

PROBABILISTIC FAILURE ANALYSIS OF PILE FOUNDATIONS

BY



OPEYEMI, DAVID AKINYIWOLA

CVE/05/7104

A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL IN
PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE
AWARD OF MASTER OF ENGINEERING (M.ENG) DEGREE
IN CIVIL ENGINEERING OF THE FEDERAL UNIVERSITY OF
TECHNOLOGY, AKURE, NIGERIA.

APRIL, 2009.

DECLARATION

I hereby declare that this thesis was written by me and that it is a record of my own research findings under the supervision of Prof. J.O. Afolayan. It has not, either in part or whole, been previously submitted by me for this or any degree or qualification at this or any other University. All sources of information are specifically acknowledged by means of references.

Opeyemi, D.A.

January, 2009.

CERTIFICATION

This thesis titled: "Probabilistic Failure Analysis of Pile Foundations" by Opeyemi, David Akinyiwola, has been read and approved as having satisfied the conditions for the award of Master of Engineering in Civil Engineering, Civil Engineering Department, Federal University of Technology, Akure, Nigeria.



Prof. J. O. Afolayan

Major Supervisor

16. 6. 09

Date



Dr. C. Arum

Co-Supervisor

16. 06. 09

Date



Prof. J. O. Afolayan

Head of Department

16. 6. 09

Date



DEDICATION

To the Only **WISE** God.



ACKNOWLEDGEMENTS

I acknowledge the fact that God is the source of all wisdom and wisdom personified. The first and greatest structural engineer – He spreads the heavens like a curfain and laid the foundations of the earth upon many waters (floods). Psalm 24:2.

I like to express my sincere appreciation to my Supervisor, Prof. J.O. Afolayan, for his guidance, teaching, patience, encouragement, suggestions and continued support, particularly for arousing my interest in Risk Analysis (Reliability Analysis of Structures).

Special thanks are extended to my beloved parents – Mr. & Mrs. Joseph Akinbulijo Ogunfesola and Felicia Modupe Ogunfesola for their love and support. Also, Reconciliation House brethren- their prayers, supports and encouragement.

I appreciate my lecturers by whose wealth of experience/knowledge made me a master of Civil Engineering, and be well informed: Prof. S.A. Ola, Prof. J.O. Afolayan, Prof. M.O. Ogedengbe, Dr. C. Arum, Dr. J. O. Babatola, Dr. A. M. Oguntuase, Dr. A. O. Owolabi, Dr.C. S. Okoli and Engr. O. Adeniyi.

I like to sincerely thank my course mates: Alhassan and Allen (Structures), Adekunle, Olarinoye, Fijabi, Salawu and Mrs. Akinwumi (Transportation) and Adenikinju (Water).

Finally, I appreciate my family members: my wife – Blessing Kehinde and the children: (a boy and 2 girls) – IniOluwa Toluwanimi and EbunOluwa for their prayers, financial and moral support.

D.A. Opeyemi.



TABLE OF CONTENTS

Title Page	...	...	...	...	...	...	...	...	i
Declaration	...	...	...	...	...	...	...	...	ii
Certification...	...	...	...	...	...	...	...	...	iii
Dedication	...	...	...	...	...	...	...	...	iv
Acknowledgements	...	...	...	...	...	...	...	...	v
Table of Contents	...	...	...	...	...	...	...	...	vi
List of Figures	...	...	...	...	...	...	...	...	x
List of Tables...	...	...	...	...	...	...	...	...	xiii
Abstract	...	...	...	...	...	...	...	...	xv

CHAPTER ONE: INTRODUCTION

1.1	Definitions	...	...	...	...	...	...	...	1
1.2	Background to the Research Work	...	...	...	...	...	...	...	2
1.3	Aim and Objectives	...	...	...	...	...	...	...	2
1.3.1	Aim	...	...	...	...	...	...	...	2
1.3.2	Objectives	...	...	...	...	...	...	...	2
1.4	Justification	...	...	...	...	...	...	...	3
1.5	Scope	...	...	...	...	...	...	...	3
1.6	Expected Contribution of the Research to Knowledge	...	...	...	...	...	...	...	4

CHAPTER TWO: LITERATURE REVIEW

2.1	Introduction	5
2.2	Accuracy of Determining Pile Capacity by Dynamic Methods	6
2.2.1	Dynamic method development	7
2.2.2	Pile capacity variations with time	9
2.2.3	Accuracy of dynamic formulae	9
2.2.4	Accuracy of dynamic testing and analysis	10
2.2.5	Points for considerations	12
2.3	Reliability and Stability Assessment of Concrete Gravity Structures	13
2.4	Reliability – Based Design for Deep Foundations	14
2.5	Probabilistic Geo-technical analysis for Off-shore Facilities	15

CHAPTER THREE: PILE FOUNDATION ANALYSIS

3.1	Introduction	18
3.2	Pile Materials (Steel & Precast Concrete)	20
3.2.1	Precast Concrete Piles	20
3.2.2	Steel Piles	21
3.3	Static Pile Capacity	22
3.3.1	Ultimate Static Pile Point Capacity	23
3.3.2	Skin Resistance Capacity	24

3.3.2.1	The δ Method	...	...	...	...	26
3.3.2.2	The λ Method	...	...	...	...	30
3.3.2.3	The β Method	...	...	...	...	30
3.3.2.4	Calculation of the carrying capacity of Piles in Soils intermediate between Sand and Clay and Layered Soils	...	...	...	...	34
3.4	Dynamic Analysis	...	...	...	...	34
3.5	Allowable Design Loads	...	...	...	...	41

CHAPTER FOUR: METHODOLOGY

4.1	Conception of Reliability Analysis	...	...	...	...	50
4.1.1	Introduction	...	...	...	...	50
4.1.2	Reliability Analysis	...	...	...	...	51
4.2	Second – Moment Concept (FOSM)	...	...	...	...	55
4.3	First Order Reliability Method (FORM)	...	...	...	...	56

CHAPTER FIVE: RELIABILITY ESTIMATES

5.1	Allowable Design Load	...	...	...	...	59
5.2	Static Pile Capacity of Concrete Piling in Cohesive Soils	...	...	...	...	59
5.3	Static Pile Capacity of Concrete Piling in Cohesionless Soil	...	...	...	...	64
5.4	Static Pile Capacity of Steel Piling in Cohesive Soils	...	...	...	...	70

5.5	Static Pile Capacity of Steel	
	Piling in Cohesionless Soils	76
5.6	Dynamic Pile Capacity Using Hiley Formula	82
5.7	Dynamic Pile Capacity Using Janbu Formula	88
5.8	Dynamic Pile Capacity Using Gates Formula	90
5.9	Comparison of Piling Options	91
5.9.1	Static Pile Capacity of Concrete	
	Piling in Cohesive and	
	Cohensionless Soils...	91
5.9.2	Static Pile Capacity of Steel	
	Piling in Cohesive and	
	Cohensionless Soils ...	91
5.9.3	Static Pile Capacity of Concrete	
	& Steel Piling in Cohesive Soils ..	92
5.9.4	Static Pile Capacity of Concrete	
	& Steel Piling in Cohesionless	
	soils.	92
5.9.5	Dynamic Pile Capacity using	
	Hiley & Janbu Formulae ...	92
5.9.6	Dynamic Pile Capacity Using	
	Hiley, Janbu & Gates Formulae.....	92

CHAPTER SIX: CONCLUSION AND RECOMMENDATIONS

6.1	Conclusions ...	93
6.2	Recommendations	93
	References	95
	Appendix	100

LIST OF FIGURES

FIGURES

3.1	Typical Pile Configurations	43
3.2	(a) General development of pile capacity in either single or stratified soil mass	44
	(b) Qualitative zone of interest for ultimate point capacity	44
3.3	Coefficient depending on pile penetration	
	Data plot from Flaate and Seines 1977	45
3.4	Plot of S_u and q versus average skin resistance	46
3.5	Reduction in angle of internal friction with increase in mean normal confining stress	47
3.6	Adhesion factors for driven piles in clay	48
3.7	Terms used in the dynamic pile-driving equations	49
5.1	Safety index (β_R) against Length of Pile with varying Diameters	60
5.2	Safety index (β_R) against Cohesion with varying Diameters	61
5.3	Safety index (β_R) against Adhesive factor with varying Diameters	61
5.4	Safety index (β_R) against Factor of Safety with varying Diameters in cohesive soils	62
5.5	Safety index (β_R) against Adhesion with varying Lengths	62
5.6	Safety index (β_R) against Factor of Safety with varying Lengths in cohesive soils	63
5.7	Safety index (β_R) against Cohesion with varying Lengths	63

5.8	Safety index (β_R) against Length of Pile with varying Diameters	66
5.9	Safety index (β_R) against Unit weight of soil with varying Diameters	66
5.10	Safety index (β_R) against Angle of internal friction with varying Diameters	67 -
5.11	Safety index (β_R) against Effective friction angle with varying Diameters.....	67
5.12	Safety index (β_R) against Factor of safety with varying Diameters in.	68
5.13	Safety index (β_R) against Unit Weight of soil with varying Lengths..	68
5.14	Safety index (β_R) against Effective friction angle with varying Lengths 58	
5.15	Safety index (β_R) against Angle of internal friction with varying Lengths.. ..	69
5.16	Safety index (β_R) against Factor of safety with varying Lengths in cohesionless soils	70
5.17	Safety index (β_R) against Cohesion with varying Pile Areas	72
5.18	Safety index (β_R) against Adhesive factor with varying Pile Areas	72
5.19	Safety index (β_R) against Factor of safety with varying Pile Areas in cohesive soils	73

5.20	Safety index (β_R) against Area of Shaft with varying Pile Areas	73
5.21	Safety index (β_R) against Cohesion with varying Shaft Areas	74
5.22	Safety index (β_R) against Factor of safety with varying Shaft Areas ..	74
5.23	Safety index (β_R) against Adhesive Factor with varying Shaft Areas ..	75
5.24	Safety index (β_R) against Shaft area with varying Pile Areas	77
5.25	Safety index (β_R) against Unit Weight of soil with varying Pile Areas	77
5.26	Safety index (β_R) against Length of Pile with varying Pile Areas	78
5.27	Safety index (β_R) against Angle of internal friction with varying Pile Areas	78
5.28	Safety index (β_R) against Effective friction angle with varying Pile Areas	79
5.29	Safety index (β_R) against Factor of safety with varying Pile Areas in cohesionless soils	79
5.30	Safety index (β_R) against Shaft area with varying Lengths of Pile	80
5.31	Safety index (β_R) against Unit weight of soil with varying Lengths of Pile	80
5.32	Safety index (β_R) against angle of internal friction with varying Lengths of Pile	81
5.33	Safety index (β_R) against Effective friction with varying Lengths of Pile	81
5.34	Safety index (β_R) against Factor of safety with varying Lengths of Pile	82

