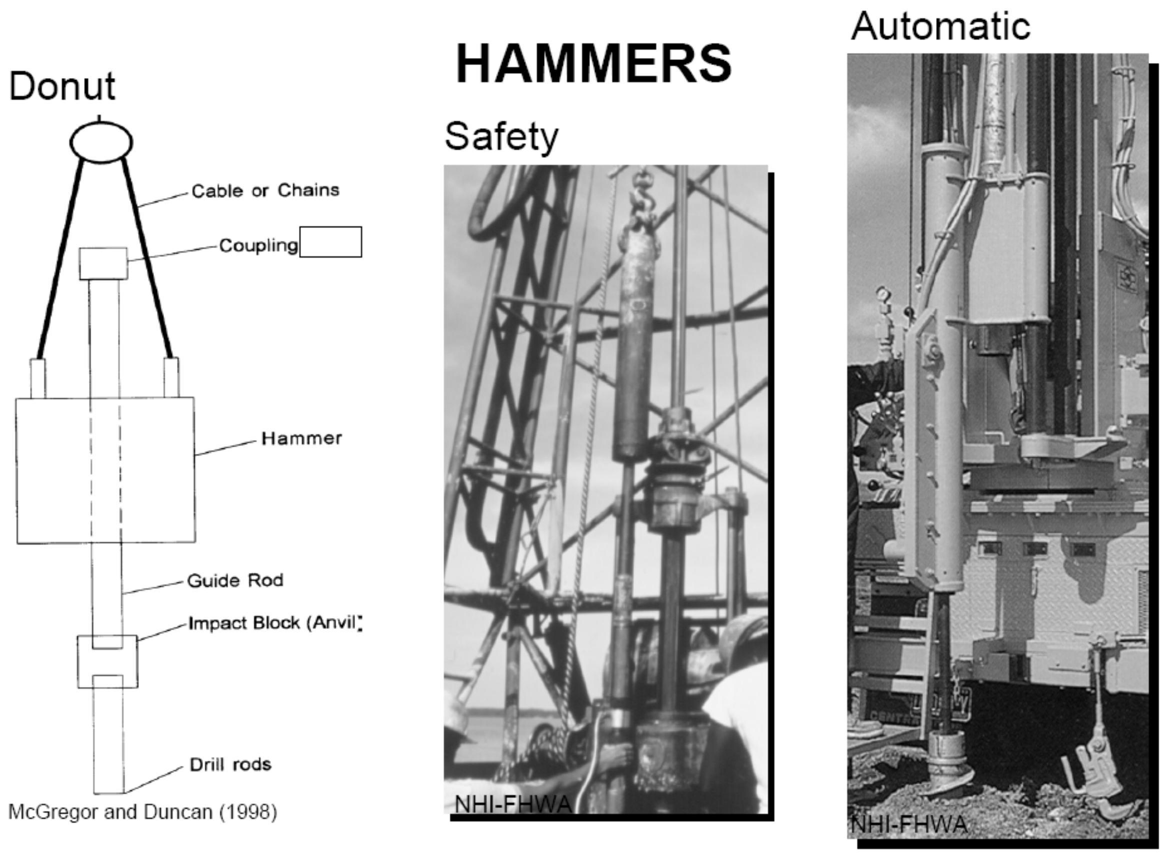

Figure 5.2: SPT hammer drop systems

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Table 5-1: Factors affecting SPT N-vales

Factors	Comments
Inadequate cleaning of the bore hole	SPT is only partially made in original soil. Sludge may be trapped in the sampler and compressed as the sampler is driven, increasing the blow count.
Not seating the sampler spoon on undisturbed material	Incorrect N-values obtained
Driving the sample spoon above the bottom of the casing	N-values are increased in sands and reduced in cohesive materials.
Failure to maintain sufficient hydraulic head in boring	The water table in the borehole must be at least equal to the piezometric level in the sand, otherwise the sand at the bottom may be transformed into loose state thereby decreasing the blow count.
Attitude of operators	Blow counts for the same soil using the same rig can vary, depending on who is operating the rig, and perhaps the mood of operator and time of drilling
Overdrive sampler	Higher blow counts usually results from an overdriven sampler.
Sampler plugged by gravel	Higher blow counts results when gravel plugs the sampler, resistance of loose sand could be highly overestimated.
Plugged casing	High N-values may be recorded for loose sand when sampling below groundwater table. Hydrostatic pressure can cause the sand to rise within the casing.
Overwashing ahead of casing	Low blow count may result for dense sand since overwashing loosen sand.
Drilling method	Drilling technique (e.g. cased holes Vs mud stabilised holes) may result in different N-values for the same soil.
Free fall of the drive weight is not attained	Using more than 1-1/2 turns of rope around the drum and or using wire cable will restrict the fall of the drive weight.
Not using correct weight	Drillers frequently supply drive hammers with weights varying from the standard.
Weight does not strike the drive cap concentrically	Impact energy is reduced, increasing N-values.
Not using a guide rod	Incorrect N-values obtained
Not using a good tip on the sampling spoon	If the tip is damaged and reduces the opening or increases the end area the N-values can be increased.
Use of drill rods heavier than standard	With heavier rods more energy is absorbed by the rods causing an increase in the blow count.

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5.4.1.4 Correlation between SPT N-vales and soil properties

The results of SPT have been used extensively to obtain soil properties for input into routine geotechnical design calculations world wide. A wide range of properties for almost all soils and weak rocks can be obtained with ease and convenience and at modest cost (Clayton, 1995). The SPT N-value is an index or measure of the in-situ firmness or denseness of the materials being penetrated. Consequently many correlations relating SPT blow count and soil properties have been developed. The first published SPT- soil properties correlation appeared in Terzaghi and Peck (1948). This was followed by correlation relating blow counts to consistency for silts and clay and relative density for sands in Peck, Hanson and Thornburn (1953). To date, many other correlation schemes have been developed.

In the 1980s, a rather new thinking to the development of SPT correlations emerged. The series of new correlations seek to correct the SPT data for a number of site specific factors (table 5.1) to improve its repeatability. Examples of published papers on the subject of correction to SPT data include: Riggs (1986), Skempton (1986), Liao and Whitman (1986), Clayton (1995), etc. Following the new line of thinking, the corrected SPT blow count is given by (Skempton, 1986):

$$N_{1(60)} = N_m C_N C_E C_B C_R C_S C_A C_{BF} C_C \qquad [5.9]$$

Where:

$N_{1(60)}$ = measured blow count corrected to 60% of the theoretical free-fall hammer energy, 100kPa effective overburden pressure and other factors.

N_m = measured SPT blow count

C_N = overburden correction factor

C_E = energy correction factor

C_B = borehole correction factor

C_R = rod length correction factor

C_S = sampling method correction factor

C_A = anvil correction factor

C_{BF} = blow counts frequency correction factor

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C_C = hammer cushion correction factor

Certainly the above correction factors are just too many to be applied in practical design situations. Nevertheless, only the overburden and hammer energy correction factors are usually considered.

Both the new and old correlations have one thing in common; they are developed from a large database of results based on past experience. Therefore when using any specific correlation, the following critical points should be borne in mind.

- The selected correlation is only as good as the data used to develop it.
- Many correlations for sands were developed for clean, uncemented, and uniform sand.
- A correlation provides an approximate answer and will undoubtedly exhibit scatter among the data points.
- The selected correlation will be most accurate if calibrated to local soil conditions.

The correlation schemes can be broadly classified into schemes for non-cohesive materials and schemes for cohesive materials.

5.4.1.5 Correlations for properties of non-cohesive materials

For design of piles in cohesionless soils, the key parameters to be obtained from SPT test data are the in-situ density and the friction angle (φ). The correlation between soil consistency, unit weight and SPT N- values is presented in table 5.2. The table is based on the correlation developed by Terzaghi and Peck (1948). The correlation has been adopted by most geotechnical codes of practice worldwide including the Guide to Practical Geotechnical Engineering in Southern Africa. The consistency field identification descriptions and the approximate unit weight values presented in table 5.2 are in accordance with the Revised Guide to Soil Profiling for Civil Engineering Purposes in Southern Africa (Jennings et.al, 1973). The unit weights required for the calculation of the ultimate pile capacity in this study were based on the correlation presented in table 5.2. The resulting unit weights for various case histories are presented in tables 5.3 and 5.4.

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Table 5-2: Correlation between SPT, consistency, and unit weight

Consistency	Filed identification	SPT (N)	γ (kN/m^3)
Terzaghi and Peck (1948)	Jennings et.al (1973)	Terzaghi and Peck (1948)	Jennings et.al (1973)
V.Loose	No resistance to shoveling	0 – 5	16
Loose	Easily penetration by 12mm bar driven by hand. Small resistance to shoveling.	5 – 10	16.5
Med. Dense	Easily penetration by 12mm bar driven with 2 kg hammer. Considerable resistance to shovel.	10 – 30	17.3
Dense	Hard penetration with 12 mm bar driven with hammer. Hand pick needed for excavation.	30 – 50	18
V.Dense	Penetration only to 75mm with 12mm bar driven with hammer. Power tools for excavation	>50	19.5

Table 5-3: Unit weight (bored piles cases)

Case No.	γ
30	19
31	19
32	19
33	19
34	18
35	18
36	18
37	18
38	18
39	20
40	17
41	16
42	17
43	17
44	17
45	17
46	17
47	16
48	16
49	17
50	16
51	16
52	18
53	18
54	18
55	18
56	18
57	18
58	16
59	16
60	17
61	18
62	18

Table 5-4: Unit weight (driven **pile)**

Case No.	γ
1	16.5
2	16.5
3	16.5
4	16.5
5	16.5
6	16.5
7	16
8	16
9	18
10	18
11	18
12	16
13	16
14	16
15	18
16	18
17	19
18	16
19	18
20	18
21	17
22	17
23	16
24	16
25	16
26	16
27	18
28	18
29	18

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Concerning the friction angle, several correlations that have been suggested by various researchers are available. Some of the correlations are based on corrected N-values ($N_{1\ (60)}$) while others are based on uncorrected values (N). Currently there is no consensus on the use of corrected versus uncorrected SPT blow count. In this study correlations based on uncorrected N-values were used. The reasons for this selection are as follows:

- Uncorrected values are used in practical designs worldwide;
- Most reliable correlations are based on uncorrected blow count;
- The required correction factors are too many;
- The models for deriving the correction factors are not perfect and therefore introduce additional uncertainties to the interpretation of SPT results. For example, several formulae and charts have been published for overburden correction with completely different results;
- State-of-the- art equipment are in use these days and therefore most equipment-related effects are eliminated;
- SPT is carried out by specialist drilling companies with sufficient experience and therefore they are conversant with the test. Hence operator/procedural effects are minimal.

With regard to the essential corrections of overburden pressure and hammer energy loss, the position taken in this study is that:

- The reference overburden stress of 100 kPa is just a hypothetical value and therefore does not reflect the operational stress around a pile. The increase in SPT N value with depth is a reflection of the increase in strength and stiffness caused by in-situ overburden stress and therefore correction for overburden stress may not be necessary.
- The correction for energy loss makes sense if the exact loss has been measured on site for each case considered. But usually a generic correction factor is applied. Surely in some sites, advanced SPT equipment might have been used with no energy loss at all while in some sites older equipment might have been used with higher energy losses. Since the exact SPT equipment used to measure the N-values as well as their site measured energy losses are not known, it is not worth while to apply the correction factor.

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Furthermore preliminary investigations on the effects of correcting the N-values on various pile design parameters were made. Using the two sets of SPT data (i.e. N and $N_{1\,(60)}$) various design parameters were derived. The parameters were used as input into the static formula to calculate the predicted capacity for each case leading to two sets of predicted capacities. The two sets of predicted capacities were then compared with the respective measured capacities evaluated in chapter 4. The comparison was based on the ratio of measured capacity to predicted capacity (i.e the bias). The bias statistics were then used to evaluate the fit of the two data sets. The bias factor statistics in this case provides an indication of the accuracy and precision of the design parameters from the two data sets. A better approach will yield a mean bias factor of close to 1 and small variability. For the 26 pile cases investigated, the bias factor statistics are presented in table 5.5.

Table 5-5: Bias factor statistics

Data set	Mean	Standard deviation	Coefficient of variation
Uncorrected N-values	1.11	0.29	0.26
Corrected N-values	1.19	0.46	0.39

The conclusions drawn from this preliminary investigation are as follows:

- In comparing the design parameters derived on the basis of corrected and uncorrected N-values, no significant improvement in terms of reduced variability was gained from correcting the N-values.
- The data set for uncorrected N-values provides the best fit to the measured capacities as shown in table 5.5 (i.e. a mean value of closer to 1 and a smaller variability).

It is therefore concluded that for reliability based design of pile foundations, correction of N-values for overburden and hammer efficiency do not add any value.

Some of the published correlation between the angle of internal friction and the uncorrected SPT N-values are presented in figure 5.3. From figure 5.3, it can be seen that curves corresponding to Peck et.al (1974) and Teng (1962) coincide which is an indication that the correlations might have been based on the same database. In terms of conservatism, φ-values by Nixon (1982) and Byrne (1995) are the most conservative while those by Meyerhof (1965) and Das (1984) are less conservative. Values by Peck et.al and Meyerhof (1974) are moderate. However, at N-values of less than 46, Nixon's φ-values are on a less

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conservative side. The correlation by Peck et.al (1974) seems to lead to reasonably conservative φ-values and was therefore selected for this study. This correlation is further shown in figure 5.4.

Many researchers including Meyerhof (1965) have drawn attention to the fact that values of angle of internal friction given by the correlation schemes in figure 5.3 are conservative estimates based on experimental data for relatively clean sands. Therefore, these values need to be adjusted to account for the presence of fines and coarser materials. The correction that has been generally adopted is to reduce the values by 3 to 5˚ for clay or silty sands and increase them by the same range for gravely sands. These adjustments were followed in this study. The procedure followed to obtain φ values from SPT N values is as follows:

a) Obtain average N values for both shaft and base materials from the borehole logs. For base material, the average N value was taken over a depth of four pile base diameters above and one base diameter below pile toe.
b) Obtain φ value corresponding to the determined average N value using Peck et.al (1974) correlation.
c) Adjust the φ value for presence of fines or gravel as follows:
    - Reduce by 3˚ for clay and silty sands;
    - Increase by 3˚ for gravely sands.

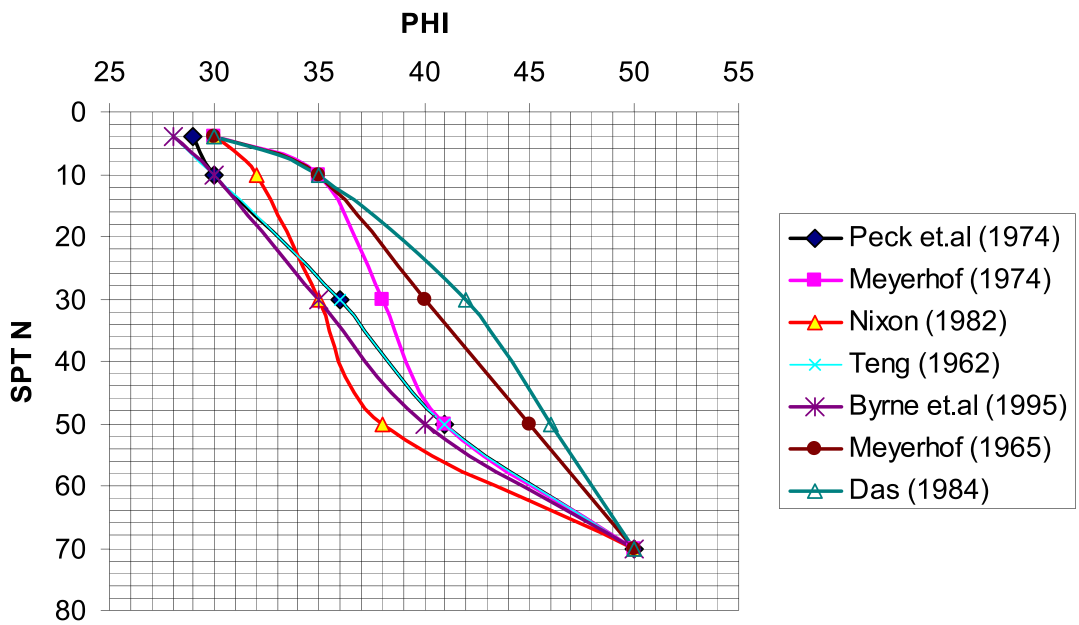

Figure 5.3: N – φ correlation by different Authors

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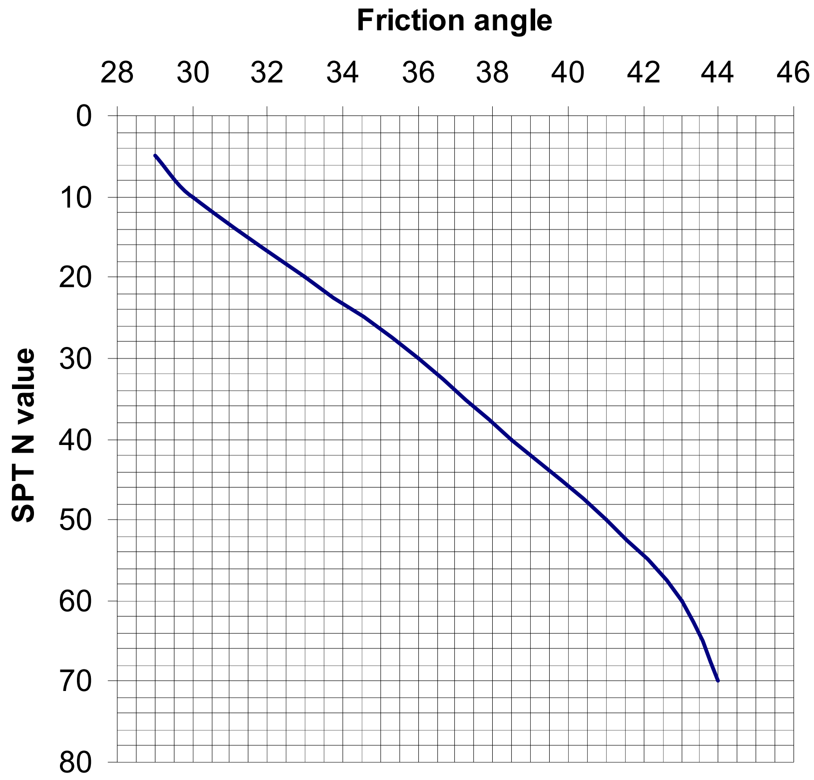

Figure 5.4: N – φ correlation (After Peck et.al (1974)

The values of the angle of friction obtained are presented in tables 5.6 and 5.7. Further examination of the φ values lead to the following observations:

- In general, for a given case φ-base values are higher than φ-shaft. This is in accordance with expectations as piles are normally founded on denser materials.
- On average, the φ values for materials in which bored piles were constructed are relatively higher than for materials in which driven piles were installed. This is attributed to the fact that under normal circumstances, bored piles are preferred in very dense materials (higher φ values) while driven piles are preferable when the granular material is not very dense.
- The magnitudes of the φ values are within the expected range of φ values for non-cohesive materials. However, some φ-shaft values are on the lower side, indicating that the pile shaft passes through some silty soils.

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Table 5-6: φ-values (driven piles)

Case No.	φ-base	φ-shaft
1	33.4	32.8
2	30.7	30.0
3	33.6	34.0
4	32.9	31.9
5	33.6	31.9
6	33.6	31.9
7	32.1	26.4
8	32.1	33.4
9	38.2	34.5
10	38.2	34.5
11	38.2	34.5
12	30.5	31.6
13	31.5	31.6
14	34.5	33.7
15	39.4	38.9
16	39.4	38.9
17	39.4	38.9
18	34.2	31.9
19	33.0	34.3
20	33.2	34.3
21	33.2	31.9
22	35.0	31.9
23	33.2	31.0
24	33.2	31.3
25	33.3	31.3
26	30.1	31.0
27	43.3	40.1
28	43.3	40.1
29	43.3	40.1

Table 5-7: φ-values (bored piles)

Case No.	φ-base	φ-shaft
30	35.9	33.1
31	35.9	33.1
32	35.9	33.1
33	40.3	31.2
34	38.9	30.8
35	38.9	30.8
36	38.9	30.8
37	38.9	30.8
38	38.9	30.8
39	40.1	36.4
40	34.5	32.2
41	34.5	31.6
42	33.7	32.2
43	32.8	31.9
44	32.8	31.9
45	33.7	32.2
46	34.5	31.9
47	34.3	31.0
48	34.5	31.3
49	34.8	32.2
50	31.0	31.6
51	31.0	32.2
52	40.3	40.3
53	47.2	47.2
54	40.3	40.3
55	47.2	47.2
56	30.3	32.5
57	30.7	31.9
58	30.7	29.7
59	32.2	29.7
60	32.2	31.3
61	37.3	36.4
62	35.9	33.1

5.4.1.6 Correlations for cohesive materials

In this study, cohesive materials do not entirely refer to clay soils. Cohesive materials predominately refer to residual soils. The area between latitudes 35˚ North and 35˚ South is characterised by the extensive occurrence of residual soils (Blight and Brummer, 1980). Southern Africa falls within these bounds and therefore the prime soil materials are of residual origin. The residual soils are overlain by transported materials and underlain by the fresh parent rock. A typical Southern African soil profile consists of (1) top horizon of transported materials, (2) middle horizon of residual materials and (3) bottom horizon of

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parent rock. The depth of the transported and residual soil horizons is influenced by the local geology, climate and geomorphology that produced the prevailing landform. In general pile foundations are founded on the residual soils rather than on the transported materials.

Residual soils exhibit special engineering properties and characteristics that distinguish it from sedimentary soils (Blight, 1991; Fookes, 1991). The definition of residual soil varies from country to country. In South Africa residual soil is defined as a soil-like material derived from the in-situ weathering (both physical and/or chemical weathering) and decomposition of rock which has not been transported from its original location (Blight, 1991). The materials are described in terms of the rock from which they have been derived.

The fundamental geotechnical property required for design of piles in cohesive materials is the undrained strength (C_u). Correlation between SPT N-values and the consistency as well as the undrained shear strength of cohesive materials have been developed. Table 5.8 presents the correlation between rock consistency and the undrained shear strength while table 5.9 presents the correlation between in-situ tests and the required design parameters.

Table 5-8: Correlation between rock consistency and undrained shear strength (Byrne et al, 1995)

Consistency	Field Identification	C_u
		kN/m^2
V. Soft	Material crumbles under firm blows with geologist's pick. Can be peeled off with knife.	350 -1500
Soft	Indentation 1 mm to 3 mm with firm blows with geologist's pick. Can just be scraped with knife.	1500 -5000
Hard	Hand held specimen breaks with hammer end of geologist's pick with single firm blow. Cannot be scraped with knife.	5000 – 10000
V.Hard	Hand held specimen breaks with hammer end of geologist's pick with more than one blow.	10000 – 35000
V.V Hard	Hand held specimen requires many blows with geologist's pick to break through intact material.	>35000

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Table 5-9: Correlation between in-situ tests and pile design parameters for cohesive materials (Byrne et al, 1995)

Consistency	Filed identification	SPT N	CPT q_c (MPa)	C_u kN/m^2	α
Very Soft	Easily moulded with fingers. Geologist's pick can be easily pushed in up to its handle	<2	<0.15	<20	1.00
Soft	Easily penetrated by thumb. Moulded with strong pressure. Geologist's pick pushed in up to 30 t0 40 mm (sharp end)	2 – 4	0.15 – 0.3	20 - 30	1.00
Firm	Intent by thumb with effort. Very difficult to mould with fingers. Geologist's pick pushed in up to 10mm (sharp end)	4 – 6 6 – 8	0.3 – 45 0.45 -600	30 -40 40 - 50	0.90 0.80
Stiff	Penetration by thumb nail. Can not be moulded with fingers. Geologist's pick makes slight indentation (sharp end).	8 – 10 10 - 12 12 -14 14 - 16	0.6 – 0.75 0.75 – 0.9 0.9 – 1.05 1.05 – 1.2	50 - 60 60 - 70 70 -80 80 -90	0.70 0.60 0.55 0.50
Very Stiff	Indentation by thumb nail difficult. Slight indentation with blow of geologist's pick. Power tools required for excavation.	16 - 18 18 - 20 20 -22 22 -24 24 -26 26 -28 28 - 30	1.2 – 1.35 1.35 – 1.5 1.5 – 1.65 1.65 – 1.8 1.8 – 1.95 1.95 – 2.1 2.1 – 2.25	90 -100 100 -110 110 -120 120 130 130 -140 140 -150 150 -160	0.45 0.40 0.38 0.36 0.34 0.32 0.31
Hard		>30 30 - 31 31 - 32 32 - 35 35 - 38 38 - 42	2.25 – 2.75 2.75 – 3.25 3.25 – 3.75 3.75 – 4.35 4.35 – 5.0	160 - 170 170 - 190 190 - 210 210 - 230 230 - 250	0.30 0.29 0.28 0.27 0.26
Very Hard		42 - 50 50 - 65 > 65	5.0 – 6.3 6.3 – 8.8 8.8 – 12	250 - 300 300 - 400 400 - 500	0.25 0.22 0.20

Another widely used correlation for obtaining the undrained shear strength from SPT results is that developed by Stroud (1989). In accordance with this correlation, the undrained strength of a cohesive material including weak rocks is given by:

$$C_u = f_1 N \qquad [5.10]$$

Where:

C_u = undrained shear strength of the material

N = number of SPT blow count

f_1 = a factor depending on the plasticity of the material. Generally f_1 is taken as 5 (Clayton, 1995).

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Due to its simplicity, Stroud's correlation was selected for estimation of the undrained shear strength of cohesive materials as well as weak rocks. However, for cases where only SPT refusal has been recorded, the undrained shear strengths were inferred from the consinstency description (Tables 5.8 & 5.9). The undrained shear strength values obtained are presented in tables 5.8.

Table 5-10: Undrained shear strength values

Driven piles			Bored piles		
Case No.	Cu-base	Cu-shaft	Case No.	Cu-base	Cu-shaft
63	175	85	122	1000	100
64	150	75	123	1000	400
65	150	75	124	1000	400
66	250	0	125	500	300
67	300	0	126	500	75
68	300	0	127	1000	75
69	300	75	128	300	75
70	300	75	129	500	100
71	200	0	130	500	100
72	250	60	131	500	100
73	250	60	132	1000	1000
74	250	60	133	1000	100
75	250	60	134	1000	100
76	1000	100	135	100	50
77	150	75	136	500	100
78	250	50	137	1000	100
79	350	100	138	1000	100
80	500	50	139	500	100
81	750	175	140	300	100
82	1000	175	141	500	100
83	500	175	142	160	140
84	500	175	143	0	140
85	1000	175	144	450	0
86	1000	175	145	0	350
87	300	100	146	1000	85
88	350	100	147	190	125
89	300	100	148	200	65
90	300	100	149	1000	175
91	300	100	150	500	85
92	250	75	151	500	85
93	250	75	152	1000	75
94	250	75	153	1000	75
95	300	200	154	1000	75
96	300	105	155	1000	75
97	300	105	156	1000	75
98	500	125	157	500	95
99	500	125	158	2000	150
100	300	200	159	450	75
101	300	75	160	1000	125
102	500	75	161	500	140
103	300	75	162	500	140
104	500	100	163	1000	155
105	175	70	164	500	155
106	160	140	165	1000	75
107	450	175	166	1000	100
108	200	60	167	1000	100
109	400	75	168	1000	100
110	400	75	169	1000	100
111	400	75	170	2000	100
112	400	75	171	1000	300
113	500	85	172	1000	300
114	500	85	173	500	200
115	500	75	174	500	200
116	200	75			
117	500	115			
118	500	115			
119	500	135			
120	500	135			
121	175	105			

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Close inspection of the undrained shear strength values show that:

- Values for C_u-base are much higher than those for C_u-shaft. This is consistent with the variation of the SPT measurements with depth and is attributed to the fact that the degree of weathering in a typical residual soil tends to decrease with depth. The weathered materials gradually merge into the unweathered rock. Accordingly the consistency varies from soft for the transported soil overlying the residual soil to very stiff for weathered rock.
- The C_u values for materials in which bored piles were constructed are generally higher than those for driven piles. These higher C_u values which are associated with higher consistencies have necessitated the construction of bored piles as it is difficult to drive piles in such materials. In fact in South Africa, the most common type of pile in residual soils is the bored cast in situ pile (Blight, 1991).
- The magnitudes of the C_u values are in accordance with what is expected of a typical profile in a residual soil. The lower values (C_u <100 kPa) denote the predominance of transported materials while middle values (i.e. C_u values of up to 500 kPa) indicate the prevalence of the residual soil. There are also a number of cases depicting rock consistency (i.e. C_u of 1000 kPa or greater). The higher values are mostly associated with C_u-base, thus confirming that piles in residual soils are usually founded on a rock consistency stratum.

5.5 Evaluation of design parameters

In the computation of pile capacity using the static formula, design parameters other than the geotechnical properties presented in table 5.4 through table 5.9 are required These design parameters can be classified as those required for piles in non-cohesive materials and those for piles in cohesive materials. For non-cohesive materials the required parameters are the bearing capacity factor (N_q) and the earth pressure coefficient (K_s) while the parameters for piles in cohesive materials are the bearing capacity factor (N_c) and the adhesion factor (α). These parameters are estimated from empirical correlations with soil properties. In any case, currently the process of determining these parameters is not standardised. Consequently different engineers follow different procedures thereby producing different design parameters for even the same site.

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To select the appropriate design parameters for the South African geological setup, two approaches were followed. The first approach entails using values commonly assumed in practice while the other approach entails conducting a parametric study of the possible values for a given parameter. The second approach is tantamount to assessment of design parameters by back analysis of load test results.

5.5.1 Selection based on commonly assumed values in practice

This approach captures the practical design situation where site specific data is limited. The selection of the key pile design parameters such as N_q, N_c, K_s and α is based on experience or adoption of published empirical correlations.

5.5.1.1 Driven piles in non-cohesive materials

Equation 5.7 constitutes the traditional theoretical equation for axial compression capacity of a pile in non-cohesive materials. To use the equation, the parameters N_q, K_s and δ need to be determined on the basis of empirical correlations with the angle of friction. Various authors have proposed various empirical correlations between the angle of friction and N_q. Figure 5.5 illustrates some of the proposed correlations. Vesic (1967) has pointed out that there is a great variation in the theoretical values of N_q, but the values by Berezantzev (1961) appear to fit the pile full-scale test data. For this reason, N_q values by Berezantzev are used by many practicing engineers and researchers as well as some geotechnical codes (Cheng 2004). Accordingly, in this study, Berezantzev N_q values have been adopted. Figure 5.6 shows an enlarged version of the φ - N_q correlation proposed by Berezantzev et.al (1961).

Also there is a wide variation of K_s values as obtained by different investigators using different theories as well as back analysis of pile tests data. In general K_s is considered to be greater than the active earth pressure coefficient but less than the passive earth pressure coefficient. It has also been found the value of K_s varies with depth such that it is approximately equal to the passive earth pressure at the top of the pile and less than the at rest earth pressure coefficient at the pile tip (Das, 2003). Further more values of K_s increases with the volume of displacement leading to smaller values for small displacement piles such as H piles and bored piles and higher values for large displacement piles. Generally the value of Ks is dependent on friction angle of the material, pile installation method and the pile characteristics (i.e. size and materials). Although the values of K_s are dependent on

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these factors, in practice generic values are used depending on whether the pile is driven or bored. For driven piles values ranging from 1 to 2 have been used. Based on this range of values a value of 1.5 was chosen.

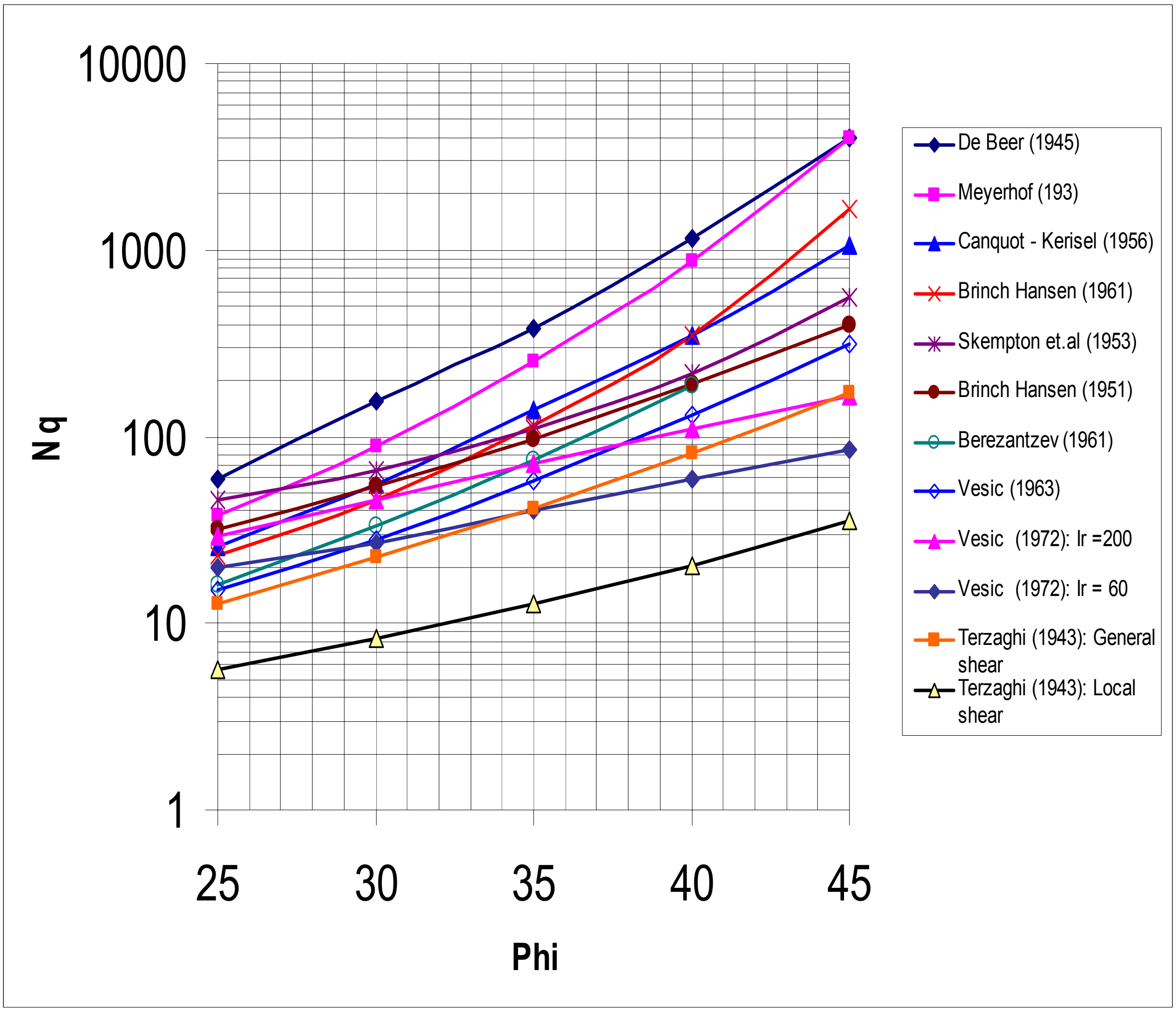

Figure 5.5: Bearing capacity factors for piles in cohesionless soils (Based on table by Coyle and Castello, 1981)

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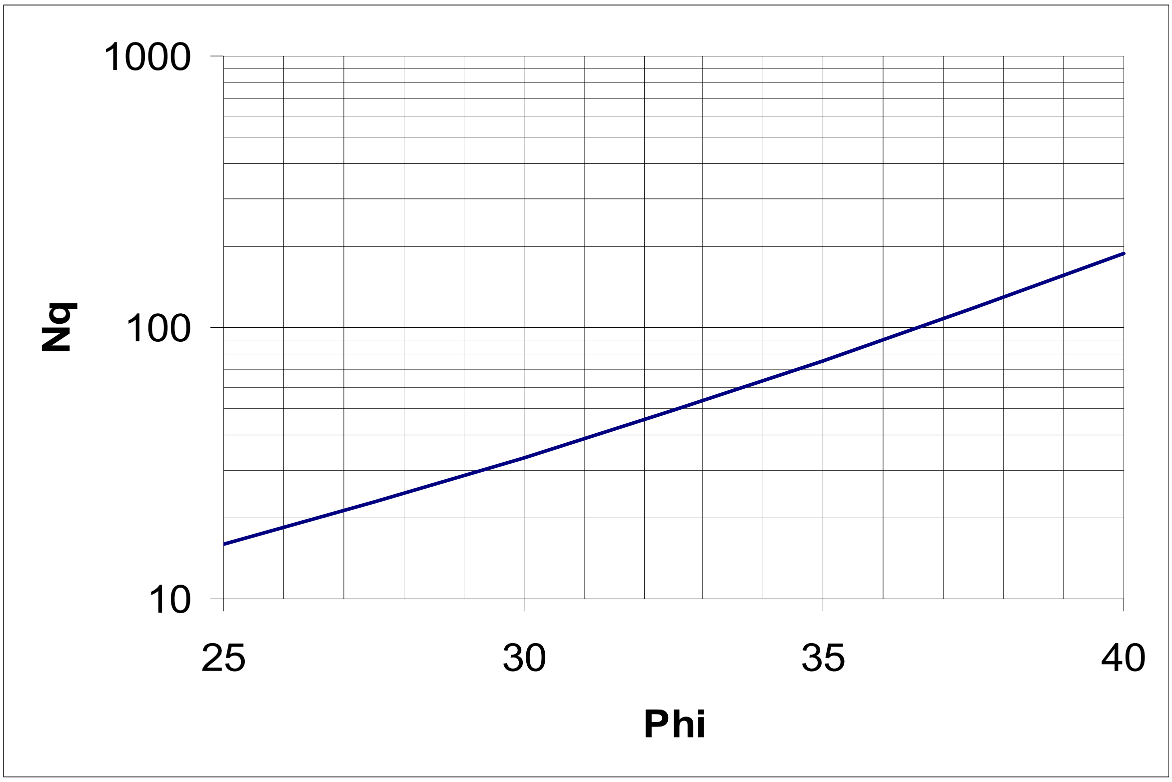

Figure 5.6: φ – N correlation by Berezantzev et.al (1961)

5.5.1.2 Bored piles in non-cohesive materials

Equation 5.7 is also used for computation of the predicted capacity of bored piles in non-cohesive materials. However, different values of N_q, K_s, and δ are used to reflect the loosening effects of the boring operation. Again, φ - N_q correlation proposed by Berezantzev et.al (1961) was selected due to its popularity. However φ values were reduced by 3˚ to account for the loosening effects of the boring operation as proposed by Paulos and Davis (1980). For bored piles, generic values of K_s range from 0.5 to 0.9. K_s value of 0.7 is commonly assumed and accordingly the same value was assumed in this analysis.

5.5.1.3 Driven piles in cohesive materials

For cohesive materials equation 5.8 is the governing theoretical equation for determination of axial compressive pile capacity. It is apparent from equation 5.8 that in addition to the undrained shear strength, the parameters N_c and α are required for the computation. The dimensionless bearing capacity factor N_c for piles has been studied by many researchers. Outcomes of some of the studies are as follows:

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- Meyerhof (1951) reported values of N_c = 9.3 and 9.8 for smooth and rough bases respectively.
- After examining various analytical and experimental values, Skempton (1951) concluded that N_c = 9 was accurate enough for practical designs.
- Whitaker and Cook (1966) suggested values of 8 to 8.5 for small diameter piles and N_c = 6.75 for large diameter piles (D≥ 600 mm). However, they suggested the adoption of N_c = 9, with a factor of 0.75 being introduced for large diameter piles.
- For expanded base piles, the local literature quote values of N_c ranging from 12 to 40. This is attributed to the high impact energy of the hammer when building the expanded base.

Based on this information, N_c = 9 and N_c = 16 were adopted for plain and expanded base piles respectively. Concerning the adhesion factor, also different values have been published. Generally the parameter α depends on the nature and strength of the material, pile dimensions, and method of pile installation. However values derived by Tomlinson (1970) are widely used. This is because the derivation of these values takes into account factors that influence α. Therefore Tomlinson's α. values were adopted.

5.5.1.4 Bored piles in cohesive materials

Generally N_c values adopted for bored piles are the same as for driven piles. Therefore again N_c = 9 and N_c = 16 were adopted for plain and expanded base piles respectively. However adhesion factors for bored piles differ from those for driven piles. Again various correlation schemes between the undrained shear strength and adhesion factors have been published (e.g. Byrne et al 1995 and Coduto, 1994). However, the use of these correlation schemes did not yield good results in term of the variability of resulting model factor statistics. Consequently a new scheme based on a combination of local and international practices was devised for bored piles. Based on this new scheme, the adhesion factors and the corresponding Cu values are presented in table 5.11.

Table 5-11: Adhesion factors for bored piles in cohesive materials

Undrained shear strength (Cu)	α-value
>75	0.8
76-150	0.6
< 150	0.45

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5.5.2 Selection based on results of a parametric study

In the previous section, it was established that for a given design parameter (e.g. N_c) a wide range of possible values have been proposed in the literature. Most of the design parameters have been derived from empirical correlations based on databases specific to certain geological setup. Extrapolation of the correlations to the Southern African distinct geologic environment may not give satisfactory results. Therefore in the absence of local correlations, it is important to establish which of the reported parameters suit the local conditions. This was accomplished by performing a parametric study on all design parameters that are selected subjectively. For a given design parameter, the parametric study entailed using the possible values that have been reported in the literature to calculate the predicted capacity. The predicted capacities were then compared with their respective measured capacities. Within a given range, the value of a given parameter that yields the best fit to the measured capacity was then considered to be the most appropriate under the Southern African soil conditions. The criterion for determining 'best fit' was based on the equation of the best fit line of predicted versus measured capacity with the corresponding coefficient of determination (R^2).

On the basis of regression analysis, the general equation of the best fit line is given by:

$$Q_{fit} = bQ_p \tag{5.11}$$

in which Q_{fit} is the least squares average measured capacity corresponding to a given predicted capacity values; b is a regression constant denoting the slope of the line; and Q_p is the predicted capacity.

Associated with each regression equation is the coefficient of determination (R^2). This is a statistical measure of goodness of fit between the predicted and measured values. More specifically, R^2 measures the proportion of the total variance in the dependent variable explained by the independent variable. For the purposes of this section, was taken as a measure of the degree of agreement between the measured capacity and predicted capacity.

For a perfect fit, b = 1 and R^2 = 1. These perfect fit values were used as the datum for evaluating the degree of fit achieved by specific values of the design parameter of interest.

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Accordingly for a range of values of a given design parameter, the value that yields a relatively high R^2 and a value of b that is relatively closer to 1 gives the best fit between predicted capacity and measured capacity.

Design values selected on the basis of best fit principle described above at least avoids the use of conservative parameters as within the framework of reliability based design, conservatism and safety are best introduced into the design process through partial factors. Given the transparency of the procedure for their derivation, such design parameters provide guidance on what constitutes appropriate design values in South Africa. Further more, such values can be tabled at a code committee for further discussion and perhaps adopted as normative values by the impending South African geotechnical code. This avoids the current chaotic situation where the selection of the design values is left to the discretion of the designer.

5.5.2.1 Parameters for driven piles in non-cohesive materials

As already noted, the required parameters are N_q, K_s, and δ. However, the use of N_q values by Berezantzev et.al (1961) and $\delta = 0.75\varphi$ have become very popular and therefore they were maintained in this analysis. Conversely, there is no common preference for K_s values. To establish which value out of the range of K_s values proposed in the literature (i.e. K_s of 1 – 2) is more appropriate for the South African conditions, a parametric study of the values was performed. The regression parameters (measures of fit) for the various K_s values are presented in table 5.12. Further inspection of table 5.12 lead to the following observations:

- The parameter b decreases with the increase in K_s value.
- The coefficient of determination slightly increases with increase in K_s value within a range of Ks of 1.0 – 1.3, and then starts to decrease with the increase in K_s values.

On the basis of R^2, it is apparent from table 5.12 that the best fit is produced by K_s value of 1.3. Therefore $K_s = 1.3$ was considered to be more appropriate. The Qi versus Qp plot for the selected case of $K_s = 1.3$ is shown in figure 5.7.

Table 5-12: Variation of the regression parameters b and R^2 with K_s

K_s	b	R^2
1.0	1.090	0.432
1.1	1.070	0.439
1.2	1.050	0.444
1.3	1.030	0.446
1.4	1.010	0.445
1.5	0.990	0.443
1.6	0.970	0.439
1.7	0.950	0.434
1.8	0.940	0.427
1.9	0.920	0.419
2.0	0.900	0.410

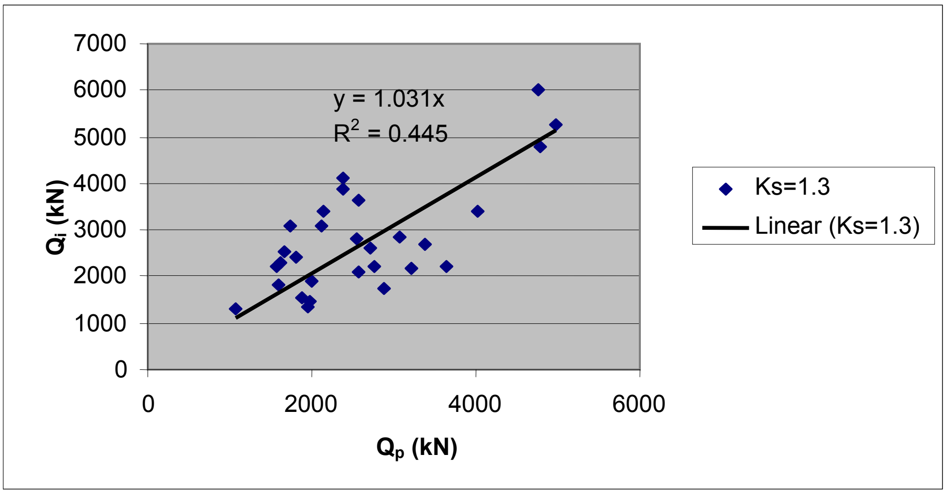

Figure 5.7: Scatter plot of Q_i Vs Q_p for K_s = 1.3

5.5.2.2 Bored piles in non-cohesive materials

Also for this design situation, N_q and δ values were fixed while the K_s values were varied from 0.5 – 0.9 in accordance with the generic values reported in the literature. The variation of the measure of fit parameters with K_s value is presented in table 5.14. Examination of table 5.13 lead to the following observations:

- As was the case for driven piles, the parameter b decreases with the increase in K_s values. On the conservative side, the b value closest to 1 corresponds to K_s value of 0.8.
- The R^2 values are almost the same for entire range. The range of values for bored piles is very narrow and accordingly a small increment of 0.1 from one point to the next can not be expected to bring drastic changes in the degree of fit.