5.35	Safety index (β_R) against Area of Pile	
	using Hiley, Janbu and Gates formulae	85
5.36	Safety index (β_R) against Hammer Efficiency	
	using Hiley, Janbu and Gates formulae	85
5.37	Safety index (β_R) against Hammer-energy rating	
	using Hiley, Janbu and Gates formulae	86
5.38	Safety index (β_R) against Point penetration per blow	
	using Hiley, Janbu and Gates formulae	86
5.39	Safety index (β_R) against Factor of safety	
	using Hiley, Janbu and Gates formulae	87
5.40	Safety index (β_R) against Weight of Pile	
	using Hiley and Janbu formulae	87
5.41	Safety index (β_R) against Length of Pile	
	using Hiley and Janbu formulae	88

LIST OF TABLES

TABLE	
3.1	Values of adhesion factors for pile driven into stiff to very stiff cohesive soil 27
3.2	Friction angles δ between various foundation materials and soil or rock 28

3.3	Values for K1 – temporary elastic compression of Pile head and cap	39
3.4	Hammer Efficiency	40
3.5	Representative values of Co-efficient of restitution for use in the dynamic pile driving equations.	41
5.1	Stochastic model for concrete piling in cohesive soils, ...	60
5.2	Stochastic model for concrete piling in cohesionless soils.	65
5.3	Stochastic model for steel piling in cohesive soils.	71
5.4	Stochastic model for steel piling in cohesionless soils	76
5.5	Stochastic model for Dynamic Pile capacity using Hiley formula ..	84
5.6	Stochastic model for Dynamic Pile capacity using Janbu formula	89
5.7	Stochastic model for Dynamic Pile capacity using Gates formula	91

ABSTRACT

The reliability assessment of the load-carrying capacities of piles based on static and dynamic approaches with special consideration to pre-cast concrete and steel as pile types is reported in this work. Steel and precast concrete piles were analyzed based on static pile capacity equations under various soils such as cohesive, cohesionless, and intermediate soils between sand and clay and layered soils. Likewise, three (3) dynamic formulae (namely: Hiley, Janbu and Gates) were closely examined and analyzed using the first-order reliability concept. Uncertainties are common phenomena in engineering, therefore all the interrelated variables in the carrying capacities of pile are treated as random variables with assumed practical probability density functions. The concept of the First-Order Reliability Method (FORM) is a powerful tool for estimating nominal probability level of failure associated with uncertainties and it is the method invoked for the reliability estimations in this work. Generally, piling reliability decreases as pile length increases but the rate of decrease is more rapid for concrete than steel. For precast concrete piling, pile length greater than 30 m will result in catastrophe while much longer steel piles that economy will permit are admissible. Hence, steel piles are recommended as alternative materials in cohesionless soils. Likewise, the dynamic predictions of Hiley and Janbu lead to similar safety levels while Gates' results in totally different implied safety levels.



CHAPTER ONE

INTRODUCTION



1.1 Definitions

Structural reliability can be expressed by defining functional relations between strength (R) and load (L) parameters as follows,

$$M = R - L = g(X_1, X_2, \dots, X_n) \quad \dots \quad \dots \quad \dots \quad \dots \quad (1.1)$$

M is the limit state function and is sometimes referred to as safety margin or performance function, X_i , $i = 1 \dots n$ denote the basic design random variables, $g(\cdot)$ denotes a function of all design variable. In general, the function $g(\mathbf{X})$ can take any form provided that the failure of the structure is defined when $M \leq 0$, and the survival of the structure is defined when $M > 0$. Therefore, the probability of failure of the structure can be calculated by performing the following integration over the region where $M \leq 0$.

$$P_f = \int \dots \int f_x(x_1, x_2, \dots, x_n) dx_1 dx_2 \dots dx_n \quad \dots \quad \dots \quad \dots \quad \dots \quad (1.2)$$

Where f_x is the joint probability density function for the basic random variables $X_1, X_2, \dots, X_n$. (Elhewy and Mesbahi, 2006)

Piles may be structural members of timber, concrete, and/or steel, used to transmit surface loads to lower levels in the soil mass. This may be by vertical distribution of the load along the pile shaft or a direct application of load to a lower stratum through the pile point. A vertical distribution of the load is made using a friction, or "floating", pile, or "end-bearing", pile. This distinction of piles is purely one of convenience since all piles function as a combination of side resistance and point bearing except when the pile penetrates an extremely soft soil to a solid base (Bowles, 1988).

Pile foundations must have sufficient structural integrity to safely transfer the applied loads from the structure to the ground. This means they must have

sufficient strength to safely sustain the applied stresses, and sufficient stiffness to keep deformations within tolerable limits (Coduto, 2001).

1.2 Background to the Research Work

Pile capacity determination is very difficult. A large number of different equations are used, and seldom will any two give the same computed capacity. Organizations which have been using a particular equation tend to stick to it especially when successful data base has been established. It is for this reason that a number of what are believed to be the most widely used (or currently accepted) equations are included in most literature.

Also, the technical literature provides very little information on the structural aspects of pile foundation design, which is a sharp contrast to the mountains of information on the geotechnical aspects. Building codes present design criteria, but often they are inconsistent with criteria for the super structure, and sometimes are incomplete or ambiguous. In many ways this is an orphan topic that neither structural engineers nor geotechnical engineers have claimed as their own.

1.3 Aim and Objectives

1.3.1 Aim:

To rate the formulae for estimating the load-carrying capacity of piles using reliability analysis concept.

1.3.2 Objectives

- (i) To examine the various predictions of load carrying capacity of piles.
- (ii) To assess the uncertainties in properties of soils, pile materials and the loads that must be resisted.

- (iii) To employ the principles of probability and statistics to describe the various parameters that affect reliability of pile foundations, and calculate the levels of reliability of piles compared to the effect of allowable design loads in design code.

1.4 Justification

Deterministic methods are very subjective and are generally not based on a systematic assessment of reliability, especially when we consider their use in the entire structure (not only the foundations). These methods can produce structures with some "over designed" components and perhaps some "under designed" components. The additional expense incurred in constructing the over designed components probably does not contribute to the overall reliability of the structure, so this is not a very cost-effective way to produce reliable structures. In other words, it would be better to redirect some of the resources used to build the over designed components toward strengthening the under designed ones. Therefore, there is increasing interest in adopting reliability-based design methods in civil engineering. These methods are intended to quantify reliability, and thus may be used to develop balance designs that are both more reliable and less expensive. Also, the methods can be used to better evaluate the various sources of failure and use this information to develop design and construction methods that are both more reliable and more robust - one that is insensitive to variations in materials, construction techniques, and environment (Coduto, 2001).

1.5 Scope

This work involves extensive literature review and analysis of the carrying capacities of piles based on static and dynamic approaches with special consideration to steel and precast concrete as pile types in cohesive,

cohesionless, and intermediate soils between sand and clay and layered soils.

Steel and precast concrete piles were analyzed based on static pile capacity equations under various soils such as cohesive, cohesionless, and intermediate soils between sand and clay and layered soils. Likewise, three (3) dynamic formulae (namely: Hiley, Janbu and Gates) were closely examined and analyzed using the first-order reliability concept.

1.6 Expected Contribution of the Research to Knowledge

The research is expected to:

- (i) Highlight the merits or demerits of the existing load carrying capacity predictions for precast and steel piles.
- (ii) Provide advice on the choice of materials for piling and suitable dimensions based on the load from the superstructure, allowable stress in the pile material, and in-situ soil properties.



CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Mankind has used pile foundations for more than 2000 years. Alexander the Great drove piles in the city of Tyre in 322 B.C., and the Romans used them extensively. Bridge builders in China during the Han Dynasty (2000 B.C. – A.D. 200) also used piles. These early builders drove their piles into the ground using weights hoisted and dropped by hand (Chellis, 1961). By the Middle Ages, builders used pulleys and leverage to raise heavier weights.

Construction methods improved more quickly during the Industrial Revolution, especially when steam power became available. Larger and more powerful equipment was built, thus improving pile driving capabilities. These improvements continued throughout the twentieth century.

Pile materials also have become better. The early piles were always made of wood, and thus were limited in length and capacity. Fortunately, the advent of steel and reinforced concrete in the 1890s enabled the construction of larger and stronger piles, and better driving equipment made it possible to install them. Without these improved foundations, many of today's major structures would not have been possible.

Today, pile foundations can support very high loads, even in hostile environments. Perhaps the most impressive are those for offshore oil drilling platforms. These are as large as 3m in diameter and must resist large lateral loads due to wind, wave and earthquake forces (Coduto, 2001).

2.2 Accuracy of Determining Pile Capacity by Dynamic Methods

Pile foundations are widely used in bridge construction, buildings and other structures. Accurate determination of pile capacity is very important for proper design, construction and estimation of these foundations. It is common in design practice to predict pile capacity by static analysis in advance of pile driving based on the results of in-situ and/or laboratory soil and rock tests. Traditionally, the static loading test is used to determine ultimate capacity of the pile-soil system or the value of a service load to be supported by a pile. In recent decades, because of advances in data acquisition during pile driving and restrikes, dynamic testing has become an integral part of pile capacity prediction and measurement.

Dynamic methods have certain advantages and some uncertainties in their application. Wave equation analysis of driven piles is a prevalent method of pile driving stress calculations. Besides driveability analysis, the wave equation method is used for determination and prediction of pile capacity during both the design stage and for construction control during pile installation. Unfortunately in most cases, computed pile capacity differs substantially from results of both static and dynamic load tests. Errors in determination of pile capacity will create insufficiencies in pile foundation selection and will decrease foundation reliability.

Dynamic measurements of force and velocity at the upper end of the pile during pile driving, followed by a signal matching procedure, is the most common method for dynamic determination of pile capacity. This method is a convenient tool in the pile driving industry. However, though dynamic methods have been used in practice for years, actual reliability of dynamic

methods is vague because their comparison with static loading tests is made incorrectly in most cases.

2.2.1 Dynamic methods development

Determination of pile capacity by dynamic formulae is the oldest and most frequently used method. All such formulas assume that the hammer kinetic energy is to be equal to the driving resistance and the soil resistance is equal to pile capacity under static loading. There are a great number of dynamic formulae available with different degrees of reliability. The derivation of most of these formulas and details of some of the parameters required are available in Whitaker (1970) and Chellis(1961).

The main goal in using the wave equation method is to provide a better prediction of the pile capacity, as a function of pile penetration resistance, than can be obtained from classical dynamic formulas. The first solution of longitudinal wave propagation in elastic rods with impact was given by St. Venant almost a century ago (Timoshenko and Goodier 1951). In the 1930s, Isaacs (1939), Fox (1932) and Granville et al. (1938) pointed out that the one-dimensional wave equation can be used to analyse pile driving. However, the first step in the practical use of wave equation analysis was pioneered by Smith (1955, 1960) when he developed a mathematical model of hammer-pile-soil system in the late 1950s. This model is the numerical idealization of load-deformation characteristics of the soil, taking into account accumulated experience of that time of the actual behaviour of driven piles and surrounding soil.

The first computer programme for pile driving analysis was developed by Smith. Although some modifications improvements of Smith's model were

subsequently necessary, results obtained by many researchers confirmed the soundness of the basic approach.

The second step was made in the middle of 1960s when Dr. G.G. Goble and Associates developed pile capacity calculations from measured force and velocity at the upper end of the pile. They have suggested a simplified close form solution: The Case Method, which yields straight forward real time results. Other simple procedures such as Impedance Method were produced later by Beringen et al. (1980). These more accurate methods utilize a numerical solution of more rigorous mathematical models of the multi-parameter hammer-pile-soil system and provide evaluation the pile and soil boundary conditions through an iterative process of signal matching. The interpretation of measured data with the signal matching technique methods is much more reliable than those with the simple methods. Dynamic testing methods are described in Holloway et al. (1979), Goble et al. (1980), Rausche et al. (1985), Hanningan (1990) and Holeyman (1992).

Dynamic Pile testing (DT) has become widely used as a replacement for or supplement to Static Loading Tests (SLT) because of its inherent savings in cost and time. These dynamic methods allow monitoring pile driving and restrikes, and also provide a method of identifying problems during driving for many kinds of piles. To obtain reliable ultimate resistance, it is necessary that the long term pile capacity be fully mobilized. Dynamic testing methods can determine static capacity at the time of testing, in other words, either at the end of driving or at restrikes. This is a substantial advantage because dynamic tests can be easily repeated and, consequently, there is an opportunity to obtain pile capacity as a function of time as well as pile embedment.

2.2.2 Pile capacity variations with time

Piles have to withstand design loads for a long period of time. Therefore, the consequences of soil modification around the pile are essential with respect to changes of pile capacity. During pile installation, the soil around the pile experiences plastic deformations, remolding, and pore pressure changes. Excess pore water pressure developed during driving reduces the effective soil shear strength and ultimate pile capacity. After the completion of pile driving, soil reconsolidation, manifested by the dissipation of excess pore pressure at the soil-pile interface zone, is usually accompanied by an increase in pile capacity (soil setup). The amount of increase in pile capacity depends on soil properties and pile characteristics. In saturated sandy soils, ultimate pile capacity may decrease (soil relaxation) after initial driving due to dissipation of negative pore pressure. Changes of strength in soil after driving and the time required for return of equilibrium conditions are highly variable and depend on soil type, and pile size and type.

2.2.3 Accuracy of dynamic formulae

The well-known dynamic formulae have been criticized in many publications. Unsatisfactory prediction in pile capacity by dynamic formulae is well characterized in the recent published Manual for Design and Construction of Driven Pile Foundations (Hannigan et al, 1996), in which it was concluded: "Whether simple or more comprehensive dynamic formulas are used, pile capacities determined from dynamic formulae have shown poor correlations and wide scatter when statistically compared with static load test result. Therefore, except where well supported empirical correlations under a given set of physical and geological conditions are available, dynamic formulas should not be used."

There are two attempts to breathe new life into dynamic formulae. First, Paikowsky and Chernauskas (1992) and Paikowsky et al. (1994) have suggested one more simplified energy approach using dynamic measurements for the capacity evaluation of driven piles. Liang and Zhou (1997) have concluded regarding this method: "Although the delivered energy is much more exactly evaluated, this method still suffers similar drawbacks of Engineering News (ENR)". In a second, criticizing the simplified energy approach, Liang and Zhou (1997) have developed a probabilistic energy approach as an alternative to the signal matching technique for effective pile-driving control in the field. Both attempts to improve dynamic formulas, comparison of pile capacity determined by the simplified and probabilistic energy methods with the results of SLT, are incorrect. Dynamic formulas, including their two new representations, using maximum energy, pile set and maximum displacement from DT do not take into account the time between SLT and DT.

2.2.4 Accuracy of dynamic testing and analysis

Since dynamic testing is often used to replace the static loading tests, it is important to ascertain the adequacy of both SLT and DT. Design methods predict pile capacity as the long term capacity after soil consolidation around the pile is complete. Independently of the time elapsed between the driving of the test pile and the static test, the ratio of the predicted ultimate load over the measured ultimate load from static loading test is for approximate evaluation of the reliability of design methods (Briaud and Tucker, 1988). According to the traditional approach, the main criterion for assessment of the pile capacity prediction based on dynamic measurements is the ratio of capacities obtained by dynamic and static tests or vice versa. A number of papers, for example, Goble et al. (1975), Goble et al. (1980), Rauchsche et al. (1985), Hannigan (1990) present good agreement between dynamic and static tests in spite of ignoring the time between compared tests. These are strange correlation results. Other papers, for example,

Hannigan and Webster (1987), Paikowsky et al. (1994), Lee et al. (1996), Liang and Zhou (1997) reveal substantial over and under prediction of pile capacity obtained by dynamic testing. Paikowsky et al. (1994) made comparison of DT and SLT for 204 pile-cases in various types of soil. The computed pile capacity from DT ranged from under prediction of about 0.4 to maximum over prediction of about 1.7. These correlation results look as more realistic.

It is necessary to point out that a ratio of DT/SLT or vice versa, taken for arbitrary time between compared tests, is not a verification of dynamic testing results. It is well-known that dynamic testing methods yield the real static capacity of piles at the time of testing, (Rausche et al., 1985). This is not a predicted value. Moreover, the papers referenced above consider the static capacity from SLT as a unique standard for assessment of dynamic testing results. Unfortunately, that is a major error. As a matter of fact, pile capacity from Static Loading Tests is a function of time and the so-called actual static capacity from SLT is not a constant value.

For a few separate piles, it is possible to find published information regarding the time between static and dynamic tests. However, for the general case of assessment of reliability of the DT, the ratio of restrikes to SLT results has been considered for various pile types, soil conditions and times of testing lumped together as in the papers referenced above. What is the real meaning of such mixture? Nobody knows. It is not a verification of dynamic testing at restrikes and it is not assessment of real set up factor because everything is lumped together without taking into account the time between different tests. Such a comparison of the pile capacities from SLT and DT is invalid for piles driven in soils with time-dependent properties because the soil properties at the time of DT do not correspond to the soil properties at the time of SLT i.e. soil consolidation is taken into account restrikes using the DT

but is not in the SLT. A statistical approach for assessment of the time between comparable SLT and DT, is also unacceptable for piles in soils with time-dependent properties because this approach demonstrates correlation of set up factors rather than correlation of dynamic methods.

Static Loading Tests and Dynamic Testing present different ways of determining pile capacity at various times after pile installation, but for valid correlations two principal conditions have to be the same for both kind of tests. 1) static and dynamic capacities must be compared at the same time after pile installation in both SLT and DT methods, and 2) the ultimate pile capacity is obtained in the SLT only if it provides the fully mobilized pile capacity (long term capacity), similar to the DT (Svinkin,1997).

The adequacy of SLT and DT has to be confirmed by proper correlation of time. Due to the consolidation phenomenon in soils, comparison of SLT and DT can only be made for tests performed immediately one after another. In practice, it is sometimes difficult to make two immediately successive tests, but nonetheless the time difference between both comparable tests should not exceed 1 – 2 days during which soil set up changes only slightly. Closely time correlated comparisons of SLT and DT have to be made in order to clarify the reliability of pile capacity by dynamic testing in soils with time-dependent properties.

2.2.5 Points for consideration

It is imperative to consider time effects for accurate determination of pile capacity by both static and dynamic methods.

The prediction of pile capacity in pre-driving wave equation analysis can be improved by the use of variable damping as a function of time. Variable damping is the key parameter to enhance accuracy of wave equation solutions because this damping takes into consideration soil consolidation after pile installation.

The main criterion for accurate assessment of pile capacity prediction based on dynamic measurements of force and velocity at the upper end of the pile during driving is the ratio of capacities obtained by dynamic and static tests. Such a ratio, taken for arbitrary time between compared tests, is not a verification of dynamic testing results.

Dynamic testing and analysis yield the real, not predicted, static capacity of piles at the time of testing. The static capacity from a static loading test is not a unique standard for assessment of dynamic testing results. Both static loading test and dynamic testing yield the pile capacity at the time of testing.

In soils with time-dependent properties, comparison of static loading test and dynamic testing must be made only for test performed immediately, in short succession, (Svinkin, 1997).

2.3 Reliability and Stability Assessment of Concrete Gravity Structures

The reliability of a structure can be viewed as the probability of its satisfactory performance according to some performance functions under specific service and extreme conditions within a stated time period. In estimating this probability, system uncertainties are modeled using random variables with mean values, variances, and probability distribution functions. Many methods have been proposed for structural reliability assessment purposes, such as First Order Second Moment (FOSM) method, Advanced Second Moment (ASM) method, and Computer-based Monte Carlo Simulation (MCS) (e.g., Ang and Tang, 1990; Ayyub and Haldar, 1984; White and Ayyub, 1985; Ayyub and McCuen, 1997) as reported by Ayyub and Patev (1998).

Reliability-based design methods could be used to address many different aspects of foundation design and construction. However, most of these efforts to date have focused on geotechnical and structural strength requirements, such as the bearing capacity of shallow foundations, the side friction and toe-bearing capacity of deep foundations, and the stresses in deep foundations. All of these are based on the difference between load and capacity, so we can use a more specific definition of reliability as being the probability of the load being less than the capacity for the entire design life of the foundation. Various methods are available to develop reliability-based design of foundations, most notably Stochastic methods, the First-Order Second-Moment Method, and the Load and Resistance Factor Design method (Coduto, 2001).

One of the primary goals of structural engineers is to assure proper levels of safety for the structures they design. This seemingly simple task is complicated by uncertainties associated with the materials with which the structure was designed and the loads they must resist, as well as our inaccuracies in analysis and design. Structural reliability and probabilistic analysis/design are tools which can be employed to quantify these uncertainties and inaccuracies, and produce designs and design procedures meeting acceptable levels of safety. Recent researches in the area of structural reliability and probabilistic analysis have centered around the development of probability-based design procedures. These include load modeling, ultimate and service load performance and evaluation of current levels of safety/reliability in design (e.g., Farid Uddin, 2000; Afolayan, 1999; Afolayan and Abubakar, 2003; Afolayan, 2003; Afolayan and Opeyemi, 2008a, 2008b). The development, implementation, and use of computational methods for the analysis of structures and structural components represent the focus area within the Structural Engineering and Materials.