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On the basis of R^2 it can be concluded the best fit is achieved with a K_s range of 0.6 – 1.0. To select the most appropriate value from within this range, use was made of the parameter b. The K_s value corresponding to the parameter b of relatively more close to 1 was taken as the most appropriate value. Inspection of table 5.13 shows that the above criterion is satisfied by K_s = 0.8 with b = 1.03 and K_s = 0.9 with b = 0.99. Comparison of these b values indicates that the b value for K_s of 0.8 is on the conservative side while that for K_s of 0.9 is on the unconservative side. Since it is better to err on the conservative side, K_s value of 0.8 was selected as the most appropriate. The plot of Q_i versus Q_p for the selected case of K_s = 0.8 is shown in figure 5.8.

Table 5-13: Variation of M statistics and resistance factors with Ks

K_s	b	R^2
0.5	1.150	0.73
0.6	1.110	0.74
0.7	1.070	0.74
0.8	1.030	0.74
0.9	0.990	0.74
1.0	0.960	0.74

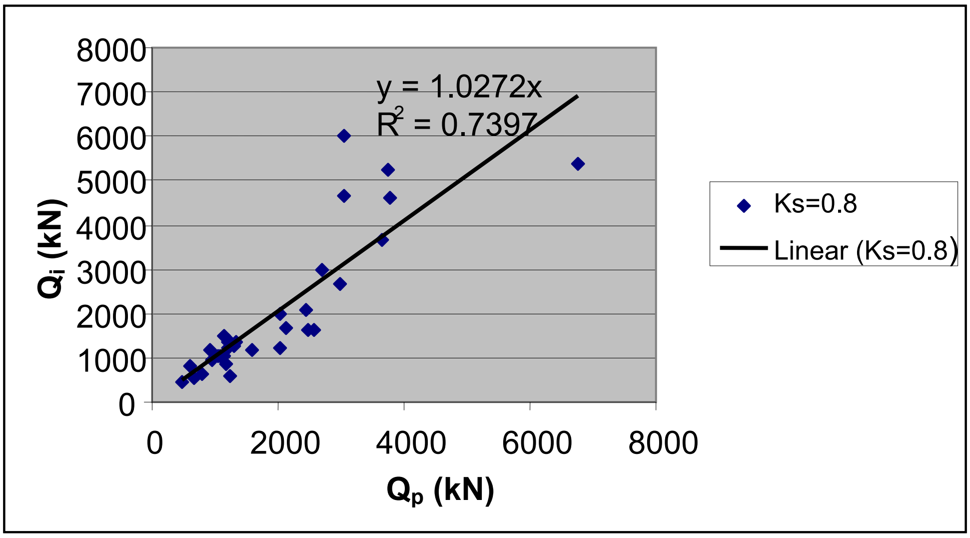

Figure 5.8: Scatter plot of Q_i Vs Q_p for K_s = 0.8

5.5.2.3 Driven piles in cohesive materials

For piles in cohesive materials, input geotechnical parameters that are not well defined include the bearing capacity factor N_c and the adhesion factor α. Due to their popularity, α values by Tomlinson have been maintained while the N_c values were varied from 9 to 30.

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Varying of N_c values was only applicable to expanded base piles (Franki piles). For the plain piles the value of $N_c = 9$ was maintained on the grounds that this is the value that is commonly used worldwide. The resultant measures of fit are presented in table 5.14. The following conclusions were drawn from the inspection of table 5.14.

- There is a rapid decrease in the parameter b with the increase in N_c values, suggesting that lower N_c values (N_c of 9 – 14) tend to under-predict the predicted capacity while higher N_c values ($N_c > 14$) tend to over-predict the predicted capacity.
- There is also a rapid decrease in R^2 values with the increase in N_c values, implying a better fit for lower Nc values.
- In general N_c values ranging from 9 to 14 seem to yield better overall prediction (i.e. under predicts theoretical capacity and relatively higher R^2 values). The results support the lower range of N_c values suggested by the local literature.

Based on the above observations, the most appropriate N_c value for the design of expanded base piles under the Southern African soil conditions falls within a range 9 – 14. Within this narrow range of N_c values, R^2 values corresponding to N_c of 9 and 10 are the highest. However the corresponding b values are appreciably greater than 1 leading to a significant degree of conservatism (33% for $N_c = 9$ and 24% for $N_c = 10$). Conversely the N_c value of 14 produces the best b value but the lowest R^2 value. The fit parameters for N_c value of 12 are somewhat between the extremes. The R^2 value is quite close to that shown by N_c values of 9 and 10 while the degree of conservatism is just 10 %. Therefore the N_c value of 12 yields the best combination of R^2 and b and was consequently selected as the most appropriate value. The Q_i versus Q_p for the selected case of $N_c = 12$ is shown in figure 5.9.

Table 5-14: Variation of fit parameters b and R^2 with N_c

N_c	b	R^2
9	1.33	0.713
10	1.24	0.717
12	1.10	0.707
14	0.98	0.687
16	0.89	0.662
18	0.81	0.635
20	0.74	0.609
22	0.68	0.584
24	0.63	0.560
26	0.59	0.538
28	0.55	0.52
30	0.52	0.498

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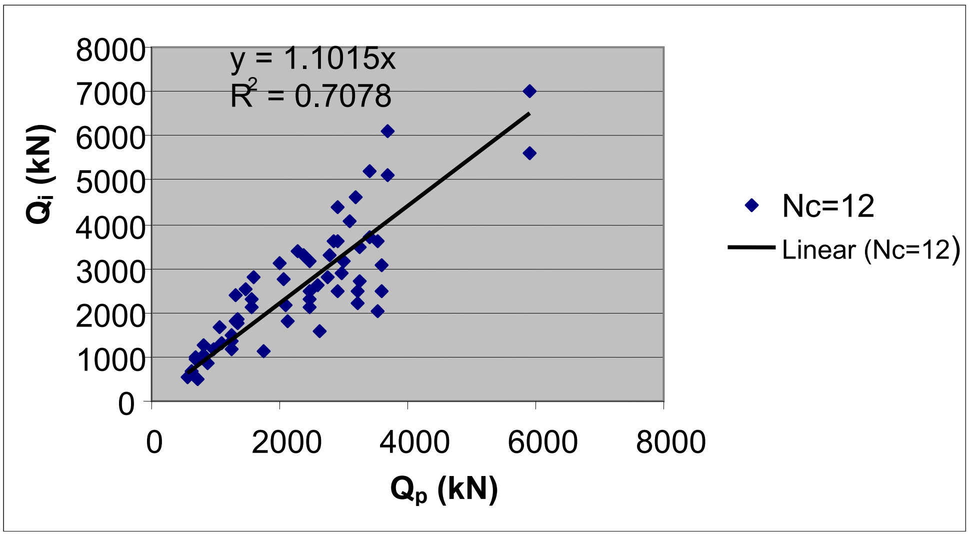

Figure 5.9: Scatter plot of Q_i Vs Q_p for $N_c = 12$

5.5.2.4 Bored piles in cohesive materials

In exception of two pile cases, all the piles in the database for this category are plain piles (i.e. without expanded bases). For piles without expanded base, normally N_c is taken as 9. For the two cases with expanded bases, an N_c value of 12 was assumed as was the case for driven piles. Again adhesion factors previously assumed were adopted. The Q_i versus Q_p plot for bored piles in cohesive materials is shown in figure 5.10. From figure 5.10, the fit parameters are; b= 1.12 and R^2 = 0.87. Relative to parameter for cases previously considered, these values were considered reasonable.

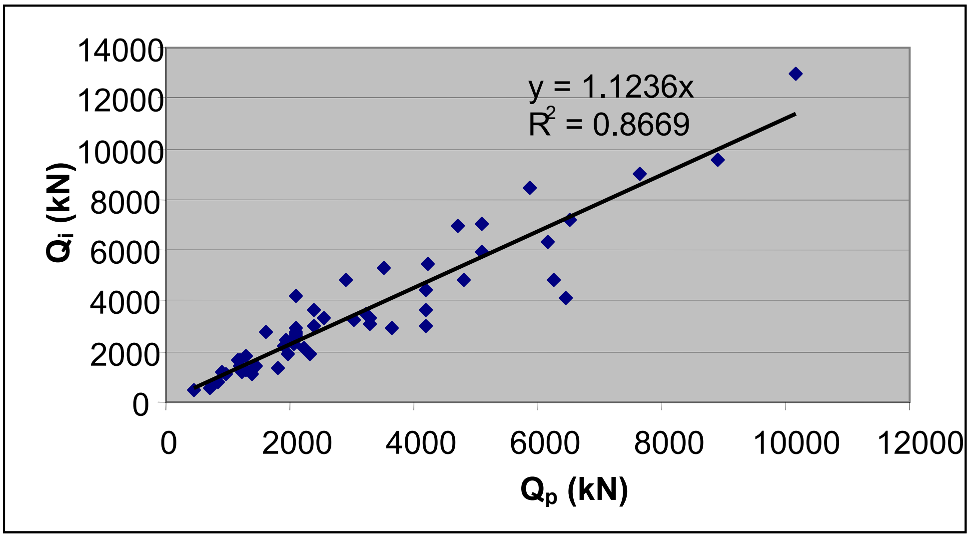

Figure 5.10: Scatter plot of Q_i Vs Q_p for BC

5.5.3 Comparison of results from the two approaches

The range of parameters for the parametric study included values that were selected in section 5.4.1. Table 5.15 presents a summary of the measure of fit parameters. The parameters have been separated into those given by the commonly assumed design parameters (approach 1) and those selected on the basis of the best fit principle (approach 2). It is apparent from this table that for a given pile class, R^2 values for design parameters selected on the basis of best fit principle are slightly higher than those for commonly assumed parameters in practice. An exception to this trend is the parameters for BC which are the same for both approaches. The results for BC are attributed to the fact that this class contains only two expanded base piles. Since the varying of N_c was applicable to expanded base piles only, only two cases were affected and this could not change the overall results. Nonetheless, on the basis of a general improvement of the R^2 values, the design parameters selected on the basis of best fit principle were considered as the most appropriate for the South African conditions. Accordingly results from the second approach were adopted for further analysis.

Table 5-15: Comparison of measure of fit parameters

Pile class	Approach 1		Approach 2	
	b	R 2	b	R 2
DNC	1.09	0.45	1.01	0.47
BNC	1.15	0.73	1.03	0.74
DC	1.02	0.69	1.14	0.71
BC	1.12	0.87	1.12	0.87

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5.5.4 Selected design parameters

The selected design parameters are summarised in table 5.16.

Table 5-16: Selected design parameters

Parameter	Value/correlation scheme
1. Driven piles in non-cohesive materials	
(a) φ	Peck, Hanson and Thornburn (1974)
(b) N_q	Berezantzev et.al (1961)
(c) K_s	1.3
(d) δ	0.75φ
2. Bored piles in non-cohesive materials	
(a) φ	Peck, Hansen and Thorburn (1974)
(b) Nq	Berezantzev et.al (1961), but φ reduced by 3
(c) Ks	0.8
(d) δ	φ
3. Driven piles in cohesive materials	
(a) Cu	Stoud (1989)
(b) α	Tomlison (1970)
(c) N_c	9 for plain piles 12 for expanded base piles
4. Bored piles in cohesive materials	
(a) Cu	Stroud (1989)
(b) α	Table 5.11
(c) N_c	9 for plain piles 12 for expanded base piles

5.6 Predicted capacities

The geotechnical design parameters determined as per table 5.16 were used as input into the relevant static formula to compute the predicted capacity for each case. The predicted capacities obtained are presented in table 5.17. The following observations were drawn from the detailed examination of table 5.17:

- Q_p ranges from 1101 – 5154 kN for driven piles in non-cohesive materials, 472 - 6754 kN for bored piles in non-cohesive materials, 556 – 5908 kN for driven piles in cohesive materials and 462 – 10172 kN for bored piles in cohesive materials.
- It is apparent from the above ranges of Q_p values that for a given soil type, the upper bound values for bored piles are higher than their respective values for driven piles. This is in line with the earlier observation that bored piles are normally installed in very stiff materials, hence the higher resistances.

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Table 5-17: Predicted capacities

Piles in non-cohesive materials				Piles in cohesive materials			
Driven		Bored		Driven		Bored	
Case	Q_p (kN)	Case	Q_p (kN)	Case	Q_p (kN)	Case	Q_p (kN)
1	3078	30	1321	63	2361	122	4807
2	1666	31	3783	64	626	123	6447
3	3389	32	2705	65	963	124	6521
4	2001	33	1084	66	1325	125	1398
5	2564	34	1202	67	1590	126	2062
6	2564	35	1202	68	2290	127	3280
7	1951	36	1131	69	3547	128	2545
8	1628	37	1202	70	3547	129	3040
9	1572	38	1178	71	1460	130	2235
10	2380	39	6754	72	2469	131	4179
11	2380	40	1287	73	2469	132	3658
12	2888	41	1145	74	2469	133	1196
13	1745	42	940	75	2469	134	2333
14	4013	43	679	76	3699	135	707
15	2109	44	1226	77	2069	136	1225
16	1592	45	1583	78	2093	137	975
17	1802	46	605	79	3100	138	3223
18	2768	47	472	80	3398	139	1221
19	3209	48	793	81	5908	140	834
20	2718	49	1060	82	2740	141	1444
21	1978	50	916	83	3186	142	2375
22	3648	51	1132	84	1325	143	1948
23	1080	52	2028	85	5908	144	1184
24	2136	53	3051	86	2770	145	462
25	2549	54	2028	87	1340	146	2392
26	1887	55	3051	88	1549	147	907
27	4766	56	2563	89	1340	148	1818
28	4797	57	2478	90	1250	149	5869
29	4968	58	2111	91	1069	150	1292
		59	2451	92	694	151	1292
		60	2974	93	1090	152	2110
		61	3644	94	2012	153	2110
		62	3739	95	697	154	2110
				96	877	155	2110
				97	556	156	2110
				98	800	157	1602
				99	800	158	8906
				100	2135	159	1156
				101	1265	160	2916
				102	3675	161	5088
				103	1265	162	5088
				104	2977	163	7648
				105	2580	164	4721
				106	3417	165	1955
				107	2986	166	4230
				108	1736	167	4196
				109	2851	168	6153
				110	2903	169	4207
				111	2903	170	10172
				112	2903	171	3294
				113	3588	172	6266
				114	3588	173	1901
				115	2621	174	3515
				116	706		
				117	3248		
				118	3248		
				119	3221		
				120	3221		
				121	1566		

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Chapter 6

STATISTICAL ANALYSIS OF THE MODEL FACTOR REALISATIONS

6.1 Introduction

For reliability calibration, the statistics (mean, standard deviation and coefficient of variation) as well as the type of distribution that best fit the data need to be determined for each random variable considered in the limit state function. Normally the various design variables can be classified into resistance and load related variables. Therefore load and resistance statistic are required. However, this study focuses on the resistance statistics as the load statistics are normally provided by loading codes. The required statistics are generated from the measured value of the random variable (resistance or load) and the predicted value yielded by the theoretical design model. Therefore the statistics are generally represented in terms of the the ratio of the measured to predicted values. This ratio is generally referred to as the bias. For this study the bias factor is defined as the ratio of the measured pile capacity over the predicted pile capacity.

6.2 Bias factors generated

Once the interpreted capacities were compiled from pile load tests as described in chapter 4, the predicted capacities were computed for the same pile type, soil conditions and installation methods using the geotechnical design parameters developed in chapter 5 as input in the static formula. The bias was then computed for each data case. A spread sheet was set to automate the computation of both the predicted capacity and the bias factor. The generated bias factors are presented in table 6.1. However, table 6.1 does not convey much information about the general characteristics of the generated bias factors, thereby necessitating a further statistical analysis of the presented data. The statistical characterisation of the bias factor realisations is presented in the subsequent sections.

6.3 The bias factor as a measure of various sources of uncertainties

In principle, the bias factor accounts for all the sources of uncertainties pointed out in Chapters 1 (i.e. model error, systematic error, inherent spatial variability, statistical error and load tests related errors). However due to the high level of expertise and experience of companies in charge of SPT tests and the pile load tests measurements errors are minimized. Also inherent spatial variability is minimized due to averaging effects along

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Table 6-1: Generated bias factors

Pile in non-cohesive materials				Piles in cohesive materials			
Driven piles		Bored piles		Driven piles		Bored piles	
Case	M	Case	M	Case	M	Case	M
1	0.926	30	1.041	63	1.394	122	0.999
2	1.530	31	1.216	64	1.086	123	0.636
3	0.797	32	1.109	65	1.205	124	1.104
4	0.950	33	0.968	66	1.358	125	0.544
5	1.423	34	1.144	67	1.761	126	1.115
6	0.811	35	1.148	68	1.485	127	0.945
7	0.692	36	0.928	69	1.015	128	1.320
8	1.401	37	1.019	70	0.578	129	0.872
9	1.399	38	0.713	71	1.733	130	0.805
10	1.722	39	0.800	72	1.276	131	0.885
11	1.630	40	0.971	73	1.008	132	0.801
12	0.597	41	1.310	74	0.931	133	1.422
13	1.777	42	1.032	75	0.867	134	0.815
14	0.847	43	0.796	76	1.644	135	0.778
15	1.458	44	0.490	77	1.329	136	0.959
16	1.131	45	0.742	78	1.042	137	1.107
17	1.332	46	1.322	79	1.307	138	1.086
18	0.795	47	0.933	80	1.530	139	1.015
19	0.678	48	0.789	81	0.948	140	0.990
20	0.956	49	0.967	82	1.014	141	1.004
21	0.748	50	1.267	83	1.444	142	1.263
22	0.603	51	1.325	84	1.796	143	1.258
23	1.213	52	0.991	85	1.185	144	1.208
24	1.592	53	1.967	86	1.191	145	1.104
25	1.099	54	0.592	87	1.306	146	1.505
26	0.822	55	1.524	88	1.485	147	1.268
27	1.259	56	0.659	89	1.388	148	1.271
28	1.001	57	0.654	90	1.200	149	1.448
29	1.057	58	0.781	91	1.544	150	0.952
		59	0.857	92	1.413	151	1.408
		60	0.901	93	1.211	152	1.223
		61	1.002	94	1.561	153	1.265
		62	1.498	95	1.378	154	1.322
				96	0.969	155	1.374
				97	0.990	156	1.991
				98	1.563	157	1.748
				99	1.313	158	1.078
				100	0.843	159	1.437
				101	0.932	160	1.646
				102	1.388	161	1.386
				103	1.059	162	1.160
				104	0.967	163	1.268
				105	1.008	164	1.483
				106	1.083	165	0.961
				107	1.055	166	1.284
				108	0.657	167	0.715
				109	1.263	168	1.016
				110	0.861	169	1.058
				111	1.516	170	1.278
				112	1.240	171	1.002
				113	0.697	172	0.768
				114	0.861	173	1.157
				115	0.595	174	1.508
				116	0.694		
				117	1.068		
				118	0.831		
				119	0.683		
				120	0.767		
				121	1.354		

the pile length. This leaves out model error as the predominant uncertainty reflected by the bias factor.

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That the bias factor mainly reflects model uncertainty is further reiterated by the following studies: Phoon and Kulhawy (2005) compared model pile test and full scale pile test results in terms of histograms and simple statistics. The model tests were conducted under controlled laboratory conditions and hence uncertainties arising from evaluation of soil parameters, construction variability and measurement errors associated with load tests were minimal. The results were such that the histograms and the simple statistics for the two sets of data were similar. Further more, p-values for equal medians statistical test showed that the null hypothesis of equal medians cannot be rejected at the customary 5% level of significance. It was concluded from these results that the ratio of measured to predicted capacity primarily represents model uncertainty. Further more, the study by Ronold and Bjerager (1992) mentioned in Chapter 1 demonstrated that model uncertainty exclusively is the most important source of uncertainty by a contribution to the total uncertainty by close to 100%. It was further concluded that other sources of uncertainties such as evaluation of soil parameters could just as well be neglected.

It is appreciated by many geotechnical engineers that in pile foundations, there is significant uncertainty about pile behaviour and calculation models available to describe this behaviour. For example, in the drafting of Eurocode 7 it was reckoned that the major uncertainty was not the strength of the in-situ ground but the way the construction would interact with it (Simpson, 2000). Therefore the partial factor required is largely a factor on the resistance model rather than on the strength of material. This led to the exception in Design Approach 1 that for piles factors should be applied to the resistance rather than on the materials as is the case for other geotechnical structures.

In accordance with the above discussion, in this study, the bias factor is considered to represent model uncertainty. Therefore the term model factor instead of bias factor will be used to refer to the ratio of the measured over predicted capacity.

6.4 Statistical analysis of the model factor data

The model factor realisations presented in table 6.1 are just a raw dataset representing a sample from the population of interest. Such raw data do not convey much information and therefore need to be reduced to manageable forms to facilitate its interpretation. In order to reduce the model factor realisation data to a manageable form, statistical analysis was

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carried out. The analysis comprised of (a) graphical representation by histograms, (b) outliers detection and correction of erroneous values, (c) using the corrected data to compute the sample moments (mean, standard deviations, skewness and kurtosis), (d) determining the appropriate distribution for the model factor and (e) investigation of correlation with underlying pile design parameters.

6.4.1 Histogram of the model factors

As already stated, the compiled model factors are just a set of raw data in a form of unorganized list of numbers. It is not easy to spot patterns and other useful information from such a data set. Therefore the data need to be organised and summarised in useful ways to make them more informative. A useful first step in the presentation of observed data is the graphical display. The large information content of pictures is aptly captured by the old adage that “a picture is worth a thousand words”. The same principle holds true for graphical displays as they can uncover hidden or at least not readily noticed features in the data. The most common method of graphical data presentation is the histogram.

The histogram condenses a set of data for easy visual comprehension of its characteristics. Visual inspection of the histogram often brings out the following features that are not immediately apparent from a given set of data:

- Immediate impression of the range of the data, its most frequently occurring values, and the degree to which it is scattered about the mean,
- Outlying observations which somehow do not fit the overall pattern of the data,
- The exhibition of two or more peaks which may imply an inhomogeneous mixture of data from different samples,
- Whether the data is symmetric or asymmetric,
- Indication of the underlying theoretical distribution for the data.

The fact that the histogram is generated by plotting the proportional frequency of observations lying within given numerical intervals renders its appearance to be influenced by the choice of the number of intervals or cells. Histograms from the same data would look very different if the number of cells is different. The produced shape affects the visual interpretation of the data. There are no universally applicable rules for determining the number of cells. However, a practical guidance has been reported in some geotechnical

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literature (e.g Smith, 1986; Baecher and Christian, 2003). In accordance with this guide, the number of intervals for a given data size is given by:

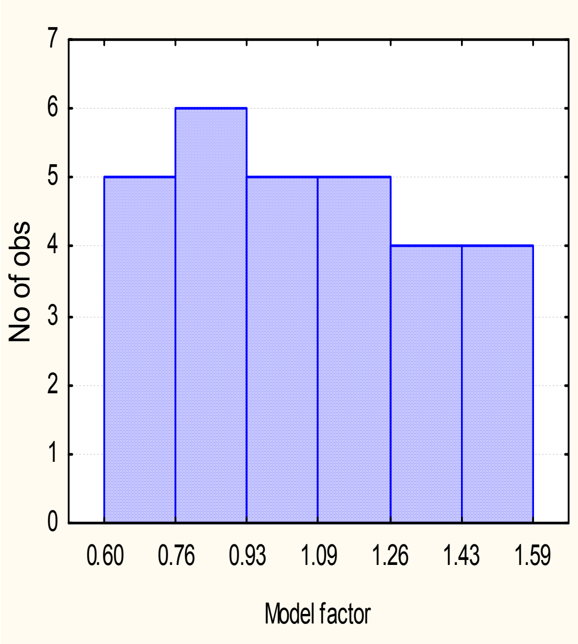
$$k = 1 + 3.3\log_{10} n$$

Where k is the number of cells; n is the number of data points.

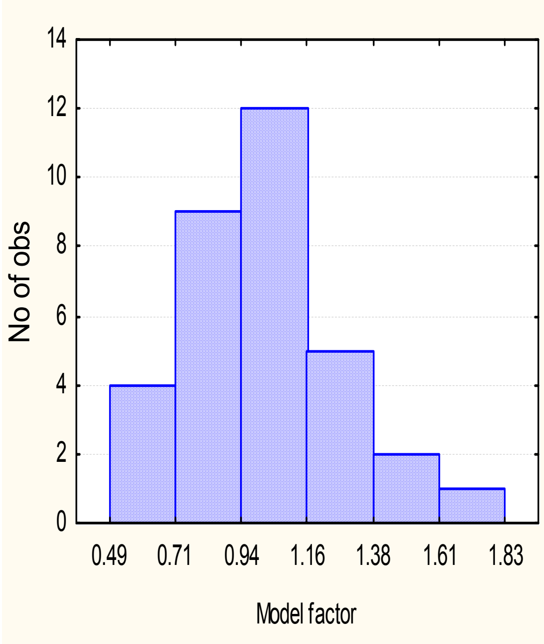
Nevertheless, experience and experimentation with different cell numbers usually provide the best guidance (Baecher and Christian, 2003). In this study the cell number for a given sample size was obtained by the combination of the above guide and experimentation. First trial cell number was by the guide and the histogram drawn. Next histograms with cell numbers slightly less and more than the guide cell number were drawn. The various histograms were then compared and the one with a well defined shape and a smooth variation of the observed frequencies was chosen. The resulting histograms are presented in figure 6.1. Visual examination of the histograms shows similar characteristics for the four histograms. These characteristics are:

- Most of the data points are clustered around the mean value.
- The histograms are unimodal, i.e. they have one point of concentration or a single peak indicating that the data is from the same source in terms of geological set up and piling practice.
- The data are not symmetrical about the peak frequency, suggesting that the underlying distribution for the data is not normal.
- There are no extreme values (outliers) in the data.
- The histograms are skewed to the right, i.e. they show a long right tail of relatively large values.
- The widths of the histograms are an indication of the scale of variability.

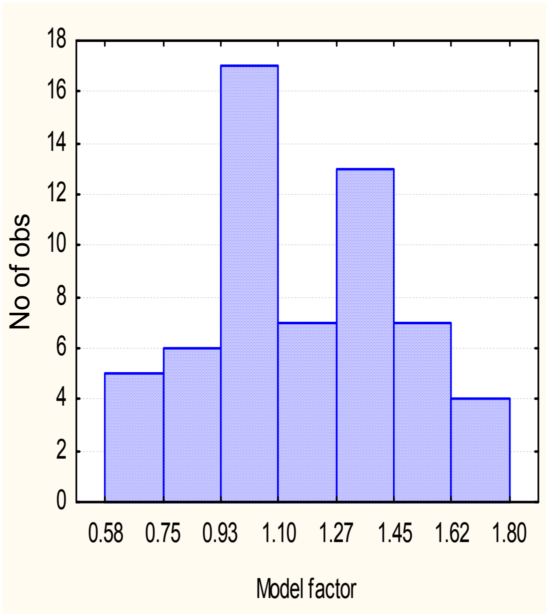
Given that the histograms would look very different if cell numbers used were different, over-interpretation of results should be avoided. For this reason the most reliable insight obtained from the histograms is immediate impression of the range of the data, its most frequently occurring values, and the degree to which it is scattered about the mean.



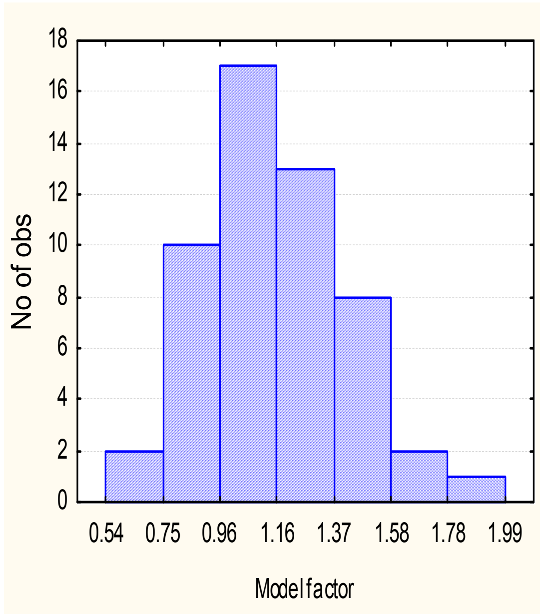
(a) Driven piles in cohesionless materials



(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 6.1: Histograms of the model factors

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6.4.2 Detection of data outliers

Data outliers are extreme (high or low) values that appear to deviate markedly from the main body of a data set. In general, outliers in data are attributed to human error, instrument error and natural deviations in populations. There is also a possibility that the outlier is a

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correct observation representing the true state of nature. In the model factors data considered in this study, the most likely causes of outliers include:

- Pile load tests related errors (inaccuracy in measuring devices calibration or data recording);
- SPT blow count related errors due to equipment or procedure;
- Data entry errors in processing pile load test results to determine the interpreted capacities;
- Data entry errors in processing the measured SPT N-values to determine design geotechnical properties.

The presence of outliers may greatly influence any calculated statistics leading to biased results. For instance, they may increase the variability of a sample and decrease the sensitivity of subsequent statistical tests (McBean and Rovers, 1998). Therefore prior to further numerical treatment of samples and application of statistical techniques for assessing the parameters of the population, it is absolutely imperative to identify extreme values and correct erroneous ones.

A number of procedures have been developed to detect outliers. The procedures can be divided into univariate and multivariate approaches. In a univariate approach, screening data for outliers is carried out on each variable while in the multivariate approach, variables are considered simultaneously. Since there may be some correlation between the variables, the multivariate approach is considered to be statistically correct as it accounts for the correlation (Robinson et.al, 2005). However in this study, both univariate and multivariate approaches will be employed to detect outliers in the data.

6.4.2.1 Univariate approach

With this approach, the ratio of the interpreted capacity to predicted capacity (model factor) is the only variable considered. Two methods were used to detect outliers in the model factor data set. The methods include (i) sample z-score approach and (ii) box plot

6.4.2.1.1 Sample z-score method

The z score is a measure of the number of standard deviations that an observation is above or below the mean. A positive z-score indicates that the observation is above the mean while

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a negative z-score that the observation is below the mean. The z-score of an observation in a given data set is given by the expression:

$$z = \frac{x - \bar{x}}{s}$$

Where: x = original data value; $\bar{x}$ = the sample mean; s = the sample standard deviation; z = the z-score corresponding to x.

According to Chebychev's rule, in any distribution, the proportion of scores between the mean and k standard deviation contains at least $1-1/k^2$ scores. This rule gives at least 75% of the scores between the mean and two standard deviations (± 2s), and 89 % of the scores would lie between the mean and three standard deviations (± 3s). Another well known data distribution rule is the Empirical Rule applicable to normally distributed data. According to the empirical rule, approximately 68% of the z-scores reside between mean and ± 1s, approximately 95% of the scores resides between mean and ± 2s, and approximately 99% of the scores reside between mean and ± 3s. Both these rules have led to the general expectation that almost all the observations in a data set will have z-score less than 3 in absolute value. This implies that all the observation will fall within the interval $(\bar{x} - 3s,\ \bar{x} + 3s)$. Therefore the observation with z-score greater than ± 3 is considered an outlier.

The above principles were applied to the data. First the z-score for each data point was determined. The z-scores were then plotted against the model factors (original data values). The plots of z-score versus model factor are presented in figure 6.2. Examination of figure 6.2 shows that:

- For driven piles in non-cohesive materials (fig. 6.2 (a)), there are no data point with a z-score of greater than 3. Therefore there are no outliers in this data set.

- For bored piles in non-cohesive materials (fig.6.2 (b)), there is one data point plotting above a z-score of 3. The data point is for pile case number 55 with a model factor of 2.55.
- For the driven piles in cohesive materials (fig. 6.2 (c)), there are no data points with z-scores of greater than 3.
- For the bored piles in cohesive materials (fig. 6.2 (d)), there are also no data points with z-scores of greater than 3.

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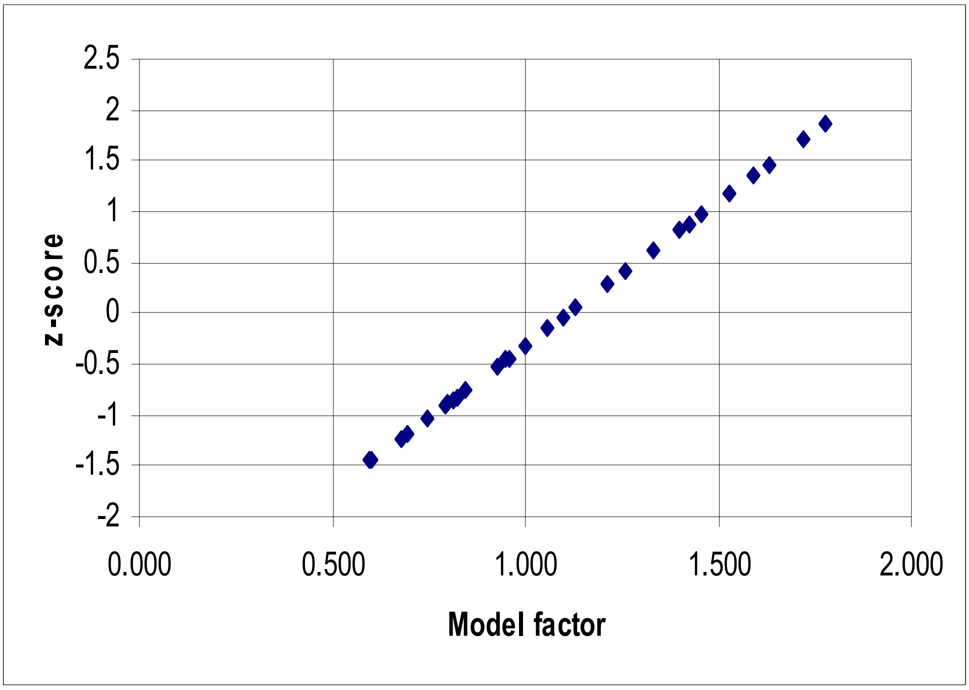
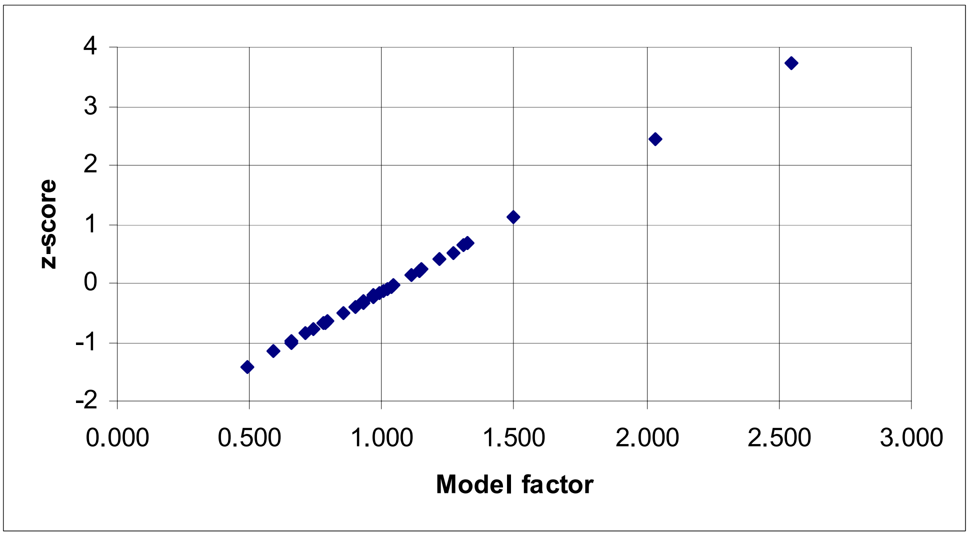
Based on the examination of the z-scores for the four data sets, it can be concluded that there are no outliers in the data sets for driven piles in non-cohesive materials, driven piles in cohesive materials and bored piles in cohesive materials. However for bored piles in non-cohesive materials, there is one outlier in the data set.



(a) Driven piles in non cohesive materials



(b) Bored piles in non-cohesive materials

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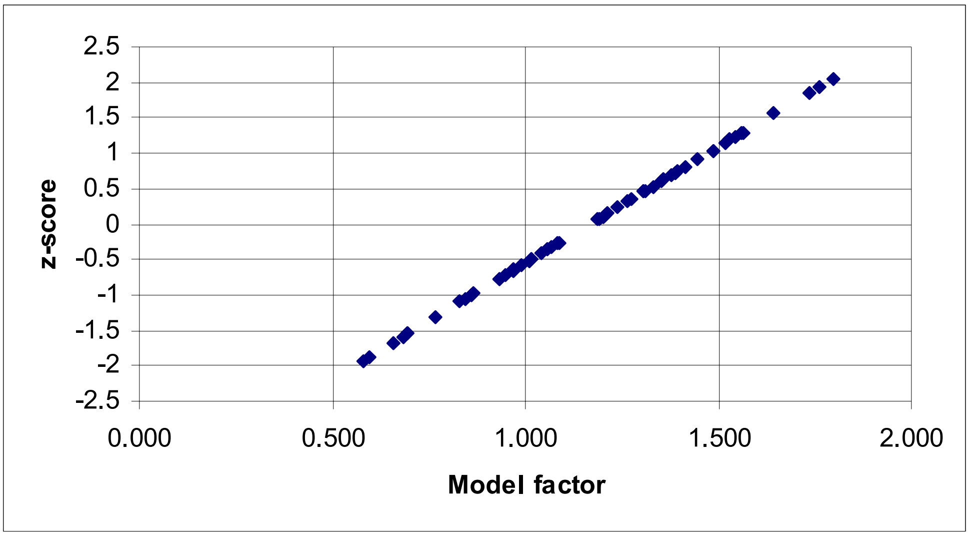
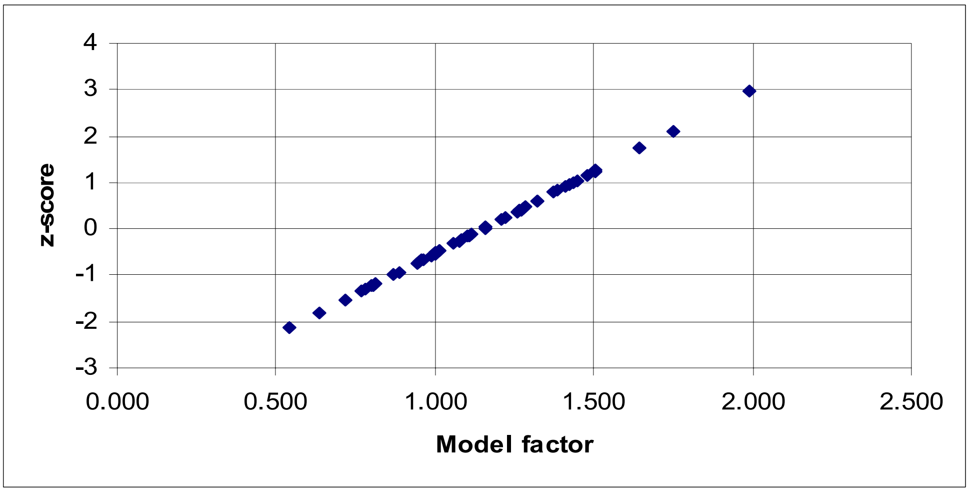


(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 6.2: Z-scores vs model factor plots

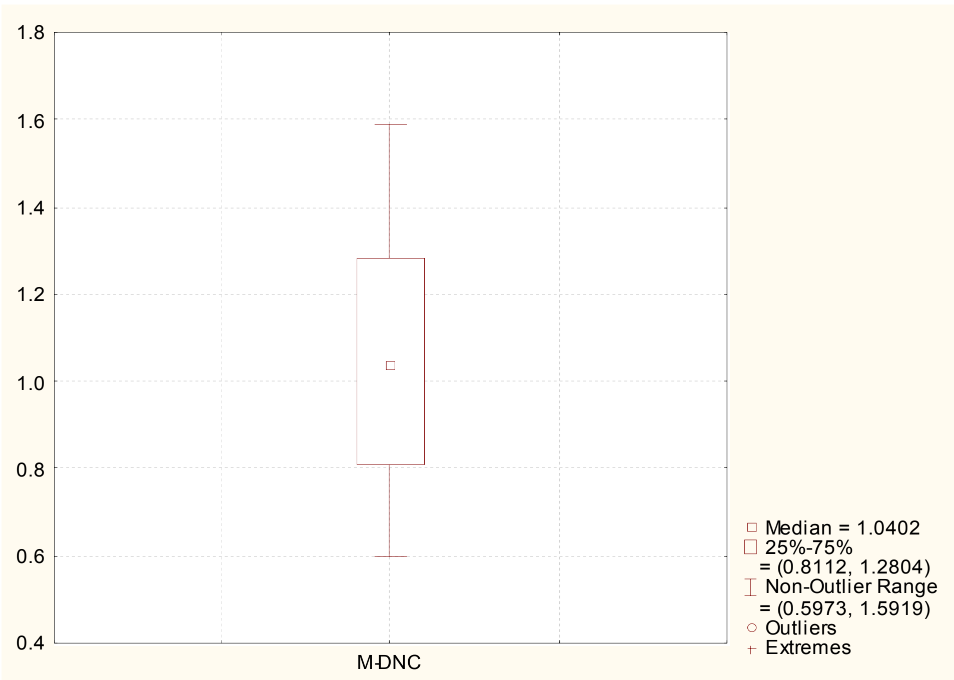
6.4.2.1.2 Box plot method

The box plot method is a more formalised statistical procedure for detecting outliers in a data set. A box plot displays a 5-number summary in a graphical form. The 5-number summary consists of; the most extreme values in the data set (the maximum and minimum values), the lower and upper quartiles, and the median. These values are presented together and ordered from lowest to highest: minimum value, lower quartile, median value, upper quartile, and largest value. Each of these values describe a specific part of a data set: the median identifies the centre of a data set; the upper and lower quartiles span the middle half of a data set; and the highest and lowest observations provide additional information about the actual dispersion of the data.

In using the box plot to identify outliers in the data set, the inter-quartile range (IQR) is required. The inter-quartile range is the difference between the upper quartile and the lower quartile. Any observation that is more than 1.5 IQR beyond the upper and lower quartiles is regarded as an outlier. In this study the programme STATISTICA was used to construct the box plots for the four data sets. The resulting box plots are presented in figure 6.3.

In figure 6.3, the vertical axis represents the response variable, which in this case is the model factor. To the right of each box plot is a legend explaining the meaning of the various symbols used to represent the data summaries. Visual inspection of figure 6.3 for outliers shows the following results:

- For driven piles in non-cohesive materials (fig. 6.3 (a)), there are no outliers in the data set.
- For bored piles in non-cohesive materials (fig. 6.3 (b), there are two data point marked as outlier or extreme value. The data points correspond to pile cases number 53 and 55.
- For driven piles in cohesive materials (fig. 6.3 (c)), there are no data points marked as outliers.
- For bored piles in cohesive materials (fig. 6.3 (d)), two data points have been tagged as an outlier or extreme value. These data points are again pile cases number 156 and 158.