The society requires increasingly that the hazard and risk associated with engineered constructions be quantified. Geotechnical hazard assessment in the context of a risk framework, using concepts of uncertainties, reliability, safety and risk shows that the use of this approach is exemplified for offshore facilities, including piled foundations, jack-up structures, gravity foundations and under water slopes. The application demonstrates that probabilistic analyses complement the conventional deterministic safety factor and deformation-based analyses, and contribute to achieving safe and optimum design. The probabilistic approach adds value to the results with a modest additional effort. The conclusions emphasize the usefulness of risk assessment, the importance of engineering judgment in the assessment and the need for involving multi-disciplinary competences to achieve reliable estimate of hazard and risk (Lacasse and Nadim, 2007).

Patev (1995) made a reliability assessment of timber pile-founded navigation structures without the loss of support. The techniques developed to perform the reliability assessment employed a capacity/demand relationship for limit states of axial capacity, lateral deflections, axial deflections, and combined bending. Monte Carlo Simulation and First Order Second Moment were utilized to calibrate the limit states to an instrumented pile load test.

The design of a geotechnical work should satisfy both the ultimate limit state and serviceability limit state requirements. The limit state design approach addresses various performance requirements and aims to accommodate uncertainties. At present, most limit-state design codes use reliability principles to design for ultimate limit states. However, serviceability limit states are still considered in some design codes using the conventional deterministic approach. To develop reliability-based serviceability criteria for

more consistent design, the probability distributions of the limiting tolerable displacements should be studied.

In a recent study (Zhang and Ng, 2007), the variability of tolerable displacements of structures was examined and a procedure for establishing probability distributions of the limiting tolerable displacements for steel bridges, concrete bridges, buildings on shallow and deep foundations was proposed.

The well-known dynamic formulas have been criticized in many publications. Unsatisfactory prediction in pile capacity by dynamic formulas is well characterized in the recent published Manual for Design and Construction of Driven Pile Foundations (Hannigan et al., 1996), in which it was concluded: "Whether simple or more comprehensive dynamic formulas are used, pile capacities determined from dynamic formulas have shown poor correlations and wide scatter when statistically compared with static load test result. Therefore, except where well supported empirical correlations under a given set of physical and geological conditions are available, dynamic formulas should not be used."

There are two attempts to breathe new life into dynamic formulas. First, Paikowsky and Chernauskas (1992) and Paikowsky et al. (1994) have suggested one more simplified energy approach using dynamic measurements for the capacity evaluation of driven piles. Liang and Zhou (1997) have concluded regarding this method: "Although the delivered energy is much more exactly evaluated, this method still suffers similar drawbacks of Engineering News (ENR)". In a second, criticizing the simplified energy approach, Liang and Zhou (1997) have developed a probabilistic energy approach as an alternative to the signal matching technique for effective pile-driving control in the field. Both attempts to improve dynamic formulas, comparison of pile capacity determined by the simplified and

probabilistic energy methods with results of Static Loading Tests (SLT), are incorrect. Dynamic formulas, including their two new representations, using maximum energy, pile set and maximum displacement from Dynamic Pile Testing (DT) do not take into account the time between SLT and DT.

PILE FOUNDATION ANALYSIS- STATIC AND DYNAMIC METHODS.**3.1 Introduction**

Piles are structural members of timber, concrete, and/or steel, used to transmit surface loads to lower levels in the soil mass. This may be by vertical distribution of the load along the pile shaft or a direct application of load to a lower stratum through the pile point. A vertical distribution of the load is made using a friction, or "floating", pile and a direct load application is made by a point, or "end-bearing", pile. This distinction of piles is purely one of convenience since all piles function as a combination of side resistance and point bearing except when the pile penetrates an extremely soft soil to a solid base

Piles are commonly used:

- to carry the superstructure loads into or through a soil stratum. Both vertical and lateral loads may be involved.
- to resist uplift, or overturning, forces as for basement mats below the water table or to support tower legs subjected to overturning.
- to compact loose, cohesionless deposits through a combination of pile volume displacement and driving vibrations. These piles may be later pulled.
- to control settlements when spread footings or a mat is on a marginal soil or is underlain by a highly compressible stratum.
- to stiffen the soil beneath machine foundations to control both amplitudes of vibration and the natural frequency of the system.
- as an additional safety factor beneath bridge abutments and/or piers, particularly if scour is a potential problem.

- in offshore construction to transmit loads above the water surface through the water and into the underlying soil. This is a case of partially embedded piling subjected to vertical (and buckling) as well as lateral loads.

Piles are sometimes used to control earth movements (as landslides). The power poles and many outdoor sign poles may be considered as partially embedded piles subject to lateral loads. Vertical loads may not be significant, although buckling may require investigation for very tall members.

A pile foundation is much more expensive than spread footings and likely to be more expensive than a mat. In any case great care should be exercised in determining the soil properties at the site for the depth of possible interest so that it can be accurately determined that a pile foundation is needed and, if so, that neither an excessive number nor lengths are specified. A cost analysis should be made to determine whether a mat or piles, in particular the type (steel, concrete, etc), are more economical. In those cases where piles are used to control the settlement at marginal soil sites, care should be taken to utilize both the existing ground and the piles in parallel so that a minimum number are required.

Piles are inserted into the soil via a number of methods:

- Driving with a steady succession of blows on the top of the pile using a pile hammer. This produces both considerable noise and local vibrations which may be disallowed by local codes or environmental agencies and, of course, may damage adjacent property.
- Driving using a vibratory device attached to the top of the pile. This is usually a relative quiet method and driving vibrations may not be excessive. The method is more applicable in deposits with little cohesion.

- Jacking the pile. This is more applicable for short stiff members.
- Drilling a hole and either inserting a pile into it or, more common, filling the cavity with concrete which produces a pile upon hardening.

When a pile foundation is decided upon, it is necessary to compute the required pile cross section and length based on the load from the superstructure, allowable stress in the pile material (usually a code value), and the in situ soil properties. This is necessary so that the required number and lengths of piles can be ordered by the foundation contractor. Dynamic formulae, pile-load tests, or a combination are used on site to determine if the piles are adequately designed and placed. It is generally accepted that a load test is the most reliable means of determining the actual pile capacity.

3.2 Pile Materials (Steel & Precast Concrete)

In this research work, only steel and precast concrete piles were analysed based on static pile capacity equations under cohesive, cohesionless, and intermediate soils between sand and clay and layered soils; as were thus considered as piling materials in this write-up.

3.2.1 Precast concrete piles

Precast Concrete Piles are formed in a central casting yard to the specified length, cured, and then transported to the construction site. If space is available and a sufficient quantity of piles needed, a casting yard may be provided at the site to reduce transportation costs. Precast piles may be made using ordinary reinforcement or prestressed. Precast piles using ordinary reinforcement are designed for bending stresses during pick up and transportation to the site, for bending moments from lateral loads, and to

provide sufficient resistance to vertical loads and any tension forces developed during driving,. Handling and driving (tensile) stresses are temporary and allowable concrete stresses are often increased as much as 50% to reflect this. The minimum pile reinforcement should be 1%.

3.2.2 Steel Piles

These members are usually rolled H shapes or pipe piles. Wide-flange beams or I beams may also be used; however, the H shape is especially proportioned to withstand the hard driving stress which the pile may be subjected. In the H pile the flanges and web are of equal thickness; the standard WF and I shapes usually have a thinner web than flange. Pipe piles are either welded or seamless steel pipes which may be driven either open-end or close-end. Pipe piles are often filled with concrete after driving, although in some cases this is not necessary.

The H pile is a small volume displacement pile since the cross sectional area is not very large. A plug tends to form between the flanges at greater depths, however, so the bottom several meters may remold the soil on the order of the volume of the plug. An open-end pipe is also considered a small volume displacement pile, however, a plug also forms inside with a depth of one or more meters below the outside ground level – probably from a combination of inside perimeter friction and driving vibrations. From the depth at which the “plug” stabilizes (not visible during driving because of the pile cap and hammer interference) to the final driving depth the lower soil may be remolded based on the volume of the plug and not the pipe area.

H piles have an advantage of sufficient rigidity that they will either break smaller boulders or displace them to one side. Open-end pipe piles have the

advantage of surface entry to break up boulders encountered by either use of a chopping bit or drilling, blasting, and removal of the rock fragments.

3.3 Static Pile Capacity

All static pile capacities can be computed by the following equations:

$$P_u = P_{pu} + \sum P_{si} \text{ (compression) } \dots \dots \dots (3.1a)$$

and

$$T_u = \sum P_{si} + W \text{ (tension) } \dots \dots \dots (3.1b)$$

where P_u = ultimate (maximum) pile capacity in compression – usually defined as that load producing a large increase (or rate) of penetration

T_u = ultimate pullout capacity

P_{pu} = ultimate point capacity; may be neglected for “floating” piles

$\sum P_{si}$ = skin (or shaft friction) resistance contribution from several strata penetrated (see Fig. 3.2a); neglect for “end bearing” piles

W = weight of pile

The allowable pile capacity P_a or T_a is obtained from applying a suitable Factor of Safety (SF) on the parts as

$$P_a = \frac{P_{pu}}{SF_p} + \frac{\sum P_{si}}{SF_s} \dots \dots \dots (3.2)$$

or using a single value SF (most common) to obtain

B = base of pile (usually used only when point is enlarged),

N'_c = bearing capacity factor for cohesion adjusted for shape and depth. When $\phi = 0$, we have $c = S_u$ and N'_c is often taken as 9,

N'_q = bearing capacity factor for overburden effects, $q = \gamma L$, and includes shape and depth effects,

N'_γ = bearing capacity factor for base width = N_γ since it is not affected by depth effects,

q = effective vertical stress (overburden pressure) at pile point,

$\eta = 1$ for all except the Vesic(1975) N factors where $\eta = \frac{1+2K_0}{3}$ and K_0

is equal to at rest earth pressure coefficient.

These bearing capacity factors are based on the initial in situ soil parameters and not the later revised soil parameter from driving effects (which would be unknown at the initial design stage anyway). Neglecting the N_q term and making adjustment for pile weight, Equation (3.4) is rewritten as follows:

$$P_{pu} = A_p \{cN'_c + q(N'_q - 1)\}$$

This equation gives for $c = S_u$ and $\phi = 0$ the values $N'_q - 1$ and the ultimate pile point capacity is

$$P_{pu} = A_p (9S_u) \quad (3.4b)$$

Most designers use N_q and not $N_q - 1$ term for $\phi > 0$ since the factor reduced by 1 is too much refinement. In design the ultimate point capacity is, divided by a suitable $SF = 1.5$ to 3.

3.3.2 Skin resistance capacity

The skin resistance part of Eq. (3.1) is currently computed using either a combination of total and effective or only effective stresses. Some evidence

exists that use of only effective stresses gives a better correlation of prediction to load tests; however, both methods are widely used. Preference will depend on the data base of successful usage in a given local/design office.

There are three procedures currently used with the two general methods for computing the skin resistance of piles in cohesive soils. These are called the α -, λ -, and β - methods for the factors used in the skin resistance capacity part of Eqn. (3.1). The method is also used for piles in cohesionless soils. In all cases the skin resistance capacity is computed as

$$P_s = \sum A_s f_s \quad (3.5)$$

where A_s = effective pile surface area on which f_s acts and commonly computed as perimeter x embedment increment L . Refer to Fig. (3.2a) for perimeters for several pile types

ΔL = increment of embedment length (to allow for pile shaft variations and soil stratification)

f_s = skin resistance to be computed using one of the following methods

$\sum$ = summation of contributions from several strata or pile segments.

Note that the following equations for f_s are in general terms so that successful use will depend on how accurately the summation process is made and the soil parameters are identified.

3.3.2.1 The α - method

This method was proposed by Tomlinson (1971), and basically the skin resistance is computed as

$$f_s = \alpha c + qK \tan \delta \quad (3.6)$$

where α = coefficient from Table 3.1,

c = average cohesion (s_u) for the soil stratum of interest

q = effective vertical stress on element ΔL (Fig. 3.2a)

K = coefficient of lateral earth pressure ranging from K_0 to about 1.75, depending on volume displacement, initial soil density, etc. Values close to K_0 are generally recommended because of long-term soil creep effects even where the pile has a large volume.

δ = effective friction angle between soil and pile material (use either ϕ (drained angle) or values from Table (3.3); use $\delta = 0$ when $\phi = 0$)

K_0 for piles is most commonly computed as

$$K_0 = (1 - \sin \phi') \sqrt{OCR}$$

The correlation reported by Tomlinson for the α - method is on the order of $\pm 25\%$.

TABLE 3.1 - Values of adhesion factors for piles driven into stiff to very stiff cohesive soils for design *

Case	Soil conditions	Penetration ratio	Adhesion factor
1.	Sands or sandy gravels overlaying stiff to very stiff cohesive soil	< 20 > 20	1.25 Fig. 3.2a
2.	Soft clays or silts overlaying stiff to very stiff cohesive soil	$8 < PR \leq 20$ >20	0.40 Fig. 3.2a
3.	Stiff to very stiff cohesive soils without overlaying strata	$8 < PR \leq 20$ >20	0.40 Fig. 3.2a

Source: Bowles, J.E. (1988)

* After Tomlinson (1971).

Penetration ratio $PR = \text{Depth of penetration into cohesive soil} / \text{Diameter of pipe}$

TABLE 3.2 Friction angles δ between various foundation materials and soil or rock

Interface materials	Friction angle δ degrees
Mass concrete or masonry on the following:	
Clean sound rock	35
Clean gravel, gravel-sand mixtures, coarse sand, silty or clayey	29-31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24-29
Clean fine sand, silty or clayey fine to medium sand	19-24
Fine sandy silt, nonplastic silt	17-19
Very stiff and hard residual or preconsolidated clay	22-26
Medium stiff and stiff clay and silty clay	17-19
Steel sheet piles against:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22
Clean sand, silty sand-gravel mixture, single-size hard-rock fill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheetpiling against:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22-26
Clean sand, silty sand-gravel mixtures, single size hard rock fill	17-22

Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14
Various structural materials:	
Masonry on masonry, igneous and metamorphic rocks:	
Dressed soft rock on dressed soft rock	35
Dressed hard rock on dressed soft rock	33
Dressed hard rock on dressed hard rock	29
Masonry on wood (cross grain)	26
Steel on steel at sheet-pile interlocks	17
Wood on soil	14-16

Source: Bowles, J.E. (1988)

3.3.2.2 The λ - method

Vijayvergiya and Focht (1972) presented a method of obtaining the skin resistance f_s of a pile in clay (second term of Eq. (3.6) neglected as common) as

$$f_s = \lambda (q + 2Su) \quad (3.7)$$

where q, Su = values defined in Eq. (3.6)

λ = coefficient which can be obtained from Fig. 3.3

The λ - coefficient was obtained from a graphical regression (best-fit) analysis of a plot with a large number of pile-load tests. The shorter piles were mostly in a stiffer clay or a clay with a stiff upper crust ($OCR > 1$) which may account for the larger values for short piles. Where long piles penetrated into soft clay the values reflect both averaging for a single value and development of a limiting skin resistance since q does not increase pile capacity without bound. The correlation was reported for this method to be about ± 10 percent.

3.3.2.3 The β - Method

This method was suggested by Burland (1973), and makes the following assumptions:

1. Soil remolding adjacent to the pile during driving reduces the effective stress cohesion intercept on a Mohr's circle to zero.
2. The effective stress acting on the pile surface after dissipation of excess pore pressures generated by volume displacement is at least equal to the horizontal effective stress (K_0) prior to pile installation.

3. The major shear distortion during pile loading is confined to a relatively thin zone around the pile shaft and drainage of this thin zone occurs rapidly during loading – or has already occurred in the delay between driving and loading.

With these assumptions Burland (1973) developed a simple design equation written as

$$f_s = Kq \tan \delta \quad (3.8a)$$

Taking $\beta = K \tan \delta$ we can rewrite the equation for skin resistance as

$$f_s = \beta q \quad (3.8b)$$

Since q = effective overburden pressures at z_i , modify for a surcharge q_s to

$$f_s = \beta(q + q_s) \quad (3.8c)$$

As previously used, q = average effective vertical stress for the i th element of length ΔL . The friction coefficient $f = \tan \delta$ so an estimate for the soil-to-pile friction angle must be made. One may use Table 3.3 to obtain an estimate in the absence of any laboratory measured values.

The lateral earth pressure coefficient K may be designer selected; however, a value of $K = K_0$ as defined in Eq. (3.6), where ϕ = effective stress angle internal friction is commonly used.

A particularly attractive feature of the β - method is that if we use K_0 and $\delta = \phi'$, the product $K_0 \tan \phi' = \beta$ ranges from about 0.25 to 0.40 in the practical range of ϕ' with an average value around 0.32. This means that we can make a reasonable estimate for ϕ' and obtain a computed skin resistance

that does not vary widely (however it still remains to be seen whether or not it is correct from a load test).

Figure (3.4) is a data plot from Flaate and Selnes (1977) which was obtained from back-computing a number of reported load tests using this method. While there is substantial scatter it does not seem as great as in using other methods including both the α - and λ -method according to Esrig and Kirby (1979).

Most authorities agree that f_s in Eq. (3.8) do not increase indefinitely with depth but rather, beyond some critical depth L/B it increases at an ever decreasing rate. Bhushan (1982) suggests for large displacement piles (closed end pipe, solid concrete, and possibly open end pipe with a plug) that a reasonable estimate for K and β be as follows:

$$\beta = K \tan \delta = 0.18 + 0.0065D_r \quad (3.9)$$

$$K = 0.50 + 0.008 D_r \quad (3.10)$$

where D_r is the relative density (as a percent). We can use SPT correlations to obtain D_r with depth.

Zeitlen and Paikowsky (1982) suggest that the "limiting f_s " can be accounted for by the decrease in ϕ with effective normal confining pressure as illustrated in Fig. (3.5) and using the following equation.

$$\phi = \phi_0 - 5.5 \log(q_{en}/q_0) \quad (3.11)$$

where ϕ is the angle of internal friction at the effective normal pressure q_{en} at depth of interest (point or along the shaft) and computed as $\eta q = \eta \gamma z$ with η defined in Eq. (3.5). Angle ϕ_0 is the reference angle obtained at an effective normal pressure q_0

We must also make a decision on what to use for $\tan \delta$, some persons suggest a maximum for δ on the order of 0.5 to 0.75 ϕ' while others routinely use ϕ' . It has already been pointed out that this depend on the normal pressure at the interface of soil and pile.

Finally, the lateral earth pressure coefficient is questioned. While we have given an equation for K in terms of relative density there are others who use somewhat arbitrary choices based on load tests performed or reported in the literature. For example, Mansur and Hunter (1970) in an extensive pile test program found values as follows:

H pile	1.4. to 1.9
Pipe piles	1.2 to 1.3
Precast square concrete piles	1.45 to 1.6
Timber (1 test)	1.25
Tension tests (8 all types)	0.4 to 0.9

Tavenas (1971) tested a series of piles and found that K was around 0.5 for H piles, 0.7 for precast concrete, and 1.25 for tapered timber. His pulling tests indicated a K on the order of 0.6. Ireland (1957) performed a series of tension

tests on step-taper piles and found K ranging from 1.11 to 3.64 (he did not accept 3.64 as correct). The API (1984) recommends $K = 0.8$ for unplugged pipe piles in either tension or compression and 1.0 for full displacement piles. From this one can readily see that there is not very good agreement on what to use for K . It is suspected that pile weight was not included in some of the pullout tests and little to no consideration was made for stratification or changes in the soil parameters with depth or effective normal confining pressure. We might note, too, that a significant variation in K can be obtained by the assumption of how load distribution is made between shaft and point resistance.

3.3.2.4 Calculation of the carrying capacity of Piles in Soils intermediate between sand and clay and layered soils

There are no available data on the skin friction or adhesion of the pile shaft in C-Ø soils. However, it may be taken as the sum of the adhesion and skin friction based on experience and using as a guide the values given in Fig. (3.6), the skin friction calculated. In the case of bored piles, due allowances should be made for likely softening or loosening effect caused by the boring operations.

3.4 Dynamic Analysis

Estimating the ultimate capacity of a pile while it is being driven in the ground at the site has resulted in numerous equations being presented to the engineering profession. Unfortunately, none of the equations is consistently reliable or reliable over an extended range of pile capacity. Because of this, the best means for predicting pile capacity by dynamic means consists in driving a pile, recording the driving history, and load testing the pile. It would be reasonable to assume that other piles with a similar driving history at that site would develop approximately the same load capacity.

Dynamic formulae have been widely used to predict pile capacity. Some means is needed in the field to determine when a pile has reached a satisfactory bearing value other than by simply driving it to some predetermined depth. Driving the pile to a predetermined depth may or may not obtain the required bearing value because of normal soil variations both laterally and vertically.

The basic dynamic pile-capacity formula termed the *rational pile formula* was derived from Fig. 3.7 and the following list of symbols. Newly all the dynamic pile formulae currently used is based on this equation – generally by simplifying certain terms. The rational pile formula depends upon impulse – momentum principles.



A	=	Pile cross-sectional area
E	=	Modulus of Elasticity
e_h	=	Hammer efficiency
E_h	=	Manufacturers' hammer-energy rating
h	=	height of all of ram
K_1	=	elastic compression of capblock and pile cap and is a form of $P_u L / AE$
K_2	=	elastic compression of pile and is of a form of $P_u L / AE$
K_3	=	elastic compression of soil, also termed quake for wave-equation.
L	=	Pile length
n	=	Co-efficient of restitution
P_u	=	Ultimate pile capacity

s = amount of point penetration per blow.