(a) Driven piles in non-cohesive materials

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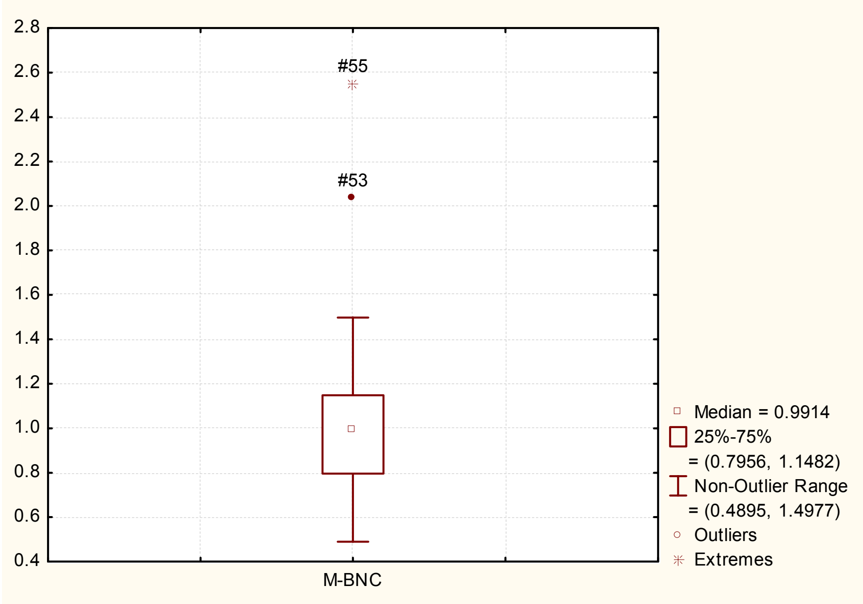
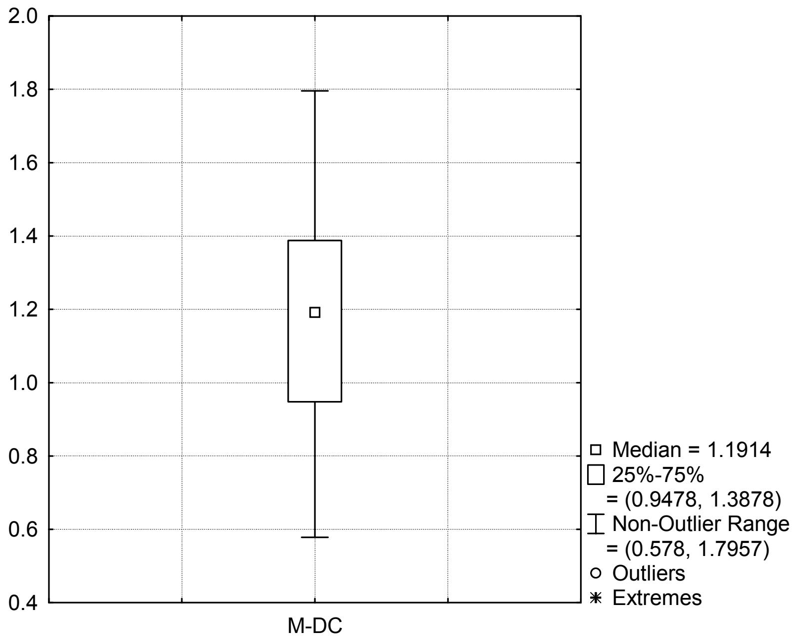
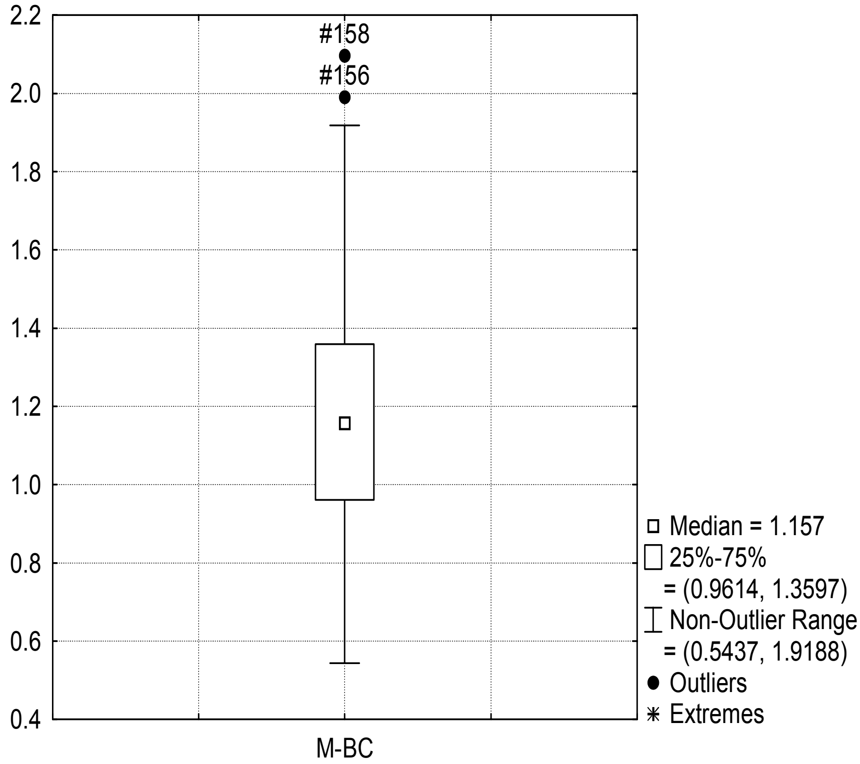


(b) Bored piles in non-cohesive materials



(c) Driven piles cohesive materials



(d) Bored piles in cohesive materials

Figure 6.3: Box plots of model factors

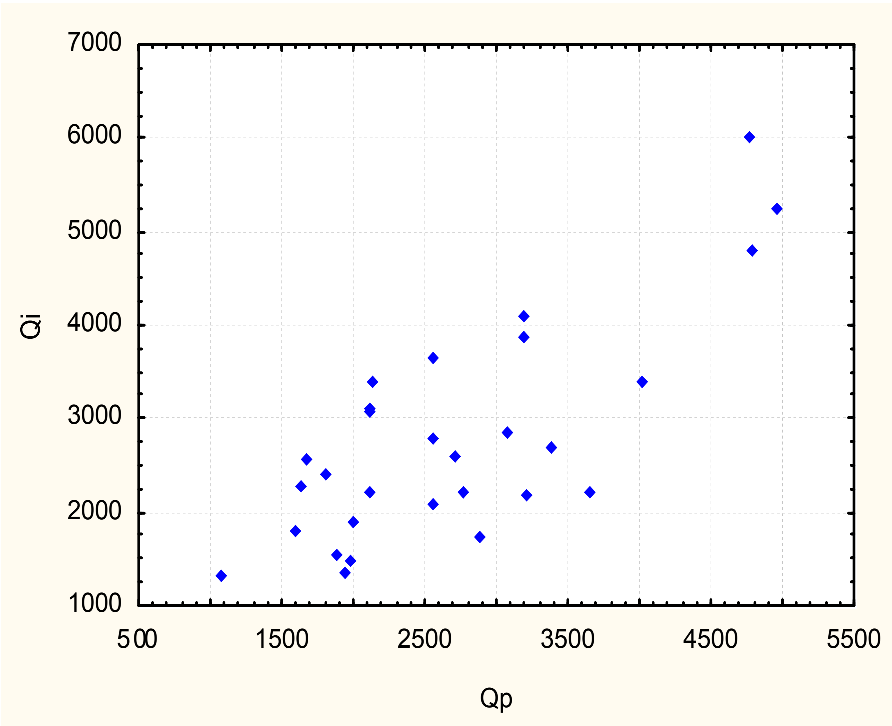
6.4.2.2 Multivariate approach

The main variables in the computation of model factor realisations are the predicted (Q_p) and interpreted (Q_i) capacities. Outliers in either of these two variables affect the calculated model factor. Based on correlative behaviour of the variables, some results with inconsistent behaviour, such as lying a significant distance from the general trend of the data is indicative that such values may be outliers. The multivariate approach adopted was limited to scatter plots of Q_i vs Q_p. The scatter plots of Q_i vs Q_p are presented in figure 6.4. Inspection of figure 6.4 shows the following:

- For driven piles in cohesionless materials (fig. 6.4 (a)), there are no data points that deviate from the general trend and therefore there are no outliers in the data. The results are consistent with the results from the z-score and Box plot approaches.
- For bored piles in non-cohesive materials, (fig. 6.4 (b)) there appear to be two observation that do not follow the general trend and therefore regarded as potential outliers. The data points correspond to pile cases 53 and 55. The pile cases were also detected by the Box plot.

- For driven piles in cohesive materials, (fig. 6.4 (c)) there are no data points that appear to deviate markedly from the main body of a data set. The results agree with the results obtained by the Box plot.
- For bored piles in cohesive materials, (fig. 6.4 (d)), pile cases 158 and 170 appear to be markedly different from the others. Case 158 was also detected by the box plot. Case 156, although tagged as an outlier by the Box plot seems to follow the general trend of the data set.

Other possible statistical approaches for detecting outliers include residuals Vs dependent variable (Q_i) and Residuals Vs Independent variable (Q_p). However, in this study the three methods considered were deemed to be sufficient.



(a) Driven piles in non-cohesive materials

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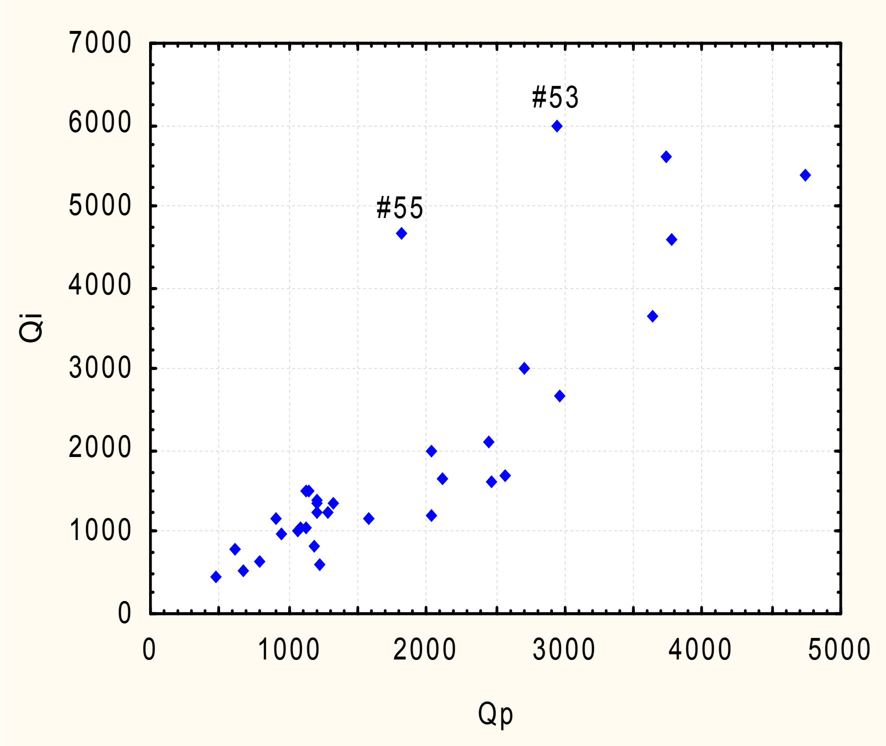
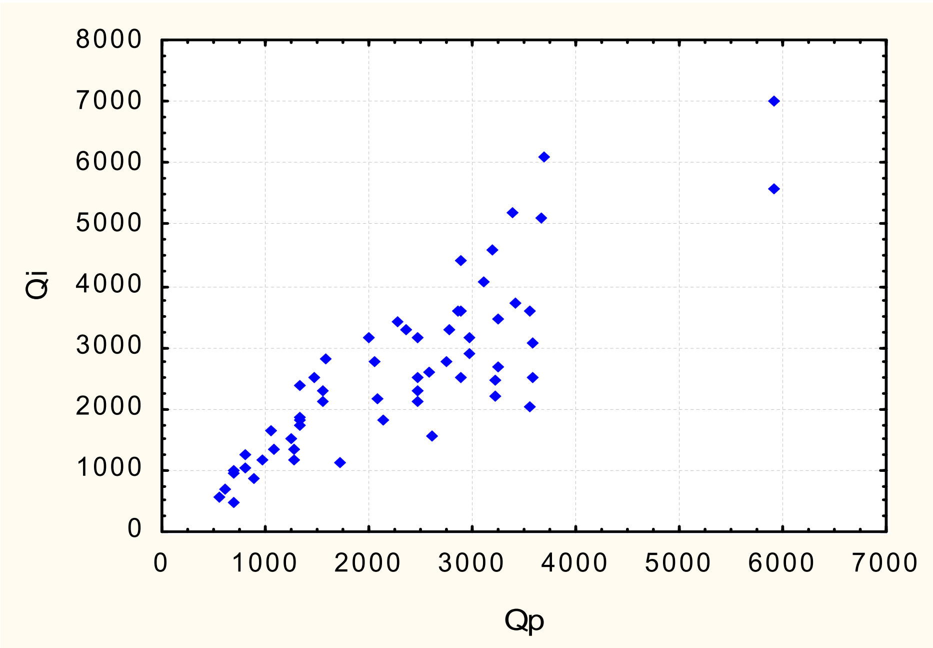
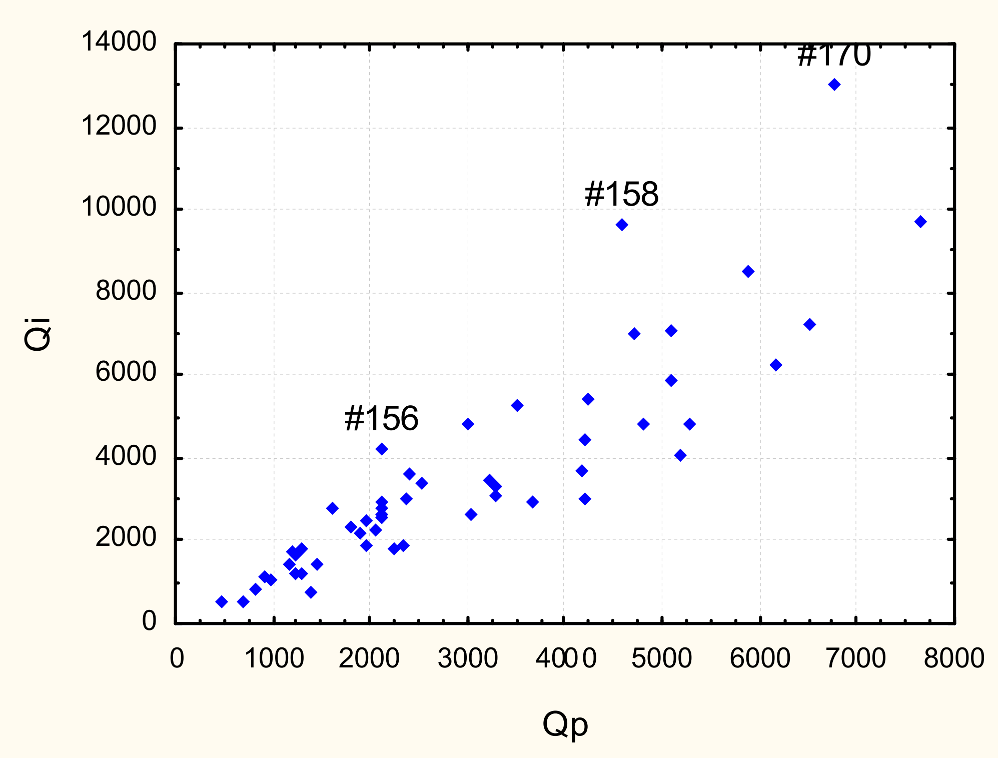


(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles cohesive materials

Figure 6.4: Scatter plots of Q_i vs Q_p

6.4.2.3 Correction to the identified outliers

A total of five observations were detected as potential outliers (i.e cases 53, 55, 156, 158 and 170). It is incorrect to automatically delete a data point once it is identified as a statistical outlier (Robinson et.al, 2005). It is only after examining why a data point is an outlier (i.e. carefully considering all factors) that a data point can be deleted. Accordingly the three data points identified as outliers were carefully examined by double checking the processes of determination of interpreted capacities and computation of predicted capacities. This entailed going back to the original data (pile testing records and derivation of soil design parameters) and checking for recording and computational errors. Following this procedure the corrections were as follows:

- Cases 53 and 55: Examination of records for these cases showed that an uncommon pile installation practice was employed. The steel piles were installed in predrilled holes and then grouted. The strength of the grout surrounding the piles contributed to the high resistance and hence the higher interpreted capacities. Therefore these data points were valid and no corrections were applied to them. However, since the

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installation procedure for these piles deviates from the normal practice, they represent a different population. These data points were therefore regarded as genuine outliers and were deleted from the data set.

- Case 156: This case was detected by the univariate approaches. However, on the scatter plots of Qi vs Qp, these data points seem to fit into the general pattern of the data sets (Fig. 6.4d). Therefore no correction was necessary for this data point.
- Case 158: No mistake was found with the determination of the interpreted capacity. However some details regarding the foundation materials were missed leading to incorrect base and shaft resistances. Revisiting the data pertaining to this test pile case revealed that the pile had a 2m socket in very soft rock followed by another 2m socket and end bearing in soft rock. In the original computation of the predicted capacity, only socket friction in very soft rock was considered leading to a smaller shaft resistance. Also the end bearing was wrongly taken to be in very soft rock. In general parameters for very soft rock were used in the computation of the predicted capacity. After correcting the rock design parameters to the appropriates ones (soft rock), the predicted capacity increased from 4580 kN to 8906 kN.
- Case 170: Examination of original data revealed a pile diameter of 600 was used in the computation of the predicted capacity thereby giving a smaller capacity of 6776 kN. The correct test pile diameter was 750 mm. With the correct test pile diameter the predicted capacity increased to 10172 kN. Correcting the predicted capacity made the data point for case 170 to fall in the general pattern of the data

6.4.3 Summary statistics

After the detection of outliers and correction of erroneous observations, the next step was to summarise the data numerically. Although graphical presentation condenses a set of data for easy visual comprehension of its general characteristics, numerical sample characteristics are required for calculations, statistical testing, and inferring the population parameters. These are quantities used to describe the salient features of the sample. The four main sample characteristics most commonly used in practical applications are:

- Mean (m)
- Standard deviation (s)
- Skewness
- Kurtosis

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These sample characteristics are also known as sample moments and summary statistics. The mean is the first moment about the origin and is correctly interpreted as the centre of gravity of the frequency distribution of the data along the x-axis. It describes the centre around which the observations in the data are distributed. The mean of a set of n data x = ($x_{i,}$ …, x_n) is given by:

$$m = \frac{1}{n}\sum_{i=1}^{n} x_i \qquad [6.1]$$

in which n is the sample size.

The standard deviation is a measure of variation and describes the dispersion of the data (i.e. how far the observations are from the centre). It is computed as:

$$s_x = \frac{\sum (x_i - \bar{x})^2}{n-1} \qquad [6.2]$$

where $\bar{x}$ is the sample mean and n is the sample size.

The skewness and the kurtosis provide hints on the shape of the underlying distribution for the data. In this regard, the skewness measures the asymmetry of the data distribution while the kurtosis is a measure of whether the data are peaked or flat relative to a normal distribution. In accordance with STATISTICA, the two parameters are computed from the following expressions:

$$\text{Skewness} = \frac{n * M_3}{[(n-1)*(n-2)*s^3]} \qquad [6.3]$$

where:

$M_3 = \sum (x_i - \bar{x})^3$

s = the standard deviation

n = the valid number of cases

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$$\text{Kurtosis} = \frac{[n*(n+1)*M_4 - 3*M_2*M_2*(n-1)]}{[(n-1)*(n-2)*(n-3)*s^4]} \qquad [6.4]$$

where:

$$M_j = \sum (x_i - \bar{x})^j$$

n = the valid number of cases

s = the standard deviation

The summary statistics for the model factors are presented in table 6.2. The key statistics are the mean (m) and the standard deviation (s). In addition to the measure of centrality and dispersion, in this study the mean and standard deviation of the bias factor were considered as indicators of the accuracy and precision of the predication method. An accurate and precise method gives $m_M = 1$ and $s_M = 0$ respectively, which means that for each pile case, the predicted pile capacity equals to the measured capacity (an ideal case). However, due to uncertainties mentioned earlier, the results of an ideal case can not be attained in practice. Therefore in reality, the method is better when m_M is close to 1 and s_M is close to 0. In general when $m_M > 1$, the predicted capacity is less than the interpreted capacity, which is conservative and safe where as when $m_M < 1$, the predicted capacity is greater than the interpreted capacity, which is unconservative and unsafe.

Table 6-2: Summary statistics for the model factor

Case	N	Mean	Std. Dev.	COV	Skewness	Kurtosis
DNC	29	1.06	0.30	0.28	0.171	-1.131
BNC	31	0.98	0.24	0.24	0.054	-0.368
DC	59	1.17	0.30	0.26	0.022	-0.713
BC	53	1.15	0.28	0.24	0.457	0.715

It is apparent fro from table 6.2, that the mean model factor is greater than one for DNC, DC and BC while for BNC, it is less than one. This implies that on average, the static formula is conservative by 6 % in DNC, 17% in DC, 15 % in BC, and unconservative by 2% in BNC. It can be concluded from these results that the use of the static formula with geotechnical parameters recommended in this study, yields slightly conservative theoretical capacities for DNC, DC, and BC. Conversely for DNC the approach is marginally unconservative.

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A comparison of the standard deviations or coefficient of variations, for the four cases indicates small differences. However there seem to be a distinct trend that is influenced by the pile installation method. In this regard, driven piles depict higher variability compared to bored piles irrespective of materials type. Further more, for a given installation method (driven or bored) the variability in non-cohesive materials is higher than that in cohesive materials. This is attributed to the fact that in cohesive materials the undrained shear strength derived from the SPT measurement is directly used in the computation of pile capacity while in non-cohesive materials, the angle of friction obtained from the SPT measurement is not directly used. Instead other parameters such as Nq are obtained from the derived angle of friction on the basis of empirical correlation and thus introducing some additional uncertainties. The trend that model factors from cohesionless materials seem to be more variable than those from cohesive materials has also been reported by Briaud and Tucker (1988) and Phoon and Kulhawy (2005a). Phoon and Kulhawy attributed the high variability exhibited by model factors in cohesionless materials to the correlation of the model factors to the nominal side or tip shear.

In general the mean model factors for the cases considered in this study are not considerably different from unity. Model factors of close to unity have also been reported in other studies based on other different other pile capacity calculation models (e.g. Li et al, 1993; Lacasse and Nadim, 1996; Tuomi and Roth, 1995). However, the coefficient of variations obtained in this study, although within the range of values reported by other studies seem to be on the lower side. For example Lo et al (1995) reported that 50% of the model factor data set studied yielded a coefficient of variation in excess of 0.4 while Phoon and Kulhawy (2005) reported values ranging from 0.30 to 0.40. The lower coefficients of variations are attributed to procedure used to determine the soil design parameters, especially the parametric study approach described in Chapter 5.

A further characterization of the data through summary statistics includes skewness and kurtosis. In table 6.2, the skewness for all pile cases are positive, suggesting that the corresponding data is right skewed (i.e. has a long right tail of relatively large observations). That the underlying distributions for the data seem to be skewed to the right was also highlighted by the histograms.

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According to the explanation provided in STATISTICA, a skewness that is clearly different from zero indicates an asymmetrical distribution. However no guidance have been provided as to what constitutes "clearly different". McBean and Rover (1998) provide a further guideline as follows; a distribution is considered highly skewed if the absolute value of skewness is greater than one; a distribution with skewness value from 0.5 to 1 is considered moderately skewed and one with skewness value of 0 to 0.5 is essentially symmetrical. In accordance with this guideline, the data for all pile cases can be considered as symmetrical. A symmetrical distribution suggests that the data is normally distributed. In any case, formal normality tests will be carried out to determine whether the underlying distributions for the data sets are normal or not.

The values of kurtosis from table 6.2 are negative for driven piles in cohesionless materials, bored piles in non-cohesive materials, and driven piles in cohesive materials. However for bored piles in cohesive materials the numerical value of the kurtosis is positive. Positive kurtosis indicates a "peaked" distribution and negative kurtosis indicates a "flat" distribution. A peaked distribution implies that most of the values of the distribution are concentrated around the mean whereas a flat distribution denotes that the values are more evenly spread throughout the range. Concerning the magnitudes of the values, generally data sets with high kurtosis tend to have a distinct peak near the mean, decline rather rapidly, and have long tails. Data sets with low kurtosis on the other hand tend to have a flat top near the mean rather than a sharp peak.

6.4.4 Probabilistic model for the model factor

The theory of reliability is based on a general principle that the basic variables (actions, material properties and geometric data) are considered as random variables having appropriate types of distribution. One of the key objectives of the statistical data analysis is to determine the most appropriate theoretical distribution function for the data. This is the governing probability distribution for the random process under consideration and therefore extends beyond the available sample (i.e. the distribution of the entire population). Once the probability distribution function is known, inferences based on the known statistical properties of the distribution can be made. Reaching conclusions that extend beyond the available sample data is the main point of sampling. It is the population that is of more interest than the sample itself. The process of mapping from sample statistics to population parameters starts with the selection of the suitable distribution for the data.

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A number of probabilistic models of random variable are available. Certain types of variables follow specific distributions. However, the probabilistic characteristics of a given random variable are sometimes difficult to discern because the appropriate model to describe these characteristics is not readily amenable to theoretical formulation (Ang and Tang, 1975). Therefore in practice, data are often assumed to come from a particular distribution. The assumption that a sample follows a particular distribution is often based on the physical considerations about the nature of the process under study. For example processes whose values are determined by an infinite number of independent random events will be distributed following the normal distribution. On the other hand, if the values of the physical process are the result of very rare events, then the variable will be distributed according to the Poisson distribution (Ang and Tang, 1975). Also there may be historical or theoretical reasons that favour a certain distribution for the phenomenon under consideration. For example past data may have consistently fit a known distribution, or theory may predict that the underlying population should be of a specific form.

There are situations where among a number of contending distributions, there is no single one that is preferred on the basis of underlying characteristics of a given phenomenon. Under such circumstances, the distribution is determined empirically from the available observational data. The technique entails fitting the observed distribution to a theoretical distribution by comparing the frequencies observed in the data to the expected frequencies of the theoretical distribution. The procedure is however curtailed by the limitation of the sample size to generate a well defined observed distribution.

Whether the distribution is assumed or derived empirically, it must be verified in the light of data using statistical tests known as goodness of fit tests for distribution. Goodness-of-fit tests indicate whether or not it is reasonable to assume that a random sample comes from a specific distribution.

From theoretical and practical point of view, the most commonly applied distributions to describe actions, materials properties and geometric data are the normal and lognormal distributions (Holicky, 2005). Therefore the candidate distributions for the model factor are the normal and lognormal distributions. In the Joint Committee on Structural Safety (JCSS) Probabilistic Model Code, model factors are modeled by the lognormal distribution. Further more, past studies (Phoon, 2005; Briaud and Tucker, 1988; Ronold and Bjerager, 1992; Titi

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and Abu-Farsakh, 1999, FHWA-H1-98-032, 2001; Rahman et.al, 2002) have indicated that the most suitable theoretical model for model factor realisations data is the lognormal distribution. Other considerations that suggest the lognormal distribution seems to be the most appropriate model include:

- Theoretically, the ratio of interpreted capacity to predicted capacity ranges from zero to infinity. This results in an asymmetric distribution with a zero lower bound and an infinite upper bound. The lognormal probability density function is often the most suitable theoretical model for such data as it is a continuous distribution with a zero lower bound and an infinite upper bound.
- As the logarithm of the value is normally distributed rather than the value itself, it provides a convenient model for random variables with relatively large cov for which an assumption of normality would imply a significant probability of negative values (Jones et al, 2002).
- In accordance with the central limit theorem, the distribution of products or ratios of random variables approaches the lognormal distribution as the number of random variables increases.

Based on the above considerations, a lognormal distribution was assumed as the most appropriate theoretical distribution for the model factor. This assumption was verified using goodness of fit statistical tests.

6.4.4.1 Verification of the assumption of lognormal distribution for the data

There are several techniques for verifying an assumed distribution. The techniques can be broadly classified into qualitative and quantitative approaches. The most commonly used qualitative approach is to construct a histogram and superimpose the assumed distribution on it. This approach suffers from an inability to distinguish between anomalies in shapes due to sampling fluctuations and theoretical considerations (Bowker and Lieberman, 1972). An alternative qualitative technique is to plot the data on a probability paper. Probability paper is a specialized graph paper with horizontal and vertical scales designed such that the cumulative frequencies of a particular form of distribution plot as straight lines. Graph papers possessing this property are available for several distributions and it is common practice to plot empirical data on the appropriate graph paper as a check for the form of the underlying distribution. This is often done for checking the assumption of normality.

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In general qualitative approaches greatly suffer from the lack of objective criteria for judging whether the data fit the assumed distribution and to what degree. As a solution to the above problem, quantitative techniques have been developed. Quantitative approaches are based on goodness of fit statistical tests. Well established statistical methods are used to test whether the difference between the empirical data frequencies and the assumed distributions are statistically significant or insignificant.

6.4.4.2 Goodness of-fit statistical tests

When a random variable X is lognormally distributed, its natural logarithm, ln(X) is normally distributed. Accordingly the natural logarithms of the model factors were determined and some normality tests supported by the available software were applied to the resulting data sets. The normality tests included Kolmogorov-Smirnov (K-S) test, Lilliefors test and Shapiro-Wilk's W test. Lilliefors test is a form of Kolmogorov-Smirnov test with the mean and standard deviation computed from the data. The interpretation of the results from the statistical significant tests is as follows:

Null hypothesis (H_0): the empirical distribution is similar to the assumed theoretical distribution (normal distribution).
Alternative hypothesis (H_1): the empirical distribution is not similar to the assumed theoretical distribution.

Results: A small p value ($p < 0.05$) means it's unlikely that the assumed distribution is correct and therefore the null hypothesis is rejected.

A big p value ($p \geq 0.05$) means that there is no evidence that the assumed distribution is not correct and therefore the null hypothesis is accepted.

Where the p value is a measure of fit and therefore the larger the p value the better the fit.

The results of the various tests are presented in table 6.3. The p-values for all the three tests are greater than 0.05 and therefore there is no evidence to reject the null hypothesis of normal distribution for the logarithms of the model factors of all the cases considered. Since the distribution of the natural logarithms of the bias factors is normal, it follows that the

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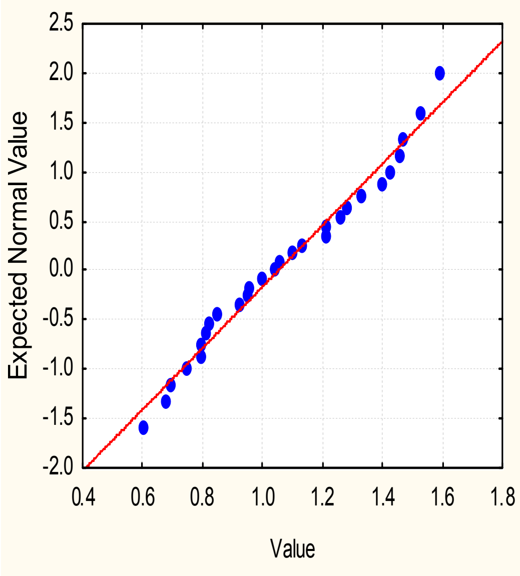
distribution of the bias factors is lognormal. It is thus verified that the lognormal distribution is an appropriate theoretical model for the bias factor. The results of the statistical tests are in agreement with that found by Lo et.al (1995, 1996). Lo et.al (1995, 1996) carried out the Kolmogorov-Smirnov test on model factor data sets and concluded that the model factor for most data sets could be well approximated by the lognormal distribution as compared to a normal distribution.

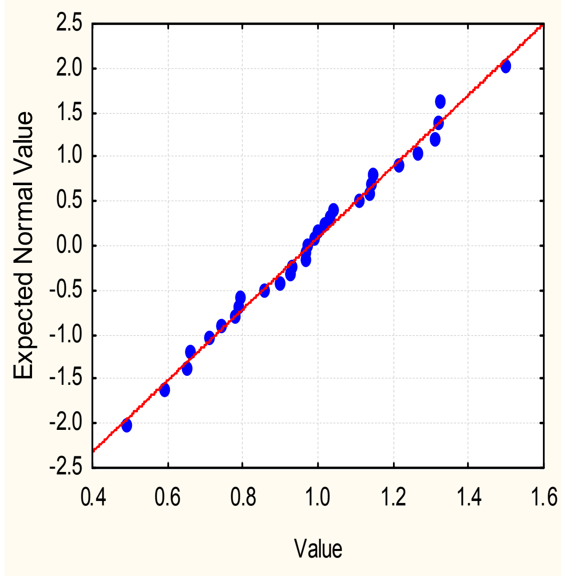
Table 6-3: Normality tests results

Variable	N	Normality tests				
		Kolmogorov-Smirnov test			Shapiro-Wilk's W test	
		Max D	K-S p	Lilliefors p	W	p
Cohesionless materials						
Driven piles	29	0.108	p > 0.20	p > 0 .20	0.957	0.27
Bored piles	31	0.073	p > 0.20	p > 0 .20	0.989	0.98
Cohesive materials						
Driven piles	59	0.079	p > 0 .20	p > 0 .20	0.981	0.51
Bored piles	53	0.072	p > 0.20	p > 0.20	0.982	0.59

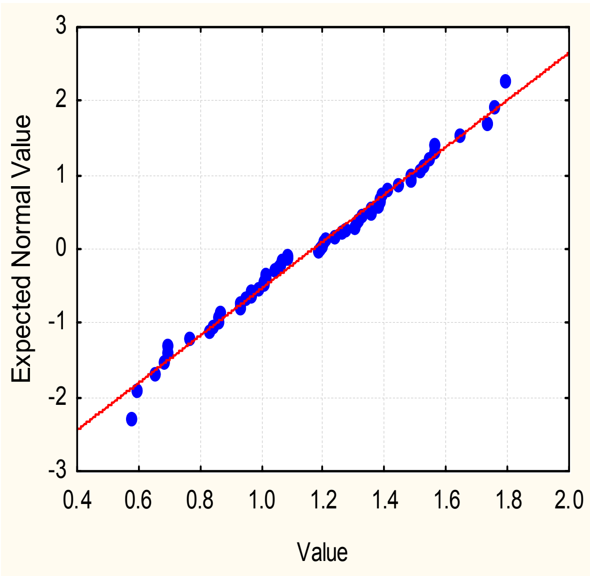
To check visually how well the assumed distribution (i.e. normal distribution) fits the data, probability-probability plots (P-P plots) were constructed. In the P-P plot, the observed cumulative distribution function is plotted against a theoretical cumulative distribution function. If the theoretical cumulative distribution function reasonably models the observed cumulative distribution function, the point pattern in this plot should be approximately linear. The P-P plots for the logarithms of the bias factors are presented in figures 6.5. Examination of these P-P plots shows a linear point pattern and therefore it can be concluded that the assumed distribution is the appropriate model for the data.



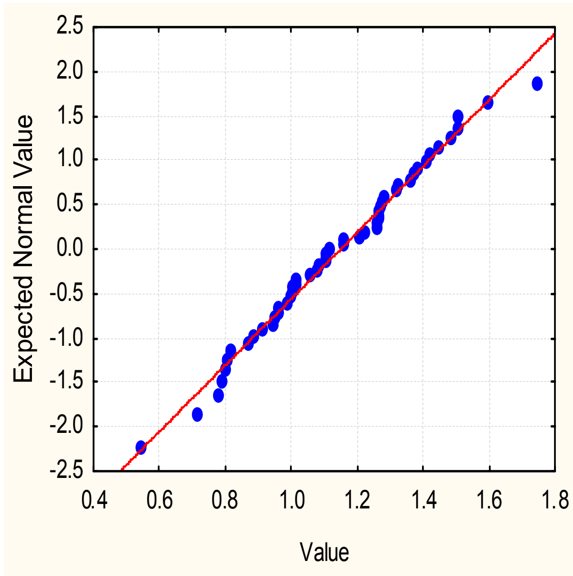
(a) Driven piles in non-cohesive materials



(b) Bored piles in non-cohesive materials



(c)Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 6.5: P-P plots for model factors

6.4.5 Investigation of correlation with underlying factors

Although the mean and standard deviation presented in table 6.2 provide a useful data summary, they combine data in ways that mask information on trends in the data. When there is a strong correlation between the dependent and independent variables, part of the total variability of the dependent variable is explainable by its association with the independent variable. Accordingly if there are hidden correlations between the model factor and some pile design parameters, then part of the variability of the model factors presented

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in table 6.2 is explained by these design parameters. The presence of correlation between the model factor and deterministic variations in the database would indicate that:

- Generally a trend in the bias factor with a design parameter implies that the method does not fully take the effects of the parameter into account.
- The pile design model does not fully take into account such parameters or
- A random variable probabilistic model is the appropriate model for the parameter as the variation of a random variable is not explainable by deterministic variations in the database.

Reliability based design is based on the assumption of randomness of basic variables. Therefore it was necessary to check for correlation between the bias factor and various pile design parameters (pile length, pile diameter, soil properties, and the predicted capacity). This was accomplished by checking presence or absence of correlation between the bias factor and various pile design parameters (pile length, pile diameter, and soil properties).

The measure of the degree of association between variables is the correlation coefficient. The basic and most widely used type of correlation coefficient is Pearson r, also known as linear or product-moment correlations. The correlation can be negative or positive. When it is positive, the dependent variable tends to increase as the independent variable increases; when it is negative, the dependent variable tends to decrease as the independent variable increases. The numerical value of r lies between the limits -1 and +1. A high absolute value of r indicates a high degree of association whereas a small absolute value indicates a small degree of association. When the absolute value is 1, the relationship is said to be perfect and when it is zero, the variables are independent.

The Pearson product moment correlation coefficient is calculated is given by the following expression:

$$r = \frac{\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_{i=1}^{n}(x_i - \bar{x})^2 \sum_{i=1}^{n}(y_i - \bar{y})^2}} \qquad [6.5]$$

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Where x_i = the i[th] realisation of the dependent variable; $\bar{x}$ = the mean of all the realisations of the variable x; y_i = the i[th] realisation of independent variable y; $\bar{y}$ = the mean of all the realisations of the variable y.

An alternative to the parametric Pearson product moment correlation coefficient is a variety of non-parametric correlations. Generally non-parametric statistics have been specifically developed to process data from low quality data. Low quality in this context refers to small samples and unknown parameters (mean, standard deviation and distribution) of the variable. Accordingly the non-parametric correlations are most suitable when the sample sizes are small. The most popular of the non-parametric correlations is the Spearman rank order correlation.

The Spearman correlation (R) is calculated by replacing x_i and y_i in equation 6.1 by their ranks in the respective list of number (i.e the smallest number takes the value of 1, the next smallest takes the value of 2, and so forth). In calculating the Spearman correlation, equation 6.1 reduces to (Phoon and Kulhawy, 2005):

$$R = 1 - \frac{6\sum(s_i - t_i)^2}{n(n-1)(n-1)} \qquad [6.6]$$

in which s_i and t_i are the ranks of x_i and y_i respectively.

In this study both the Pearson and Spearman correlations were considered. However, in case of discrepancies between the results of the two correlations, the results of the Spearman correlation were used to reach the final decision. The preference of the Spearman correlation is based on the account of limited size of the database and unknown population parameters of the model factors. Further more, the Spearman correlation is less affected by outliers (Phoon and Kulhawy, 2005).

Whether the correlation is computed on the basis of the Pearson or Spearman correlation coefficient, the technical question that needs to be answered is "when is the numerical value of the correlation coefficient considered high and when is it low and negligible"? Several authors in various fields have suggested guidelines for the interpretation of the correlation

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coefficient. One such typical interpretation uses five easy rules of thumb (Franzblau, 1958) as follows:

- *r* ranging from 0 to about ±0.2 may be regarded as indicating no or negligible correlation.
- *r* ranging from about ±0.2 to ±0.4 may be regarded as indicating a low degree of correlation
- *r* ranging from about ±0.4 to ±0.6 may be regarded as indicating a moderate degree of correlation.
- *r* ranging from about ±0.6 to ±0.8 may be regarded as indicating a marked degree of correlation.
- *r* ranging from about ±0.8 to ±1 may be regarded as indicating a high correlation.

Other more recent scholars suggest that as a rule of thumb, a correlation coefficient of less than 0.3 indicate no or negligible correlation. However, Cohen (1988) observed that all such criteria are in some ways arbitrary and should not be observed too strictly. Generally the interpretation of the correlation coefficient depends on the context and purpose of the analysis and the field of research.

Other statisticians prefer the use of r^2. Technically r^2 is called the coefficient of determination and represents the proportion of shared variance between the variables. However even with this approach, the criterion for deciding what r^2 value is significant is still subjective.

In geotechnical reliability, Smith (1986) suggested the interpretation presented in table 6.4. Due to its relevancy to the current study, this is the interpretation adopted for the study.

Table 6-4: Practical significance of correlation coefficient

Range of r values	Interpretation
≤ ±0.2	Weak correlation and the variables can be assumed to be independent
±0.8 > r > ±0.2	Marked correlation between variables
≥ ±0.8	Strong correlation and the variables can be assumed to be completely depended

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Another widely used convention to determine the significance of the correlation is the concept of statistical significance tests. There is always a possibility that the computed correlation could have risen by chance and begs the question, how probable it is that a similar relation would be found if the experiment was replicated with other samples drawn from the same population? The doubt regarding the reliability of the computed correlation coefficient necessitates that the results be subjected to some statistical significance tests. The statistical significance of a result is an estimated measure of the degree to which it is "true" in the sense of "representative of the population". Most recent statistical softwares including STATISTICA give the results of the significance tests in terms of the p-value. When used to check the significance of the correlation, the p-value is interpreted as follows:

Null hypothesis (H_0): there is no correlation between the variables (indicative of statistical independence).

Alternative hypothesis (H_1): there is a correlation between the variables (statistical dependency).

Results: A small p value ($p < 0.05$) means that the null hypothesis is not valid and should be rejected.

A big p value ($p \geq 0.05$ indicates that there is no relationship and therefore the null hypothesis is valid.

It should be noted that the significance of the correlation as expressed by the p-value simply means that there is statistical evidence that it is unlikely that the computed correlation have occurred by chance. It does not necessarily mean that the correlation is large, important or significant in the usual sense of the word. Therefore through out the analysis, significance of correlation inferred on the basis of practical considerations (table 6.3) supersedes that reached on the basis of statistical significance.

The existence of a correlation was therefore assessed by (i) visual inspection of the scatter plots of the model factor versus various pile design parameters, (ii) the numerical value of the correlation coefficient and (iii) the statistical significance of the correlation expressed in terms of the p-value. The results of correlation assessment between M and pile design parameters are presented in the subsequent sections.

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6.4.5.1 Correlation with pile length

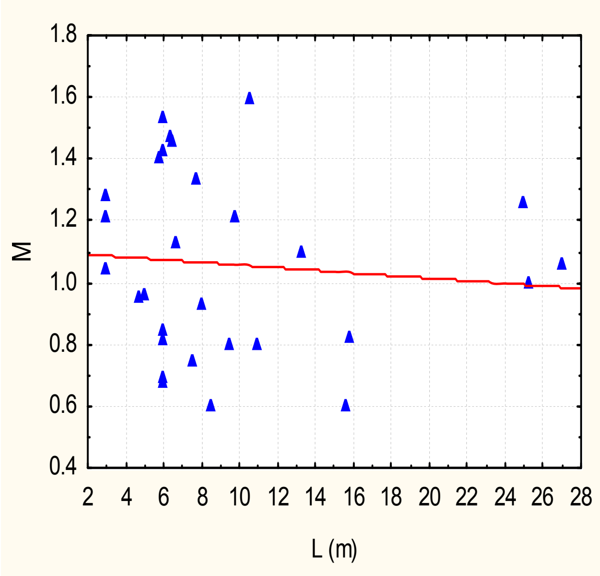
Figure 6.6 presents the scatter plots of the model factor (M) versus pile lengths while Table 6.5 presents the correlation coefficients together with their associated p-values. Visual inspection of the scatter plot shows

- A weak negative correlation for driven piles in non cohesive materials
- A weak positive correlation for bored piles in non-cohesive materials
- A significant negative correlation for driven piles in cohesive materials and
- A weak negative correlation for bored piles in cohesive materials

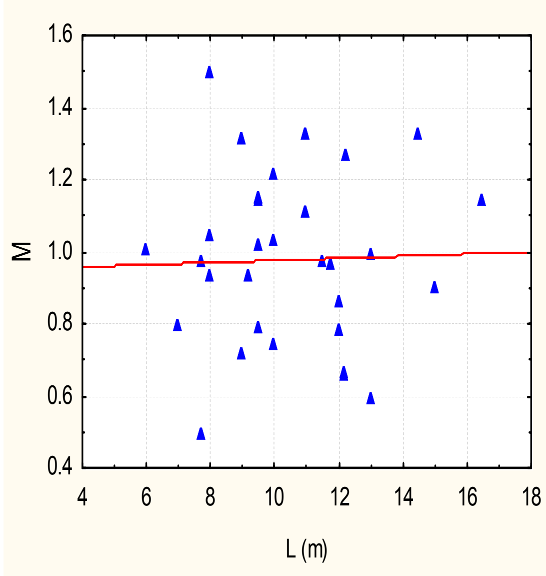
The quantitative analysis of the degree of the correlation is presented in table 6.5. The results indicate:

- Both Pearson and Spearman correlations show a positive correlation for BNC and negative correlation for the other pile classes.
- The correlation coefficients are less than 0.2, hence the parameters are considered to be independent.
- The p-values for all the cases are greater than 0.05, suggesting that at the customary significant level of 5%, there is no evidence to reject the null hypothesis of no correlation between the model factor and pile lengths.

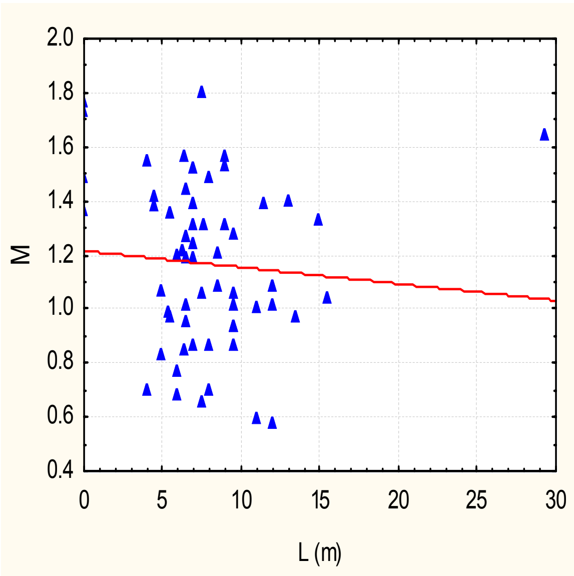
Both the practical significance and statistical significance of the correlation coefficients suggest that the degree of correlation is negligible and the parameters can be assumed to be independent. However, on the basis of the slope of the best fit line, visual inspection of the scatter plots for driven piles seem to suggest that the correlation is appreciable. This discrepancy is an indication that judgement based on visual inspection of the scatter plots can be misleading. Therefore more credence is given to the quantitative results and accordingly, it can be concluded that for all four cases there is no correlation between the model factor and pile lengths.



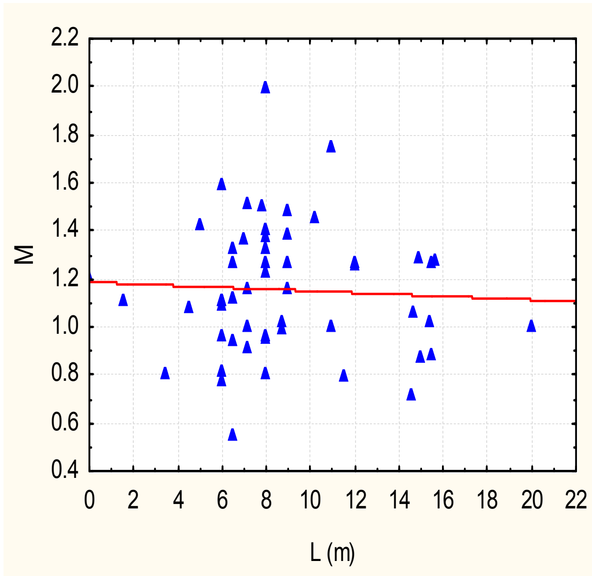
(a) Driven piles in non-cohesive materials



(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 6.6: Scatter plots of M vs pile length

Table 6-5: Correlation coefficients and associated p-values for M vs pile length

Case	N	Pearson correlation			Spearman rank correlation	
		r	r^2	p-level	R	p-level
DNC	29	-0.09	0.009	0.63	-0.15	0.45
BNC	31	0.03	0.001	0.87	0.01	0.97
DC	59	-0.09	0.008	0.51	-0.15	0.26
BC	53	-0.05	0.003	0.71	-0.05	0.73

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6.4.5.2 Correlation with pile diameter

For expanded base piles, the shaft diameter differs from the base diameter. The base diameter contributes to the base capacity while the shaft diameter contributes to the shaft capacity. Therefore the correlation is investigated for both base and shaft diameters.

6.4.5.2.1 Correlation with shaft diameter

The correlation analysis results are presented in figure 6.7 and table 6.6. The scatter plots of the model factor versus shaft diameters are presented in figure 6.7. Qualitative inspection of the scatter plots indicates:

- Weak negative correlation for driven piles in non-cohesive materials
- Marked positive correlation for bored pile in non-cohesive materials
- Appreciable negative correlation for driven piles in cohesive materials
- Low positive correlation for bored piles in cohesive materials

The quantitative measure of the strength and significance of the correlation are presented in table 6.6. The following observations can be drawn from this table:

- The signs of the correlation coefficients agree with the qualitative observations.
- Except for bored piles in non-cohesive materials, the correlation coefficients from both Pearson and Spearman approaches are less than 0.2 and therefore negligible. However, for bored piles in non-cohesive materials, the Spearman rank correlation is -0.215 which is marginally greater than 0.2. In accordance with the interpretation followed the correlation for bored piles in non-cohesive materials is practically significant. Therefore the qualitative observations are in complete agreement with the visual inspection results.
- The p-values for all the cases are greater than 0.05, indicating that the hypothesis that the correlation between the two parameters is not significant is valid. Even for bored piles in non-cohesive materials which on the basis of visual inspection of the scatter plot as well as the practical significance of the numerical value seemed to depict a significant correlation, the p-values are still greater than 0.05 (0.172 and 0.102 for Pearson and Spearman respectively).