W_p = Weight of pile including weight of pile, cap, driving shoe, and capblock (also includes anvil for double-acting steam hammers)

W_r = Weight of ram (for double-acting hammers include weight of casing).

The available dynamic pile formulae include:

(a) Canadian National Building Code (use SF = 3)

$$P_u = \frac{e_h E_h C_1}{s + C_2 C_3} \quad (3.12)$$

in which

$$C_1 = \frac{W_r + n^2 (0.5W_p)}{W_r + W_p}$$

$$C_2 = \frac{3P_u}{2A}, \text{ and}$$

$$C_3 = \frac{L}{E} + 0.0001$$

(b) Danish formula (use SF = 6)

$$P_u = \frac{e_h E_h}{s + C_1} \quad (3.13)$$

where

$$C_1 = \sqrt{\frac{e_h E_h L}{2AE}}$$

(c) Gates formula (use SF = 3)

$$P_v = a\sqrt{e_b} E_h (b - \log s) \quad (3.14)$$

(d) Janbu (use SF = 3 to 6)

$$P_v = \frac{e_b E_h}{k_a s} \quad (3.15)$$

where

$$k_a = C_d \left(1 + \sqrt{1 + \frac{\lambda}{C_d}}\right)$$

and

$$C_d = 0.75 + 0.15 \frac{W_p}{W_r}$$

$$\lambda = \frac{e_b E_h L}{A E s^2}$$

(e) Modified ENR formula (use SF = 6)

$$P_v = \frac{1.25 e_b E_h}{s + 0.1} \frac{W_r + n^2 W_p}{W_r + W_p} \quad (3.16)$$

(f) AASHTO (use SF = 6); primarily for timber piles.

$$P_v = \frac{2h(W_r + A_r p)}{s + 0.1} \quad (3.17)$$

(g) Navy-McKay (use SF = 6)

$$P_u = \frac{e_h E_h}{s(1 + 0.3C_1)} \quad (3.18)$$

where

$$C_1 = \frac{W_p}{W_r}$$

(h) Pacific Coast Uniform Building Code (PCUBC) (use SF = 4)

$$P_u = \frac{e_h E_h C_1}{s + C_2} \quad (3.19)$$

$$C_1 = \frac{W_r + kW_p}{W_r + W_p}$$

$k = 0.25$ for steel piles

$= 0.10$ for all others

$$C_2 = \frac{P_u L}{AE}$$

(i) Hiley

$$P_u = \frac{e_h E_h}{s + \frac{1}{2}(k_1 + k_2 + k_3)} \frac{W + n^2 W_p}{W + W_p} \quad (3.20)$$

Table 3.3 – Values for K_1 – temporary elastic compression of pile head and cap.

For driving stresses larger than 14 Mpa use K_1 in last column.

	Driving stresses P/A on pile head or cap, MPa (Ksi)			
	3.5(0.5)	7.0(1.0)	10.5(1.5)	14(2.0)
Pile material	K_1 , mm (in)			
Steel piling or pipe				
Directly on head	0	0	0	0
Directly on head of timber pile	1.0(0.05)	2.0(0.10)	3.0(0.15)	5.0(0.20)
Precast concrete pile with				
75–100mm packing inside cap	3.0(0.12)	6.0(0.25)	9.0(0.37)	12.5(0.50)
Steel-covered cap containing wood				
Packing for steel H pile piling	1.0(0.04)	2.0(0.05)	3.0(0.12)	4.0(0.16)
5-mm fiber disk between two				
10-mm steel plates	0.5(0.02)	1.0(0.04)	1.5(0.06)	2.0(0.08)

† After Chellis (1961)

Source: Bowles, J.E. (1988)

Table 3.4 – Hammer Efficiency

Type	Efficiency e_h
Drop hammers	0.75 – 1.00
Single acting hammers	0.75 – 0.85
Double-acting or differential	0.85
Diesel Hammers	0.85 – 1.00

Source: Bowles, J.E. (1988)

The term K_2 is computed as $P_u L / AE$, and one may arbitrarily take the K_3 term (quake) as

$K_3 = 0.0$ for hard soil (rock, very dense sand, and gravels)

$= 0.1$ to 0.2 in or 2.5 to 5mm for other soils.

Table 3.5 – Representative values of coefficient of restitution for use in the dynamic pile driving equations

Material	n
Broomed wood	0
Wood piles (nondeteriorated end)	0.25
Compact Wood cushion on steel pile	0.32
Compact wood cushion over steel pile	0.40
Steel-on-steel anvil on either steel or concrete pile	0.50
Cast-iron hammer on concrete pile without cap	0.40

Source: Bowles, J.E. (1988)

† After ASCE (1941)

3.5 Allowable Design Loads

(a). Steel Pipes

The allowable design load for a steel pile based on material is

$$P_a = A_p f_s \quad (3.21)$$

Where A_p = cross-sectional area of pile at cap

f_s = allowable steel stress (code or specification)

$$f_s = 0.35 - 0.5 f_y$$

(b). **Concrete Piles**

Concrete piles may be precast, prestressed, cast-in place, or of composite construction.

The allowable design load for all concrete (not prestressed) is

$$P_a = A_c f_c + A_s f_s \quad (3.22)$$

Where A_c, A_s = area of concrete and steel respectively

f_c, f_s = allowable material stresses

$$\begin{aligned} P_a &= A_c f_c + A_s f_s \\ &= 0.33 f_{cu} A_c + 0.4 f_y \pi D^2 / 4 \end{aligned} \quad (3.23)$$

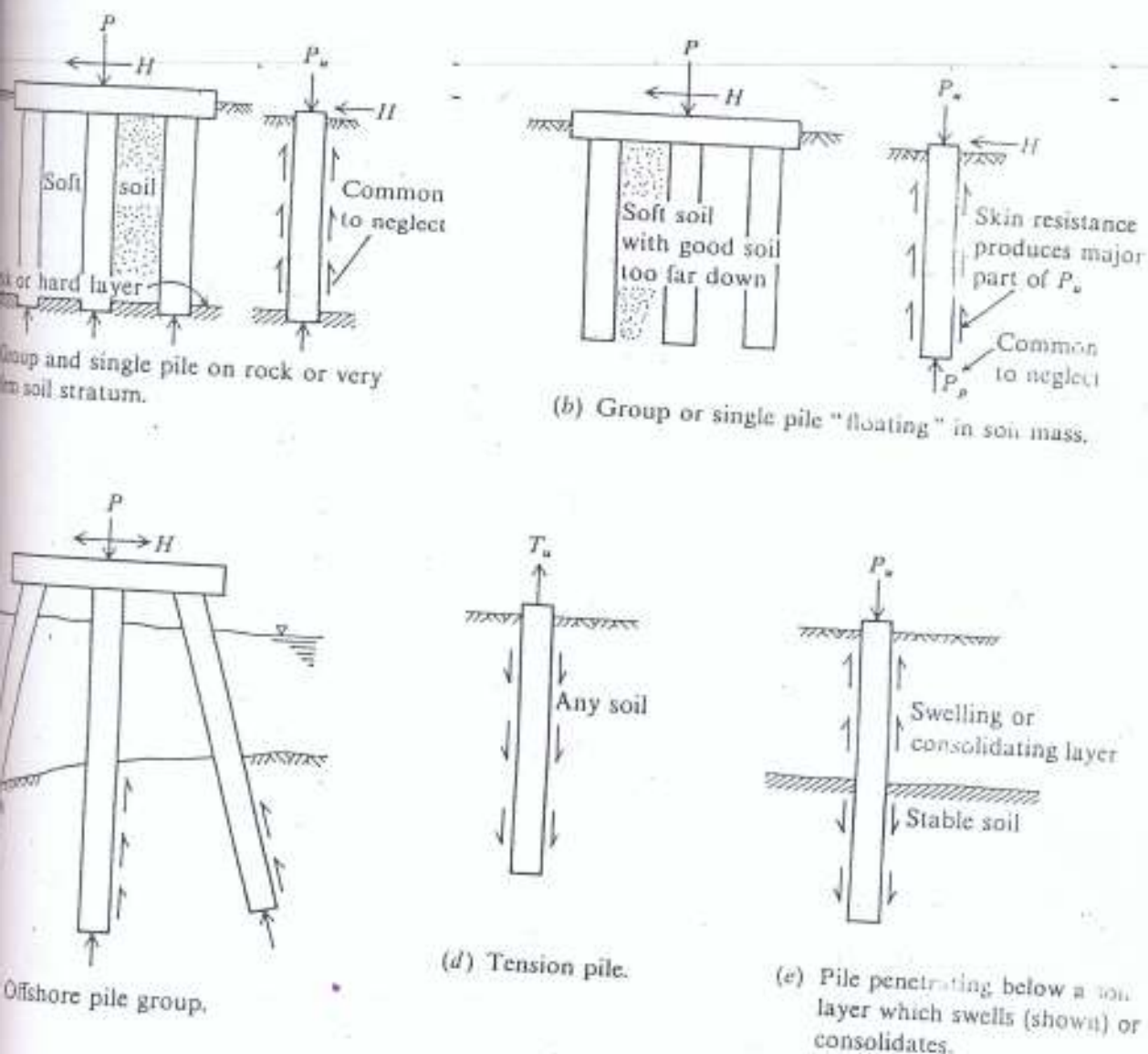


Fig. 3.1 – Typical Pile Configurations (Bowles, 1988)