Since more credibility is given to the practical significance of the correlation coefficient rather than to its statistical significance, it can be concluded that the correlation for bored

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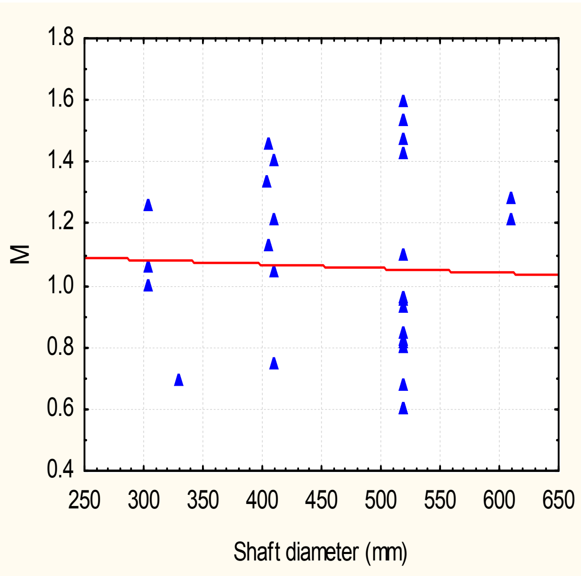
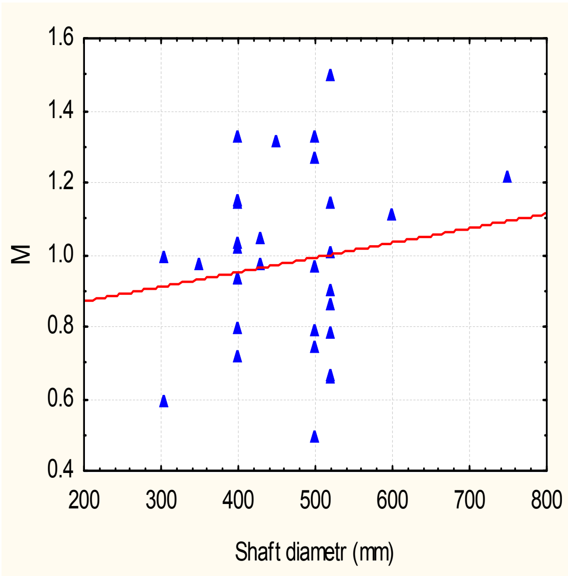
piles in non-cohesive materials is significant while that for the other three cases is insignificant.

Table 6-6: Correlation coefficients and associated p-values for M vs pile shaft diameter

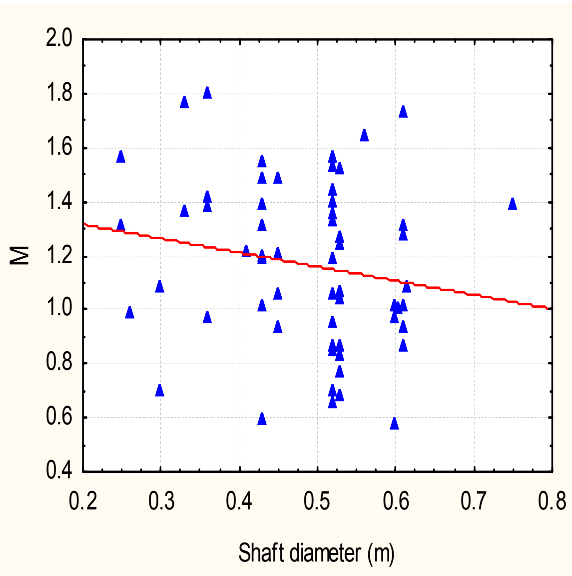
Case	N	Pearson correlation			Spearman rank correlation	
		r	r^2	p-level	R	p-level
DNC	29	-0.04	0.002	0.83	-0.09	0.63
BNC	31	0.15	0.023	0.42	0.04	0.84
DC	59	-0.18	0.033	0.17	-0.22	0.10
BC	53	0.06	0.004	0.67	0.05	0.73



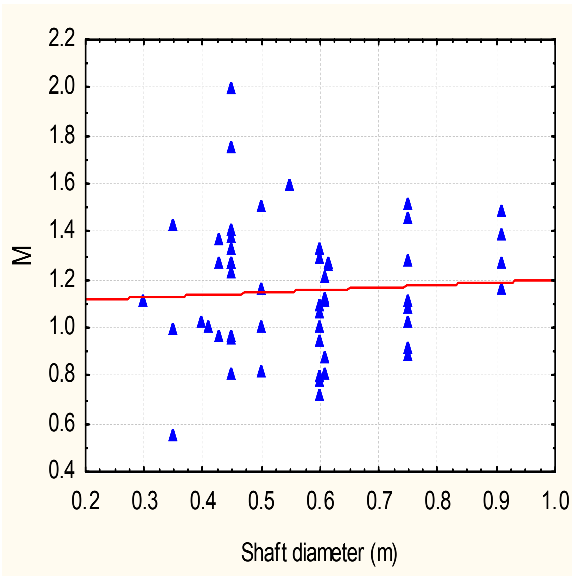
a) Driven piles in non-cohesive materials



(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 6.7: Scatter plots of M vs shaft diameter

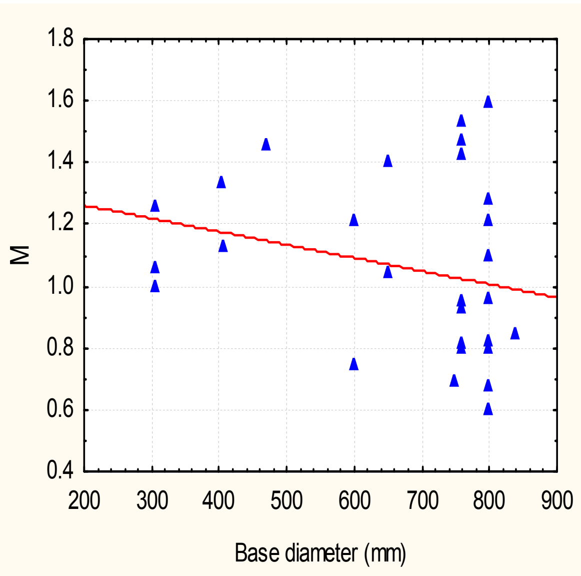
6.4.5.2.2 Correlation with base diameter

The scatter plots are presented in figure 6.8 while the correlation coefficients and the associated p-values are presented in table 6.7. Visual examination of the scatter plots shows:

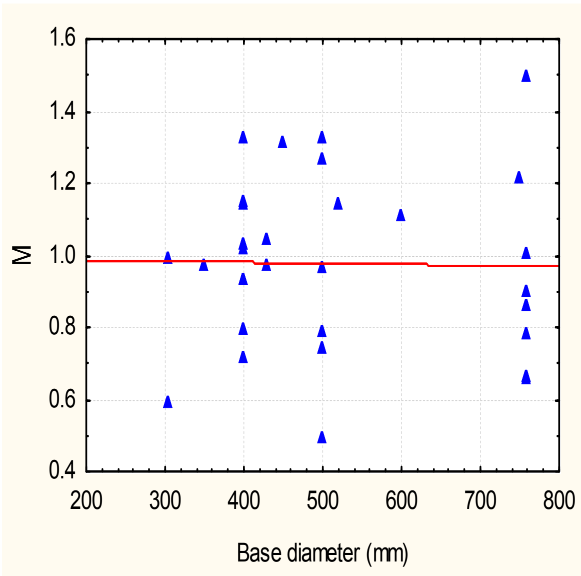
- Significant negative correlation for driven piles and negligible correlation for bored piles.

Observations based on the correlation statistics (table 6.7) indicate that:

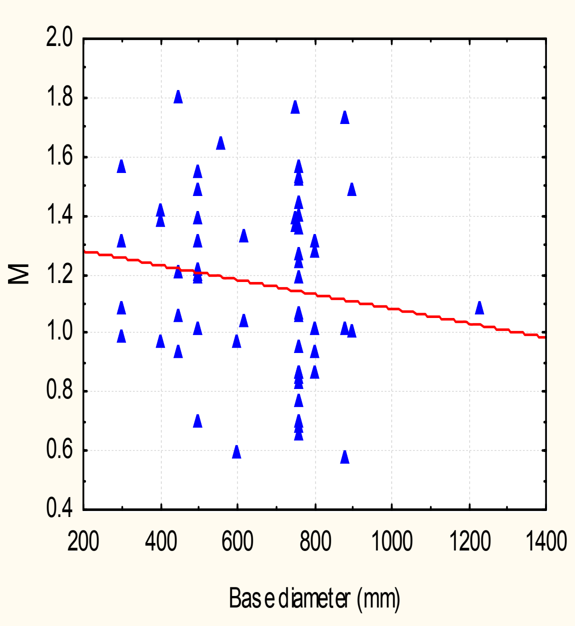
- In agreement with the visual observations, the magnitudes of correlation coefficients for driven piles are much higher than those for bored piles. On the basis of Spearman correlation, the correlation coefficient for driven piles in non-cohesive materials is greater than 0.2 while that for driven piles in cohesive materials is on the borderline. Accordingly the correlation for driven piles can be regarded as practically significant. Conversely the correlation coefficients for bored piles are from practical point of view negligible.
- However, the p-values for all the cases are greater than 0.05, suggesting that there is no evidence to reject the null hypothesis.



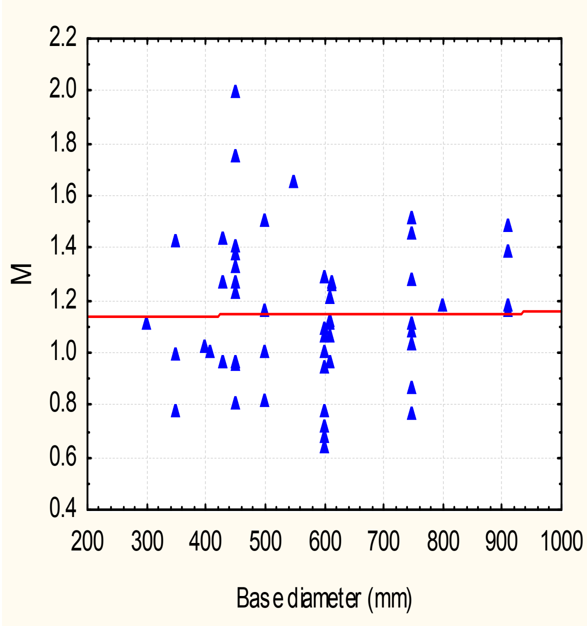
(a)) Driven piles in non-cohesive materials



(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 6.8: Scatter plots of M vs base diameter

Table 6-7: Correlation coefficients and associated p-values for M vs pile base diameter

Case	N	Pearson correlation			Spearman rank correlation	
		r	r^2	p-level	R	p-level
DNC	29	-0.25	0.063	0.19	-0.29	0.12
BNC	31	-0.02	0.0004	0.92	-0.05	0.80
DC	59	-0.15	0.023	0.25	-0.19	0.14
BC	53	0.01	0.0001	0.94	0.05	0.71

6.4.5.3 Correlation with soil properties

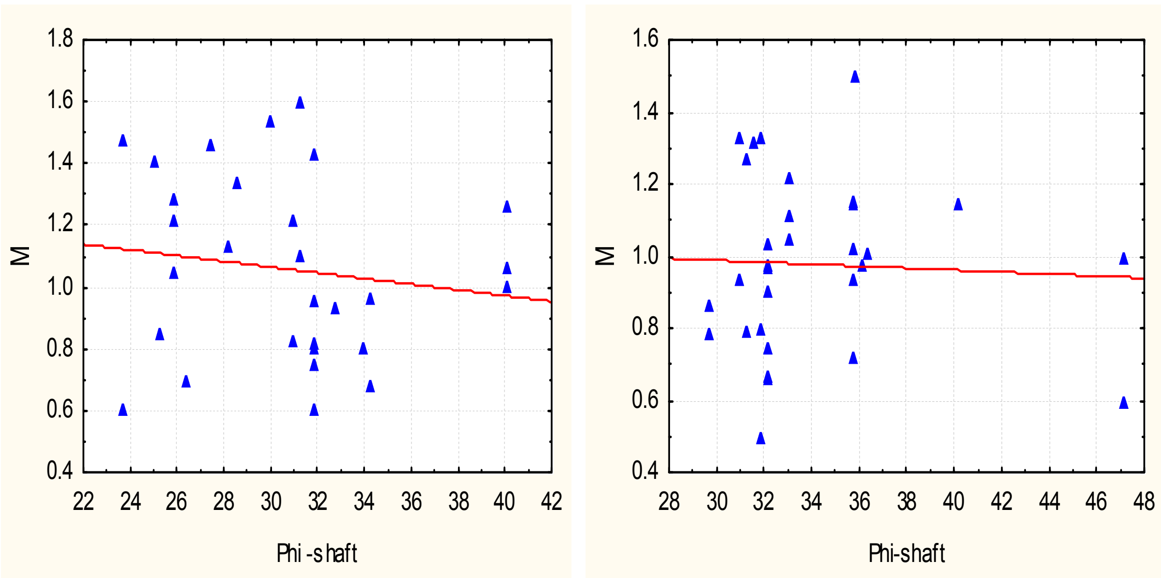
The soil properties considered for the correlation investigations are the angle of friction for non-cohesive materials and the undrained shear strength for cohesive materials.

6.4.5.3.1 Correlation with friction angle (φ)

Both the base friction angle and the shaft friction angle were considered. The outcome of the regression and correlation analysis between the model factor and the friction angle are presented in figure 6.9, figure 6.10 and table 6.8. The following observations were drawn from the results:

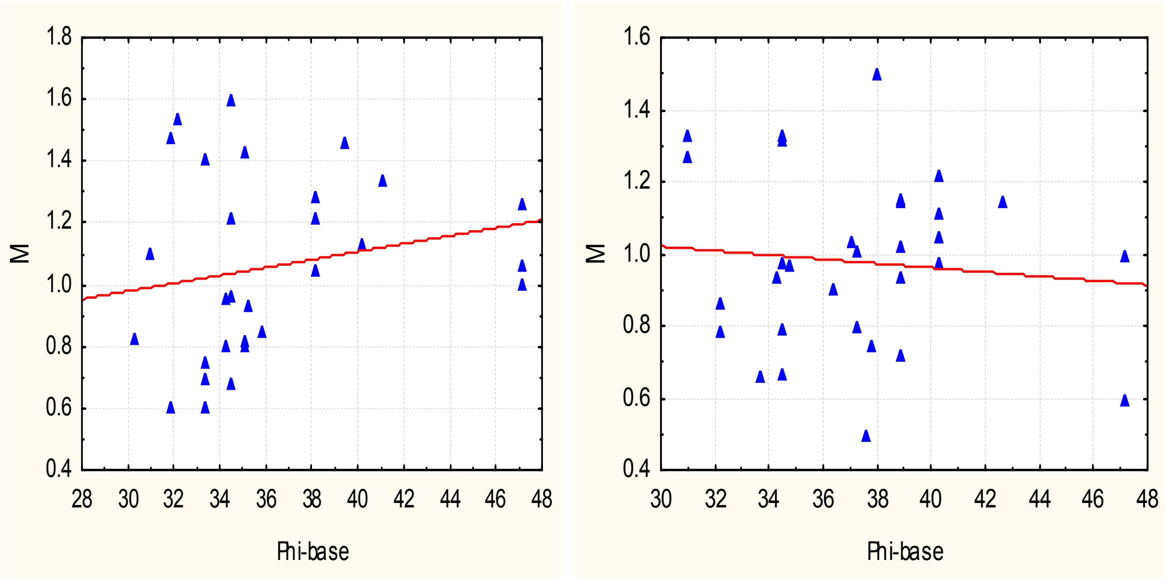
- For φ-shaft, visual inspection of the scatter plots shows an appreciable correlation for driven piles and a weak correlation for bored piles. Consistent with this observation, the correlation coefficients indicate appreciable correlation for driven piles and weak correlation for bored piles. However, the p-values for both cases are greater than 0.05, suggesting that the computed correlations are statistically insignificant.
- For φ-base, visual inspection of the scatter plots shows a significant positive correlation for driven piles and a weak negative correlation for bored piles. The quantitative analysis results (i.e. correlation coefficients values) also imply a noticeable correlation for driven piles and a weak correlation for bored piles. At a confidence level of 5%, the degree of correlation portrayed by both Pearson and Spearman correlation coefficients are considered to be statistically insignificant as indicated by p-values of greater than 0.05.

On the basis of practical significance of the correlations, it is concluded that there exist an noticeable correlation between the model factor and the friction angle in driven piles but not in bored piles.



(a) Driven piles (b) Bored piles

Figure 6.9: Scatter plots of M vs φ-shaft



(a) Driven piles (b) Bored piles

Figure 6.10: Scatter plots of M vs φ-base

Table 6-8: Correlation coefficients and associated p-values for M vs φ

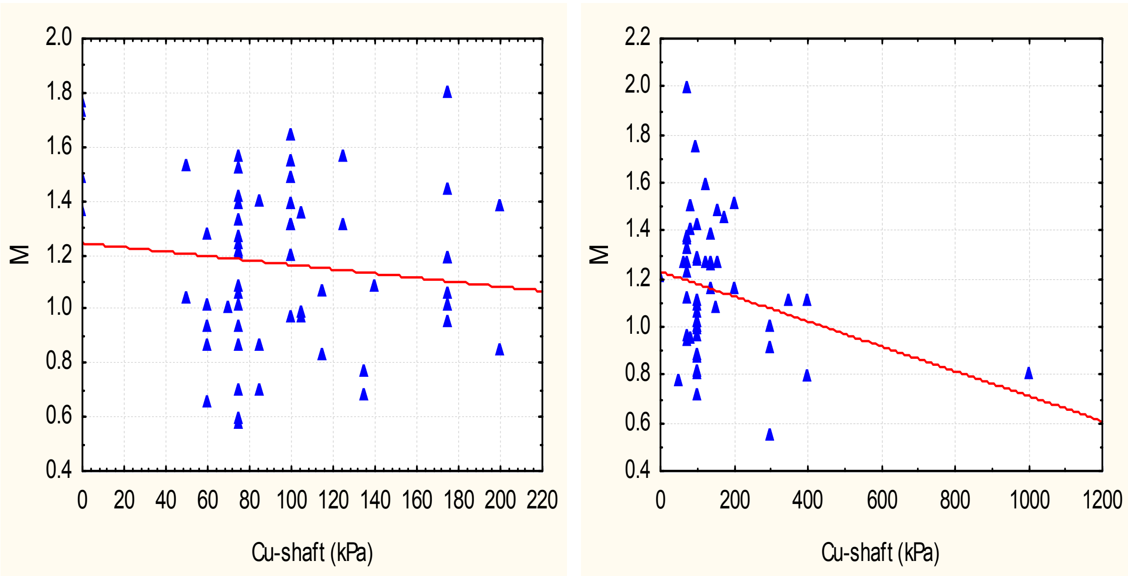
Case	N	Pearson correlation			Spearman rank correlation	
		r	r^2	p-level	R	p-level
φ-shaft						
DNC	29	-0.143	0.020	0.46	-0.25	0.19
BNC	31	-0.048	0.002	0.80	-0.10	0.60
φ-base						
DNC	29	0.200	0.040	0.30	0.25	0.19
BNC	31	-0.098	0.010	0.60	-0.05	0.77

6.4.5.3.2 Correlation with undrained shear strength

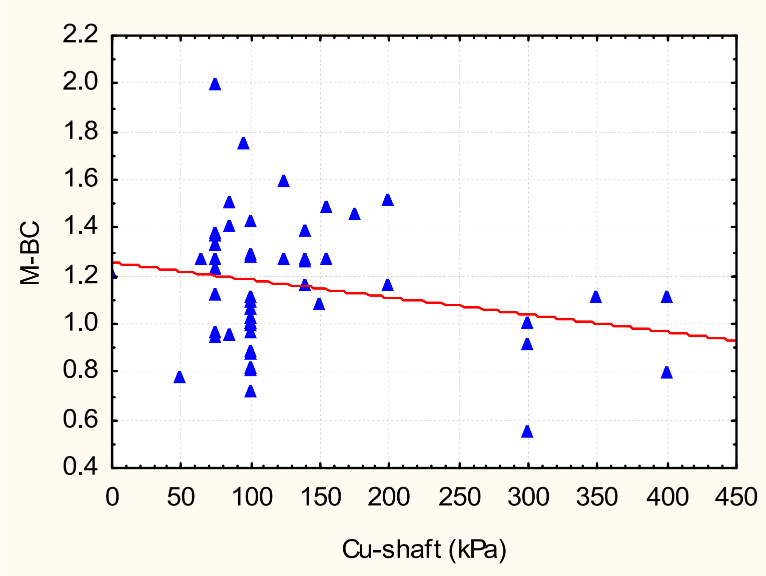
Results for the investigation of correlation between the model factor and undrained shear strength are presented in figure 6.11, figure 6.12, and table 6.9. Visual examination of the scatter plots indicates an appreciable negative correlation for C_u-shaft and a negligible correlation for C_u-base. Further scrutiny of figure 6.11a reveals a divergent data point to the right which might have led to the observed appreciable correlation. Accordingly this data point was excluded and the resulting scatter plot is shown in figure 6.11c. Even after the exclusion of the divergent data point, the correlation for C_u-shaft is still noticeable (Fig. 6.11c). Therefore it can be concluded that indeed the correlation for C_u-shaft is significant.

The quantitative assessment of the correlation (table 6.9) indicates that:

- Only the correlation coefficients (both Pearson and Spearman) for C_u-shaft in bored piles are practically significant ($r \geq 0.2$) while the values for the other cases are practically insignificant. To investigate the effects of the divergent data point, the correlation coefficients were re-calculated with the divergent point excluded. The results are highlighted in table 6.9. It is evident from this results that the exclusion of the divergent point slightly reduces the correlation coefficients. With this reduction, the Spearman rank correlation now less than 0.2 after rounding off to one decimal point while the Pearson correlation is still greater than 0.2. In any case, the value for Spearman was upheld.
- After exclusion of the divergent data point, the p-values for both Spearman and Pearson correlation coefficients are greater than 0.05, indicating a statistically insignificant correlation.

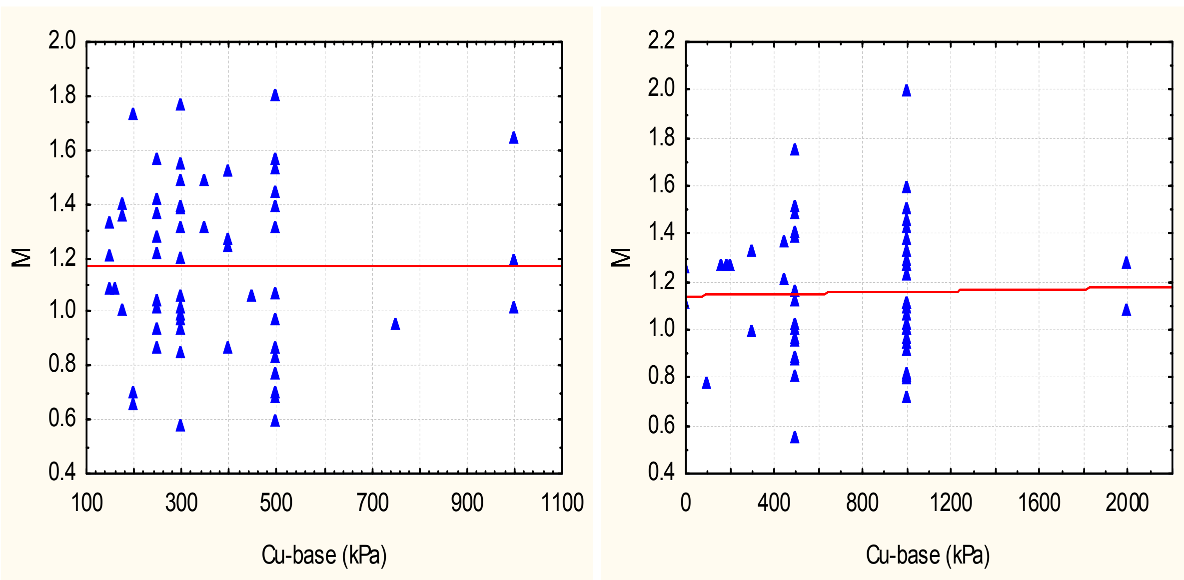


(a) Driven piles (b) Bored piles



(c) Bored piles after exclusion of the divergent data point

Figure 6.11: Scatter plots of M vs C_u-shaft



(a) Driven piles (b) Bored piles

Figure 6.12: Scatter plots of M vs C_u-base

Table 6-9: Correlation coefficients and associated p-values for M vs C_u

Case	N	Pearson correlation			Spearman rank correlation	
		r	r^2	p-level	R	p-level
Cu-shaft						
DC	59	-0.13	0.016	0.35	-0.07	0.61
BC	53	-0.28	0.073	0.04	-0.16	0.26
	52	-0.23	0.052	0.10	-0.12	0.41
Cu-base						
DC	59	-0.003	0.000	0.98	-0.05	0.74
BC	53	0.03	0.001	0.86	0.01	0.97

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6.4.5.4 Correlation with predicted and interpreted pile capacities

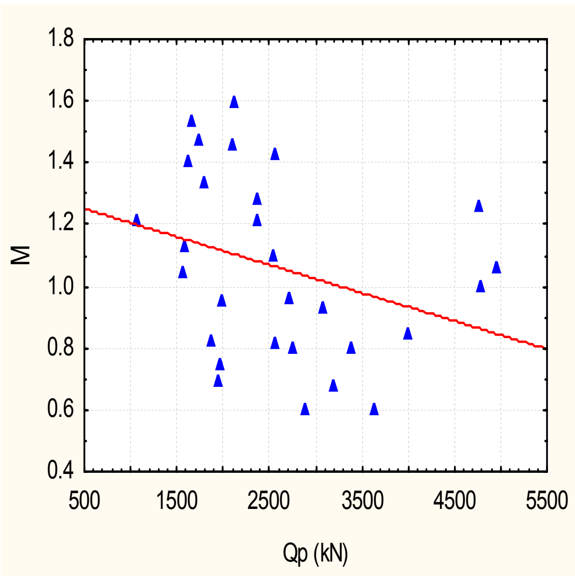
The model factor was previously defined as the ratio of the measured to predicted capacity (i.e $M=Q_i/Q_p$). In this definition, the measured and the predicted capacities appear as the numerator and denominator respectively. Therefore by virtue of the way the model factor is computed, it may be correlated to these two parameters. The correlation with Q_p if it exists is of more practical significance in the calculation of reliability index. This arises from the fact that in practice the model factor is applied as an independent corrective random variable to predicted capacity. To apply the model factor as an independent random variable on the predictive capacity equation, it is important to verify that it does not vary systematically with the predicted capacity. Correlation can have significant impact on the calculated reliability index if not accounted for. Further more, all the pile design parameters investigated so far, appear in the equation for calculating the ultimate bearing capacity. Therefore the sensitivity of the bias factor to the combined effects of the pile design parameters can be explored through its correlation with the predicted capacity. On the other hand the correlation with the interpreted capacity has no effects on the reliability calculation and therefore it was not considered. The results of the correlation analysis between the model factor and the predicted capacities are presented in figure 6.13 and table 6.10

For driven piles in non-cohesive materials (fig. 6.13a), the scatter plot shows a significant negative correlation between the two parameters. Based on the Spearman correlation coefficient, the statistical measures of the strength of the correlation are; $R = -0.413$ and $p = 0.026$. The magnitude of correlation coefficient supports the visual observation that the correlation is significant. The significance of the correlation is further captured by p-value of less than 0.05. A p-value of less than 0.05 indicates that the null hypothesis of zero correlation is not valid. Therefore at a confidence level of 5%, the model factor is considered to be statistically correlated to the predicted capacity for driven piles in non-cohesive materials.

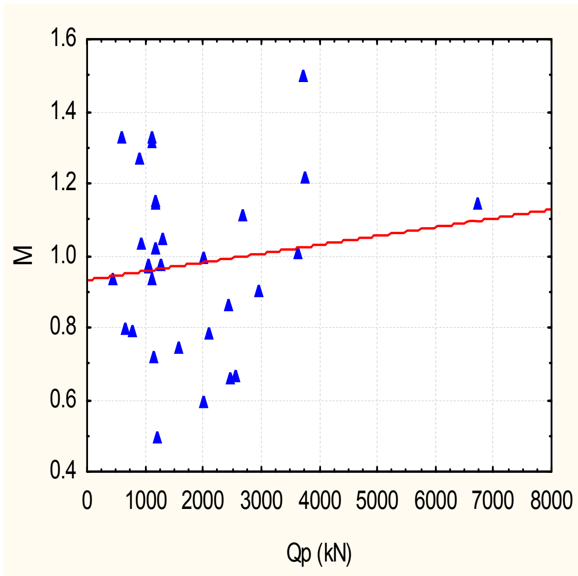
For the case of bored piles in non-cohesive materials, the scatter plotter (fig. 6.13b) shows a relatively weak positive correlation between the model factor and predicted capacity. The Spearman rank correlation coefficient and the associated p-value confirm that the correlation between the two parameters is not statistically significant ($p > 0.05$).

For piles in cohesive materials, visual inspection of the scatter plots indicate that the correlation between the model factor and predicted capacity appears to be strong in driven piles and weak in bored piles. For driven piles (fig. 6.13c) there two extreme data points to the right might have influenced the observed correlation. These points were excluded and the resulting scatter plot is presented in fig. 6.13e. Inspection of fig. 6.13e still shows a noticeable correlation. Therefore with or without the divergent data points, the correlation for driven piles in cohesive materials is appreciable. Both the Spearman and Pearson correlation coefficients are greater than 0.2, implying that the correlation is practically significant. Even after the exclusion of the two divergent data points the correlation coefficients are still greater than 0.2 (highlighted results). However the associated p-values are marginally greater than 0.05, indicating that the correlation is statistically insignificant.

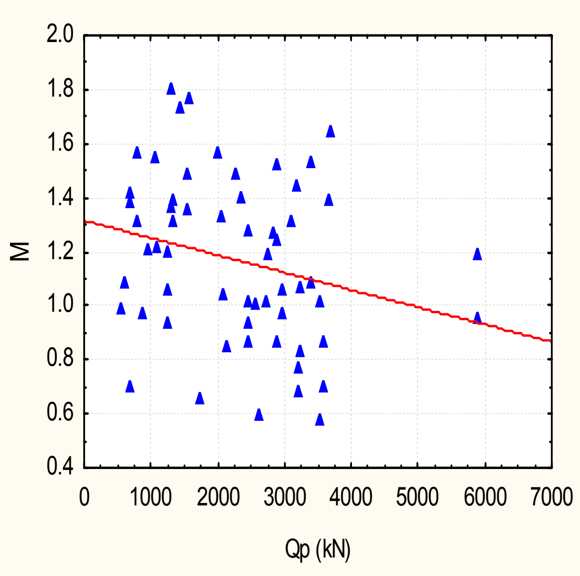
For the case of bored piles in cohesive materials, visual examination of the scatter plot (fig. 6.13d) shows a negligible correlation. In agreement, quantitative results indicate practically and statistically insignificant correlation.



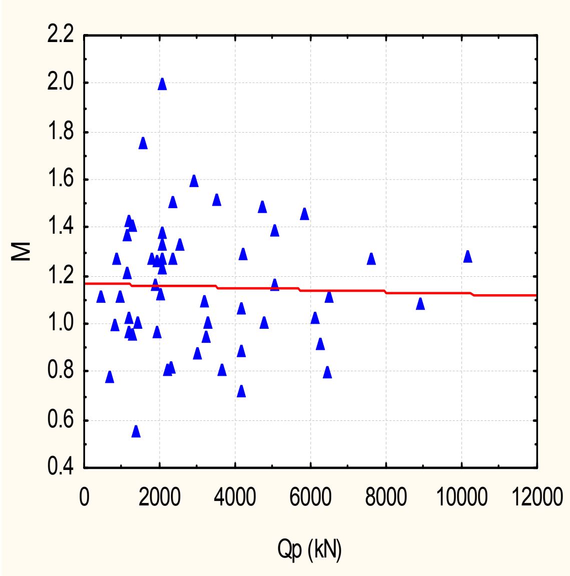
a) Driven piles in non-cohesive materials



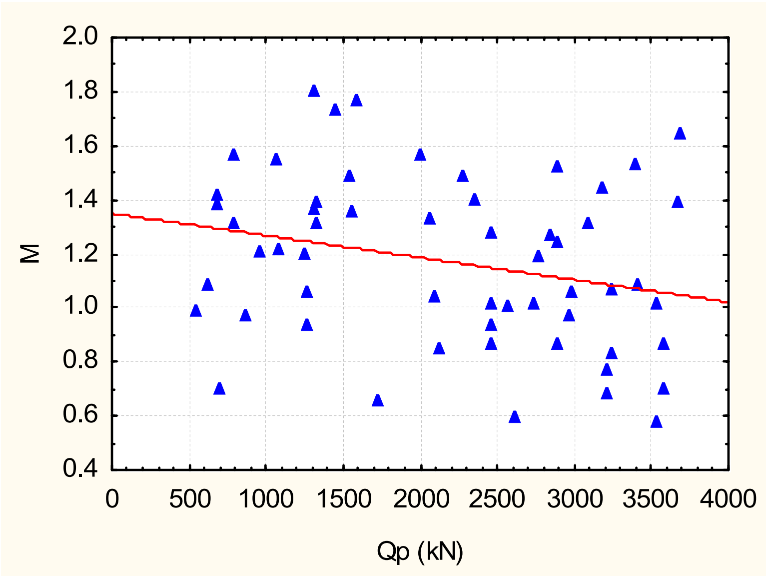
(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials



(e) Driven piles cohesive materials after removal of divergent data points

Figure 6.13: Scatter plots of M vs Q_p

Table 6-10: Correlation coefficients and associated p-values for M vs Q_p

Case	N	Pearson correlation			Spearman rank correlation	
		r	r^2	p-level	R	p-level
DNC	29	-0.32	0.099	0.09	-0.41	0.03
BNC	31	0.13	0.018	0.48	-0.05	0.81
DC	59	-0.25	0.062	0.06	-0.26	0.05
	57	-0.26	0.068	0.05	-0.24	0.07
BC	53	-0.04	0.001	0.81	-0.01	0.94

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6.4.5.5 Summary and discussion of correlation investigation results

On the basis of statistical significance (p-value) it can be concluded that at the customary confidence level of 5%, none of the pile design parameters is significantly correlated with the model factor. From the probabilistic perspective, this implies that the variation in the model factor is not caused by the variations in the key pile design parameters. Therefore it is correct to model the model factor as a random variable. With regard to predicted capacity, it was found that the model factor is correlated to Q_p for driven piles but not bored piles. These results differ from that reported by Dithinde et.al (2006) where on the basis of 5% confidence level it was found that the model factor was correlated to base diameter for driven piles in non-cohesive materials, φ-shaft in driven piles, φ-base in driven piles, Cu-base in driven piles, and Q_p for driven and bored piles in non-cohesive materials as well as Q_p for driven piles in cohesive materials. The difference in results is attributed to the differences in the schemes for selecting the geotechnical design parameters. The former was based on commonly assumed parameters while the later was based the outcome of a parametric study as discussed in chapter 5. The differences in the two results lead to the conclusion that the use of more realistic design parameters significantly reduces the systematic dependence of the model factor on the pile design parameters.

If the interpretation of the numerical values of correlation coefficients is based on practical considerations (table 6.4), then the model factor was found to be correlated with:

- Shaft diameter for driven piles in cohesive materials
- Base diameter for driven in non-cohesive materials
- Base diameter for driven piles in cohesive materials
- φ-shaft for driven piles in non-cohesive materials
- φ-base for driven piles in non-cohesive materials
- Predicted capacity for driven piles in non-cohesive materials
- Predicted capacity for driven piles in cohesive materials

The numerical values of the correlation coefficients for the above cases ranged from 0.2 to 0.4. In general this range of correlation indicates a low degree of correlation. However for the purposes of this study this range of correlation was considered appreciable and therefore warrants a further investigation. This further investigation is carried out in Chapter 7.

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It is evident from the above results that for the individual pile design parameters, significant correlation occurs only in driven piles and not in bored piles. This scenario suggests that the static formula is more accurate and precise in bored piles. This deduction is further supported by the results of the summary statistics (Table 6.2). As observed earlier, the variability of the model factor is slightly higher in driven piles compared to bored piles irrespective of the materials type. The non-existence of correlation in bored piles was also reported by Tuomi and Roth (1995) who investigated the sensitivity of the model factor for bored piles in clay with respect to pile dimensions, predicted capacity, and soil properties. The results indicated that the model factors were not significantly influenced by the pile design parameters. For correlation with soil properties, the results show correlation with shear strength parameters for non-cohesive materials and negligible correlation with undrained shear strength. This trend was also observed by Phoon and Kulhawy (2005). Phoon and Kulhawy plotted model factors for undrained and drained analysis against side and tip shears. The results indicated a significant correlation for drained analysis and no correlation for the undrained analysis. With regard to predicted capacity, it can be seen that correlation occurs only in driven piles and not in bored piles, reiterating that correlation is more prevalent in driven piles than in bored piles.

With more weight given to the practical significance of the value of the correlation coefficient rather than its statistical significance, it can be concluded that the conventional model factor exhibits some statistical dependencies with some underlying factors. Therefore to apply the model factor as an independent variable in reliability analysis, these statistical dependencies need to be removed or taken into account. Treatment of the statistical dependencies is presented in Chapter 7 of the dissertation.

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Chapter 7

REMOVAL OF STATISTICAL DEPENDENCY BETWEEN THE MODEL FACTOR AND THE PREDICTED CAPACITY

7.1 Introduction

To apply a model factor as an independent random variable on the predictive capacity equation as it is currently done, it is desirable to verify that the model factor does not vary systematically with some underlying factors. However, it was established in Chapter 6, that the model factor is correlated to various underlying factors. Correlation can have significant impact on the calculated reliability index if not accounted for. Even though correlation can be incorporated into the reliability calculations, it makes the calculation to be cumbersome as it involves transforming the original variables to a set of uncorrelated variables. Therefore to apply the model factor as an independent random variable in reliability analysis, the statistical dependencies need to be removed. Alternatively, the model factor should be presented as a function of Q_p to reflect the its dependency upon Q_p. Accordingly the two approaches to the treatment of correlation were employed. Removal of statistical dependencies is presented under the generalised model factor approach while accounting for the correlation is presented under conditioned model factor approach.

7.2 Generalised model factor approach

The generalised model factor approach is based on a sound regression theory and it therefore entails performing regression using the predicted values as the predictor variable. It was established in Chapter 6 that the appropriate probabilistic model for the model factor is the lognormal probability distribution. This being the case, the generalised model factor was derived from the regression of $LN(Q_i)$ on $LN(Q_p)$. The resulting functional relationship between $LN(Q_i)$ and $LN(Q_p)$ is given by a general regression model of the form:

$$LN(Q_i) = a + b\,LN(Q_p) + \varepsilon \qquad [7.1]$$

in which a and b are regression constants and ε is a normal random variable with zero mean and non-zero variance.

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Taking antilog on both sides of equation 7.1 yields;

$$Q_i = \exp(a).\exp(\varepsilon).Q_p^b \quad [7.2]$$

Note that in equation 7.2, the error term (i.e. ε) is now modeled as multiplicative as compared to equation 7.1 where it was additive. Therefore regression model in the form of equation 7.2 removes systematic effects and the remaining component tends to appear random (Phoon and Kulhawy 2005b).

Equation 7.2 can be re-written as:

$$Q_i = \exp(a + \varepsilon)Q_p^b \quad [7.3]$$

Let $\exp(a + \varepsilon) = \mathrm{M}$ [7.4]

Then;

$$Q_i = MQ_p^b \quad [7.5]$$

Equation 7.5 is the generalised representation of the model factor M. This equation is immediately recognised as being of the same form as that for the conventional model factor (i.e. $Q_i = MQ_p$). In fact, the conventional model factor is a special case of the generalised model factor with $b = 1$.

In equation 7.4, ε is a random variable and therefore M will likewise be random. Theoretically the probability distribution of a function of random variables such as M can be derived from the probability distributions of the basic random variables. However, such derivations are generally difficult especially when the function is non-linear. Therefore for practical purposes the moments, in particular the mean and variance are used to describe the function. Accordingly a linear function for M can be described by its mean and variance. In accordance with the computational rules for lognormal random variables, the mean and variance for M are as follows:

$$\mu_M = \exp\left(a + 0.5\xi^2\right) \quad [7.6]$$

$$\sigma_M^2 = \mu_M^2\left[\exp\left(\xi^2\right) - 1\right] \quad [7.7]$$

The generalised model factor as presented in equation 7.5 is not dimensionless in contrast to conventional model factor equation. The force unit adopted throughout this study is

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kiloNewton (kN) and this unit is applicable to equation 7.5. To make the generalised model factor dimensionless, both the interpreted and predicted capacities need to be normalised.

7.2.1 Normalisation schemes

The objective of normalisation is to make Q_i and Q_p dimensionless so that the resulting model factor is dimensionless. In principle any quantity or combination of quantities associated with pile design, can be used as a normalizing factor. Obviously different model factor statistics will be obtained for different normalisation schemes. The dependence of the generalised model factor on the normalisation scheme is the greatest disadvantage of this approach. In this study the following three normalization schemes were experimented with:

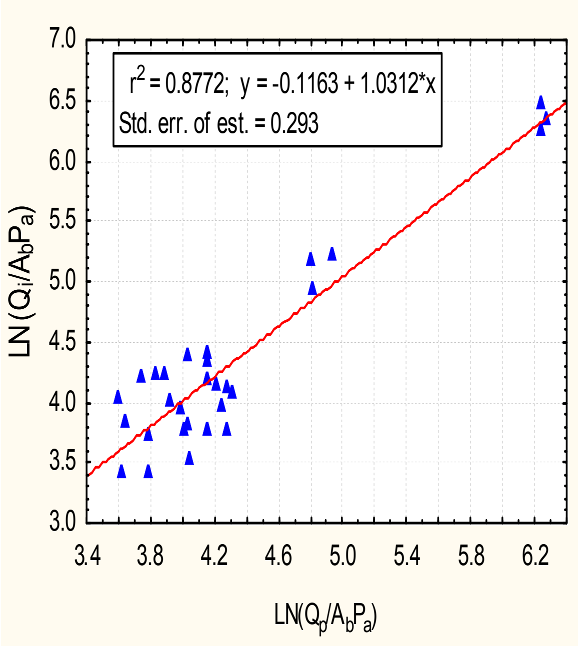
- Scheme 1: dividing LN(Q_i) and LN(Q_p) by area of pile base x atmospheric pressure (A_bP_a)
- Scheme 2: dividing LN(Q_i) and LN(Q_p) by volume of water displayed by the pile (i.e. volume of piles x unit weight of water (V_w))
- Scheme 3: dividing LN(Q_i) and LN(Q_p) by weight of pile shaft (W_s)

7.2.2 Regression results after normalisation

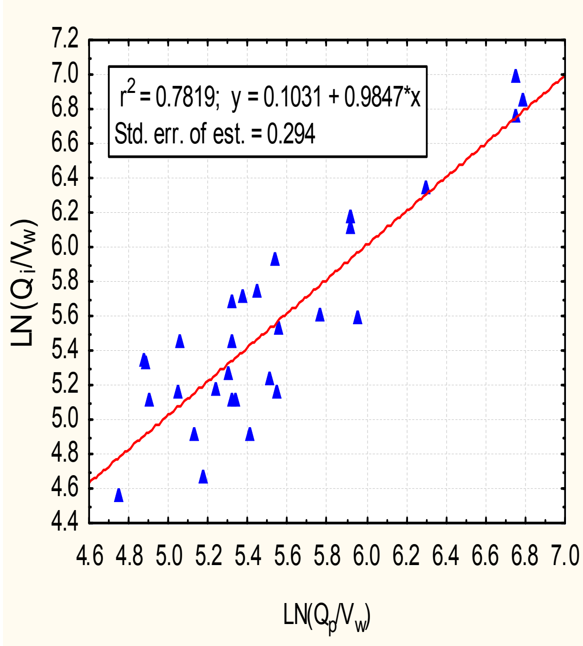
The data for each pile class were normalised by the three approaches. The regression results for each pile class are presented in the subsequent subsections. The normalisation schemes were evaluated on the basis of the resulting fit between Q_i and Q_p. In this regard, the coefficient of determination (r^2) was used as the measure of fit.

7.2.2.1 Results for DNC

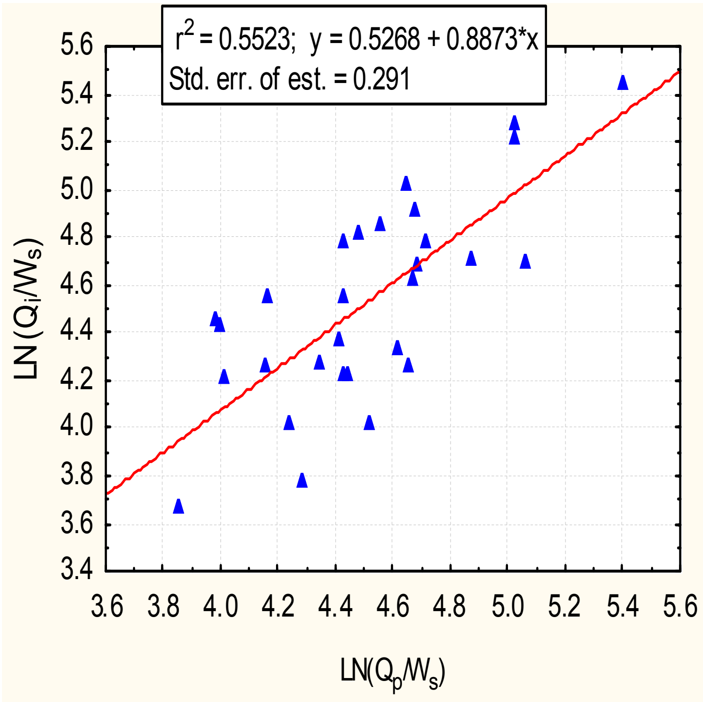
The regression results for DNC are presented in figure 7.1. A comparison of the r^2 indicates that normalisation scheme 1 gives the best fit while scheme 3 yields the poorest fit. However the standard deviations about the regression lines (standard error of estimate) are comparable.