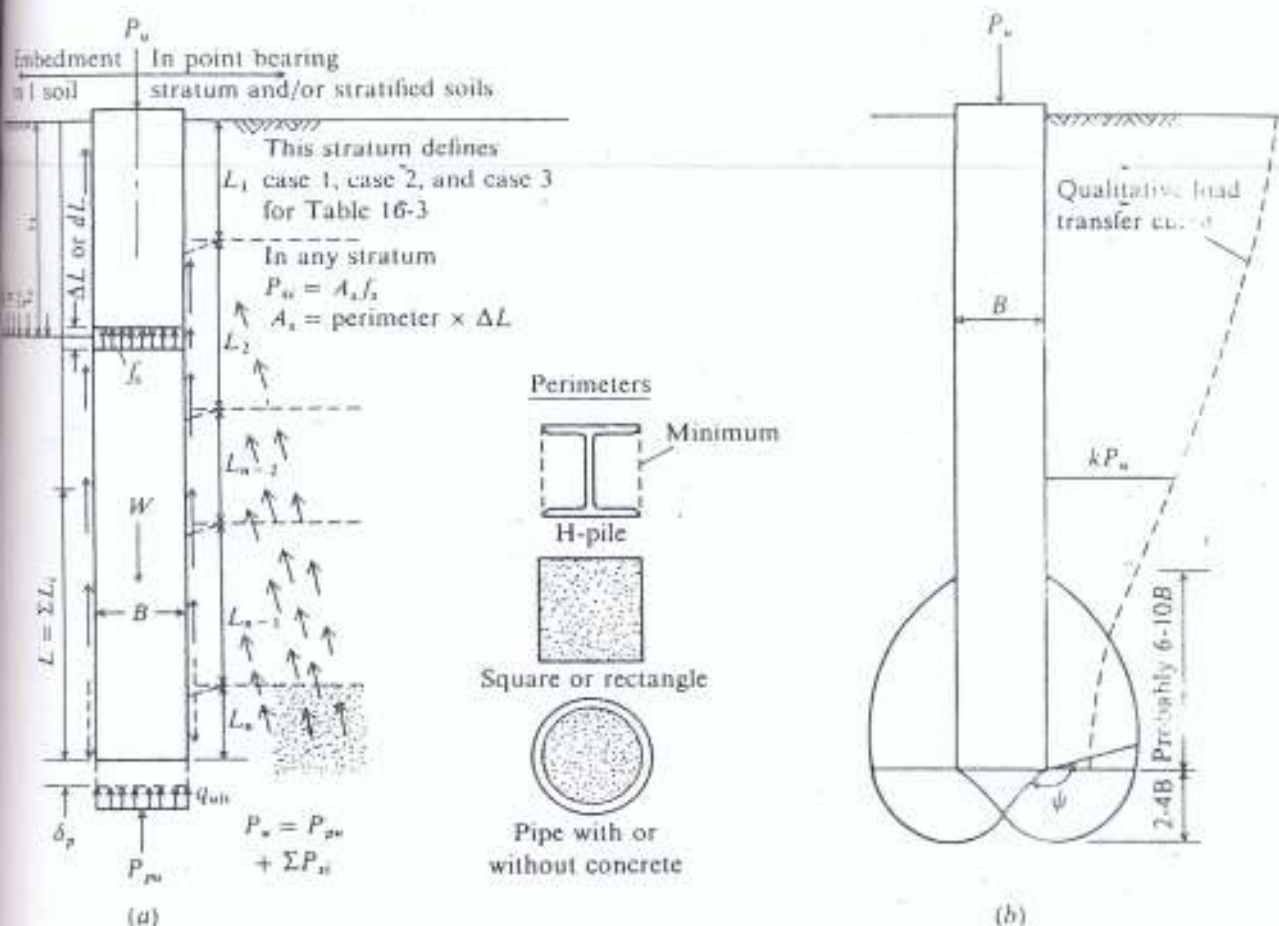


Fig. 3.2 – (a) General development of pile capacity in either single or stratified soil mass; also pile perimeter defined. (b) Qualitative zone of interest for ultimate point capacity (Bowles, 1988)

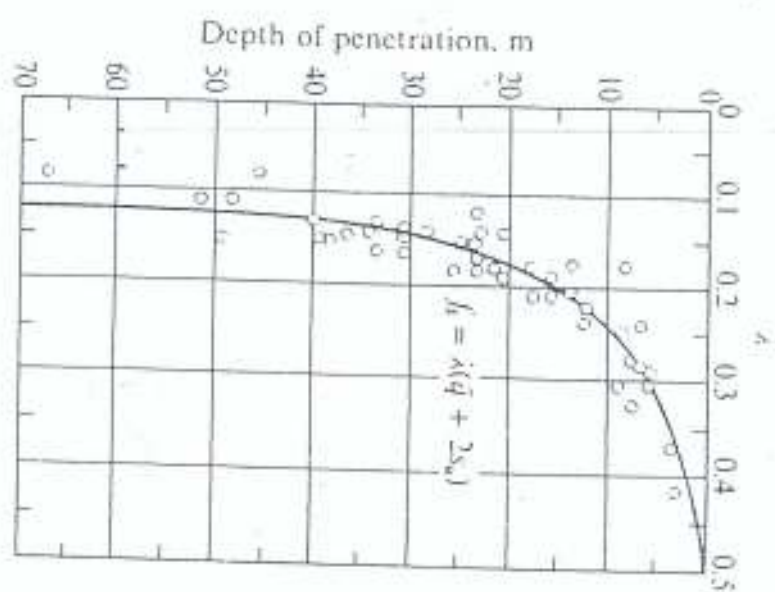


Fig. 3.3 – λ Coefficients depending on pile penetration. Data replotted and depths converted to meters by author from Vijayvergia and Focht (1972) (Bowles, 1988)

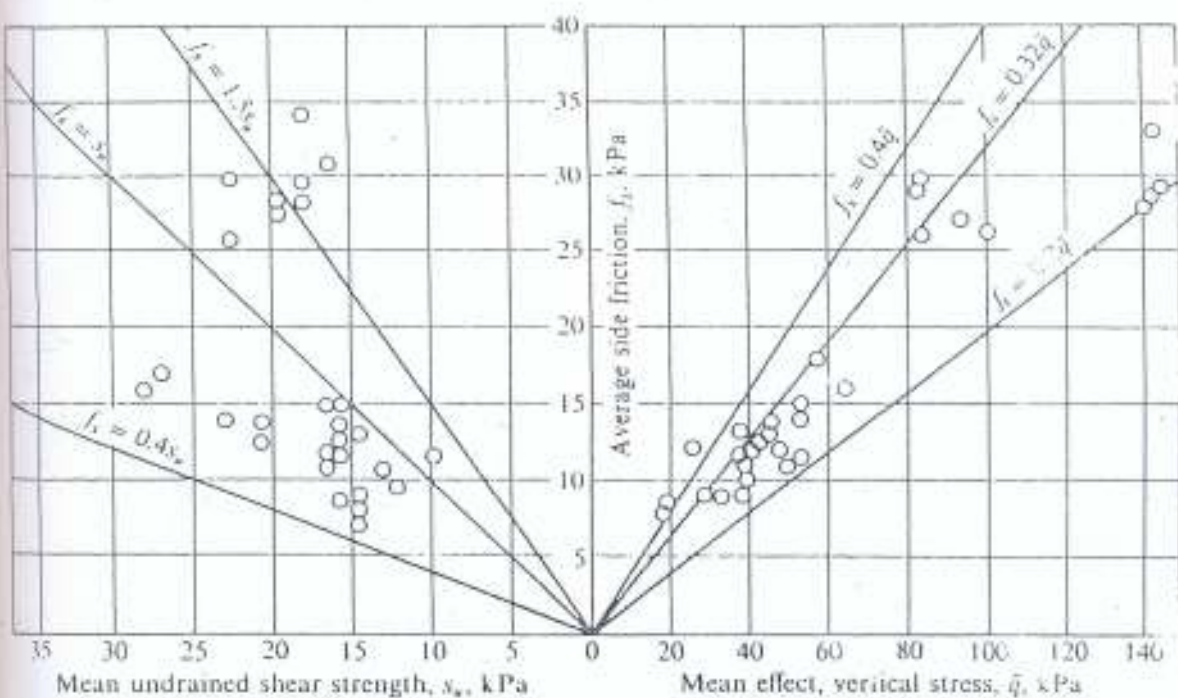


Fig. 3.4 – Plot of s_u and $\bar{q}$ versus average skin resistance f_s to illustrate scatter. Plot of f_s versus $\bar{q}$ uses the factor. After Flaate and Selnes (1977). (Bowles, 1988)



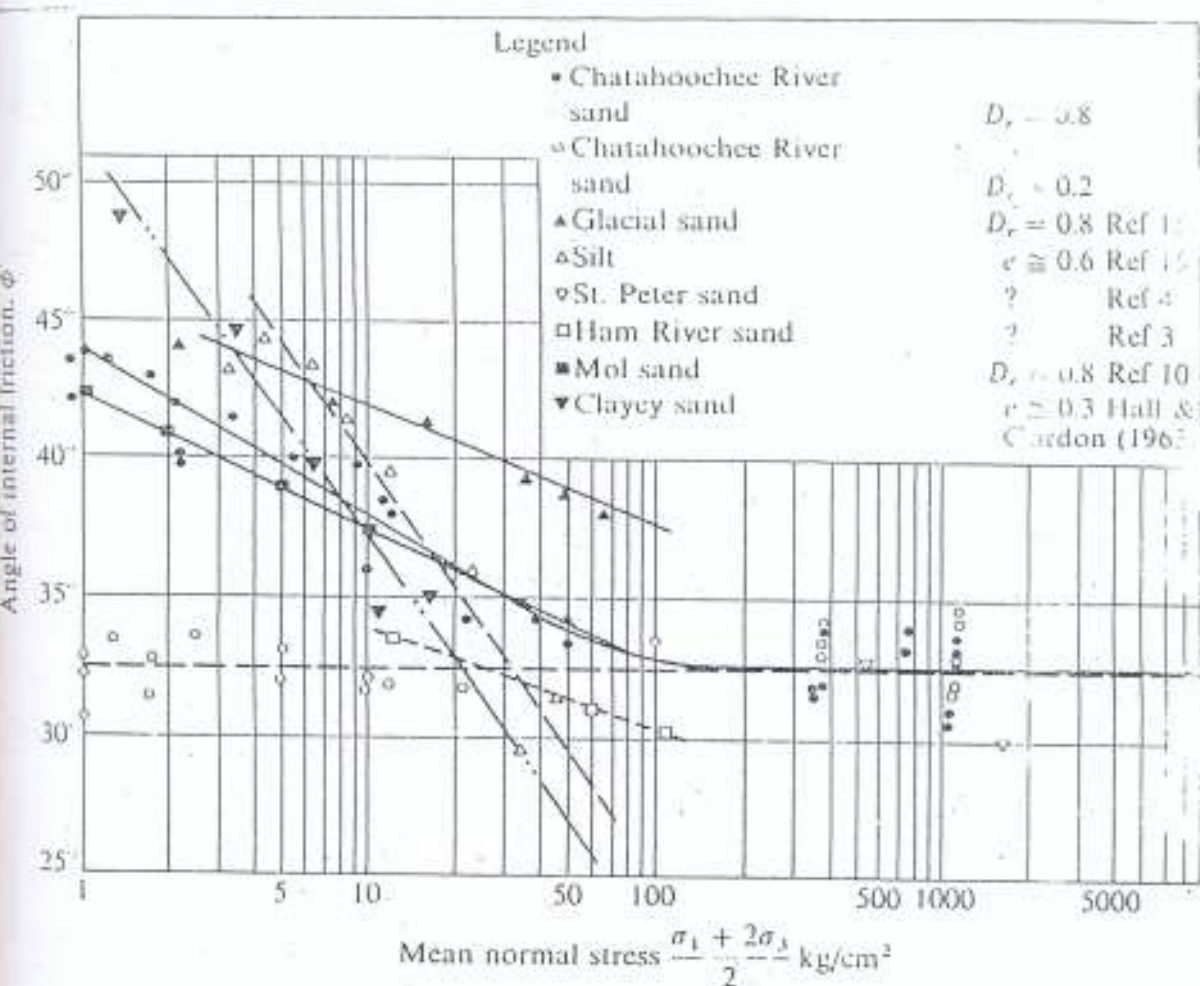


Fig. 3.5 – Reduction in angle of internal friction with increase in mean normal confining stress. (Bowles, 1988)