(a) Scheme 1



(a) Scheme 2

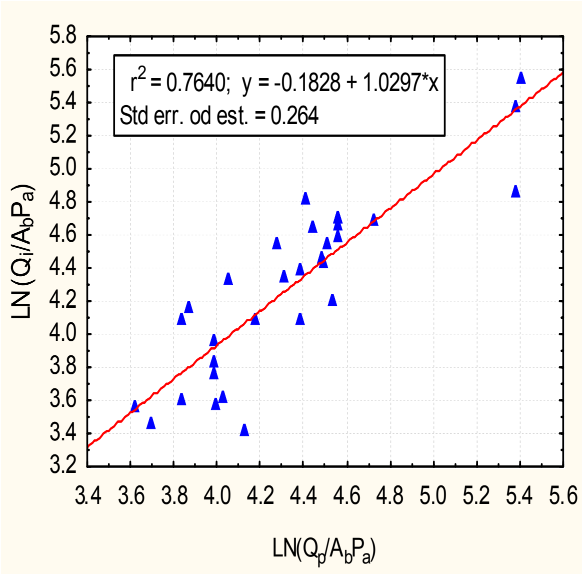


(c) Scheme 3

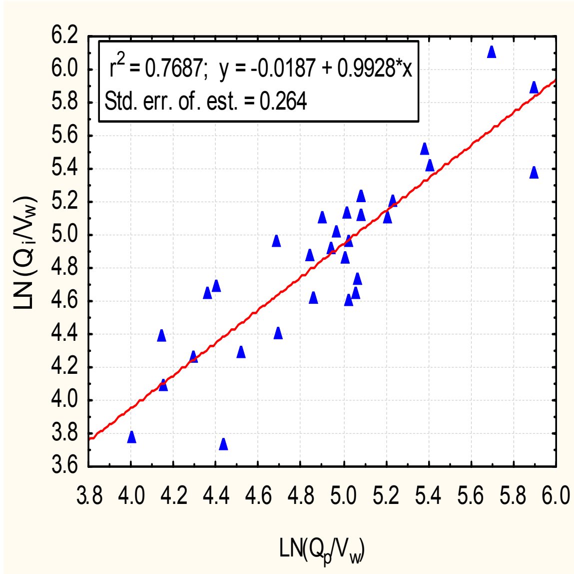
Figure 7.1: Normalised regression results for DNC

7.2.2.2 Results for BNC

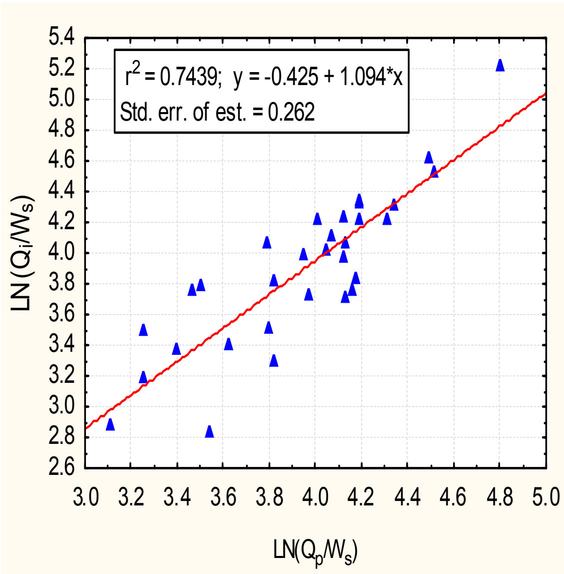
The results for BNC are presented in figure 7.2. Inspection of figure 7.2 shows that r^2 and the standard error of estimate (ξ) for the three schemes are quite close.



(a) Scheme 1



(b) Scheme 2



(c) Scheme 3

Figure 7.2

Figure 7.2: Normalised regression results for BNC

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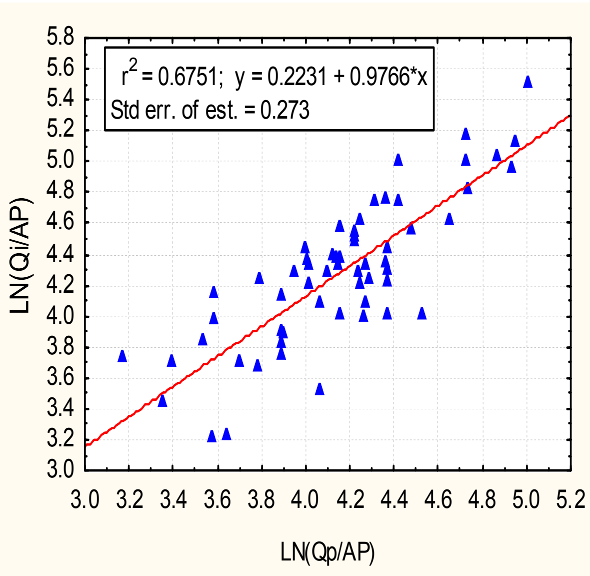
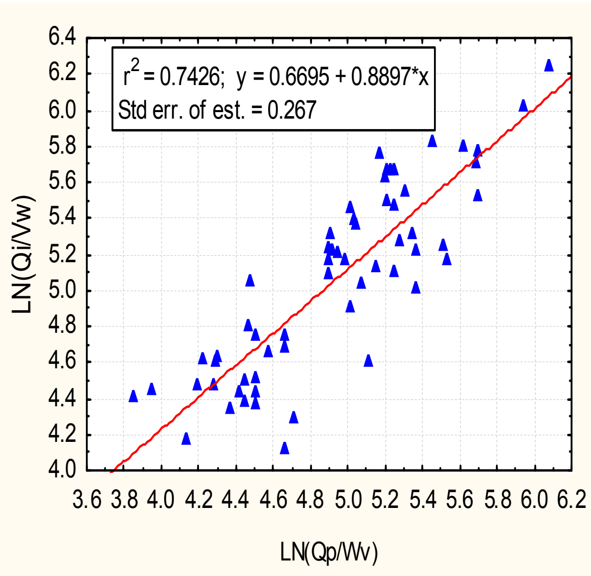
7.2.2.3 Results for DC

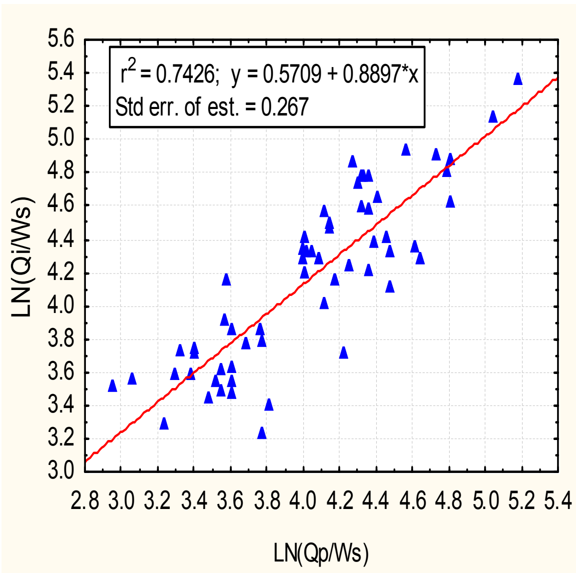
The regression results for DC are presented in figure 7.3. The values for coefficient of determination and standard error of estimate suggest that normalisation schemes 2 and 3 produces a better fit compared to scheme 1.



(a) Scheme 1



(b) Scheme 2

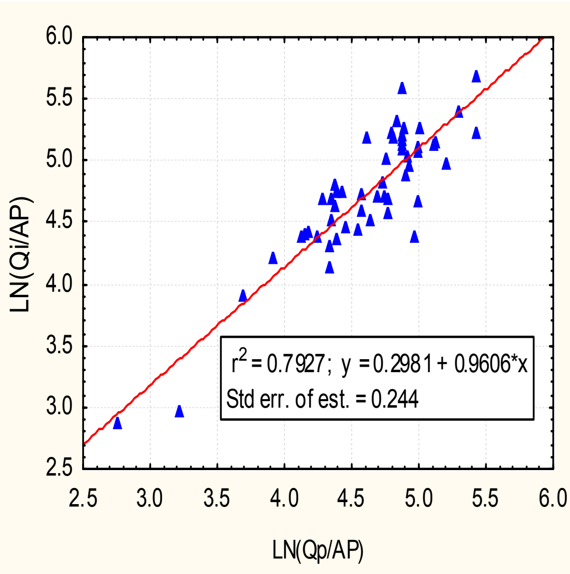


(c) Scheme 3

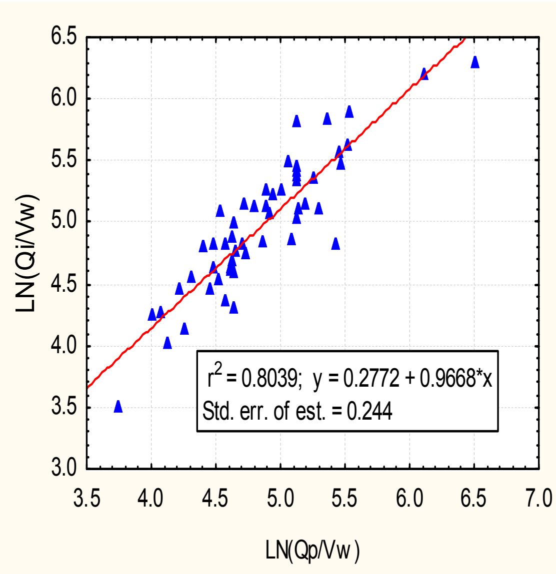
Figure 7.3: Normalised regression results for DC

7.2.2.4 Results for BC

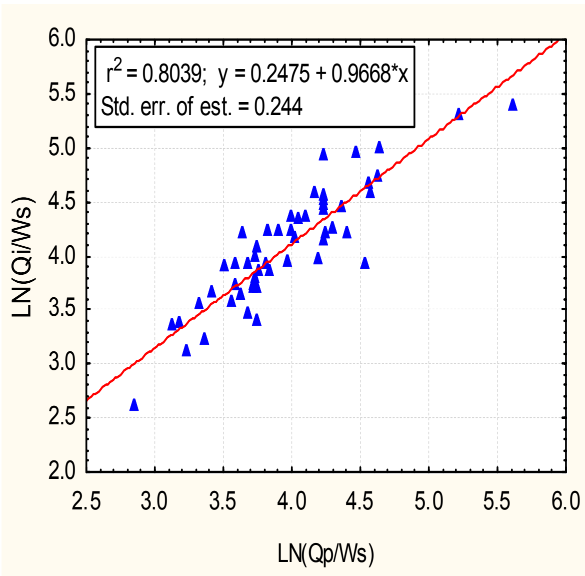
The regression results for BC are shown in figure 7.4. Both measures of fit (r^2 and ξ) suggest that the three schemes produce similar results.



(a) Scheme 1



(b) Scheme 2



(c) Scheme 3

Figure 7.4: Normalised regression results for BC

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7.2.2 Generalised model factor statistics and discussion of the regression results

The regression results presented in figure 7.1 through figure 7.4 were then used in conjunction with equations 7.6 and 7.7 to compute the required generalised model factor statistics. The regression parameters and the ensuing generalised model factor statistics are summarised in table 7.1. Since both the statistics and the regression parameter "b" are used as input into the performance function, they influence the calculated value of the reliability index.

Table 7-1: Generalised model factor statistics for various normalisation schemes

Case	Normalisation	Regression parameters				Generalised M statistics		
		R^2	a	b	ξ	μ	σ	COV
DN	A_bP_a	0.88	-0.1163	1.03	0.293	0.93	0.28	0.30
	Vw	0.78	0.1031	0.985	0.294	1.16	0.35	0.30
	Ws	0.55	0.5268	0.887	0.291	1.77	0.53	0.30
BN	A_bP_a	0.76	-0.1828	1.03	0.264	0.86	0.23	0.27
	Vw	0.77	-0.0187	0.993	0.264	1.02	0.27	0.27
	Ws	0.74	-0.4250	1.094	0.262	0.68	0.18	0.27
DC	A_bP_a	0.68	0.2231	0.977	0.273	1.30	0.36	0.28
	Vw	0.74	0.6695	0.89	0.267	2.02	0.55	0.27
	Ws	0.74	0.5709	0.89	0.267	1.83	0.50	0.27
BC	A_bP_a	0.79	0.2981	0.961	0.244	1.39	0.34	0.25
	Vw	0.80	0.2772	0.967	0.244	1.36	0.34	0.25
	Ws	0.80	0.2475	0.967	0.244	1.32	0.33	0.25

Scrutiny of table 7.1 lead to the following observations:

- The numerical values of the generalised model factor statistics (mean and standard deviation) reduces with the decrease in the regression constant "a". Consequently when the regression constant is negative, the mean generalised model factor tends to be less than or quite close to one (i.e. becomes unconservative). The standard deviation also reduces significantly. This trend is apparent in DNC with respect to normalisation scheme 1 and BNC with respect to the three normalisation schemes. Conversely for higher positive values of "a", the mean and standard deviation of the generalised model factor becomes significantly high. Typical examples are DNC with respect to normalisation scheme 3 and DC with respect to normalisation schemes 2 and 3. The reduction in the mean and standard deviation with degrease of the regression constant is attributed to the calculation models (Eq. 7.6) and Eq. 7.7).

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- As expected, the regression parameter "b" increases with the decrease of the regression constant "a" and vice-verse. For negative values of "a", the regression parameter "b" becomes greater or quite close to one. Conversely for higher positive values, "b" becomes appreciably less than one. Accordingly the parameter "b" can be related to the generalised model factor statistics as follows:
    - For mean model factor of less than one (with small standard deviation), "b" becomes greater than one
    - For relatively high mean generalised M (with higher standard deviation), "b" becomes appreciably less than one
- The above relationship between the regression parameter "b" and the generalised M statistics has a significant influence on calculated value of reliability index. A smaller standard deviation and a value of "b" that is greater than one tend to increase the reliability index. In this regard, the lower mean value is compensated for by the smaller standard deviation and higher b value. High values of standard deviation and values of the "b" that are less than one tend to reduce the reliability index. This combination counter balances the effects of the high mean value. Therefore for a given pile class, the different normalisation schemes will result in comparable values of reliability indices.
- Because for a given pile class, the mean and the standard deviation change in the same direction, the coefficient of variation remains the same for the three schemes.
- As already noted, for a given pile class, the r^2 values for the different normalisation schemes are generally comparable. An exception is DNC with respect to scheme 3. The r^2 value for this case is notably smaller. Also for a given pile class, values of the standard error of estimate are quite close. It can therefore be concluded that, in general the three normalisation schemes give comparable results.

Based on the above results, the best normalisation scheme was selected. The best normalisation scheme is that which produces the best fit between Qi and Qp. The best fit was evaluated in terms of the results of a perfect fit. For the perfect fit, b = 1 and $R^2 = 1$. Accordingly a normalisation scheme that yields a relatively high R^2 and a value of b that is relatively closer to 1 gives the best fit. The two parameters have been extracted from table 7.1 and presented in Table 7.2 in a manner that facilitates easy comparison. Inspection of Table 7.2 shows that scheme 3 yield the poorest overall results (smallest r^2 for DNC and values of b that are appreciably different from 1 for DNC, BNC and DC). On the basis of r^2 schemes 2 and 3 give very close results. However, in terms of "b", scheme 2 gives a value of appreciably less than one for DC whiles scheme 1 produces values that are close to one

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for all the pile classes. In this respect, scheme 1 is slightly better than scheme 2. Accordingly the generalised M statistics for scheme 1 were selected for further analysis. For ease of reference the selected generalised M statistics are presented in table 7.3.

A comparison of the generalised M statistics to the standard M statistics (Table 6.2) show reasonable agreement. The standard deviations and coefficients of variation are comparable. Furthermore, the two sets of model factors show similar trends such as:

- The mean model factors in non-cohesive materials are less than those in cohesive materials
- Driven piles exhibit higher variability compared to bored piles irrespective of materials type

Table 7-2: Comparison of r^2 and b from the three normalisation schemes

Pile Class	Scheme 1		Scheme 2		Scheme 3	
	r^2	b	r^2	b	r^2	b
DNC	0.88	1.031	0.78	0.985	0.55	0.887
BNC	0.76	1.030	0.77	0.993	0.74	1.094
DC	0.68	0.977	0.74	0.890	0.74	0.890
BC	0.79	0.961	0.80	0.967	0.80	0.967

Table 7-3: Selected generalised M statistics

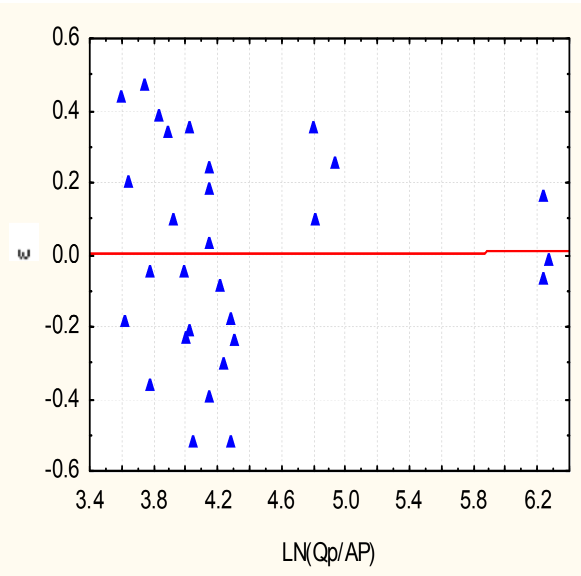
CASE	μ	σ	COV	b
DNC	0.93	0.28	0.30	1.031
BNC	0.86	0.23	0.27	1.030
DC	1.30	0.36	0.28	0.977
BC	1.39	0.34	0.25	0.961

7.2.3 Verification of removal of systematic dependency

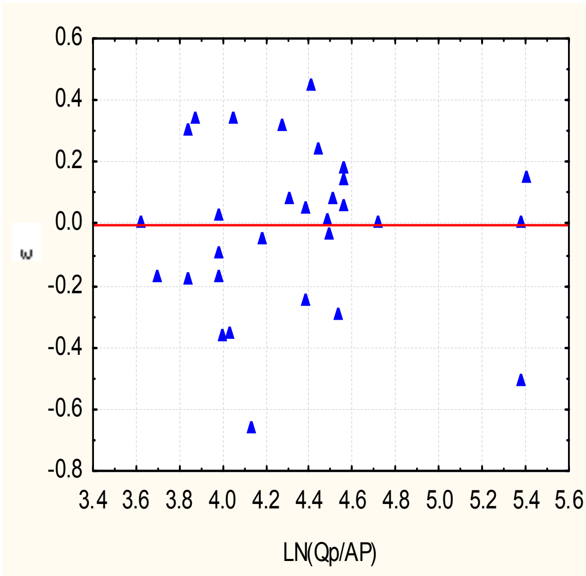
As noted earlier, the correlation of practical significance is that between the model factor and the predicted capacity. To verify the removal of such correlation, the regression model error (ε) is plotted against LN(Qp/AP). For a given case the model error for a given data point was determined from equation 7.1 as follows:

$$\varepsilon = LN(Q_i / A_b P) - a - bLN(Q_p / A_b P) \qquad [7.8]$$

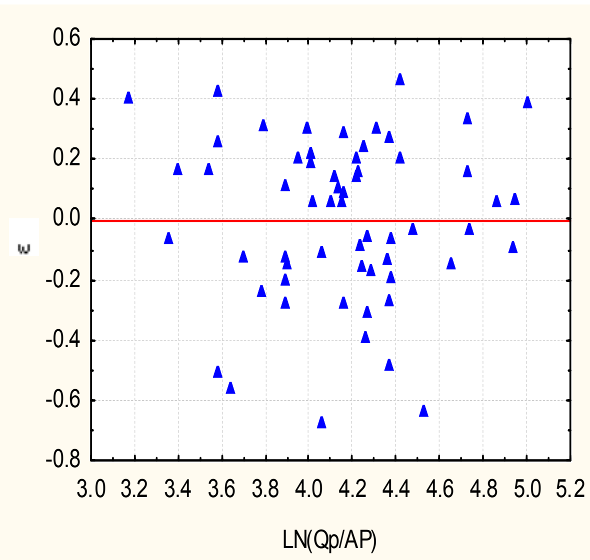
The scatter plots of the model error (ε) versus predicted capacity are presented in figure 7.5. Visual inspection of the scatter plots shows that for all the four cases, the best fit line is horizontal, suggesting that the correlation between the two parameters is insignificant. The quantitative measure of the correlation in terms of the correlation coefficients and the associated p-values is presented in table 7.4. Again for all the four cases, the p-values are much greater than 0.05, thus implying that the null hypothesis of zero correlation is valid. Therefore both qualitative and quantitative results are in agreement that the correlation between the parameters is negligible. It is thus verified that model factor defined as the regression model error does not suffer from statistical dependencies with underlying parameters.



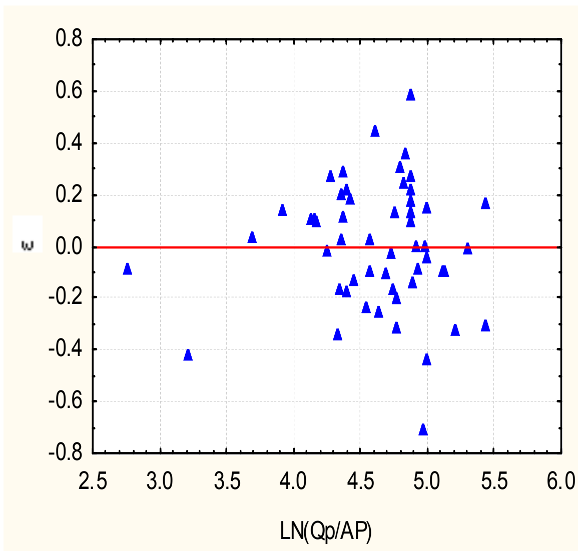
(a) Driven piles in non-cohesive materials



(b) Bored piles in non-cohesive materials



(c) Driven piles in cohesive materials



(d) Bored piles in cohesive materials

Figure 7.5: Scatter plots of ε Vs LN(Q_p/AP)

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Table 7-4: Correlation coefficients and associated p-values for ε Vs LN(Q_p/AP)

Case	N	r	p-value
DNC	29	-0.003	0.987
BNC	31	-0.002	0.990
DC	59	-0.001	0.996
BC	53	-0.001	0.995

7.3 Conditioned M

This approach seeks to take into account the fact that the model factor varies with the predicted capacity. Since for a given M value, there is a range of possible values of Q_p, a probabilistic description of the functional relationship between M and Q_p is required. A probabilistic description of the relationship between variables is generally achieved by regression analysis. Accordingly regression analyses of M on Q_p were performed for each piles class. The resulting regression equation gives the functional relationship between the mean model factor and Q_p. The general equation for the model factor is given by:

$$M = a + bQ_p \qquad [7.9]$$

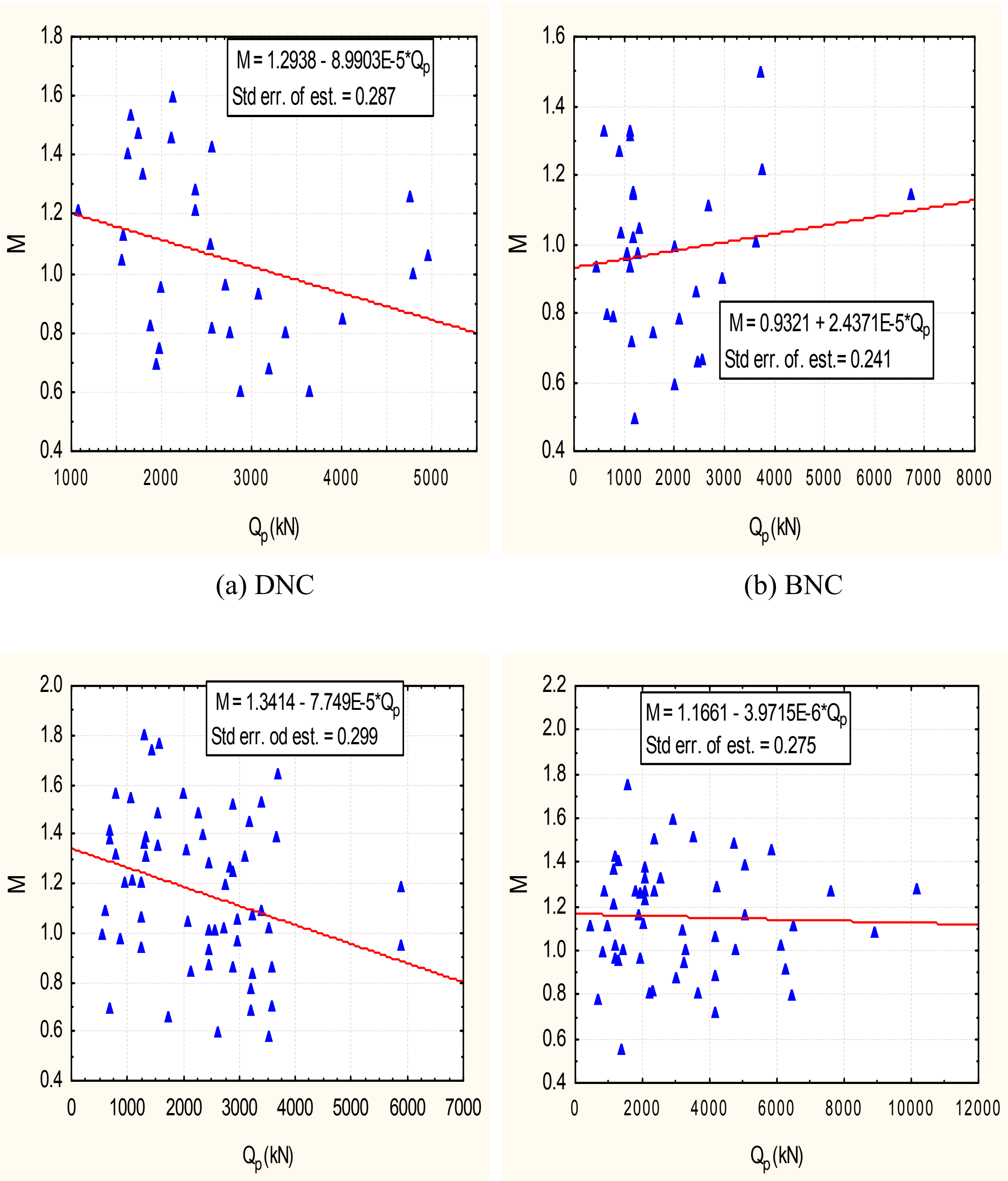
Where *M*= mean model factor, *a* = y-intercept, *b* = slope, Q_p = interpreted capacity.

Since the computation of the mean and the standard deviation of M takes into account its correlation with Q_p. The resulting M statistics are referred as conditioned mean and standard deviation (i.e. the standard deviation given Q_p). Because the trend is accounted for through the regression line of Eq. 7.9, the variance about this line is the measure of dispersion of interest, which is the conditional variance. For the case where the conditional variance is constant within the range of Q_p values of interest, the conditional standard deviation of the model factor is given by the standard error of the estimate (ξ), which is the variation of the model factor values about the regression line.

7.3.1 Results for Regression of M on Q_p

The scatter plots of regression of M on Q_p for the various cases are presented in figure 7.6. The regression equations and the associated standard error of estimate are also shown in this figure. The conditional mean and standard deviation of the model factor for a given pile class are represented by the regression equation and standard error of estimate respectively.

The representation of the mean model factor by an equation captures the fact that the mean model factor is a function of Q_p. Although the mean model factor varies with $Q_{p,}$ for a given pile class values of the standard deviation are generally assumed to be constant.



(a) DNC (b) BNC

(c) DC (d) BC

Figure 7.6: Regression of M on Q_p

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7.3.2 Derivation of model factor statistics

The general regression equation of M on Q_p (i.e. Eq. 7.9) is taken as a function of a random variable. From the computational rules of random variables, the moments of M are given by:

$$E(M) = E(a + bQ_p) = a + b\mu_{Q_p} \quad [7.10]$$

$$\sigma_M^2 = b^2\sigma_{Q_p}^2 \quad [7.11]$$

From Eq. 71, the standard deviation is given by:

$$\sigma_M = b\sigma_{Q_P} = \xi \quad [7.12]$$

The regression parameters and the ensuing conditioned model factor statistics are presented in table 7.5. It should be noted that the expression for computing the mean model factor is not dimensionless. Therefore the conditioned mean model factors presented in table 7.5 are not dimensionless. However, comparison of the conditioned and standard (Table 6.2) model factor statistics shows very little difference. In fact, it is only for driven piles in non-cohesive materials that the standard deviation reduced from 0.30 to the conditional value of 0.29. For BNC and DC, the values of conditional standard deviations are the same as their respective absolute values. Theoretically accounting for the trend significantly reduces the variability provided there is a significant correlation between the parameters under consideration. Accordingly in this study it was shown that the model factor was only statistically significantly correlated to the predicted capacity in driven piles in non-cohesive materials, which explains the slight reduction of variability for this case. For BC, the conditional standard deviation is slightly higher than the absolute standard deviation. This suggests that in the scatter plot of M vs Q_p, a horizontal line about the mean model factor best fit the data compared to the regression line. Visual inspection of figure 7.6(d) seems to support that a horizontal line drawn at M=1.2 will fit the data better.

Due to the weak correlation between M and Q_p, it can be concluded that accounting for the trend between the two parameters does not lead to reduced variability. Since there is no noticeable reduction of variability, it follows that no significant improvement to the calculated reliability index will be achieved by using conditional model factor statistics

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rather than the standard model factor statistics. Accordingly the approach was not considered any further.

Table 7-5: Conditioned model factor statistics

Pile class	Regression parameters		Mean	Model factor statistics		
	a	b	Q_p	M	STD	COV
DNC	1.2938	-8.9903E-05	2702.59	1.05	0.29	0.28
BNC	0.9321	2.4371E-05	1787.16	0.98	0.24	0.25
DC	1.3414	-7.7490E-05	2286.97	1.16	0.30	0.26
BC	1.1661	-3.9715E-06	3100.48	1.15	0.28	0.24

7.4 Comparison in terms of reliability indexes

In this study the objective of accounting for the correlation was to minimize its effects on the calculated reliability index. Accordingly this section investigates the extent of improvement to the calculated beta values imparted by the removal or incorporation of the correlation. Regarding the conditioned model factor approach, the model factor statistics are similar to those for standard model factor approach. Given that the performance functions for the two approaches will be identical, the two approaches will give similar results. Therefore there is no need to compute the beta values based on the conditioned M approach. For the generalised model factor approach, the statistics are different for that of the standard approach and on the basis of the statistics, it can be seen that no significant reduction of the variability has been realized. However, the performance function for the approach incorporates the regression parameter “b” as an exponent to the Qp. The effects of “b” might lead to improve the calculated reliability. This necessitated the comparison of beta values derived on the basis of the standard and generalised model factor statistics.

Instead of computing reliability indexes for a few selected pile design cases, the reliability indexes implied by the current working stress design method were evaluated using the compiled database. A motivation for computing the reliability indexes inherent in the current practice is that, the results can be used as the basis for the selection of target beta for pile foundations in South Africa. For a given pile class, the reliability index implied by the current practice was evaluated by the advanced first-order second moment method using the nonlinear optimisation function in Excel as discussed in Chapter 3.

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7.4.1 Design equation and performance function for the working stress design approach

Currently, in South Africa geotechnical design is based on the working stress design approach. With this approach, the resistance is factored while the loads are unfactored. If only dead load plus live load are considered with wind load ignored as it is the case for design of pile foundations for most common structures, the design equation is given by:

$$\frac{R_n}{FS} = D_n + L_n \qquad [7.10]$$

Where: R_n = Nominal resistance (predicted pile capacity), D_n = nominal dead load, L_n = nominal life load and FS = factor of safety.

For simplicity the calculation was done in the load space which entails expressing L_n in terms of D_n. By so doing, it is not necessary to deal specifically with pile diameter and length. When L_n is expressed in terms of D_n, Eq. 7.10 becomes:

$$\frac{R_n}{FS} = D_n\left(1 + \frac{L_n}{D_n}\right) \qquad [7.11]$$

From Eq. 7.11 the expressions for D_n and L_n are as follows:

$$D_n = \frac{R_n}{FS\left(1 + {L_n}/{D_n}\right)} \qquad [7.12]$$

$$L_n = \frac{R_n}{FS} - D_n \qquad [7.13]$$

The limit state function is given by:

$$R - D - L = 0 \qquad [7.14]$$

Where *R*, *D*, and *L* are random variables defined as follows:

R = measured resistance

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D = measured permanent load and

L = measured variable load

In this study the resistance and load were modelled as lognormal variable. Usually the measured load and resistance are presented in terms of their respective predicted values and mean model factors as follows:

$$R = M_R R_n ; \quad D = M_D D_n ; \; L = M_L L_n \qquad [7.15]$$

Where: M_R = mean model factor for resistance, M_D = mean model factor for dead load, M_L = mean model factor for live load and the other symbols are as previously defined.

Substituting Eq. 7.15 into Eq. 7.14, the performance function becomes:

$$M_R R_n - M_D D_n - M_L L_n = 0 \qquad [7.16]$$

For the generalised model factor approach, the design equation is the same as that for the standard model factor approach (i.e. Eq. 7.11 through Eq. 7.13). However the performance function becomes:

$$M_R R_n^b - M_D D_n - M_L L_n = 0 \qquad [1.17]$$

in which the exponent b is the appropriate regression parameter given in table 7.3.

The formulations were set on a spread sheet to compute beta values implied by the current design practice. The spreadsheets for computation of beta values as well as the description of the Excel functions for both standard M and generalised M approaches are presented in Appendix C.

The model factor statistics for load were deduced from the South African loading code (Chapter 2). The factor of safety was taken as 2.5 in accordance with common practice in South Africa.

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7.4.2 Beta values based on the standard model factor statistics

Beta values were sensitive to the variation in L_n/D_n ratio and not the variation in the nominal resistance, implying that the calibration points are only defined by the L_n/D_n ratio. Accordingly beta values were calculated for a range of L_n/D_n ratio. The results are presented in figure 7.7.

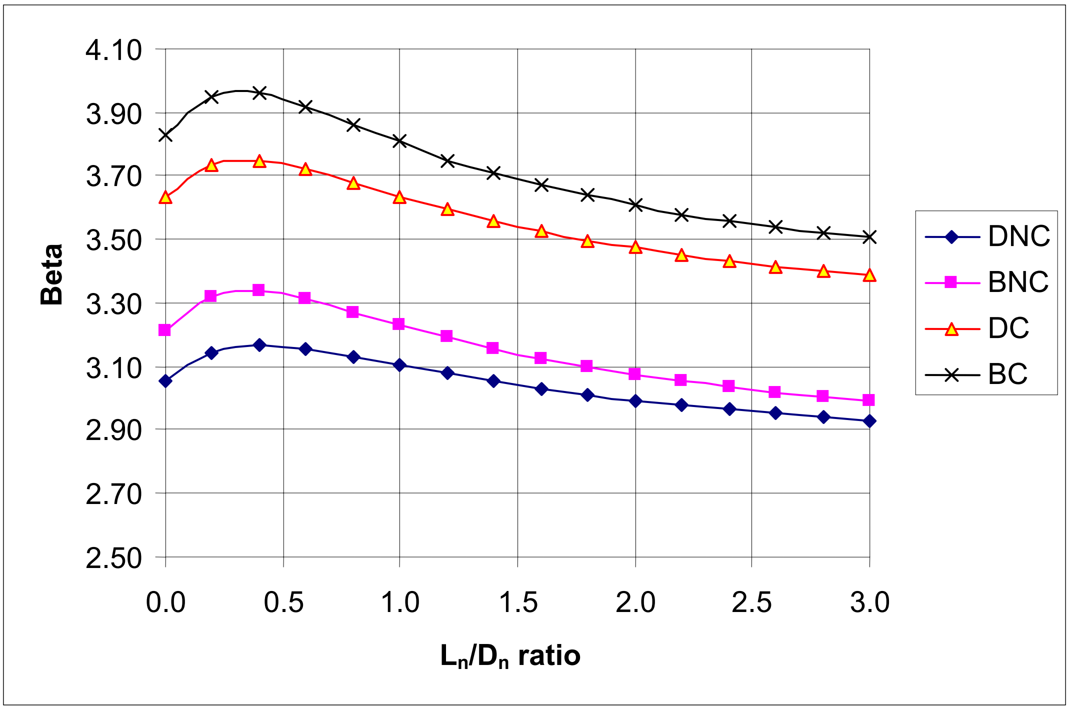

Figure 7.7: Variation of beta values with L_n/D_n ratio for standard M approach

The following observations were drawn from figure 7.7:

- The beta values vary from one pile class to the other indicating that the current global factor of safety approach does not produce a consistent level of reliability across different design situations. Further more even within the same pile class beta values vary with individual cases represented by the L_n/D_n ratios. This suggests that even within the same design situation the global factor approach fails to attain a consistent level of reliability.
- It is evident from figure 7.7 that piles in cohesive materials depict higher reliability compared to piles in non-cohesive materials. The scenario is attributed to higher inherent conservatism of the static formula in non-cohesive materials compared to non-cohesive materials.

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- For a given material type, the beta values for bored piles are higher than that for driven piles. This is attributed to the observation noted in chapter 6 that for a given material type the variability of the model factor in driven piles was higher than that in bored piles
- In general, the variation of beta values with the L_n/D_n ratio seems to be dependent on the nature of the structure. For structures that are predominated by dead loads (Ln/Dn < 0.6), beta values tends to increase with the increase in the L_n/D_n ratio. Conversely as live loads become significant (i.e. Ln/Dn ≥ 0.6), beta values decreases with the increase in the L_n/D_n ratio.
- Within the range of L_n/D_n ratios where beta values decrease with increase in the ratio, the change in the beta values are not substantial.
- For a given material type, the beta values for bored piles are higher than that for driven piles. This is attributed to the observation noted in chapter 6 that for a given material type the variability of the model factor in driven piles was higher than that in bored piles.

To facilitate the comparison between beta values derived by the various model factor statistics, a single representative beta from each approach is required. Ideally such a value could be determined from a weighted average of all the calculated beta values (Melchers, 1999). In practice this approach is seldom feasible owing to the large number of calibration points and the difficulty of assigning a suitable weight to each point. A practical approach is to note the complexity of the issue and select representative beta values on a semi intuitive basis (Melchers, 1999). In this study, a critical value was used instead of the representative value. The critical value was the smallest value within a typical range of Ln/Dn ratios encountered in practice. Typical range of L_n/D_n ratios are 0.5 – 1.5 for concrete structures and 1 – 2 for steel structures (Melccres, 1999). Based on this information, a practical range of L_n/D_n ratio of 0.5 to 2 was adopted. Since within this practical range, the beta values decrease with the increase in Ln/Dn ratio, the lowest values correspond to L_n/D_n ratio of 2. From figure 7.7 the critical values are 2.99 for DNC, 3.07 for BNC, 3.47 for DC and 3.61 for BC.

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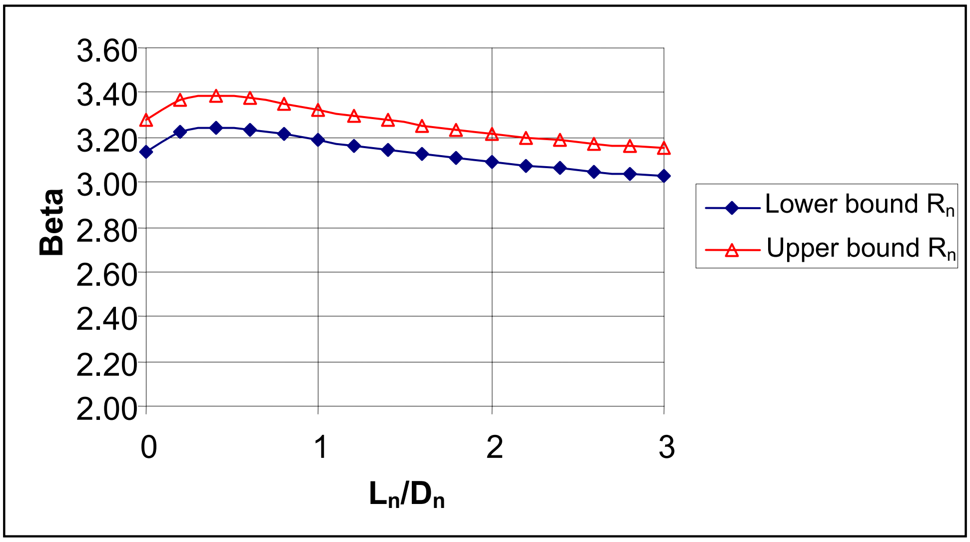
7.4.3 Beta values based on the generalised model factor statistics

Beta values based on the generalised M statistics vary with L_n/D_n ratio as well as nominal resistance values (R_n). This necessitates the determination of beta values for representative R_n values. To obtain the range of beta values, only values corresponding to the lower and upper bounds R_n need to be determined. The Lower and upper bounds R_n values were in accordance with values in the pile load tests database for the respective cases. The range of R_n values for the respective cases are presented in table 7.6.

Table 7-6: Range of R_n (Q_p) values in the database

Case	**Range** (kN)
Driven piles in non-cohesive materials (DNC)	1080-4969
Bored piles in non-cohesive materials (BNC)	472-6754
Driven piles in cohesive materials (DC)	556-5908
Bored piles in cohesive materials (BC)	462-10172

The beta values for the lower and upper bounds Rn corresponding to a range of L_n/D_n ratio, are presented in figure 7.8a through 7.8d.



(a) DNC

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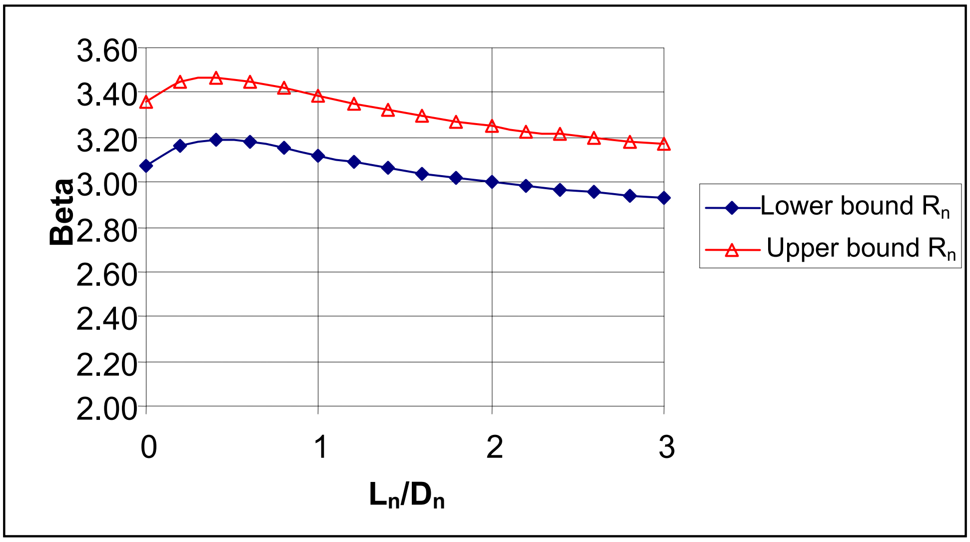
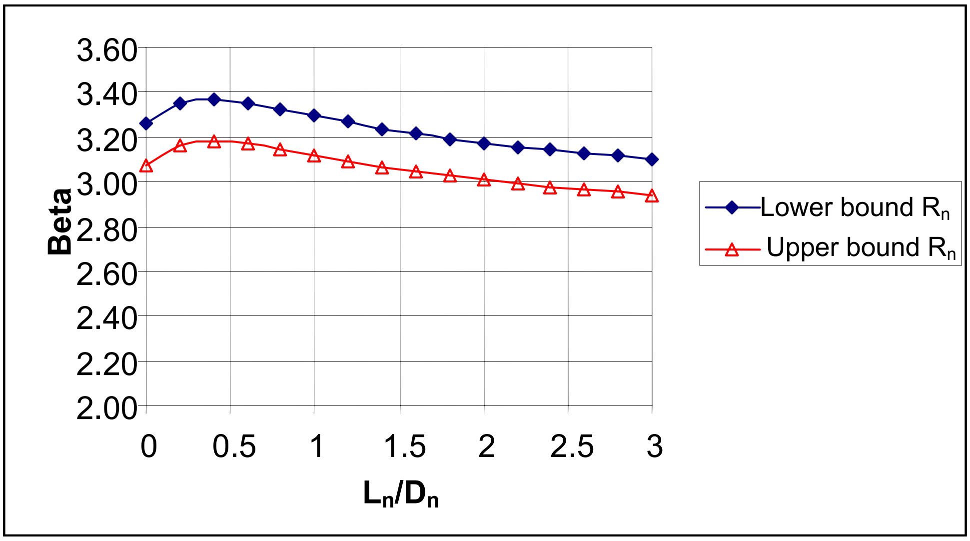
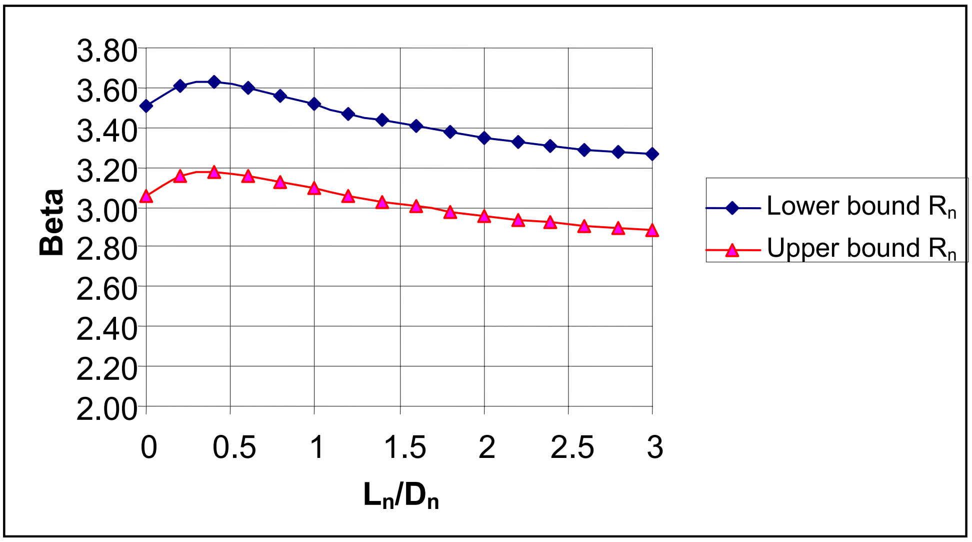


(b) BNC



(c) DC



(d) BC

Figure 7.8: Variation of beta values with L_n/D_n ratio for generalised M approach

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The analysis of figure 7.8 yielded the following observations:

- For driven piles in non-cohesive materials, beta values increases with the increase in R_n values.
- For bored piles in non-cohesive materials, also beta values increase with the increase in Rn values.
- For driven piles in cohesive materials, beta values decreases with the increase in R_n values.
- Also for bored piles in cohesive materials there is a decrease in beta values with the increase in R_n.
- The variation in beta values with R_n value depends on the magnitude of the exponent b. When b is greater than 1, beta increases with the increase in R_n as was the case for DNC and BNC. The b values for these two cases were 1.031 for DNC and 1.030 for BNC. Conversely when b is less than 1 the beta values decreases with the increase in R_n as was the case for DC and BC (b=0.977 for DC and 0.961 for BC). This comes from the mathematical set up of the performance function ($g = MR_n^b - MQ$).
- The variation of beta values with L_n/D_n ratio follows a similar trend to the one shown by beta values based on standard M statistics (i.e. in general beta values increases with the increase in L_n/D_n ratio for $L_n/D_n < 0.6$ and there after, beta values decrease with the increase in L_n/D_n ratio).

Again, the range in beta values implied by the working stress design approach varies widely. This wide variation of beta values reiterate the fact that the current working stress design approach does not produce uniform beta values even in the same design case (e.g. bored piles in cohesive materials).

As was the case for standard M approach, within the practical range of L_n/D_n ratio (i.e. 0.4 to 2), beta values decrease with the increase in L_n/D_n ratio. Therefore the critical beta value corresponds to an L_n/D_n ratio of 2 for either lower or upper bound R_n value, depending on which gives smaller beta values. From figure 7.8, the critical values are 3.09 for DNC, 3.00 for DNC, 3.02 for DC and 2.96 for BC.

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7.4.4 Comparison of magnitudes of beta values

The critical beta values corresponding to the practical range of L_n/D_n of 0.5 to 2 are summarised in table 7.7. Close inspection of the critical beta values for the two approaches lead to the following observations:

- The critical beta values are close to the specified target beta of 3. Therefore all the three approaches yield beta values that meet the target reliability index specified by SANS 10160. This further suggests that the three approaches produce satisfactory results; hence any of them can be adopted.
- A comparison of beta values shows that values for the standard M are significantly higher than that for generalised M in cohesive materials. However, in non-cohesive materials, the difference between the results of the two approaches is negligible. Therefore the generalised M approach does not produce improved reliability indexes.

Table 7-7**:** Critical beta values for L_n/D_n ratio of 0.5 to 2

Case	Standard M	Generalised M
DNC	2.99	3.09
BNC	3.07	3.00
DC	3.47	3.02
BC	3.61	2.96

From the foregoing, it can be concluded that the results of the supposedly more refined approach is not radically different from those of the conventional approach (standard M approach). This state of affairs can be traced back to the degree of the correlation established in chapter 6. It was shown that except for DNC, the correlation between the model factor and predicted capacity was statistically insignificant. Therefore the removal of the existing weak correlation can not lead to considerable improvement in the beta values. Therefore, while acknowledging the potential of the generalised M approach as an alternative method for deriving model factor statistics, in this study the resistance factors will be calibrated on the basis of the standard model factor statistics. Additional advantages of adopting the standard model factor approach include:

- The approach is simple to use as the calibration points are limited to L_n/D_n ratio only.

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- The approach has been used in other calibration studies, thereby facilitating the comparison of the calibration results with those obtained in other studies. This is crucial in the case of Southern Africa as this is the first exercise of its kind.
- Various calibration equations based on standard model factor statistics have been developed. This will make it possible to derive the resistance factors using well developed and tested calibration equations.

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Chapter 8

RELIABILITY CALIBRATION OF RESISTANCE FACTORS

8.1 Introduction

As alluded to in Chapter 2, the partial factors format has become the orthodox method for verification of non-accidence of the limit states. In the context of reliability based design, the partial factor method constitutes an important step from the complex direct probabilistic methods towards simplified design procedures (i.e. level 1 reliability methods). In conjunction with the calibration principles set in Chapter 3 and the model factor statistics developed in the study, this Chapter is primarily focused on deriving the resistance partial factors.

8.2 Target beta

The primary factors influencing the selection of target beta values were identified in Chapter 3 as:

- Reliability indexes implied by the current design approach
- Target beta value set by regulatory authorities for a given limit state
- Redundancy inherent in the system

Reliability indexes inherent in the current design practice captures the long successful South African design experience. The beta values implied in the current working stress design approach were developed Chapter 7. The critical beta values corresponding to the practical range of Ln/Dn of 0.5 to 2 were presented in table 7.7. The same value, but now rounded to one dismal place are presented in table 8.1.

On the basis of beta values presented in table 8.1, it can be concluded that reliability indexes implied by the current working stress design practice are comparable to the prescribed target beta of 3. Therefore the two selection criteria of beta values implied by the current practice and the target beta value set by the South African loading code suggest a target beta of 3. However, the redundancy inherent in pile foundation system needs to be considered.

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Table 8-1: Critical beta values

Case	β value
DNC	3.0
BNC	3.1
DC	3.5
BC	3.6

In pile foundations, it is not common for a single pile to support a structural load on its own because of overturning effects, lateral loading and difficulty of transferring the load axially down the pile (Barnes, 1995). In most cases piles are used in groups with the structural load shared between the piles. In the pile group, failure of an individual pile does not necessary imply that the group will fail as adjacent piles that may be more lightly loaded could take some of the additional load (Zhang, et al 2001). Therefore beta for pile groups can be significantly higher than that of single piles. Further more, in a pile system of several groups (most common practical case) supporting a structure, failure of a pile group does not necessarily mean the pile system will fail. Load redistribution among the pile groups through superstructure elements may occur due to system effects resulting in an additional reliability. The flexibility of the soil contributes to the ability of the pile-soil system to share and redistribute load. Therefore redundancy is prevalent in the field operation of pile foundations. Because of presence of redundancy, pile foundation can be designed for a lower target beta value than superstructure components.

In the case of pile foundations, the general target beta value prescribed by regulatory authorities should be considered as a target value for pile groups. The task now is to determine the corresponding target beta values for single pile design (β_{TS}) to achieve the target reliability of the pile group (β_{TG}). In general (β_{TS}) values are smaller than the (β_{TG}) value for the pile group. In this regard Zhang et.al (2001) showed that for a (β_{TG}) value of 3 the corresponding (β_{TS}) values were in the range of 2.0 – 2.8 if no system effects are considered and further decreased to 1.7 to 2.5 when a system factor of 1.25 is considered. The (β_{TS}) values reduced even further to 1.7 – 2.0 when a larger system factor of 1.5 is considered.

From the foregoing it can be inferred that representative beta values implied in the current practice of about 3 for piles in non-cohesive materials and about 3.5 for piles in cohesive materials will significantly increase if group effects and system effects are taken into account. This further implies that the target reliability of the pile group implied in the current design practice is higher than 3. In other words current practice is over-conservative.

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Therefore the representative beta values of 3.0 and 3.5 need to be reduced so that when group and system effects are considered β_{TG} should be a minimum of 3 but not much higher than 3.

Calculation of single pile reliability on the basis of group reliability or vice-versa is possible if pile test data on pile groups are available. However in this study such data is not available and therefore mapping from single pile reliability to group reliability or vice-versa is based on intuition and values reported in the literature. Hence a target beta of 2.5 is considered sufficient for building structures and other common structures (i.e. Class RC 2). For class RC 3 a target beta of 3 is adopted while for class RC 1 structures a target beta of 2 is considered. Table 8.2 compares the target values recommended for pile foundations to values given in the revised South African loading code (SANS 10160-Draft).

Table 8-2: Comparison of β_T in SANS 10160 the β_T adopted for the study

Class	β_T –SANS 10160	β_T - recommended for pile foundations
RC 3	3.5	3.0
RC 2	3.0	2.5
RC 1	2.5	2.0

Depending on the failure consequence class the recommended target beta values range from 2.0 – 3.0. This range of target beta values has also been adopted in other calibration studies around the world. Baecher and Christian, (2003) asserted that in modern foundation codes, target reliability indexes ranging from 2.0 to 3.0 are common. The lower range is for cases of non-essential designs with high redundancy while the upper range is for critical designs with little redundancy. Paikowsky et.al (2004), as a revision to the driven piles and drilled shaft section of the AASHTO LRFD Bridge Design Specifications, recommended $\beta_T = 2.33$ for redundant piles (5 or more piles per pile cap), and $\beta_T = 3.0$ for non-redundant piles (≤ 4 piles per pile cap). Becker, et.al (1991) selected β_T of 2.0 to 2.5 for calibration of resistance factors of driven piles and Withiam et.al (1998) confirmed that this range of target beta is reasonable for a single pile design considering that piles are usually used in groups. Rahman et.al (2002), asserted that β_T of 2.0 to 2.5 was within conformity with β values implied by their current design practice and hence selected this range of β_T for the calibration of resistance factors for their study.

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8.3 Calibration Methods

A number of expressions for reliability calibration of resistance factors proposed in the literature were presented in Chapter 3. In addition, direct calibration to the existing practice was carried out for the purposes of comparison of the results. The following five approaches were explored:

d) Advanced first-order second moment approach (AFOSM)
e) First-order second moment approach (FOSM)
f) Approximate to first-order second moment approach
g) Design point approach
h) Calibration by fitting to WSD

8.3.1 Advanced first-order second moment approach

As pointed out in Chapter 3 AFOSM analysis entails an iteration process. For calibration of partial factors, limit state design equations are required as opposed to the working stress design equations used for deriving beta values implied by the current practice. With the limit state design equations, both the resistance and the loads are factored. Accordingly, the design equation is given by:

$$\frac{R_n}{\gamma_R} = \gamma_G G_k + \gamma_Q Q_k \qquad [8.1]$$

Where: R_n = predicted pile capacity; γ_R = partial resistance factor; G_k = permanent action; γ_G = partial factor for permanent action; Q_k = variable action; and γ_Q = partial factor for variable action.

It was pointed out in Chapter 2, that for Design Approach 1 two combinations of load and resistance partial factors need to be considered. However it has become customary that combination 1 governs the structural design of the elements while combination 2 governs the geotechnical of the elements. Accordingly for the GEO limit state, the values of the load factors γ_G and γ_Q are 1.0 and 1.3 respectively. Therefore Eq. 8.1 becomes:

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$$\frac{R_n}{\gamma_R} = 1.0G_k + 1.3Q_k \qquad [8.2]$$

Expressing Q_k in terms of G_k

$$\frac{R_n}{\gamma_R} = G_k\left(1 + \frac{1.3Q_k}{G_k}\right) \qquad [8.3]$$

From Eq. 8.3, the expressions for G_k and Q_k are as follows:

$$G_k = \frac{R_n}{\gamma_R\left(1 + {}^{1.3Q_k}\!/_{G_k}\right)} \qquad [8.4]$$

$$Q_k = \frac{1}{1.3}\left(\frac{R_n}{\gamma_R} - G_k\right) \qquad [8.5]$$

The performance function is the same as that developed for the working stress design approach and is given here as Eq. 8.6.

$$M_R R_n - M_D D_n - M_L L_n = 0 \qquad [8.6]$$

In equation 8.6 D_n and L_n have the same meaning as G_k and G_k respectively.

The design equation, the performance function, and the model factor statistics were then set on a spread sheet to facilitate the computation of the partial resistance factor that will yield a prescribed target beta value for a given set of model factor statistics. The spreadsheet and the description of Excel functions for the various cells are presented in Appendix D. The basic algorithm of the AFOSM for resistance factor calibration is similar to that of the AFOSM reliability analysis presented in Chapter 7. Therefore the spreadsheet in figure 8.1 is similar to that presented in chapter 7. However the process of obtaining the resistance factors is a reverse of what was used to determine the beta values for a given set of model factor statistics and factor of safety. The procedure was as follows:

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- Enter a trial resistance factor value on the spread sheet;
- Evoke the solver function to compute the beta value corresponding to the entered resistance factor;
- Repeat steps 1 and 2 till the computed beta value is equal to the desired target beta value.

The resistance factors were calibrated for β_T = 2.5 (for class RC 2 structures), β_T = 3.0 (for class RC 3 structures) and βT = 2.0 (for class RC 3 structures). The model factor statistics used in the calibration are presented in table 8.3. These are the same model factor statistics derived in chapter 6.

Table 8-3**:** Model factor statistics used in the calibration of resistance factors

Case	Mean	Std. dev.	COV
DNC	1.06	0.30	0.28
BNC	0.98	0.24	0.24
DC	1.17	0.30	0.26
BC	1.15	0.28	0.24

8.3.1.1 Resistance factors for a target beta of 2.5

The resistance factors as a function of the Ln/Dn ratio are is shown in figure 8.1.

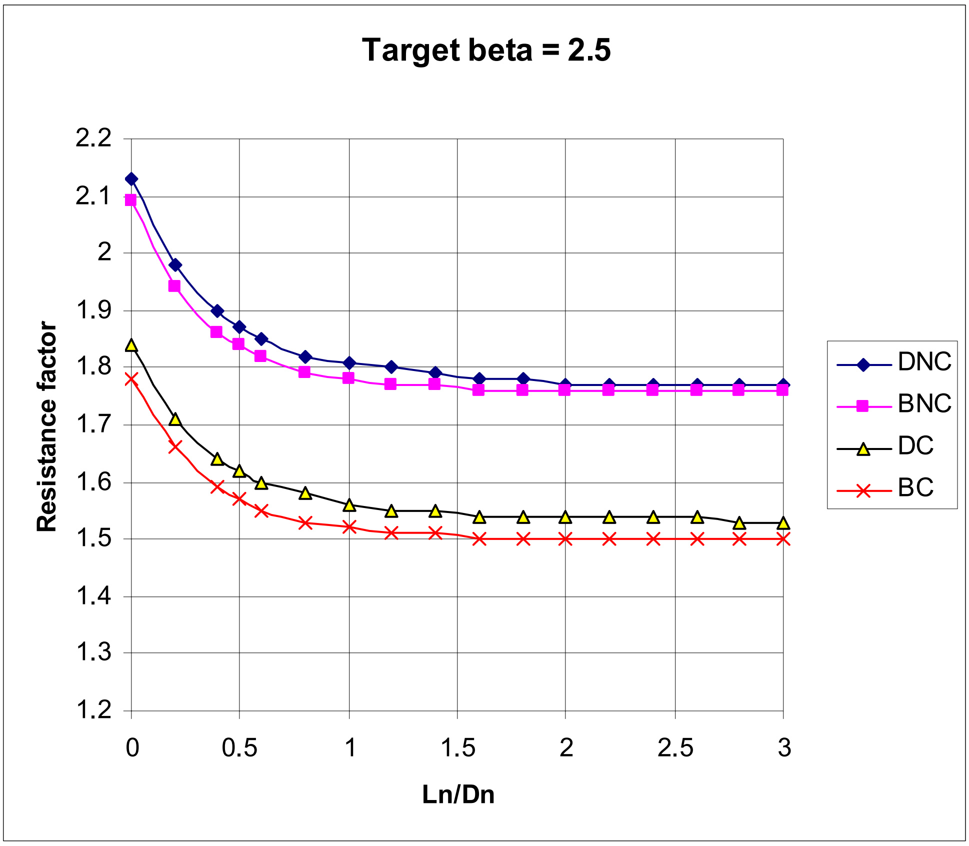

Figure 8.1: Variation of resistance factors with Ln/Dn ratio

The following observations can be drawn from figure 8.1:

- For all the cases, there is a general decrease in values of resistance factors with the increase of the L_n/D_n ratio. However, the rate of decrease reduces with the increase in L_n/D_n ratio. From an L_n/D_n ratio of 0 – 1 the rate of decease is appreciable while at an L_n/D_n ratio of greater than 1, the rate of decrease is very small.
- The values of the resistance factors appear to be influenced by the materials types. In this regard, the partial factors for piles in cohesionless materials are quite close irrespective of the pile installation method. The same trend is exhibited by piles in cohesive materials.

The variation of resistance factor with the various calibration points is an indication of the variation in reliability indexes across the calibration points or design situations within a given class of pile foundations. Despite the variation in resistance factors across the calibration points, a single value applicable to all the design situations is required for the purposes of codified design. The application of a single resistance factor to all the design situations will inevitably lead to some deviation from the target reliability index for some of

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the calibration points. Therefore the partial factors should be calibrated such that the beta values for the various calibration points are as close as possible to the target reliability index.

In principle the partial factor which best approximate the uniform target reliability can be obtained by minimizing the deviation from the target beta using an objective or penalty function penalising the deviation from the target reliability index. A number of objective functions have been proposed in the literature. Ideally such functions should be based on economic terms (i.e. cost benefit analysis). Detailed discussion on this form of objective function can be found in Ditlevsen and Madsen (2005). However, in most current calibration exercises, the mean square deviation of the calculated reliability index from the target value has been used (e.g. Gayton et al, 2004). The most common and simplest one is the weighted least function given by:

$$S = \sum_{i}^{n} w_i \left(\beta_i - \beta_T\right)^2 \qquad [8.7]$$

Subject to the constraint:

$$\sum_{i}^{n} w_i \left(\beta_i\right) = \beta_T \text{ with } \sum_{i}^{n} w_i = 1$$

Where S = measure of closeness to the target reliability index; β_T = target reliability index; β_i = the value of the ith reliability index computed on the basis of the partial factor under consideration; w_i = a weighting factor to account for the importance of the calibration point (Ln/Dn ratio) relative to design practice.

It is common to express equation 8.6 in terms of probability of failure. In this regard, logarithm of the probability of failure is used to make S relatively more sensitive to very low values of failure probabilities. The resulting expression is as follows:

$$S = \sum_{i}^{n} w_i \left(\log P_{fC} - \log P_{fT}\right)^2 \qquad [8.8]$$

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Where $P_{fT} = \Phi(-\beta_T)$ is the target probability of failure; $P_{fC} = \Phi(-\beta_i)$ is the probability of failure for a given calibration point.

The main disadvantage of above penalty function is that it is symmetrical about β_T and hence calibration points with a reliability index smaller than the target are not more penalised than structures with a higher reliability index. As a solution to this problem, skewed objective functions giving relatively more weight to reliability indices smaller than target compared to those larger than the target have been proposed in the literature. A commonly used function in this category is that proposed by Lind (1977). The function is given by:

$$S = \sum_{1}^{i} w_i (k(\beta_i - \beta_T) + \exp(-k(\beta_i - \beta_T)) - 1) \qquad [8.9]$$

in which k>0 is the curvature parameter and the other symbols are as defined earlier.

When the parameter k increases the Lind's function becomes more penalising for reliability indexes smaller than the target value. Gayton et.al (2004) observed that for k=1 the Lind's function gives results that are close to the least square function while still penalising under-designs more than over-designs.

In general, it has been found that the final result is not very sensitive to the choice of the penalty function (Ditlevsen and Madsen, 2005; Gayton et.al, 2004). This suggests that any of the three expressions discussed above (Eq. 8.6, Eq. 8.7 and Eq. 8.8) can be adopted. However, for this study the three expressions were investigated using data from one of the pile classes. The function leading to conservative results was then chosen and applied to all the other pile classes. The solution to equations 8.6, 8.7 and 8.8 becomes a problem of constrained minimization for which a number of standard techniques and computer programs are available. In the absence of a ready made computer programme for the optimization process, the following procedure was adopted in this study:

- The number of calibration points was reduced by limiting the L_n/D_n ratio to a range that is most frequently encountered in practical designs. Theoretically, Ln/Dn range from 0 - ∞. However, as noted in Chapter 7, in practical designs typical ranges of L_n/D_n are: 0.5 – 1.5 for reinforced concrete structures and 1-2 for steel structures. On the basis of the

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typical ranges, a range of 0.5 – 2.0 was considered to constitute a commonly employed design range.

- Weighting factors were assigned to the chosen calibration points to reflect the relative occurrence of each point in practical design practice. Greater weight has been given to the more frequent design cases. The most common design cases are represented by Ln/Dn range of 1-1.5. This is the range applicable to both concrete and steel structures. The weights assigned to the respective design cases or calibration points are presented in table 8.4.

Table 8-4: Importance weighting for the respective calibration points

Ln/Dn ratio	Weight
0.5	0.1
1.0	0.4
1.5	0.4
2.0	0.1

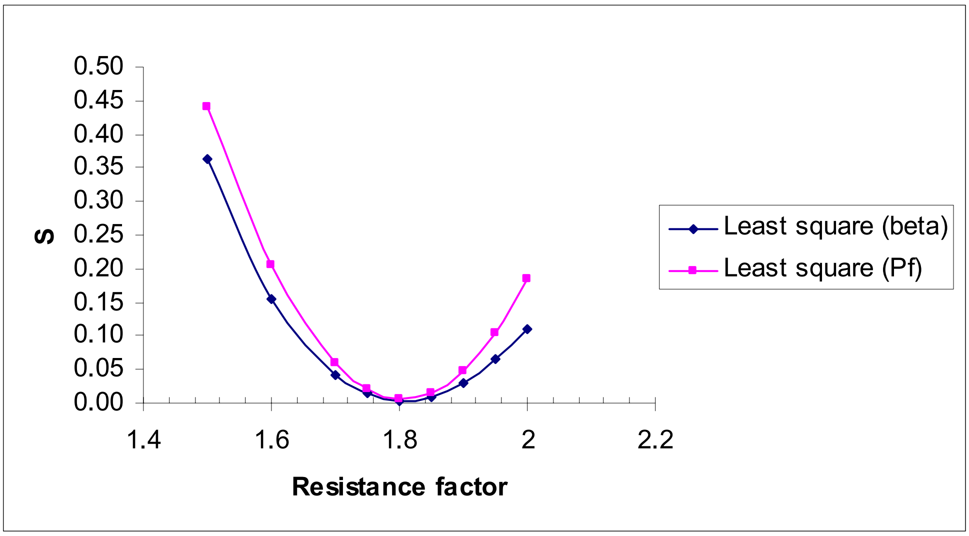
- Trial values of partial factors were used to calculate β_i for each calibration point. The results were substituted in equation 8.6 to 8.8 to calculate the S.
- After a number of trials, a set of partial resistance factors and their respective S values were obtained.
- The partial resistance factors were plotted against their respective S values (S_1= least square in terms of beta, S_2= least square in terms of probability of failure, S_1= Lind's objective function) to obtain a curve from which optimal resistance factor can be read. The optimal resistance factor corresponds to the minimum S value.

Table 8.5 presents the S values for the three objective functions for the DNC case. The corresponding weighted average beta values are also shown in the last column of the table. From table 8.5 it can be seen that for the objective functions based on the least squares the minimum S values correspond to a resistance factor of 1.8 while for the Lind's objective function, the minimum S value corresponds to a partial factor of 1.85. These values are approximate. To obtain more accurate values graphs of S versus resistance factors were plotted (figure 8.2).

Graphs of the trial partial factors against their respective S values for the three functions show the same trend. That is, initially the S values decrease with the increase in the partial factors, then reaches a minimum value and then increases with the increase in the partial factor values. The partial factor corresponding to this minimum S value is taken as the optimum value. For DNC, the lowest S value corresponds to a partial factor of 1.82 for the two least square functions (fig. 8.2 a) and 1.87 for Lind's function (fig. 8.2 b). The values are slightly different from those obtained from table 8.5. It is evident from these results that the Lind's function is on the conservative side. Since it is better to err on the conservative side, Lind's objective function was used in the selection of the optimum partial factor for all the pile classes.

Table 8-5: S values for respective trial γ values

γ	S1	S2	S3	wβ
1.5	0.3619	0.4416	0.0734	1.9
1.6	0.1560	0.2040	0.0336	2.1
1.7	0.0419	0.0581	0.0116	2.3
1.75	0.0143	0.0202	0.0054	2.4
1.8	0.0041	0.0059	0.0017	2.5
1.85	0.0098	0.0153	0.0001	2.6
1.9	0.0303	0.0484	0.0003	2.7
1.95	0.0642	0.1051	0.0020	2.7
2	0.1106	0.1853	0.0049	2.8
2.2	0.402848	0.737793	0.0257	3.1
2.5	1.0844	2.2327	0.0719	3.5



(a) least square penalty functions

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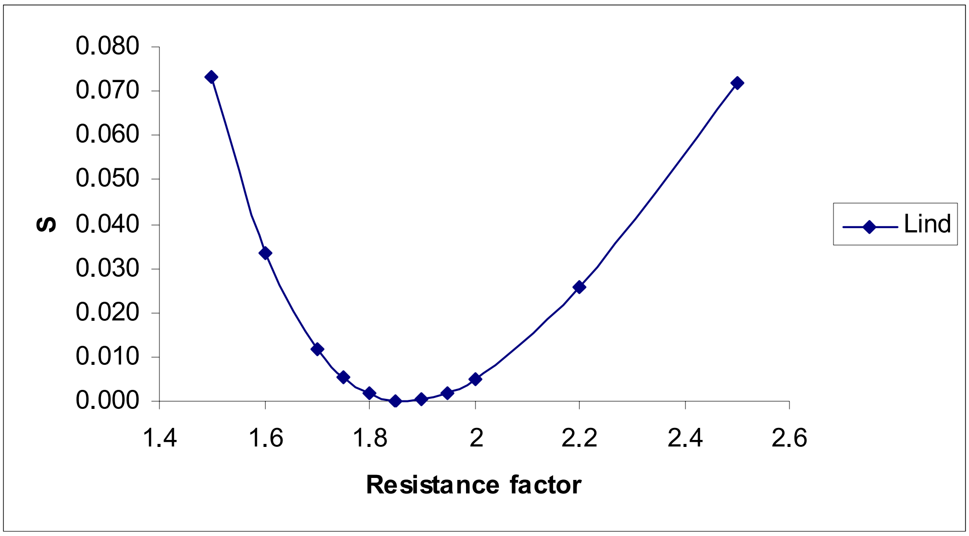
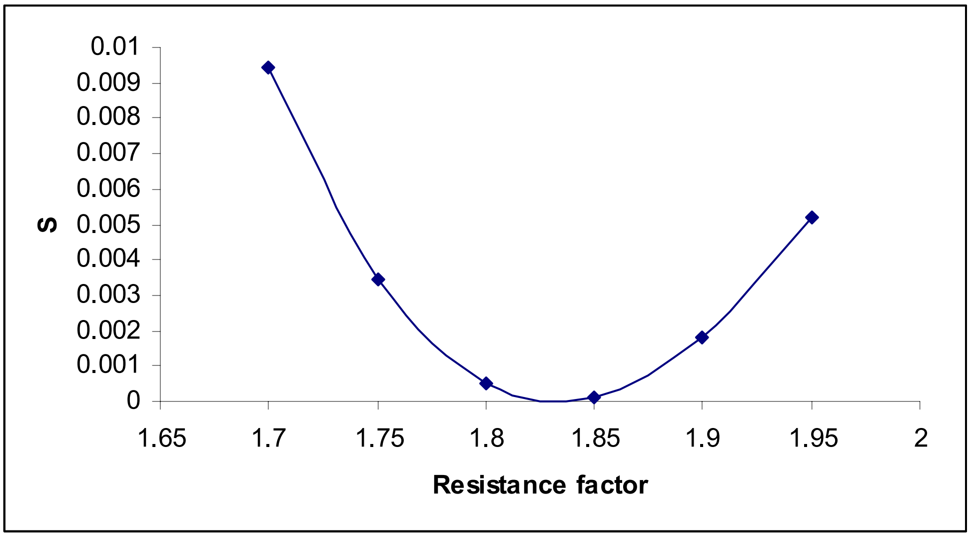


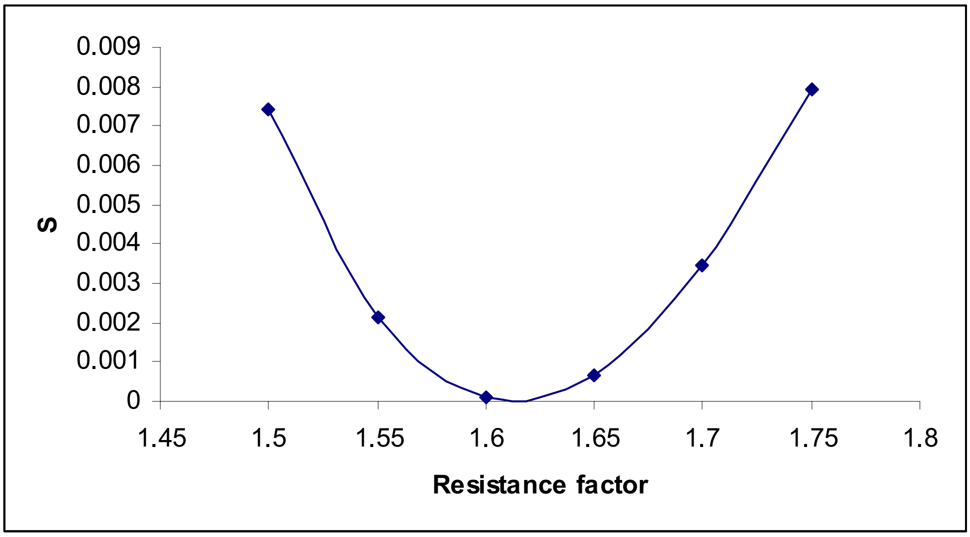
(b) Lind's penalty function

Figure 8.2: Optimal partial resistance factor for DNC

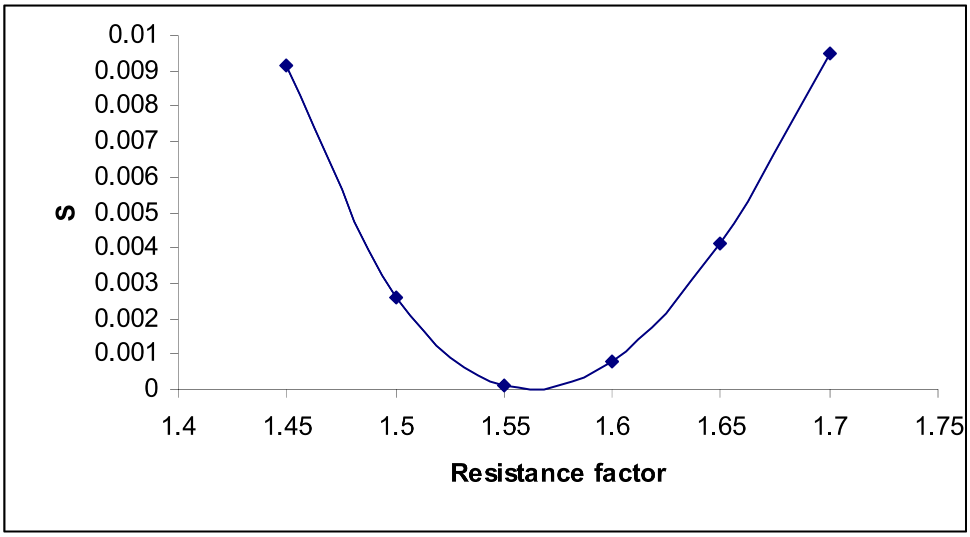
The graphs of S versus partial resistance factors based on Lind's objective function for BNC, DC, and BC are presented in figure 8.3. The ensuing optimal partials factors are 1.84 for BNC, 1.62 for DC and 1.57 for BC.



(a)



(b)



(c)

Figure 8.3: Optimal partial resistance factors; (a) BNC, (b) DC, (c) BC

Scrutiny of the optimal partial factors obtained reveals that these values are the same as the partial factor values for L_n/D_n of 0.5 in all the pile classes. L_n/D_n of 0.5 is the minimum value of the commonly employed design range. Within the practical range of L_n/D_n (i.e 0.5 to 2), the largest partial resistance factor correspond to L_n/D_n of 0.5 as shown in figure 8.2. In terms of reliability index, the calibration point represented by L_n/D_n of 0.5 has the lowest beta value within the range under consideration. In essence, the partial factor for L_n/D_n of 0.5 provides the minimum required reliability index (target beta). Therefore Lind's objective function yields optimal partial factors that ensure that reliability indexes for all calibration points are equal or greater than the target value.

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From the foregoing, it can be concluded that the optimal partial factors could have as well been selected on the basis of values that meet the required minimum reliability index within the range of selected calibration points rather than the tedious optimisation process. For all pile classes, such partial factor values correspond to the calibration point denoted by L_n/D_n of 0.5.

Comparison of the numerical values of the optimal partial factors indicates that the optimal resistance factors for the four pile classes are comparable. Consequently, it is tempting to adopt one single partial factor for all the four classes. Such a partial factor could be established from an optimisation process similar to the one described previously or by simply using the partial factor for the least reliable pile class. The least reliable pile class was identified as BNC (i.e. one with the highest γ_R). Accordingly the single partial factor applicable to all the cases is 1.87. The use of a partial factor of 1.87 for all the four cases gives a minimum beta value of 2.50 for DNC, 2.58 for BNC, 3.04 for DC and 3.19 for BC and an average beta of 2.83. For DC and BC the beta values are appreciably higher than the target value of 2.5, leading to high conservatism. Even the average beta value for the four cases is significantly higher than the target beta by 13.2 %. To avoid high conservatism for piles in cohesive materials (DC and BC), it seems reasonable to apply separate partial factors for piles in cohesive materials and piles in non-cohesive materials. Differentiation in terms of material types is further motivated by the closeness of the partial factors for a given material type irrespective of the installation method (i.e. 1.87 and 1.84 for piles in cohesionless materials; 1.62 and 1.57 for piles in cohesive materials).

Separation of partial factors on the basis of materials type implies further subdividing the four pile classes into two categories of two pile classes each. For each category, the partial factor belonging to the least reliable pile class was selected as the overall partial factor for piles in that specific material. The resulting partial factors are: γ_R = 1.87 for piles in cohesionless materials and γ_R = 1.62 for piles in cohesive materials. Adopting γR = 1.87 for piles in non-cohesive materials yield beta values of 2.5 for BNC and 2.58 for DNC with an average value of 2.54. These values are not very different from the target beta and hence not much additional conservatism has been introduced. For piles in cohesive materials, the use of γ_R = 1.62 produces beta values of 2.52 for DC and 2.64 for BC with an average value of 2.58. As it was the case for piles in non-cohesive materials, the beta values for the two pile classes in cohesive materials are not appreciably different from the target value. Therefore differentiation in terms of material types satisfies the requirements of (i) achieving a

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minimum target reliability index across a specific domain of interest and (ii) increasing the uniformity of reliability across the domain of interest. For ease of reference and comparison with values for other target beta, the optimal values for the target beta of 2.5 are compiled in table 8.6.

Table 8-6: Optimal resistance factors for a target beta of 2.5

Pile class	Optimal γ_R
DNC	1.87
BNC	1.84
DC	1.62
BC	1.57

8.2.1.2 Resistance factors for a target beta values 3.0

The variation of resistance factors with the calibration points (L_n/D_n ratios) are presented in figure 8.4. The analysis of figure 8.4 shows similar characteristics to those shown by resistance factors for the target beta of 2.5. However, the numerical values of resistance factors have slightly increased as expected.

It has been previously observation that the optimal partial resistance factor obtained by minimization of Lind's objective function is very close to partial factors corresponding to the calibration point represented by L_n/D_n of 0.5. Therefore instead of performing the optimisation process, the partial factors for L_n/D_n of 0.5 in figure 8.4 were taken as the optimal partial resistance factors. The resulting optimal partial resistance factors for the various pile classes are presented in table 8.7.

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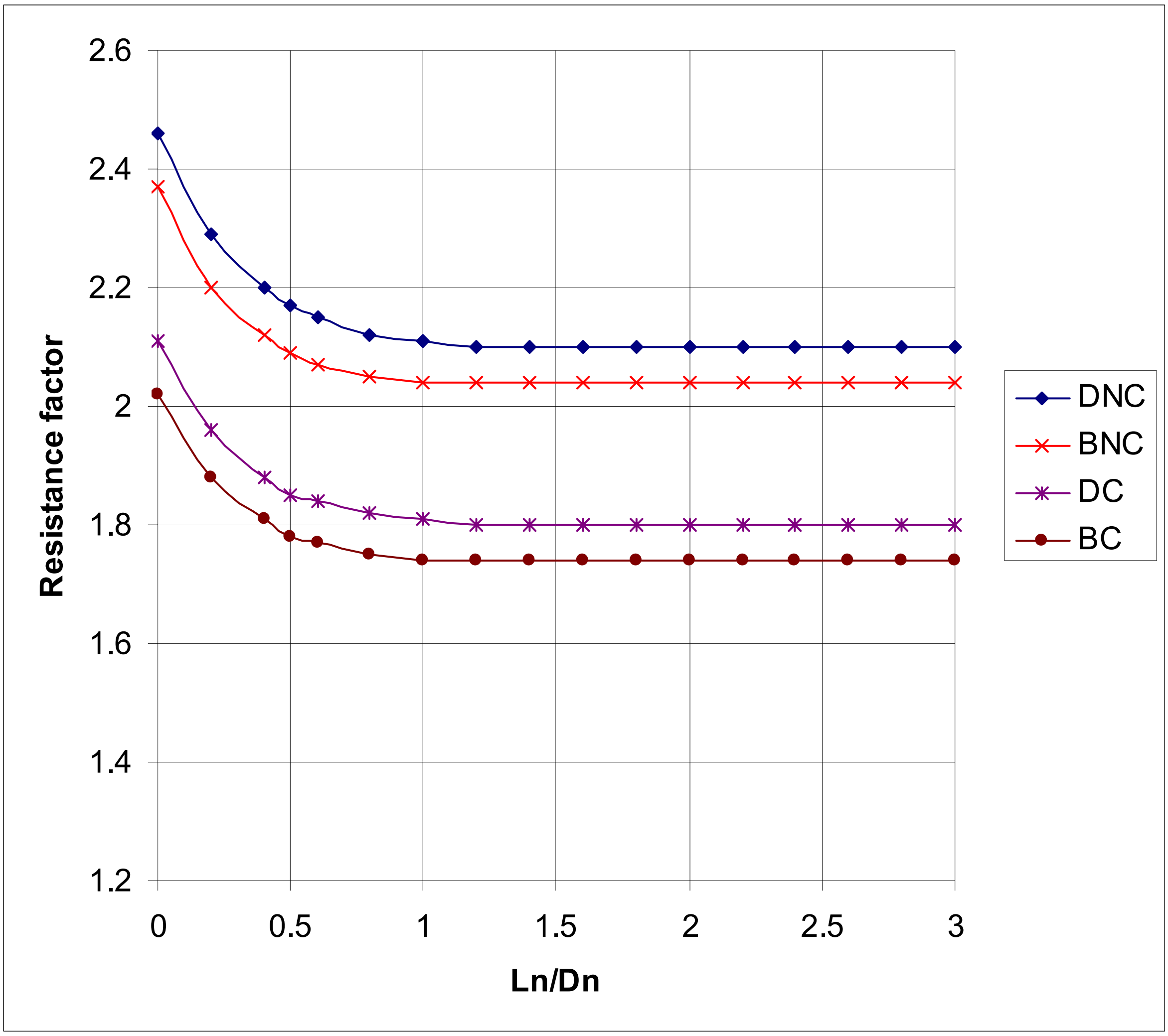

Figure 8.4: Variation of resistance factors with L_n/D_n ratio for $\beta_T = 3.0$

Table 8-7: Optimal resistance factors for a target beta of 3.0

Pile class	γ_R
DNC	2.17
BNC	2.09
DC	1.85
BC	1.78

Differentiating only in terms of material types yields γ_R = 2.17 for piles in non-cohesive materials and γ_R = 1.85 for piles in cohesive materials.

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8.2.1.3 Resistance factors for a target beta values 2.0

The calibration results for the target beta of 2.0 are presented in figure 8.5. Examination of the results reveals characteristics shown by results for other target beta values. The only noticeable difference is in the values of the partial resistance factors which have now been reduced as expected. Comparison of figures 8.2, 8.4, and 8.5 indicate that for a given material type, the partial factor for driven and bored piles become quite close as the target reliability index reduces. This trend is more apparent in figure 8.5 (β_T=2.0) in which curves for driven and bored piles in non-cohesive materials coincide.

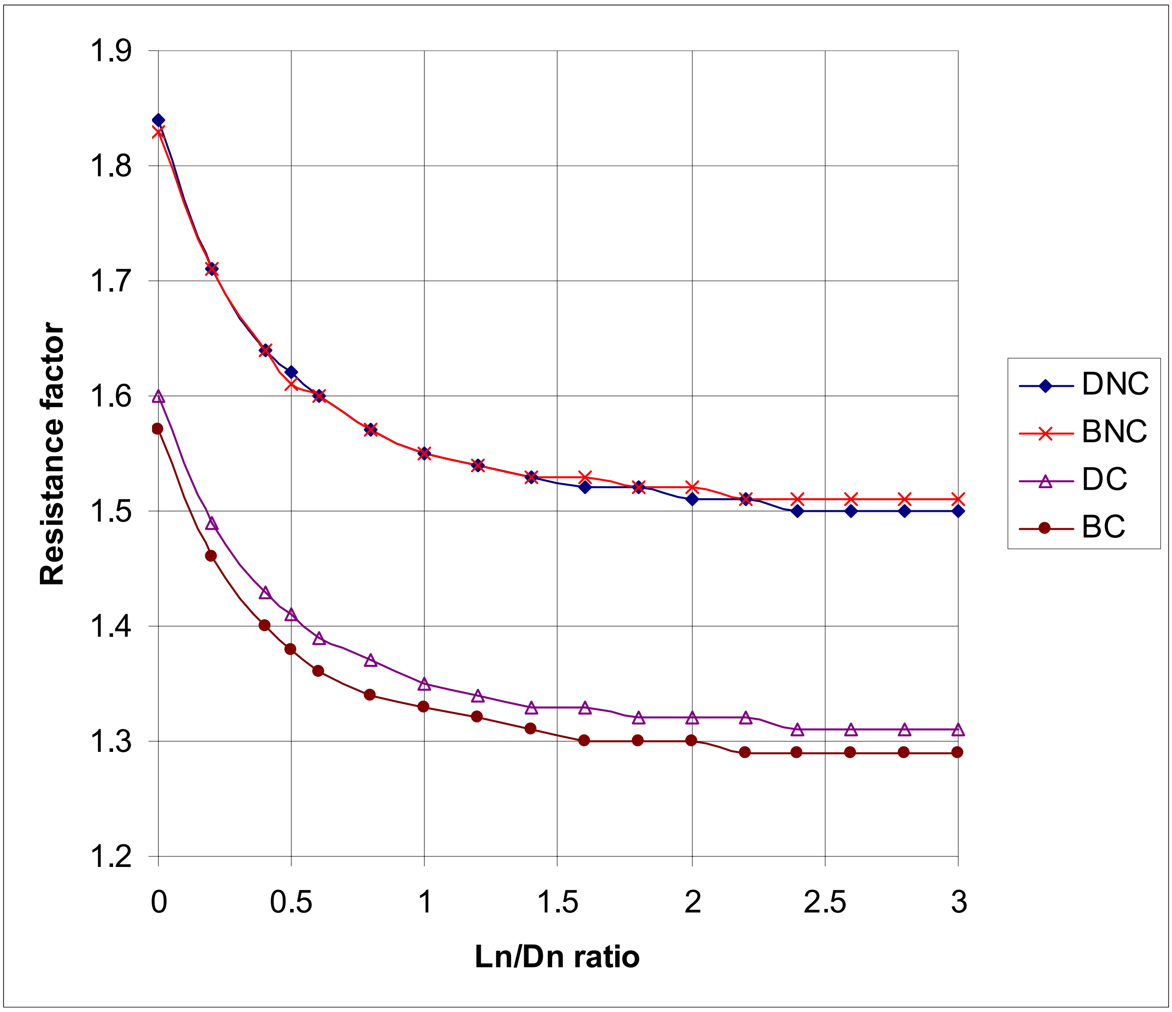

Figure 8.5: Variation of resistance factors with L_n/D_n ratio for $\beta_T = 2.0$

The selection of the optimal resistance factors for the different pile classes was based on values corresponding to L_n/D_n of 0.5 as was the case for target beta of 3.0. The ensuing partial resistance factors are presented in table 8.8. Further classification in terms of material

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types only, yields γ_R = 1.62 for piles in non-cohesive materials and γ_R = 1.41 for piles in cohesive materials.

Table 8-8: Optimal resistance factors for a target beta of 2.0

Pile class	γ_R
DNC	1.62
BNC	1.61
DC	1.41
BC	1.38

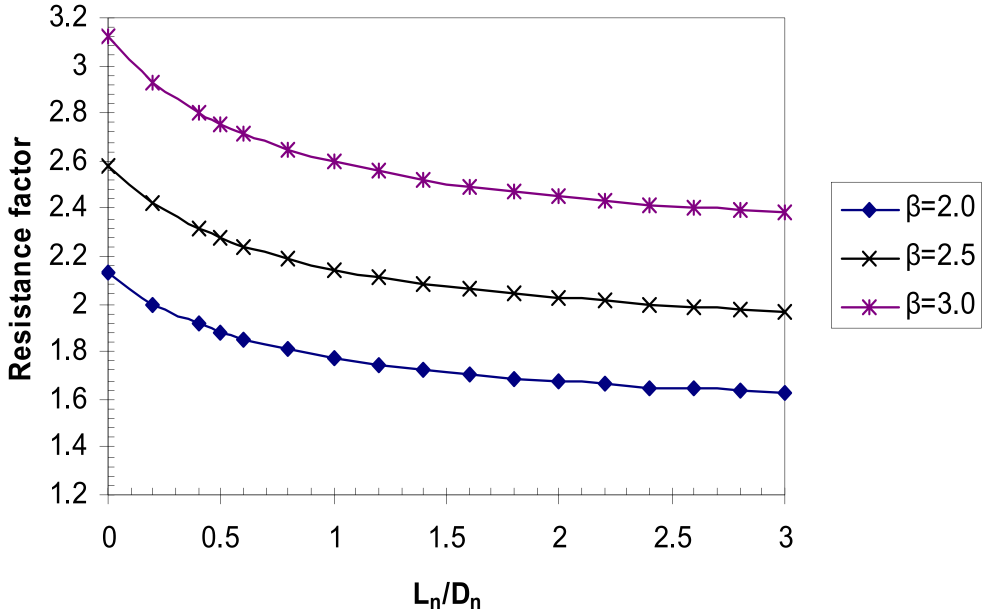
8.3.2 Mean value first-order second moment approach (MVFOSM)

The principal equation for determining resistance factor in accordance with the MVFOSM approach was derived in Chapter 3. This principal equation is given by:

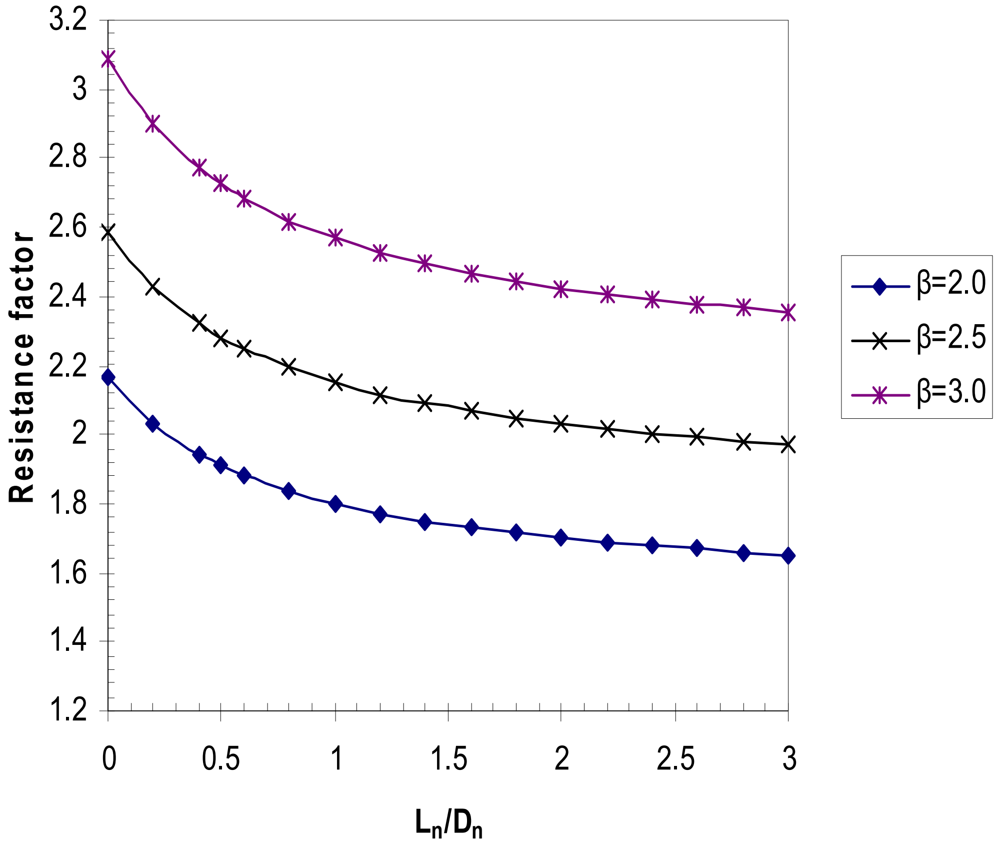
$$\phi = \frac{\lambda_R\left(\gamma_{QD} + \gamma_{QL}\dfrac{Q_L}{Q_D}\right)\sqrt{\dfrac{1+COV_{QD}^2+COV_{QL}^2}{1+COV_R^2}}}{(\lambda_{QD} + \lambda_{QL}\dfrac{Q_L}{Q_D})\exp\left[\beta_T\sqrt{\ln(1+COV_R^2)(1+COV_{QD}^2+COV_{QL}^2)}\right]} \qquad [8.10]$$

All the statistics in Eq. 9.10 are known. Accordingly Eq. 9.10 was set on a spread sheet to automate the calculations. The results are presented in figure 9.7a through 9.7d. The figures show similar characteristics as follows:

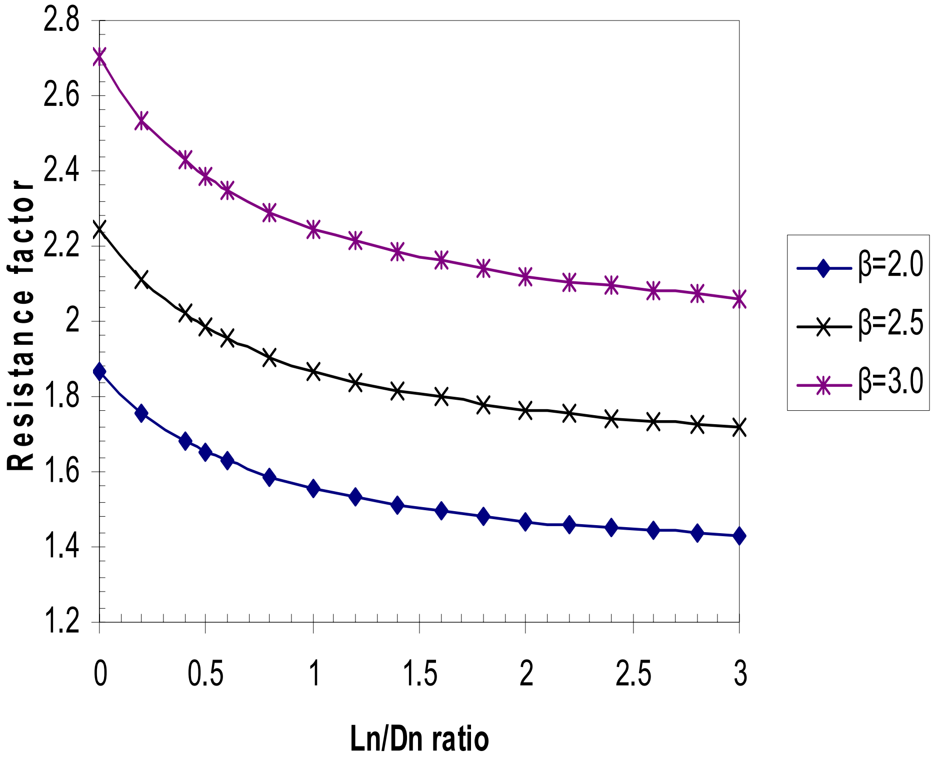
- In all the cases, for a given Ln/Dn ratio, resistance factor increases with the value of the target beta.
- As was the case for the AFOSM approach, there is a general decrease in values of resistance factors with the increase of the L_n/D_n ratio. Again for L_n/D_n ratio of 0 – 1 the rate of decease is rapid while for L_n/D_n ratio of greater than one the rate of decrease is gentle.


Resistance factor
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
0
0.5
1
1.5
2
2.5
3
L_n/D_n
β=2.0
β=2.5
β=3.0

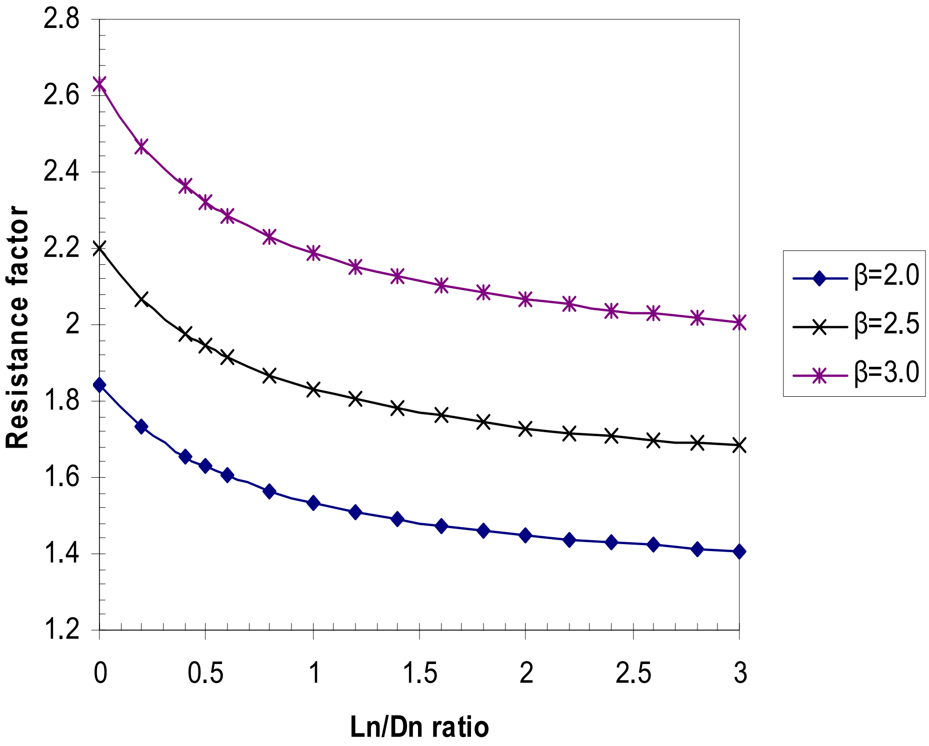
(a)


Resistance factor
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
0
0.5
1
1.5
2
2.5
3
L_n/D_n
β=2.0
β=2.5
β=3.0

(b)



(c)



(d)

Figure 8.6: Resistance factors for MVFOSM method, (a) DNC, (b) BNC, (c) DC, (d) BC

For a given pile class, a single representative value of the resistance factor was taken as the highest value within the practical range of L_n/D_n ratio (i.e 0.5 to 2) as was the case for the AFOSM method. The resultant resistance partial factors are presented in table 8.9. It is evident from table 8.9 that partial factors for piles classes in the same material type are quite close, thereby reiterating the need to separate the partial factors on the basis of materials types only. In comparison with the values for the same pile class and target beta presented in tables 8.5 to 8.7, the partial factors in table 8.9 are generally higher. For the reference class of structures with a target beta of 2.5, these partial factors are high by 22 % for DNC, 24% for BNC, 22 % for DC and 24 % for BC. On average the MVFOSM approach gives resistance factors that are 23% higher.

Table 8-9: Resistance factors for the MVFOSM approach

Pile class	β=2.0	β=2.5	β=3.0
DNC	1.88	2.28	2.75
BNC	1.91	2.28	2.73
DC	1.65	1.98	2.38
BC	1.63	1.94	2.32

8.3.3 Approximate to first-order second moment approach

The approximation allows for separate determination of resistance and load factors. This is an advantage given that for a particular geotechnical application, resistance and load statistics are not readily available and it would be a lengthy process to collect the necessary data. In this study, only resistance statistics were collected and therefore calibrating resistance and load factors independently seems to be the most logical approach. The expression for determining resistance factors using this approach was derived in Chapter 3 and is given by:

$$\gamma_R = \lambda_R \exp(-\alpha\beta_T\, COV_R) \qquad [8.11]$$

The use of equation 8.11 requires the selection of a fitting factor (α). Although there is consensus about the range of α (i.e 0.7 to 1), no specific value is universally used. For example Becker, (1996) and Scott et.al (2003) used α = 0.75 while FHWA H1-98-032 (2001) recommends a value of α = 0.87. FHWA H1-98-032 (2001) recommended value was derived on the basis of extensive trial and error exercise involving ranges of values for

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loading and resistance encountered in foundation design. Because of its strong basis, the value of α = 0.87 was adopted for this study.

The resistance factors obtained from Eq. 8.11 and α = 0.87 for the various pile classes are presented in table 8.10. The values follow the trend that for pile classes in the same soil type, the partial factors are very close. For all the pile classes the partial factors are 24% less than values for the full MVFOSM approach (Table 8.9). When compared to partial factors from the advanced first-order second moment approach, the values in table 8.12 are generally lower. Using the reference case with target beta of 2.5 as an example, the values are lower by 7% in all cases. On the basis of closeness of the values to that obtained by rigorous reliability calibration, it can be asserted that the approach gives reasonable results. However, from a practical point of view, the results of this approach are unconservative and therefore will lead to unsafe designs. From this perspective, the results of the MVFOSM method are better as they err on the conservative side.

Table 8-10: Resistance factors for the approximate MVFOSM method

Case	Resistance factors		
	β =2.0	β =2.5	β =3.0
DNC	1.54	1.73	1.96
BNC	1.55	1.72	1.91
DC	1.34	1.50	1.68
BC	1.32	1.47	1.63

8.2.4 Design value approach

In accordance with the design value format discussed in chapter 3, the partial resistance factor is given by:

$$\gamma_R = \frac{R_k}{R_d} \qquad [8.12]$$

Where: R_k is the characteristic resistance and R_d is the design value of the resistance given by:

$$R_d = \mu_R (1 - \alpha_R \beta V) \qquad [8.13]$$

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in which α_R = sensitivity factor taken as -0.8 (EN 1990) and V = coefficient of variation.

Since geotechnical performance is governed by the average resistance, the characteristic resistance (R_k), is the lower bond of the 95% confidence interval of the mean value. Form Chapter 2, the characteristic value as a mean value at 95% confidence level is given by:

$$X_k = \overline{X}\left(1 - t_{n-1}^{0.95} V \sqrt{\frac{1}{n}}\right) \qquad [8.14]$$

Where: Where; X_k is the characteristic value, $\overline{X}$ is the arithmetic mean of the test results, V is the coefficient of variation of the desired property, n is the number of test results, and t is the value of the student distribution corresponding to a confidence level of 95% and a degree of freedom of n-1.

From Eq. 8.13 and 8.14;

$$\gamma_R = \frac{\mu_R\left(1 - t_{n-1}^{0.95} V \sqrt{\frac{1}{n}}\right)}{\mu_R(1 - \alpha_R \beta V)}$$

[8.15]

Given that R is a lognormal variable, Eq. 8.32 can be expressed as:

$$\gamma_R = \frac{\mu_R \exp\left(t_{n-1}^{0.95} V \sqrt{\frac{1}{n}} - 0.5V^2\right)}{\mu_R \exp(\alpha_R \beta V - 0.5V^2)} \qquad [8.16]$$

Most of the terms in Eq. 8.16 counsels out and the expression reduces to:

$$\gamma_R = \frac{\exp\left(t_{n-1}^{0.95} V \sqrt{\frac{1}{n}}\right)}{\exp(\alpha_R \beta V)} \qquad [8.17]$$

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The resistance factors obtained from Eq. 8.17 are presented in table 8.11. These values are comparable with those for the AFOSM approach using an Excel spreadsheet (Table 8.9). For a beta value of 2.5, the resistance factors from the two approaches differ by 2% (DNC), 6% (BNC), 10% (DC) and 9% (BC). Theoretically the two approaches should give similar results as they are both based on evaluating the performance function at the design point. The difference in results is attributed to the approximation of the sensitivity factor to -0.8 which might be different from the actual values.

Table 8-11: Resistance factors for the design point approach

Case	Resistance factors		
	β =2.0	β =2.5	β =3.0
DNC	1.71	1.91	2.14
BNC	1.58	1.74	1.91
DC	1.60	1.78	1.97
BC	1.55	1.71	1.88

8.2.5 Calibration by fitting with WSD approach

The selection of final resistance factors is influenced by the resistance factors obtained from calibration by fitting to working stress design. This is of paramount importance if the pile load tests data are limited or not of high enough quality to produce reliable statistics and thereby casting doubt on the reliability of the resistance factors determined based on reliability theory. Since questions surrounding the quantity and quality of the available pile load test database are always there, it is necessary to at least compare resistance factors derived on the basis of reliability theory to resistance factors obtained by fitting to the current practice. This approach considers that if the factors of safety used in the past or current practice have resulted in consistently successful designs, one will at least maintain that degree of success at the same cost as required to meet previous practice.

Although in this study the quantity and quality of the data are considered to be sufficient, calibration by fitting is also considered. However, in the selection of the final resistance factors, more weight will be given to results obtained from reliability calibration.

The general equation for fitting to WSD was derived in Chapter 3 and is given by:

$$\phi = \frac{\gamma_D + \gamma_L (L_n / D_n)}{FS(1 + L_n / D_n)} \qquad [8.18]$$

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Eq. 8.17 was used to determine the resistance factors that need to be used in limit state design equation to obtain a factor of safety equal to that of the working stress design approach. Accordingly Eq. 8.17 was set on a spread sheet to calculate the resistance factors for a range of Ln/Dn ratio and a given factor of safety. The same load factors as those used for the reliability calibration (i.e. $\gamma_D = 1.0$ and $\gamma_L = 1.3$) were adopted while the safety factors ranged from 2 to 3. The calibration results are presented in figure 8.6.

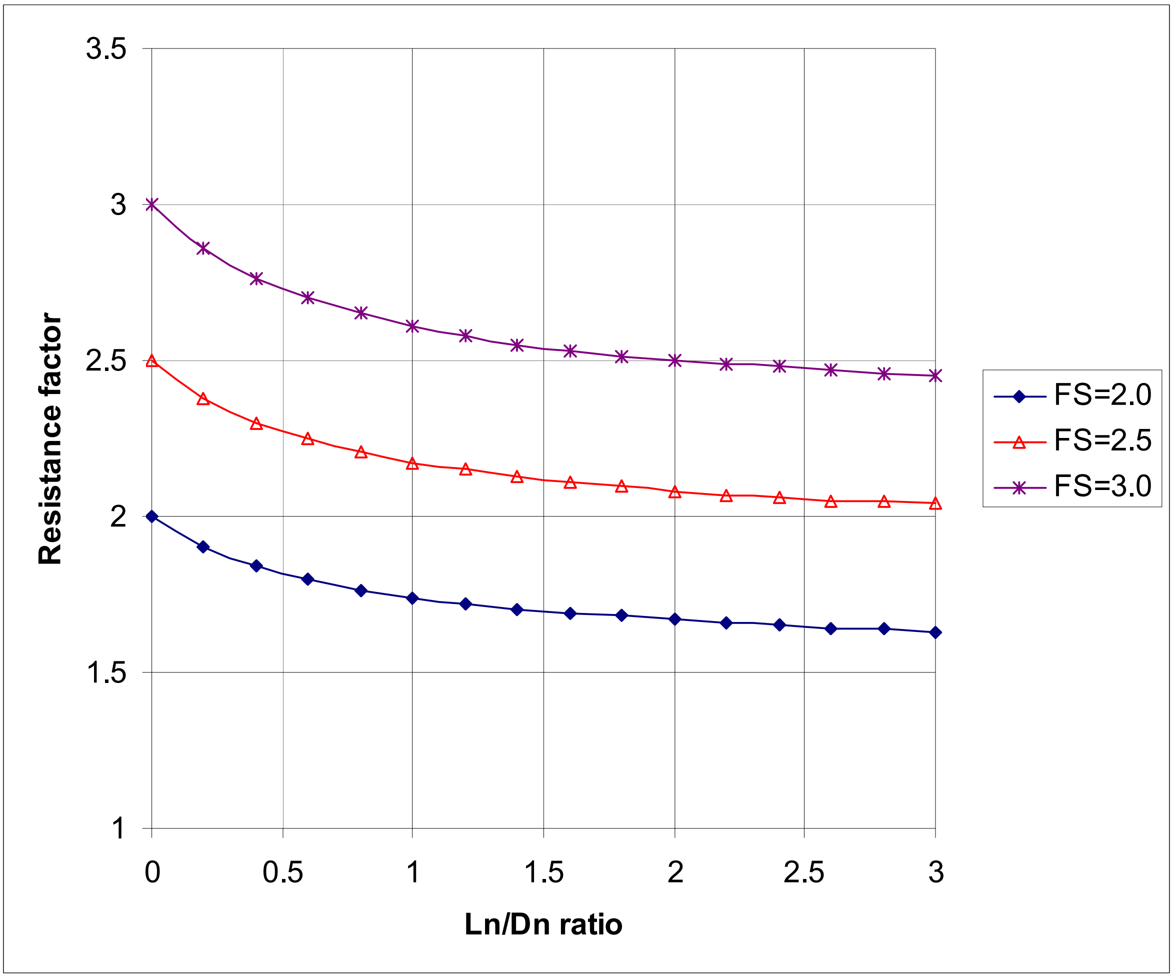

Figure 8.7: Resistance factors for fitting to WSD approach

Consistent with the working stress design approach, no distinction is made between resistance factors for different pile classes. Therefore the partial factors shown are applicable to all the pile classes. It is evident from figure 8.7 that the variation of the resistance factors with L_n/D_n ratio follows the general trend depicted by resistance factors derived on the basis of reliability calibration. In accordance with the procedure adopted in the preceding calibration methods, the representative partial factor value for a given pile

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class was taken as the highest value within the practical range of L_n/D_n ratio. The representative values are 1.82 for FS=2.0, 2.27 for FS= 2.5, and 2.73 for FS=3.0. These values mean that in a limit state design format, a resistance factor of 1.82 ie required to achieve a factor of safety of 2.0 and so forth.

A comparison of these resistance factors with those obtained by reliability calibration lead to the following observations:

- The resistance factor corresponding to FS=2.0 fall within the resistance factors for beta of 2.0 (table 8.8),
- The resistance factor for FS=2.5 is comparable with the values for β_T =2.5 (table 8.6),
- The value for FS=3.0 fits the results for β_T =3.0 (table 8.7).

The results seem to suggest that the use of factors of safety of 2.0, 2.5 and 3.0 lead to reliability indexes of 2.0, 2.5 and 3.0 respectively. As alluded to earlier, calibrating limit state design to existing practice leads to design much the same as that obtained using the working stress design. Therefore the approach does not lead to improved design.

8.2.6 Summary of resistance factors from various calibration approaches

Table 8.12 presents a compilation of the resistance factors obtained from the various approaches.

Table 8-12: Compilation of resistance factors from various approaches

β_T	Pile class	AFOSM	Design point	FOSM	FOSM approx.	Cal. to WSD
2.0	DNC	1.62	1.71	1.88	1.54	1.82
	BNC	1.61	1.58	1.91	1.55	
	DC	1.41	1.60	1.65	1.34	
	BC	1.38	1.71	1.63	1.32	
2.5	DNC	1.87	1.91	2.28	1.73	2.27
	BNC	1.84	1.74	2.28	1.72	
	DC	1.62	1.78	1.98	1.5	
	BC	1.57	1.71	1.94	1.47	
3.0	DNC	2.17	2.14	2.75	1.96	2.73
	BNC	2.09	1.91	2.73	1.91	
	DC	1.85	1.97	2.38	1.68	
	BC	1.78	1.88	2.32	1.63	

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Comparison of the resistance factors from various calculation schemes (table 8.12) indicates that:

- The resistance factors based on the approximate reliability methods are comparable to those derived by the rigorous reliability approach.
- Regarding the results for calibration to working stress design, it appears that the results for FS =2.0 match the resistance factors for β_T =2.0 while those for FS of 2.5 and 3.0 falls within the factors for β_T =2.5 and β_T =3.0 respectively. Within these subdivisions, the resistance factors from calibration to working stress design are more close to resistance factors from the MVFOSM method.

8.4 Comparison with partial factors given in Eurocode 7

Since Eurocodes are the reference codes in South Africa, it is important to compare the partial factors derived in this study with those given in Eurocode 7. The design of pile foundation is covered in Section 7 of Eurocode 7. The principle pile design methods in Eurocode 7 are (i) full scale pile load tests approach and (ii) semi-empirical analysis using standard field tests results directly.

In keeping with the rules of the Eurocodes, the design pile capacity is obtained by dividing the characteristic resistance by a partial factor. The characteristic capacity is obtained as follows:

For pile load tests characteristic value of the compressive ground resistance (R) is taken as the lesser of:

$$R_k = \frac{(R_m)mean}{\xi_1} \text{ and } R_k = \frac{(R_m)\min}{\xi_2}$$

Where:

$(R_m)mean$ = the measured resistance, $(R_m)\min$ = the minimum measured resistance, ξ_1, ξ_2 = correlation factors given in table 8.13.

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Table 8-13: Correlation factors based on static load tests (After EN 1997-1)

ξ for n =	1	2	3	4	≥5
ξ_1	1.40	1.30	1.20	1.10	1.00
ξ_2	1.40	1.20	1.05	1.00	1.00

On the basis of in-situ ground tests the characteristic resistance is given by the lesser value of:

$$R_k = \frac{(R_c)mean}{\xi_3} \text{ and } R_k = \frac{(R_c)\min}{\xi_4}$$

Where:

$(R_c)mean$ = the mean calculated resistance, $(R_c)\min$ = the minimum calculated resistance, ξ_3, ξ_4 = correlation factors given in table 8.14.

Table 8-14: Correlation factors based on semi-empirical methods (After EN 1997-1)

ξ for n =	1	2	3	4	5	7	10
ξ_3	1.40	1.35	1.33	1.31	1.29	1.27	1.25
ξ_4	1.40	1.27	1.23	1.20	1.15	1.12	1.08

For design approach 1 which has been adopted in South Africa, the partial factors with which to divide the determined characteristic resistance are given in table 8.15.

Table 8-15: Partial factors on total compressive resistance

Pile type	γ_R
Driven piles	1.3
Bored piles	1.5
CFA piles	1.4

It is evident from the foregoing discussion that two sets of partial factors are applied to calculated or measured resistance. One is from table 8.13 or table 8.14 depending on the method of pile capacity determination and the other is obtained from table 8.15. Therefore for a given pile type the total partial factor applied is given by:

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$$\gamma_R = \xi \gamma_t \qquad [8.19]$$

Where; γ_R = total partial factor applied, ξ = appropriate correlation factor (ξ_1 *to* ξ_4), γ_t = appropriate partial factor on the total resistance from table 8.16.

Using the above approach, the total applied partial factor for the full scale pile tests and semi-empirical design methods are presented in tables 8.16 and 8.17 respectively.

Table 8-16: Total resistance factors for the full scale pile test design method

Pile Type and ξ	Total partial resistance factor for test piles of n =				
	1	2	3	4	≥5
Driven					
ξ_1	1.82	1.69	1.56	1.43	1.30
ξ_2	1.82	1.56	1.37	1.30	1.30
Bored					
ξ_1	2.10	1.95	1.80	1.65	1.50
ξ_2	2.10	1.80	1.58	1.50	1.50
CFA					
ξ_1	1.96	1.82	1.68	1.54	1.40
ξ_2	1.96	1.68	1.47	1.40	1.40

Table 8-17: Total resistance factors for the semi-empirical design method

Pile type and ξ	Total resistance factor (γ_R) for profiles of tests of n =						
	1	2	3	4	5	7	10
Driven							
ξ_3	1.82	1.76	1.73	1.70	1.68	1.65	1.63
ξ_4	1.82	1.65	1.60	1.56	1.50	1.46	1.40
Bored							
ξ_3	2.10	2.03	2.00	1.97	1.94	1.91	1.88
ξ_4	2.10	1.91	1.85	1.80	1.73	1.68	1.62
CFA							
ξ_3	1.96	1.89	1.86	1.83	1.81	1.78	1.75
ξ_4	1.96	1.78	1.72	1.68	1.61	1.57	1.51

The partial factors presented in tables 8.16 and 8.17 are for the reference class of structures (β_T = 3.8). Since the partial factors were determined by judgement and fitting to the existing practice the target reliability index has no influence on values of the partial factors. Direct

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comparison of the partial factors in tables 8.16 and 8.17 with the values obtained in this study is not possible as these values are based on pile installation method only irrespective of the soil type. However the numerical values of the partial factors presented in table 8.16 and 8.17 are comparable to values of partial factors derived in this study for the reference class of structure with $\beta_T = 2.5$ (table 8.6).

Further more, Eurocode 7 state that if the calculated ultimate capacity is obtained by an analytical approach (i.e. static analysis using engineering properties of the soil as determined from laboratory or in-situ field testing), then the partial factors presented in table 8.13 may need to be corrected by a model factor larger than 1 (clause 7.6.2.3(11)). However no tentative values of the model factors have been provided. Orr (2005) recommended a model factor of 1.4 when designing a pile from ground test results. Assuming that 1.4 is the appropriate model factor for European pile design practice and soil conditions, then the partial factors in table 8.15 should be multiplied by 1.4. The resulting partial factors are presented in table 8.18.

Table 8-18: Resistance factors after inclusion of a model factor of 1.4.

Pile type	γ_R
Driven piles	1.82
Bored piles	2.1
CFA piles	1.96

Although direct comparison is not possible for the reason given earlier, in general the values of the partial factors presented in table 8.18 are again comparable to the partial factors derived in this study.

8.5 Comparison with other published resistance factor values

The introduction of the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications in North America necessitated the consideration of reliability methods to the geotechnical aspect of the specifications (FHWA-NHI-05-052). Major reliability calibration studies associated with this project are presented in NCHRP Report 343 by Backer et al. (1991) NCHRP Report 507 by Paikowsky et al. (2004). Another notable resistance factor calibration study is that contacted by Kim et al. (2002). All these major resistance factor calibration studies relate to pile foundation for bridges and therefore

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the derived partial factors can not be compared with that for building structures due to different load factors. In general the partial factors from these studies were higher than those derived in this study.

With regard to building structures the calibration work carried out by Becker (1996b) for Development of Limit State Design for Foundations in the National Building Code of Canada is notable. Resistance factors for the ultimate limit state of shallow and deep foundations were derived from calibration with working stress design and from reliability based calibration. For pile foundations, the proposed partial factors ranged from 3.3 to 1.67 depending on the method of pile design and site investigation. For the semi-empirical and full scale pile load test design methods, the proposed factors were 2.5 and 1.67 respectively. These values were for achieving a reliability index of 3.4 and 3.2 respectively. Although these partial factors are for higher reliability indexes, they are not significantly different from the partial factors developed in the current study for a target beta of 3.0 (Table 8.7).

On the basis of comparison of partial factors derived in this study with those given in Eurocode 7 Annex A and those recommended by Becker (1996b), it can be concluded that the reliability calibration results are within the range of values used in other countries.

8.6 Recommended Resistance factors

In this study the selection of the final resistance factors is made simple by the fact that the results of the principal calibration methods (i.e. AFOSM method) are comparable to results obtained from other reliability methods (MVFOSM, approximate MVFOSM and the design point approaches) and from calibration with working stress design approach. Further more, the resistance factors were within the range of values given Eurocode 7, which is the reference code for the future geotechnical code for South Africa. Under the circumstances, there is no compelling reason to adjust the resistance factors obtained by reliability calibration. Since the AFOSM method is more accurate than the FOSM method, the results from the of the AFOSM calibration are taken as the recommended resistance factors Table 8.12 column 3. If differentiation in terms of materials type is taken into consideration as previously discussed, the recommended resistance factors corresponding to target beta of 2.0 – 3.0 are as given in table 8.19.

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Table 8-19: Recommended resistance factors based on materials type

Material type	β=2.0	β=2.5	β=3.0
Non-cohesive	1.62	1.87	2.17
Cohesive	1.41	1.62	1.85

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Chapter 9

SUMMARY AND CONCLUSIONS

9.1 SUMMARY

The importance of uncertainty quantification is increasingly recognised in geotechnical engineering as reliability-based methods are assuming a prominent role in the calibration of new generation design codes for geotechnical engineering practice. Accordingly this study has quantified model uncertainty in the classic static pile design formula under the Southern African geological environment. Model uncertainty was represented in terms of model factor statistics. The generated model factor statistics were used to calibrate partial resistance factors in a reliability-based design framework. The subsequent paragraphs summarises the key results obtained.

9.1.1 Geotechnical limit state design

Chapter 2 introduced the general limit state design philosophy. It was established that:

- Although the original concept of limit state design does not specify a particular way in which non-violation of the relevant limit states are ensured, verification by partial factors has become the general design approach.
- Limit state design in the partial factor format can be implemented in both a deterministic and probabilistic framework. The fundamental difference is in the derivation of the associated partial factors. With the deterministic limit state design, partial factors are determined by engineering judgement and by fitting to the existing design practice while for the probabilistic limit state partial factors are based on reliability calibration.
- Limit state design in a non-probabilistic framework suffers from the same draw-backs as the working stress design it replaces. Consequently the geotechnical profession could not see the benefits of converting to limit state design, hence the severe criticism of the approach.
- Despite the initial reluctance to the use of limit state in the partial factors format, the geotechnical profession has now embraced this design approach. Three main factors has motivated this change in hearts: (i) the need to achieve compatibility between structural

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and geotechnical design, (ii) the application of statistics, probability and reliability theory to analysis and design which led to a transparent and rational treatment of uncertainties as well as the derivation of partial factors, (iii) limit state design awareness campaign by the Technical Committee (TC 23) of the International Society of Soil Mechanics and Geotechnical Engineering on Limit State Design in Geotechnical Engineering.

- The limitations of the deterministic limit state design serves as a motivation to adopt the more rational and transparent probabilistic limit state design in geotechnical engineering.
- Globalisation and economic co-operation requires harmonisation of technical rules for design of buildings and civil engineering works. The harmonisation of design rules for all structures dictates that geotechnical design being based on the same limit state design as for structural design involving other materials.
- The need for common design rules has led to establishment of a basis of design in the Eurocodes (EN 1990, 2002) and the revised South African Loading Code (SANS 10160-Draft). Basis of design provides a common basis and general principle for the design of building and civil engineering works within the limit state design framework.
- The common rules relate to performance requirements, specification of the limit states, design situations to be checked, reliability requirements, and treatment of basic variables (actions, materials properties, and geometric data).
- Within the Eurocodes framework, the design of elements involving geotechnical actions (e.g. foundations and retaining walls) can be carried out by any of the three alternative Design Approaches. Furthermore, the selection of a particular design approach is a matter for national determination.
- Through comparison of worked design examples, Design Approach 3 produces the most conservative designs, Design Approach 2 the least conservatism designs, and Design Approach 1 generally yields designs between the other two approaches. Accordingly Design Approach 1 was selected for this study.

9.1.2 Reliability background for the geotechnical limit state design

Having established the need for the probabilistic limit state in Chapter 2, Chapter 3 presented the fundamental reliability background required for reliability-based geotechnical design. This entails an overview of reliability theory and reliability calibration principles. Highlights of this chapter include:

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- In a probabilistic setting, the reliability of an engineered system is the probability of its satisfactory performance for a specific period under specific service conditions (i.e. the probability of survival).
- The basic measure of reliability is the probability of failure. In evaluating the probability of failure, the behaviour of the system is described by a set of basic variables (i.e. actions, material properties, geometric data and model uncertainty).
- Theoretically the probability of failure can be determined from the integration of the joint probability density function of n basic variables over the failure region. However, to directly evaluate the n-fold integral using standard methods of integration is a formidable task and closed form solutions do not exist except for very simple cases. Furthermore, the joint probability density function of random variables is practically not possible to obtain, and the PDF of the individual random variables may not always be available in explicit form. Therefore for practical purposes, analytical approximation of the integral is employed to simplify the computations of the probability of failure. The most common methods of approximation are the First Order Second Moment reliability methods (FOSM).
- In the FOSM approximation, the probability of failure is expressed in terms of the reliability index (β). In the computation of the reliability index, all that is required are the statistics of the random variables (mean and coefficient of variation).
- FOSM reliability methods can be further classified into the Mean Value First-Order Second Moment method (MVFOSM) in which the limit state function linearised at the mean values of the random variables and the Advanced First-Order Second Moment method (AFOSM) in which limit state function linearised at the design point.
- There are two contrasting interpretations of the calculated probability of failure (i.e. frequentist and degree of belief). However for civil engineering designs the calculated probability should be interpreted as a degree of belief.
- Given the complexity of the computations for the failure probability or the reliability index, this approach, although rigorous, might not be suitable for routine designs, hence the need for simplified reliability methods. Such design methods are referred to as level 1 reliability design methods.
- With the level 1 design approach, the appropriate degree of reliability is provided through the use of partial factors. Such partial factors are derived using level II reliability methods and are associated with the major sources of uncertainties in the basics variables. The process of assigning partial factors to resistances or loads is generally termed calibration.

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- Calibration of partial factors can be carried out by engineering judgement and experience, fitting to existing design practice, and by reliability theory.
- The main draw-back of the calibration by engineering judgement and experience and fitting to existing design practice is that they result in non-uniform level of conservatism. Accordingly this study was based on reliability calibration.

9.1.3 Data collection, processing and evaluation of interpreted capacities

In general, model uncertainty quantification entails comparing predicted performance with measured performance. The approach requires a substantial amount of local pile load tests results with the associated geotechnical data. Chapter 5 detailed the collection and processing of the pile load test data. Analysis of the database led to the following observations:

- The majority of the piles were made of concrete and only a few steel piles, indicating that the principal pile material in South Africa is concrete.
- The pile lengths range from 3 to 27 m for DNC, 6 to 16.5 for BNC, 3.5 to 29.3m for DC and 4.5 to 24 m for BC. Generally the extremely long piles (e.g. 27 and 29 m) were steel piles.
- The concrete pile diameters vary from 330 to 610 mm for DNC, 360 to 520 for BNC, 250 to 750 mm for DC and 300 to 910 mm for BC. Except for one case, all the steel piles comprises of H-piles of 305 x 305 mm in size. The exception is a steel tube pile of 560 mm diameter.
- The pile types include Franki (expanded base) piles, Auger piles, Continuous Flight Auger (CFA) piles and steel piles. For driven piles the Franki pile is the most popular while for bored piles the Auger and CFA are more prevalent.

For each test pile (case 1 – 174), there was some accompanying soil data. The following observations were drawn from study of the geotechnical data:

- The materials fall into one of the following four natural categories: transported soil, residual soil, pedogenic material, and rock.
- In cohesive materials the piles are generally founded in the residual materials horizon.

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- The soil test results comprises mainly of Standard Penetration Tests (SPT) measurements and a few Cone Penetration Test (CPT) results, implying that the SPT is the most popular field soil test in Southern Africa.
- In general, for a given case the SPT N-value for the base materials is higher than that for the shaft materials. This trend is marked in cohesive materials, indicating that in residual materials, the pile passes through soil strata of increasing consistency and is founded on a rock consistency stratum.

The load tests data were then used to evaluate the ultimate pile capacities. This was accomplished by plotting the load versus the head deflection to produce load-deflection curves. The majority of the test piles in the database are working piles tested to a maximum load of one and half times the design load. This limits the movement to which the pile head is subjected and requires an extension to the load-settlement curve to determine the ultimate capacity. The Chin/Davisson procedure was adopted for the current study to determine ultimate capacity of proof tests results.

9.1.4 Evaluation of engineering soil properties and design parameters

The theoretical computations of the bearing capacity of geotechnical materials require the engineering properties of the materials as input in the calculation model. Accordingly in Chapter 5, the specific site measurements presented in Chapter 4 were transformed to the desired engineering soil properties. The penetration test results included Standard Penetration Test (SPT) blow-counts, Dynamic Probe Super Heavy (DPSH) results and Cone Penetration Test (CPT) results. For derivation of soil properties, the DPSH and CPT measurements were converted to equivalent SPT N-values. The SPT N-values were then used to estimate the desired geotechnical properties.

Many correlations relating SPT blow count and soil properties have been developed. Some of the correlations are based on corrected N-values ($N_{1\ (60)}$) while others are based on uncorrected values (N). It was observed that currently there is no consensus on the use of corrected versus uncorrected SPT blow count. In this study correlations based on uncorrected N-values were used. The reasons for this selection are as follows:

- Uncorrected values are used in practical designs worldwide;
- Most reliable correlations are based on uncorrected blow count;

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- The required correction factors are too many;
- The models for deriving the correction factors are not perfect and therefore introduce additional uncertainties to the interpretation of SPT results. For example, several formulae and charts have been published for overburden correction with completely different results;
- State of the art equipment are in use these days and therefore most equipment-related effects are eliminated;
- SPT is carried out by specialist drilling companies with sufficient experience and therefore they are conversant with the test. Hence operator/procedural effects are minimal.
- The reference overburden stress of 100 kPa is just a hypothetical value and therefore does not reflect the operational stress around a pile. The increase in SPT N value with depth is a reflection of the increase in strength and stiffness caused by in-situ overburden stress and therefore correction for overburden stress may not be necessary.
- The correction for energy loss makes sense if the exact loss has been measured on site for each case considered. But usually a generic correction factor is applied. Surely in some sites, advanced SPT equipment might have been used with no energy loss at all while in some sites older equipment might have been used with higher energy losses. Since the exact SPT equipment used to measure the N-values as well as their site measured energy losses are not known, it is not worth while to apply the correction factor.
- Results of preliminary investigations on the effects of correcting the N-values on various pile design parameters showed that:

    - In comparing the design parameters derived on the basis of corrected and uncorrected N-values, no significant improvement in terms of reduced variability was gained from correcting the N-values.
    - The data set for uncorrected N-values provides the best fit to the measured capacities (i.e. a mean value of closer to 1 and a smaller variability).

On the basis of the foregoing, it was concluded that for reliability-based design of pile foundations, correction of N-values for overburden and hammer efficiency do not add any value.

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The main parameters derived from the SPT measurements were the internal angle of friction (φ) for non-cohesive materials and undrained cohesion for cohesive materials. Analysis of the φ values led to the following conclusion:

- In general, for a given case φ-base values are higher than φ-shaft. This is in accordance with expectations as piles are normally founded on denser materials.
- On average, the φ values for materials in which bored piles were constructed are relatively higher than for materials in which driven piles were installed. This is attributed to the fact that under normal circumstances, bored piles are preferred in very dense materials (higher φ values) while driven piles are preferable when the granular material is not very dense.
- The magnitudes of the φ values are within the expected range of φ values for non-cohesive materials. However, some φ-shaft values are on the lower side, indicating that the pile shaft passes through some silty soils.

Analysis of the undrained shear strength values showed that:

- Values for C_u-base are much higher than those for C_u-shaft. This is consistent with the variation of the SPT measurements with depth and is attributed to the fact that the degree of weathering in a typical residual soil tends to decrease with depth. The weathered materials gradually merge into the unweathered rock. Accordingly the consistence varies from soft for the transported soil overlying the residual soil to very stiff for weathered rock.
- The C_u values for materials in which bored piles were constructed are generally higher than those for driven piles. These higher C_u values which are associated with higher consistencies have necessitated the construction of bored piles as it is difficult to drive piles in such materials. In fact in South Africa, the most common type of pile in residual soils is the bored cast in situ pile.
- The magnitudes of the C_u values are in accordance with what is expected of a typical profile in a residual soil. The lower values (C_u <100 kPa) denote the predominance of transported materials while middle values (i.e. C_u values of up to 500 kPa) indicate the prevalence of the residual soil. There are also a number of cases depicting rock consistency (i.e. C_u of 1000 kPa or greater). The higher values are mostly associated with C_u-base, thus confirming that piles in residual soils are usually founded on a rock consistency stratum.

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In the computation of pile capacity using the static formula, design parameters other than the geotechnical properties are required. These include N_q, K_s, N_c, δ, and α. These parameters are estimated from empirical correlations with soil properties. It was observed that the process of determining these parameters is not standardised. Consequently different engineers follow different procedures thereby producing different design parameters for even the same site. To select the appropriate design parameters for the South African geological setup, two approaches were followed. The first approach entails using values commonly assumed in practice while the other approach entails conducting a parametric study of the possible values for a given parameter. The second approach is tantamount to assessment of design parameters by back analysis of load test results. The results were such that for a given pile class, r^2 values for design parameters selected on the basis of best fit principle are slightly higher than those for commonly assumed parameters in practice. Therefore, the design parameters selected on the basis of best fit principle were considered as the most appropriate for the South African conditions.

The selected geotechnical design parameters were used as input into the static formula to compute the predicted capacity for each case. Examination of the calculated predicted capacities led to the following conclusions:

- Q_p ranges from 1100 to 5150 kN for driven piles in non-cohesive materials, 470 to 6750 kN for bored piles in non-cohesive materials, 560 to 5910 kN for driven piles in cohesive materials and 460 to 10170 kN for bored piles in cohesive materials.
- It is apparent from the above ranges of Q_p values that for a given soil type, the upper bound values for bored piles are higher than their respective values for driven piles. This is in line with the earlier observation that bored piles are normally installed in very stiff materials, hence the higher resistances.

9.1.5 Model factor statistics

For reliability calibration, the statistics (mean, standard deviation and coefficient of variation) as well as the type of distribution that best fit the data need to be determined for each random variable considered in the limit state function. Using the results obtained from Chapters 4 and 5, Chapter 6 detailed the computations of the model factors as well as the statistical analysis of the model factor realisations. It was noted that in principle, the bias factor accounts for all the sources of uncertainties (i.e. model error, systematic error,

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inherent spatial variability, statistical error and load tests related errors). However due to the high level of expertise and experience of companies in charge of SPT tests and the pile load tests, measurement errors are minimized. Also inherent spatial variability is minimized due to averaging effects along the pile length. Therefore the calculated model factor mainly represents model uncertainty.

The statistical analysis comprised of graphical representation by histograms, outliers detection and correction of erroneous values, using the corrected data to compute the sample moments (mean, standard deviations, skewness and kurtosis), determining the appropriate distribution for the model factor, and investigation of correlation with underlying pile design parameters. The key results of the statistical analysis were as follows:

- Based on the mean bias factors it was observed that the static formula with geotechnical parameters recommended in this study, yields slightly conservative theoretical capacities for DNC, DC, and BC. Conversely for BNC the approach is marginally unconservative.
- Although the standard deviations of the four pile classes were not significantly different, there was a distinct trend that driven piles depicted higher variability compared to bored piles irrespective of materials type.
- For a given pile installation method (driven or bored) the variability in non-cohesive materials is higher than that in cohesive materials.
- The lognormal probability distribution was found to be the appropriate distribution for the model factor.

Concerning correlation with pile design parameters, the significance of the correlation was interpreted in terms of practical and statistical significance. On the basis of statistical significance (p-value):

- None of the pile design parameters was significantly correlated with the model factor.

With regard to predicted capacity, it was found that the model factor is correlated to Q_p for driven piles but not for bored piles, suggesting that the static formula is more accurate in bored piles than driven piles.

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When the interpretation of the numerical values of correlation coefficients was based on practical considerations (i.e. $r \geq 0.2$ considered significant), then the correlation was found to be significant as follows: model factor was found to be correlated with:

- In cohesive materials, the correlation was significant for shaft diameter, base diameter, and predicted capacity in driven piles.
- In non-cohesive materials, the correlation was significant for base diameter, φ-shaft, φ-base, and predicted capacity in driven piles.

It was noted that correlation coefficients of ≥ 0.2 occurred only in driven piles and not in bored piles. It was observed that the numerical values of the correlation coefficients for the ranged from 0.2 to 0.4 thereby implying a low degree of correlation. With more weight given to the practical significance of the value of the correlation coefficient rather than its statistical significance, it was concluded that the conventional model factor exhibits some statistical dependencies with some underlying factors.

9.1.6 Treatment of correlation

To apply the model factor as an independent variable in reliability analysis, these statistical dependencies need to be removed. In Chapter 7, the statistical dependencies between the model factor and the predicted capacity were either removed or taken into account. Removal of statistical dependencies was considered under the generalised model factor approach while accounting for the correlation was considered under conditioned model factor approach. The fact the generalised M was not dimensionless necessitated the normalisation of the interpreted and predicted capacities. Accordingly three normalization schemes were experimented with. The normalisation schemes were evaluated on the basis of the resulting fit between Q_i and Q_p. In this regard, the coefficient of determination (r^2) was used as the measure of fit. The results were as follows:

- For a given pile class, the r^2 values and values of the standard error of estimate were quite close for the different normalisation schemes. It was therefore concluded that, in general the three normalisation schemes yield comparable regression results.
- Basing the selection of the best fit on r^2 and the regression constant denoting the slope of the best fit line ("b") it was found that scheme 1 produced the best overall results. Accordingly the generalised M statistics for scheme 1 were selected for further analysis.

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- A comparison of the generalised M statistics to the standard M statistics showed reasonable agreement. The standard deviations and coefficients of variation were comparable. Furthermore, the two sets of model factors showed similar trends.
- Comparison of the conditioned and standard model factor statistics showed very little difference.
- On the basis of the model factor statistic it was concluded that there was no noticeable reduction in variability gained from removing and accounting for correlation. This was attributed to the weak correlation.

The objective of accounting for the correlation was to minimize its effects on the calculated reliability index. Accordingly the extent of improvement to the calculated beta values imparted by the removal or incorporation of the correlation was investigated. Since for the conditioned M approach, the model factors statistics and the performance function were similar to that of the standard M, it was obvious that no significant improvement to the calculated reliability index could be achieved. Therefore the beta values were determined on the basis of standard and generalised M. The results were as follows:

For the standard M:

- Beta values were sensitive to the ratio of variable to permanent load (L_n/D_n) and not the variation in the nominal resistance, implying that the calibration points are only defined by the L_n/D_n ratio.
- The beta values varied from one pile class to the other indicating that the current global factor of safety approach does not produce a consistent level of reliability across different design situations. Further more even within the same pile class beta values varied with individual cases represented by the L_n/D_n ratios. This suggested that even within the same design situation the global factor approach fails to attain a consistent level of reliability.
- Piles in cohesive materials depicted higher reliability compared to piles in non-cohesive materials. The scenario was attributed to higher inherent conservatism of the static formula in non-cohesive materials compared to non-cohesive materials.
- For a given material type, the β values for bored piles were higher than that for driven piles. This was attributed to the observation noted in Chapter 6 that for a given material type the variability of the model factor in driven piles were higher than that in bored piles

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For generalised M:

- Beta values based on the generalised M statistics varied with L_n/D_n ratio as well as nominal resistance values (R_n). This necessitated the determination of beta values for representative R_n values.
- The variation in beta values with R_n value depended on the magnitude of the exponent b. When b was greater than 1, beta increased with the increase in R_n, which was the case for DNC and BNC. The b values for these two cases were 1.031 for DNC and 1.030 for BNC. Conversely when b was less than 1 the beta values decreased with the increase in R_n, which was the case for DC and BC (b=0.977 for DC and 0.961 for BC).
- The variation of β-values with L_n/D_n ratio followeed a similar trend to the one shown by values based on standard M statistics.

For both approaches, a critical value (i.e. the smallest value within a typical range of L_n/D_n ratio) was determined for each pile class. The following conclusions were drawn for examination of the critical beta values:

- For a safety factor of 2.5 both approaches produced values of close to the specified target beta of 3. Therefore all the three approaches yield beta values that met the target reliability index specified by SANS 10160-Draft.
- A comparison of the critical beta values showed that values for the standard M were significantly higher than that for generalised M in cohesive materials. However, in non-cohesive materials, the difference between the results of the two approaches was negligible. Therefore the generalised M approach did not produce improved reliability indexes.

It was concluded that due to the weak correlation between M and Q_p, removal or accounting for the trend between the two parameters does not lead to improved reliability. It was therefore concluded that the calibration of the partial factors be based on the standard M.

9.1.7 Calibration of resistance factors

Using the model factor statistics developed in Chapter 6 and the calibration methods discussed in Chapter 3, Chapter 8 developed the required resistance factors for the ultimate limit state. Five calibration methods were employed (i) Advanced first-order second moment approach (AFOSM), (ii) Mean value first-order second moment approach

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(MVFOSM), (iii) Approximate MVFOSM, (iv) Design point approach and (v) Fitting to working stress design. The following conclusions were drawn from the calibration study:

- Based on considerations of reliability indexes implied by the current design approach, target beta values set in the current and draft South African loading codes, and redundancy inherent in pile groups, target beta values of 2.0 for RC 1, 2.5 for RC 2 and 3.0 for RC 3 were recommended for pile foundations.
- In general the resistance factors decreased with the increase of the L_n/D_n ratio, indicating variation in reliability indexes across the calibration points or design situations within a given class of pile foundations. This further implied that the application of a single resistance factor to all the design situations will inevitably lead to some deviation from the target reliability index for some of the calibration points.
- To achieve consistent reliability within a range of calibration points for a given pile class, an optimum partial factor which best approximate the uniform target reliability is needed. In principle this can be obtained by minimizing the deviation from the target beta using an objective or penalty function penalising the deviation from the target reliability index.
- Investigation of the optimisation schemes showed that Lind's function gives results that are close to the least square function while still penalising under- designs more than over-designs, hence providing conservative results. Since it is better to err on the conservative side, Lind's objective function was used in the selection of the optimum partial factor for all the pile classes.
- Further analysis revealed that optimal partial resistance factors obtained by minimization of Lind's objective function were very close to partial factors corresponding to the calibration point represented by L_n/D_n of 0.5 (i.e. design situation with the highest partial factor). Therefore instead of performing the optimisation process, the partial factors corresponding to L_n/D_n ratio of 0.5 were taken as the optimal partial resistance factors.
- The resistance factors appeared to be influenced by the materials types. In this regard, values of partial factors for a given calibration method and material type were quite close irrespective of the pile installation method.
- On average the resistance factors from the MVFOSM approach were 23% higher than those from the AFOSM method.
- The resistance factors for the approximation MVFOSM method were 24% less than those for the full MVFOSM method.

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- Resistance factors from the AFOSM and design point methods were quite close. For a beta value of 2.5, the resistance factors from the two approaches differed by 2% (DNC), 6% (BNC), 10% (DC) and 9% (BC).
- A comparison of resistance factors obtained through calibration by fitting with those obtained by reliability calibration showed that:

  - The optimal resistance factor corresponding to FS=2.0 fell within the resistance factors for $\beta_T = 2.0$.
  - The resistance factor for FS=2.5 was comparable with the values for β_T =2.5.
  - The value for FS=3.0 fits the results for β_T =3.0.

- Numerical values of the partial factors derived in this study were within those recommended in EN 1997-1 and the Canadian building code.

9.2 CONCLUSIONS

On the basis on results obtained from various Chapters, the following overall conclusions can be drawn from the study:

- The variability exhibited by the model factors is an indication that there are significant model uncertainties in the classical static pile design method. To explicitly account for the variability and bias reflected in this study, a design approach based reliability considerations seems to be the most reasonable alternative for pile foundation design in South Africa.

- The similarity of the model factor statistics for the static formula with those reported for other static pile analysis methods (e.g. Meyerhof method, CPT methods, Nordlund methods, Vesic method) is an indication that static analysis using engineering properties gives equally good results. Therefore, with the application of a resistance factor to cater for the model uncertainty, the method should lead to safe designs and perhaps lessen the need for the customary practice of verification through proof pile tests.

- The statistically insignificant correlation between the model factor and various pile design parameters is an indication that the variation in the model factor is not explainable by deterministic variations in the database. Therefore, the model factor is a random variable warranting a probabilistic description. In this regard the lognormal distribution was found to be the most appropriate theoretical model for the model factor

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- Although correlation between variables generally affects the calculated reliability index, weak correlation ($r \leq 0.4$) do not significantly affect the results and could be ignored.

- Values of reliability indexes implied by existing practice meet the minimum beta value of 3.0 specified in the current South African loading code. However, due to redundancy in pile groups, lower target beta values are recommended as acceptable for pile foundations.

- Results of the principal calibration methods (i.e. AFOSM method) are comparable to results obtained from other reliability methods (MVFOSM, approximate MVFOSM and the design point approaches). This implies that the approximate methods yield reasonable results, further suggesting that reliability calibration can as well be based on the simple approximation procedures.

- The resistance factors were within the range of values given in Eurocode 7, which is the reference code for the future geotechnical code for South Africa. This provides further confidence in the use of the partial factors developed in this study.

- The general trend that for a given material type, the resistance factors were quite close irrespective of the pile installation method suggests differentiation of partial factors in terms of materials types only.

- Results of calibration by fitting give resistance factors that need to be used in the limit state design equation to obtain a factor of safety equal to that of the working stress design. From this premise, calibrating limit state design to existing practice results in the same minimum permissible foundation sizes as those obtained by the working stress design. Therefore the use of limit state on non-reliability based procedures does little to improve design.

9.3 RECOMMENDATIONS FOR FURTHER RESEARCH

- Owing to the nature of the available data, resistance factors were calibrated for the total capacity only. However a derivation of separate resistance factors for total, shaft and base capacity provides more insight. Accordingly a separate calibration is considered valuable.

- The load factors of 1.0 for permanent actions and 1.3 for variable actions recommended in SANS 10160-Draft and EN 1997-1 were adopted in this study. However these load

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factors have not been developed on the basis of reliability calibration as is the case for load factors for structural design. Therefore it will be of interest to calibrate load factors for geotechnical design.

- Both the resistance and the load were taken as lognormal variables in this study. However, studies have shown that variable loads follow an Extreme value type 1 distribution. Since the reliability index is sensitive to the distribution assumed, it will be interesting to investigate the sensitivity of the resulting reliability to the distribution applied to the variable action, considering a lognormal distribution as was done here and an Extreme value type 1 as is generally applied.

- Although the correlation with various pile design parameters was generally weak, it appeared that the correlation in driven piles was higher than in bored piles. This suggests that the static formula applied to driven piles does not adequately account for the various pile design parameters. Therefore further study directed in the refinement of the static formula in driven piles should be a fruitful investigation.

- Refinement of the model factor statistics and the ensuing partial resistance factors can be achieved through an increment of the database developed in this study. Therefore there is need to expand the current database with more effort directed to instrumented piles and piles tested to failure.

- In order to introduce probabilistic limit state design to geotechnical design as a whole in South Africa, the procedures developed in this study need to be extended to other geotechnical applications such as shallow foundations, retaining walls and slope stability.
- For the resistance, the design point is typically located in the lower tail and therefore it is important that the lognormal parameters selected produce the best fit possible in the region of the tail. Accordingly a worthwhile further research is to determine the best fit statistical parameters of the assumed distribution and use such statistical parameters in the calibration exrcise.

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Appendix A

AVAILABLE STATISTICS ON GEOTECHNICAL PARAMETER UNCERTAINTIES

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A.1: COV of inherent variability for index soil properties (Phoon, 1995)

Property[a]	Soil Type	No. Data Groups	No. Tests/Group		Property Value (units[b])		Property COV (%)	
			Range	Mean	Range	Mean	Range	Mean
w_n	Fine-Grained Soils	40	17 - 439	252	13 - 105	28.7	7 - 46	18
w_L	Fine-Grained Soils	38	15 - 299	129	27 - 89	50.7	7 - 39	18
w_P	Fine-Grained Soils	23	32 - 299	201	14 - 27	22.1	6 - 34	16
PI	Fine-Grained Soils	33	15 - 299	120	12 - 44	25.0	9 - 57	29
LI	Clay, Silt	2	32 - 118	75	-	9.4	60 - 88	74
γ	Fine-Grained Soils	6	5 - 3200	564	14 - 20	17.5	3 - 20	9
γ_d	Fine-Grained Soils	8	4 - 315	122	13 - 18	15.7	2 - 13	7
D_r	Sand	5	-	-	30 - 70	50	11 – 36[c]	19[c]
D_r	Sand	5	-	-	30 - 70	50	49 – 74[d]	61[d]

a - w_n = natural water content; w_L = liquid limit; w_P = plastic limit; PI = plasticity index; LI = liquidity index; γ = total unit weight, γ_d = dry unit weight; D_r = relative density

b - units of w_n, w_L, w_P, PI, LI, and D_r = %; units of γ and γ_d = kN/m^3

c - total variability for direct method of determination

d - total variability for indirect determination using SPT values

1 kN/m^3 = 6.37 pcf

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A.2: COV for inherent variability for shear strength parameters
(After Jones et al, 2002)

Property (units)	Soil Type	No. of Data Groups	No. of Tests Per Group		Property Value		Property COV (%)		Note
			Range	Mean	Range	Mean	Range	Mean	
$\bar{\phi}$ (°)	Sand	7	29 – 136	62	35 – 41	37.6	5 – 11	9	1
	Clay, silt	12	5 – 51	16	9 – 33	15.3	10 – 56	21	
	Clay, silt	9	*	*	17 – 41	33.3	4 – 12	9	
	*	20	*	*	*	*	*	12.6	4
$\tan\bar{\phi}$	Clay, silt	4	*	*	0.24 – 0.69	0.509	6 – 46	20	1
$\tan\bar{\phi}$	Clay, silt	3	*	*	*	0.615	6 – 46	23	
$\tan\bar{\phi}$	Sand	13	6 – 111	45	0.65 – 0.92	0.744	5 – 14	9	1
	*	7	*	*	*	*	*	11.3	4
$\bar{\phi}$ (°)	Sand	*	*	*	*	*	2 – 5	*	2
ϕ (°)	Gravel	*	*	*	*	*	7	*	3
	Sand	*	*	*	*	*	12	*	
$s_u^{(a)}$ (kPa)	Fine-grained	38	2 – 538	101	6 – 412	100	6 – 56	33	1
$s_u^{(b)}$ (kPa)	Clay, Silt	13	14 – 82	33	15 – 363	276	11 – 49	22	
$s_u^{(c)}$ (kPa)	Clay	10	12 – 86	47	130 – 713	405	18 – 42	32	
$s_u^{(d)}$ (kPa)	Clay	42	24 – 124	48	8 – 638	112	6 – 80	32	
	*	38	*	*	*	*	*	33.8	3
$s_u^{(e)}$ (kPa)	Clay	*	*	*	*	*	5 – 20	*	2
$s_u^{(f)}$ (kPa)	Clay	*	*	*	*	*	10 – 35	*	
$s_u^{(d)}$ (kPa)	Clayey silt	*	*	*	*	*	10 – 30	*	
$c^{(g)}$	*	*	*	*	*	*	40	*	3
s_u/σ'_{v0}	Clay	*	*	*	*	*	5 – 15	*	2

* Not reported.
(a) Unconfined compression test.
(b) Unconsolidated-undrained triaxial compression test.
(c) Consolidated isotropic undrained triaxial compression test.
(d) Laboratory test not reported.
(e) Triaxial test.
(f) Index s_u.
(g) No specification on how the parameter was defined.
Notes:
(1) Phoon and Kulhawy (1999).
(2) Lacasse and Nadim (1996). No comments made on whether measurement variability was included.
(3) Harr (1987). No comments made on whether measurement variability was included.
(4) Kulhawy (1992). No comments made on whether measurement variability was included.

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A3: COV of inherent variability for consolidation and permeability parameters (After Jones et al, 2002)

Property (units)	Soil Type	No. of Data Groups	No. of Tests Per Group		Property Value		Property COV (%)		Note
			Range	Mean	Range	Mean	Range	Mean	
C_c	Sandy clay	*	*	*	*	*	26	*	1
	Clay	*	*	*	*	*	30	*	
	*	*	*	*	*	*	37	*	2
p_c'	*	*	*	*	*	*	19	*	1
OCR	*	*	*	*	*	*	10 – 35	*	3
k	*	*	*	*	*	*	240[(a)]	*	1
	*	*	*	*	*	*	90[(b)]	*	
c_v	*	*	*	*	*	*	33 – 68	*	4
e, n, e_0	All soil types	*	*	*	*	*	7 – 30	*	5
n	*	*	*	*	*	*	10	*	1

* Not reported.
(a) 80% saturation.
(b) 100% saturation.
Notes:
(1) Harr (1987).
(2) Kulhawy (1992). No comments made on whether measurement variability was included.
(3) Lacasse and Nadim (1996). No comments made on whether measurement variability was included.
(4) Duncan (2000).
(5) Lacasse and Nadim (1996).

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A 4: Measurement error for laboratory tests (After Phoon, 1995)

Property[a]	Soil Type	No. Data Groups	No. Tests/Group Range	No. Tests/Group Mean	Property Value (units[b]) Range	Property Value (units[b]) Mean	Property COV (%) Range	Property COV (%) Mean
s_u(TC)	Clay, Silt	11	-	13	7 - 407	124.6	8 - 38	19
s_u(DS)	Clay, Silt	2	13 - 17	15	108 - 130	118.8	19 - 20	20
s_u(LV)	Clay	15	-	-	4 - 123	28.7	5 - 37	13
$\bar{\phi}$(TC)	Clay, Silt	4	9 - 13	10	2 – 27°	19.1°	7 - 56	24
$\bar{\phi}$(DS)	Clay, Silt	5	9 - 13	11	24 – 40°	33.3°	3 - 29	13
	Sand	2	26	26	30 – 35°	32.7°	13 - 14	14
tan $\bar{\phi}$(TC)	Sand, Silt	6	-	-	-	-	2 - 22	8
tan $\bar{\phi}$(DS)	Clay	2	-	-	-	-	6 - 22	14
w_n	Fine-Grained Soils	3	82 - 88	85	16 - 21	17.7	6 - 12	8
w_L	Fine-Grained Soils	26	41 - 89	64	17 - 113	36.4	3 - 11	7
w_P	Fine-Grained Soils	26	41 - 89	62	12 - 35	20.9	7 - 18	10
PI	Fine-Grained Soils	10	41 - 89	61	4 - 44	22.7	5 - 51	24
γ	Fine-Grained Soils	3	82 - 88	85	16 - 17	17.0	1 - 2	1

a - s_u = undrained shear strength; $\bar{\phi}$ = effective friction angle; TC = triaxial compression test; DS = direct shear test; LV = laboratory vane shear test; w_n = natural water content; w_L = liquid limit; w_P = plastic limit; PI = plasticity index; γ = total unit weight

b - units of s_u = kN/m^2; units of w_n, w_L, w_P, and PI = %; units of γ = kN/m^3

1 kN/m^2 = 0.0104 tsf ; 1 kN/m^3 = 6.37 pcf

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A 5: COV of inherent variability of field measurements

Property[a]	Soil Type	No. Data Groups	No. Tests/Group		Property Value (units[b])		Property COV (%)	
			Range	Mean	Range	Mean	Range	Mean
q_c	Sand	57	10 - 2039	115	0.4 - 29.2	4.10	10 - 81	38
	Silty Clay	12	30 - 53	43	0.5 - 2.1	1.59	5 - 40	27
q_T	Clay	9	-	-	0.4 - 2.6	1.32	2 - 17	8
s_u (VST)	Clay	31	4 - 31	16	6 - 375	104.7	4 - 44	24
N	Sand	22	2 - 300	123	7 - 74	35.1	19 - 62	54
	Clay, Loam	2	2 - 61	32	7 - 63	32.3	37 - 57	44
A reading	Sand to Clayey Sand	15	12 - 25	17	64 - 1335	511.6	20 - 53	33
	Clay	13	10 - 20	17	119 - 455	358.2	12 - 32	20
B reading	Sand to Clayey Sand	15	12 - 25	17	346 - 2435	1336.7	13 - 59	37
	Clay	13	10 - 20	17	502 - 876	690.3	12 - 38	20
E_D	Sand to Clayey Sand	15	10 - 25	15	9.4 - 46.1	25.39	9 - 92	50
	Sand, Silt	16	-	-	10.4 - 53.4	21.63	7 - 67	36
I_D	Sand to Clayey Sand	15	10 - 25	15	0.8 - 8.4	2.85	16 - 130	53
	Sand, Silt	16	-	-	2.1 - 5.4	3.89	8 - 48	30
K_D	Sand to Clayey Sand	15	10 - 25	15	1.9 - 28.3	15.11	20 - 99	44
	Sand, Silt	16	-	-	1.3 - 9.3	4.13	17 - 67	38
p_L	Sand	4	-	-	1617 - 3566	2283.8	23 - 50	40
	Cohesive Soils	5	10 - 25	17	428 - 2779	1083.6	10 - 32	15
E_{PMT}	Sand	4	-	-	5.2 - 15.6	8.97	28 - 68	42

a - q_c = cone tip resistance; q_T = corrected cone tip resistance; s_u(VST) = undrained shear strength from vane shear test; N = standard penetration test blow count; A and B = dilatometer A and B readings; E_D = dilatometer modulus; I_D = dilatometer material index; K_D = dilatometer horizontal stress index; p_L = pressuremeter limit stress; E_{PMT} = pressuremeter modulus

b - units of q_c, q_T, E_D, and E_{PMT} = MN/m^2; units of s_u(VST), A, B, and p_L = kN/m^2; units of N = blows/ft or 305 mm

1 MN/m^2 = 1000 kN/m^2 = 10.4 tsf

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A 6: COV of measurement uncertainty of field tests

Test	COV[a] Equip. (%)	COV Proc. (%)	COV Random (%)	COV[b] Total (%)	COV[c] Range (%)
Standard Penetration Test (SPT)	5[d] - 75[e]	5[d] - 75[e]	12 -15	14[d] -100[e]	15 - 45
Mechanical Cone Penetration Test (MCPT)	5	10[f] - 15[g]	10[f] - 15[g]	15[f] - 22[g]	15 - 25
Electrical Cone Penetration Test (ECPT)	3	5	5[f] - 10[g]	7[f] - 12[g]	5 - 15
Vane Shear Test (VST)	5	8	10	14	10 - 20
Dilatometer Test (DMT)	5	5	8	11	5 - 15
Pressuremeter Test (PMT)	5	12	10	16	10 - 20[h]
Self-Boring Pressuremeter Test (SBPMT)	8	15	8	19	15 - 25[h]

a - COV = standard deviation/mean
b - $COV(Total) = [COV(Equip.)^2 + COV(Proc.)^2 + COV(Random)^2]^{1/2}$
c - Because of limited data and the judgment involved in estimating COVs, ranges represent probable magnitudes of field test measurement error
d - Best case scenario for SPT test conditions
e - Worst case scenario for SPT test conditions
f - Tip resistance CPT measurements
g - Side resistance CPT measurements
h - It is likely that results may differ for p_0, p_f, and p_l, but the data are insufficient to clarify this issue

Source: Orchant, et al. (1988), p. 4-63

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A 7: Scales of fluctuation for common geotechnical properties

Fluctuation Direction	Property[a]	Soil Type	No. of Studies	Scale of Fluctuation (m) Range	Mean
Vertical	s_u	Clay	5	0.8 - 6.1	2.5
	q_c	Sand, Clay	7	0.1 - 2.2	0.9
	q_T	Clay	10	0.2 - 0.5	0.3
	s_u(VST)	Clay	6	2.0 - 6.2	3.8
	N	Sand	1	-	2.4
	w_n	Clay, Loam	3	1.6 - 12.7	5.7
	w_L	Clay, Loam	2	1.6 - 8.7	5.2
	$\bar{\gamma}$	Clay	1	-	1.6
	γ	Clay, Loam	2	2.4 - 7.9	5.2
Horizontal	q_c	Sand, Clay	11	3.0 - 80.0	47.9
	q_T	Clay	2	23.0 - 66.0	44.5
	s_u(VST)	Clay	3	46.0 - 60.0	50.7
	w_n	Clay	1	-	170.0

a - s_u = undrained shear strength from laboratory tests; s_u(VST) = s_u from vane shear test; q_c = cone tip resistance; q_T = corrected cone tip resistance; N = standard penetration test blow count; w_n = natural water content; w_L = liquid limit; $\bar{\gamma}$ = effective unit weight; γ = total unit weight

1 m = 3.28 ft

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Appendix B

TYPICAL PILE TEST RECORDS, LOAD-DEFLECTION CURVE, AND ASSOCIATED GEOTECHNICAL DATA

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B1: Typical load-deflection record

FRANKI

TEST PILE REPORT

CONTRACT **University of Botswana Hostels** CONTRACT NO. **BC 1028**

START DATE **14/07/2000**

Time Interval (minutes)	DATE & TIME	LOAD CHANGE %WL	TOTAL LOAD %WL	LOAD CHANGE KN	TOTAL LOAD KN	Pressure Gauge MPa	DIAL GAUGE No. 1 READING	DIAL GAUGE No. 2 READING	Settlement mm	TEMP °C
0	14/7/00 14:25	0	0%	0	0	0.00	0.00	0.00	0.00	
0	14/7/00 14:25	25%	25%	300	300	9.09	0.50	0.55	0.53	
5	14/7/00 14:30	0	25%	0	300	9.09	0.60	0.61	0.61	
5	14/7/00 14:35	0	25%	0	300	9.09	0.60	0.61	0.61	
10	14/7/00 14:45	0	25%	0	300	9.09	0.60	0.61	0.61	
10	14/7/00 14:55	0	25%	0	300	9.09	0.60	0.61	0.61	
0	14/7/00 14:55	25%	50%	300	600	18.18	1.28	1.26	1.27	
5	14/7/00 15:00	0	50%	0	600	18.18	1.31	1.31	1.31	
5	14/7/00 15:05	0	50%	0	600	18.18	1.33	1.32	1.33	
10	14/7/00 15:15	0	50%	0	600	18.18	1.34	1.33	1.34	
10	14/7/00 15:25	0	50%	0	600	18.18	1.36	1.35	1.36	
0	14/7/00 15:25	25%	75%	300	900	27.27	2.04	2.04	2.04	
5	14/7/00 15:30	0	75%	0	900	27.27	2.08	2.09	2.09	
5	14/7/00 15:35	0	75%	0	900	27.27	2.14	2.13	2.14	
10	14/7/00 15:45	0	75%	0	900	27.27	2.16	2.14	2.15	
10	14/7/00 15:55	0	75%	0	900	27.27	2.18	2.17	2.18	
0	14/7/00 15:55	25%	100%	300	1200	36.36	2.96	2.95	2.96	
5	14/7/00 16:00	0	100%	0	1200	36.36	2.98	2.96	2.97	
5	14/7/00 16:05	0	100%	0	1200	36.36	3.00	2.99	3.00	
10	14/7/00 16:15	0	100%	0	1200	36.36	3.10	3.08	3.09	
10	14/7/00 16:25	0	100%	0	1200	36.36	3.15	3.13	3.14	
15	14/7/00 16.40	0	100%	0	1200	36.36	3.17	3.15	3.16	
0	14/7/00 16:40	-25%	75%	-300	900	27.27	2.85	2.85	2.85	
10	14/7/00 16:50	0%	75%	0	900	27.27	2.80	2.80	2.80	
0	14/7/00 16:50	-25%	50%	-300	600	18.18	2.43	2.42	2.43	
10	14/7/00 17:00	0%	50%	0	600	18.18	2.41	2.39	2.40	
0	14/7/00 17:00	-25%	25%	-300	300	9.09	1.79	1.80	1.80	
10	14/7/00 17:10	0%	25%	0	300	9.09	1.72	1.70	1.71	
0	14/7/00 17:10	-25%	0%	-300	0	0.00	0.80	0.62	0.71	
10	14/7/00 17:20	0	0%	0	0	0.00	0.53	0.38	0.46	
10	14/7/00 17:30	0	0%	0	0	0.00	0.53	0.29	0.41	
10	14/7/00 17:40	0	0%	0	0	0.00	0.53	0.23	0.38	
800	15/7/00 7:00	25%	25%	300	300	9.09	0.40	0.41	0.41	
5	15/7/00 7:05	0	25%	0	300	9.09	0.42	0.42	0.42	
5	15/7/00 7:10	0	25%	0	300	9.09	0.42	0.42	0.42	
0	15/7/00 7:10	25%	50%	300	600	18.18	0.90	0.94	0.92	
5	15/7/00 7:15	0	50%	0	600	18.18	0.97	1.00	0.99	
5	15/7/00 7:20	0	50%	0	600	18.18	1.00	1.02	1.01	
0	15/7/00 7:20	25%	75%	300	900	27.27	1.76	1.79	1.78	
5	15/7/00 7:25	0	75%	0	900	27.27	1.82	1.83	1.83	
5	15/7/00 7:30	0	75%	0	900	27.27	1.83	1.86	1.85	
0	15/7/00 7:30	25%	100%	300	1200	36.36	2.57	2.60	2.59	
5	15/7/00 7:35	0	100%	0	1200	36.36	2.61	2.61	2.61	

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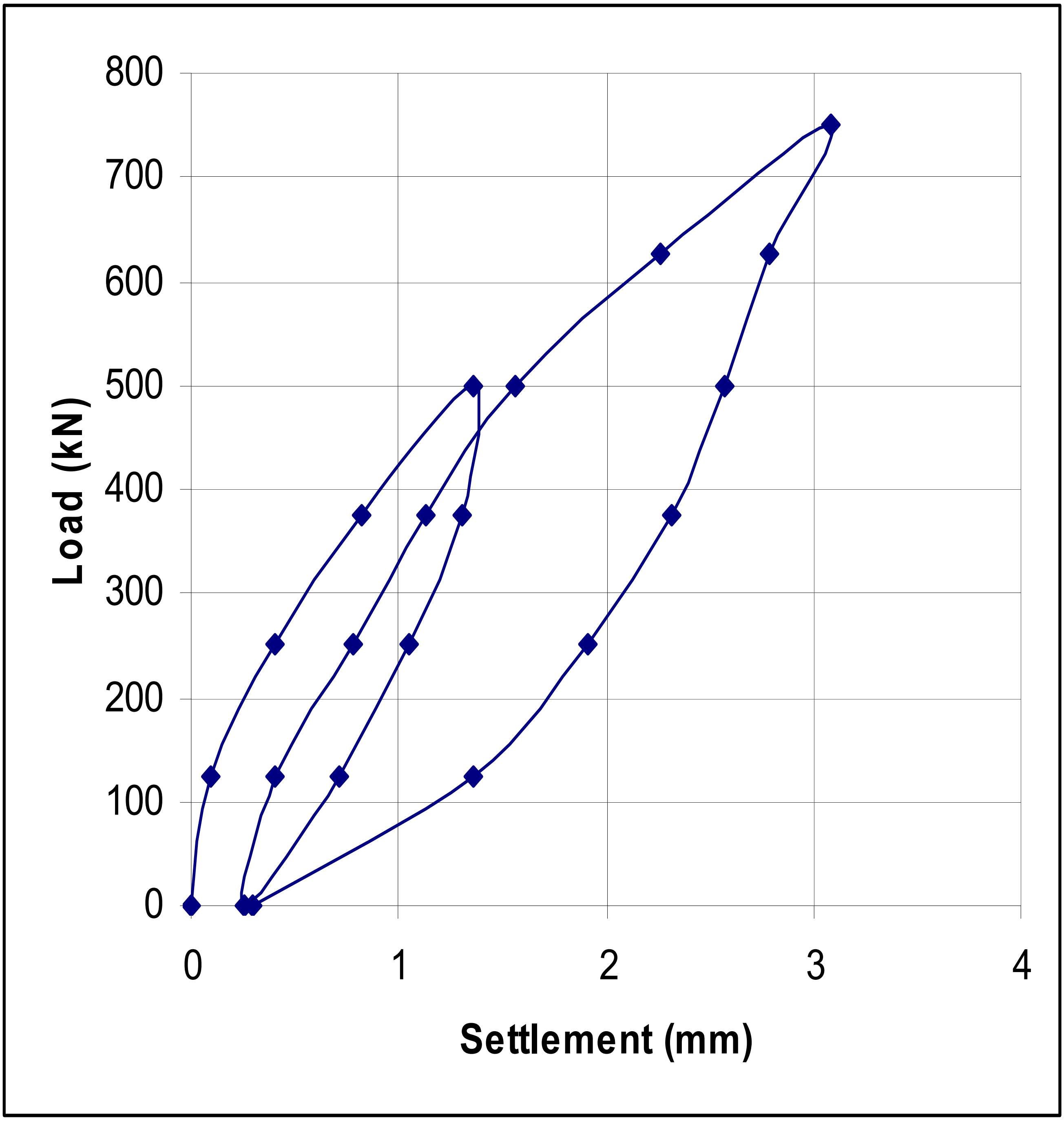
B 2: Typical load-deflection curve



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B 3: Typical soil data

BOREHOLE PROFILE

CONTRACT No	2755	LOGGED BY	B. BARRATT	SOIL PROFILE AT NATURAL MOISTURE CONTENT	BOREHOLE NO RS2 (S)
CONTRACTOR	SOILTECH	DRILLING STARTED	1998·10·28	SHEET	2 OF 2
DRILLER	SAMUEL	DRILLING COMPLETED	1998·10·29	CO-ORDINATE W =	617,183
MACHINE	TOHO	ORIENTATION	VERTICAL	CO-ORDINATE S =	893,504
				ELEVATION	27,729m M.S.L.

NOTE
ESR - EXTREMELY SOFT ROCK
VSR - VERY SOFT ROCK

CLASSIFICATION FOR PILE DESIGN
A UNCEMENTED - IMPROVED
B UNCEMENTED - UNIMPROVED SPT<25
C WEAKLY CEMENTED - SPT 25 - 40
D STRONGLY CEMENTED - SPT >40

Drilling Method and Size	Core Recovery %	R Q D %	Fracture Frequency	Test or Sample Type	Test Result	Depth m	Symbolic Log	Description
NX WASH BORE				↓	N = 52			Dry to slightly moist, dark reddish brown, dense, very slightly clayey, SILTY, coarse, fine and medium SAND.
				↓	N = 41	11,0		11,45 C/D
				↓	N = 37	12,0		Dry, dark orange brown, dense, trace fine subangular to subrounded gravelly, very slightly clayey, SILTY, fine and medium SAND.
				↓	N = 42	13,0		
				↓	N = 45	14,0		14,45 D/C
				↓	N = 50	15,0		Dry, orange brown to yellowish orange, very dense, trace fine subangular to subrounded gravelly, silty, coarse, fine to medium SAND.
				↓	N = 56	16,0		16,45 D
				↓	N = 54	17,0		Moist to wet, yellowish brown, very dense, trace fine subrounded gravelly, silty, fine medium to coarse SAND.
				↓	N = 72	18,0		18,45
NWD4 CORE	81	17	3			19,0	ESR / ESR / VSR	Yellowish brown, highly weathered, medium to widely jointed, soft rock, trace find subrounded gravelly, coarse, fine to medium SAND 18.60-18.68 } 18.89-18.95 } completely weathered, extremely soft rock, fine to medium SANDSTONE 18.95-19.45 very soft rock
						20,0	End of Borehole	19,45

MOZAL PROJECT	DAVIES LYNN & PARTNERS (PROPRIETARY) LIMITED CONSULTING ENGINEERS & ENGINEERING GEOLOGISTS P.O. Box 586, Kloof 3640 Telephone (031)7647335 Telefax (031)7647385
SPECIAL TRIAL PILE SITE REDUCTION SERVICES	REF NO 2755
	FIG NO P22·3

	Drilling Progress/Shift	↑	Standard Penetration Test
═	Casing Depth	○	Disturbed Sample
I	Standing Water Level	·	In-situ Shear Vane Test
S	Strength Test	□	Piston Sample
C	Consolidation Test	■	U4 Tube Sample

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Appendix C

RELIABILITY INDEX CALCULATION SPREADSHEETS AND THE ASSOCIATED EXCEL FUNCTIONS DESCRIPTION

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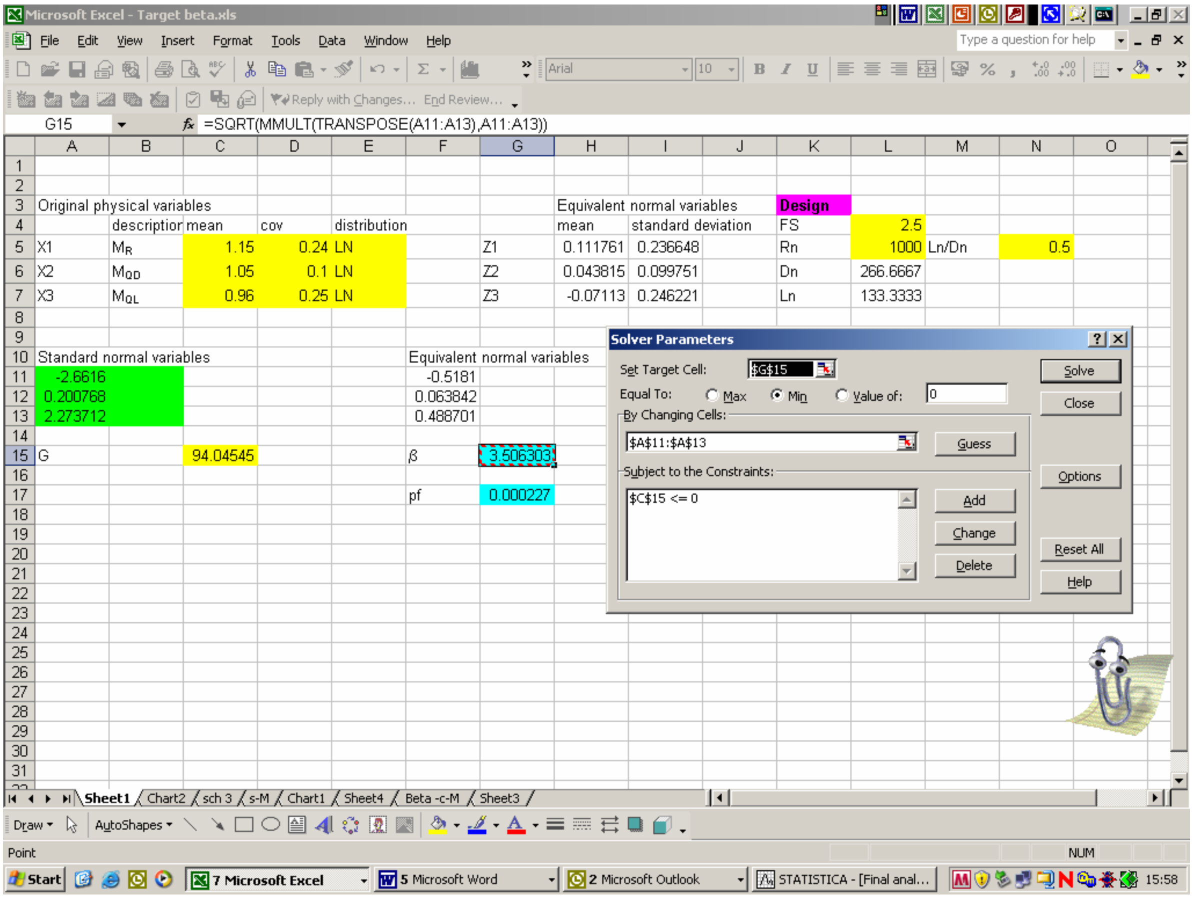
C1: Spreadsheet for computation of beta (standard M approach)



C2: Excel functions for the various cells of the calibration spreadsheet

Cell	Excel function
H5 : H7	H5 = LN(C5)-0.5*I5^2; H6=LN(C6)-0.5*I6^2; H7 = LN(C7)-0.5*I7^2
I5 : I7	I5 = SQRT(LN(1+D5^2); I6 = SQRT(LN(1+D6^2); I7 = SQRT(LN(1+D7^2)
F11 : F13	F11 = A11*I5+H5; F12 =A12*I6+H6; F13 = A13*17+H7
J11: J13	J11 = EXP(F11); J12 = EXP(F12); J13 = EXP(F13)
C15	J11*L5-J12*L6-J13*L7
G15	{=SQRT(MMULT(TRANSPOSE(A11:A13),A11:A13))}
G17	=NORMSDIST(-G15)
L6	L5/L4/(1+N5)
L7	L5/(L4-L6)

Solver:

Set target Cell: $G$15 equal to: Min

By changing Cells: $A$11:$A$13

Subject to the constraints: $C$15<=0

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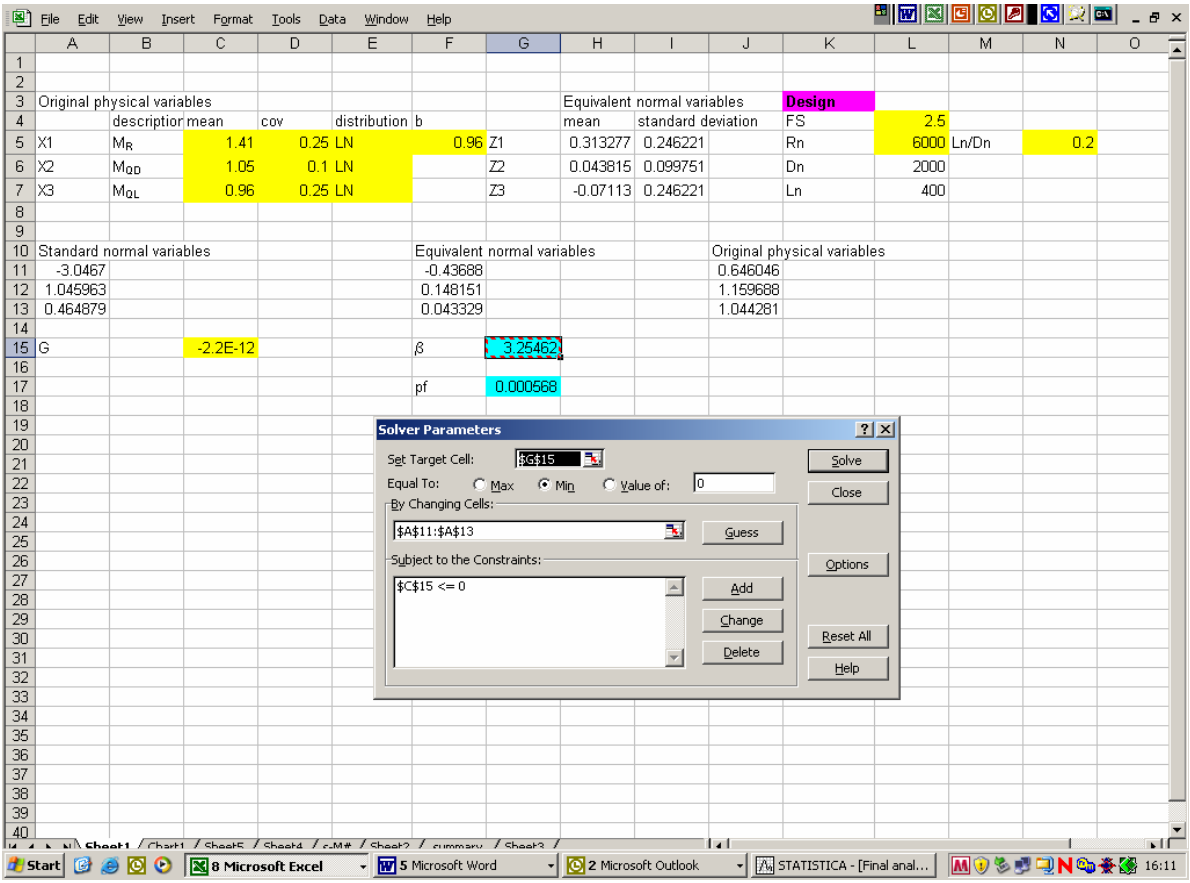
C.3: Spreadsheet for computation of beta (generalised M approach)



Chapter One **C4:** Excel functions for the various cells of the calibration spreadsheet

Cell	Excel function
H5 : H7	H5 = LN(C5)-0.5*I5^2; H6=LN(C6)-0.5*I6^2; H7 = LN(C7)-0.5*I7^2
I5 : I7	I5 = SQRT(LN(1+D5^2); I6 = SQRT(LN(1+D6^2); I7 = SQRT(LN(1+D7^2)
F11 : F13	F11 = A11*I5+H5; F12 =A12*I6+H6; F13 = A13*17+H7
J11: J13	J11 = EXP(F11); J12 = EXP(F12); J13 = EXP(F13)
C15	J11*L5^F5-J12*L6-J13*L7
G15	{=SQRT(MMULT(TRANSPOSE(A11:A13),A11:A13)))}
G17	=NORMSDIST(-G15)
L6	L5/L4/(1+N5)
L7	L5/(L4-L6)

Solver:

Set target Cell: $G$15 equal to: Min

By changing Cells: $A$11:$A$13

Subject to the constraints: $C$15<=0

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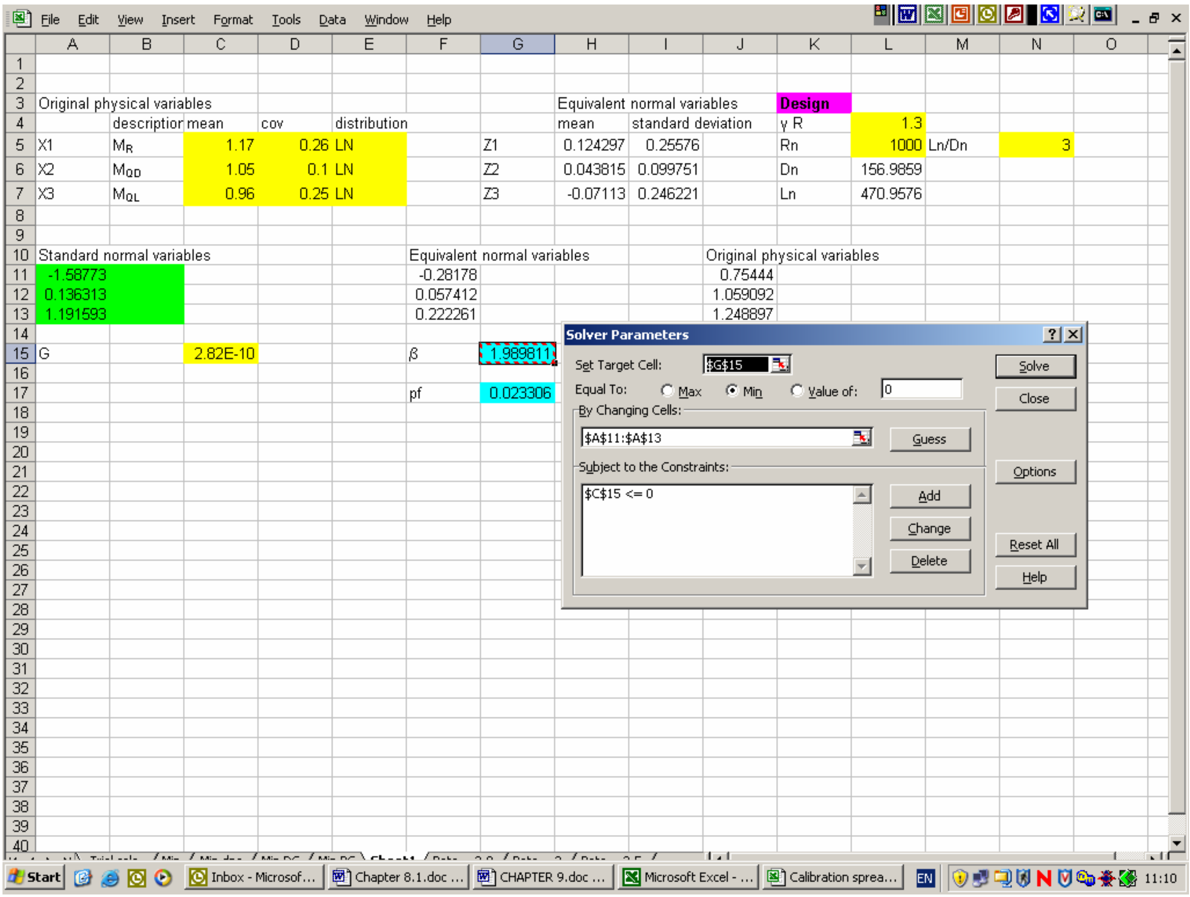
Appendix D

CALIBRATION SPREADSHEETS

D1: Calibration Spreadsheet



D2: Excel functions for the various cells of the calibration spreadsheet

Cell	Excel function
H5 : H7	H5 = LN(C5)-0.5*I5^2; H6=LN(C6)-0.5*I6^2; H7 = LN(C7)-0.5*I7^2
I5 : I7	I5 = SQRT(LN(1+D5^2); I6 = SQRT(LN(1+D6^2); I7 = SQRT(LN(1+D7^2)
F11 : F13	F11 = A11*I5+H5; F12 =A12*I6+H6; F13 = A13*17+H7
J11: J13	J11 = EXP(F11); J12 = EXP(F12); J13 = EXP(F13)
C15	J11*L5-J12*L6-J13*L7
G15	{=SQRT(MMULT(TRANSPOSE(A11:A13),A11:A13)))}
G17	=NORMSDIST(-G15)
L6	L5/L4/(1+1.3*N5)
L7	L5/(L4*1.3)-L6/1.3

Solver:

Set target Cell: $G$15 equal to: Min

By changing Cells: $A$11:$A$13

Subject to the constraints: $C$15<=0