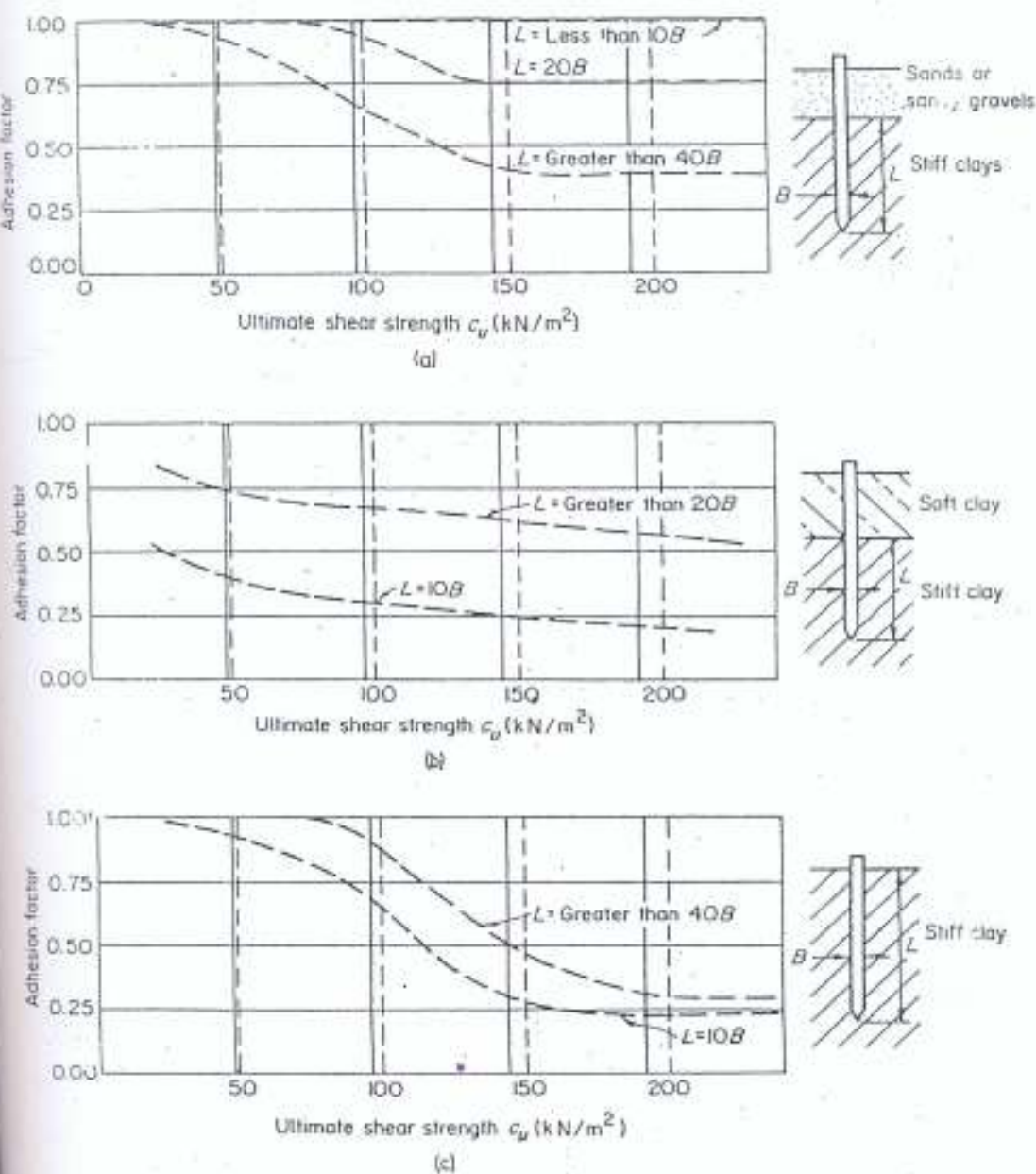


Fig. 3.6 – Adhesion factors for driven piles in clay. (a) Case 1: piles driven through overlying sands or sandy gravels. (b) Case 2: piles driven through overlying weak clay. (c) Case 3: piles without different overlying strata. (Bowles, 1988)

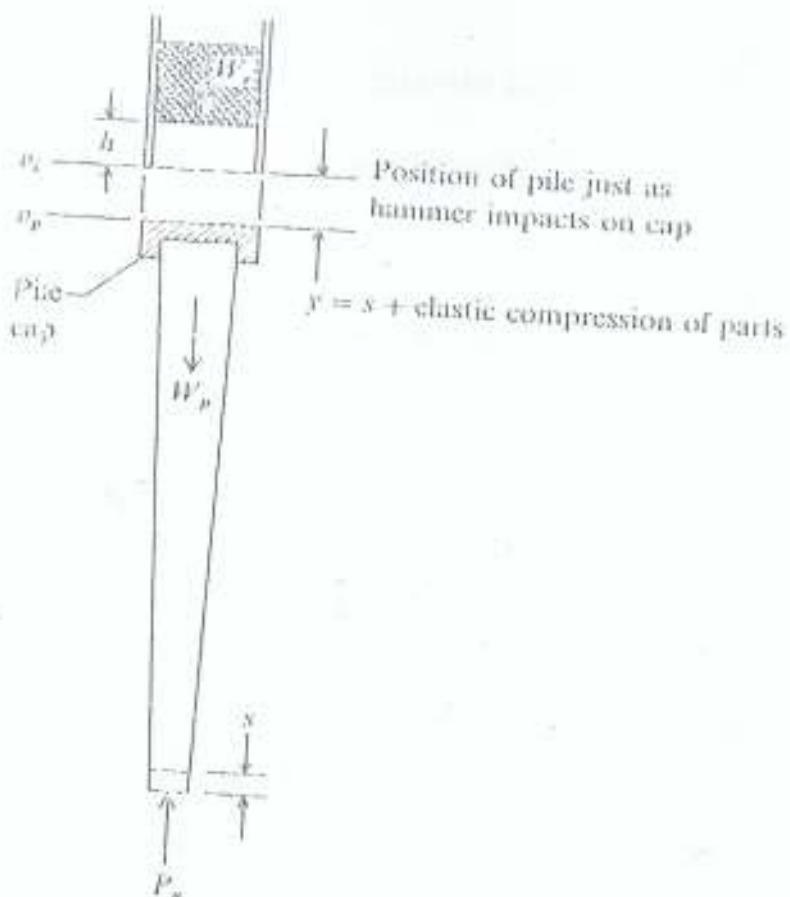


Fig. 3.7 – Terms used in the dynamic pile-driving equations. (Bowles, 1988)



4.1 Conception of Reliability Analysis

4.1.1 Introduction

The manner in which an engineering structure will respond to loading depends on the type and magnitude of the applied load and the structural strength and stiffness. Whether the response is considered satisfactory or not depends on the requirements which must be satisfied. These include safety of the structure against collapse, limitations on damage, or on deflections or other criteria. Each of such requirement may be termed a limit state. The "violation" of a limit state can then be defined as the attainment of an undesirable condition for the structure.

From observation it is known that very few structures collapse, or require major repairs, etc, so that the violation of the most serious limit states is a relatively rare occurrence. When violation of a limit state does occur, the consequences may be extreme, as exemplified by the spectacular collapses of structure such as the Tay Bridge (wind loading), Roman Point Flats (gas explosion), Kielland offshore platform (local strength problems), Kobe earthquake (ductility), etc.

Structural reliability is concerned with the calculation and prediction of the probability of limit state violation for an engineered structural system at any stage during its life. In particular, the study of structural safety is concerned with violation of the ultimate or safety limit states for the structure. In probabilistic assessments any uncertainty about a variable expressed in terms of its probability density function is taken into account explicitly. This is not the case in traditional ways of measuring safety, such as the "factor of

safety" or "load factor". These are "deterministic" measures since the variables describing the structure: its strength and the applied loads are assumed to take on known (if conservative) values about which they assumed to be no uncertainty.

The loads which are applied to a structure fluctuate with time and are of uncertain value at any one point in time. This is carried over directly to the load effects (or internal actions) S . Somewhat similarly the structural resistance R will be a function of time (but not a fluctuating one) owing to deterioration and similar actions. Loads have a tendency to increase, and resistances to decrease, with time. It is usual also for the uncertainty in these quantities to increase with time. This means that the probability density functions $f_s(\cdot)$ and $f_R(\cdot)$ become wider and flatter with time and that the mean values of S and R also change with time. The safety limit state will be violated whenever, at any time t ,

$$R(t) - S(t) < 0 \text{ or } \frac{R(t)}{S(t)} < 1 \quad (4.1)$$

The probability that this occurs for any one load application (or load cycle) is the probability of limit state violation, or simply the probability of failure P_f .

4.1.2 Reliability Analysis

The research is carried out using Structural Reliability Analysis and Prediction Concept as outlined below:

- (a) For this basic structural reliability problem only one load effect S resisted by one resistance R is considered. Each described by a known probability density function $F_s(x)$ and $F_R(x)$ respectively. $\bar{S}$ is obtained from the applied loading Q through a structural analysis, but R and S are expressed in the same units.

For convenience, but without loss of generality, only the safety of a structural element is considered and as usual, that structural element will be considered to have failed if its resistance R is less than the stress resultant S acting on it. The probability of failure, P_f of the structural element will be stated in any of the following ways:

$$P_f = P(R \leq S) \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.2)$$

or $P_f = P(R - S \leq 0) \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.3)$

or $P_f = P(R/S \leq 1) \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.4)$

or $P_f = P(\ln R - \ln S \leq 0) \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.5)$

or in general

$$P_f = P(G(R, S) \leq 0) \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.6)$$

where $G(\)$ is termed the "limit state function" and the probability of failure is identical with the probability of limit state violation.

For many problems the simple formulations (4.2) – (4.6) are not entirely adequate, since it may not be possible to reduce the structural reliability problem to a simple R versus S formulation with R and S independent random variables. In general, R is a function of material properties and element or structure dimensions, while S is a function of applied loads Q , material densities and perhaps dimensions of the structure, each of which may be a random variable. Also, R and S may not be

independent, such as when some loads act to oppose failure (e.g. overturning) or when

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} F_R(x) f_S(x) dx \quad \dots \quad (4.7)$$

- (i) The first step is to define the variables involved in the generalized reliability problem. The fundamental variables which define and characterize the behaviour and safety of a structure, termed "basic" variables, usually are variables employed in conventional structural analysis and design. Examples are dimensions, densities or unit weights, materials, loads, material strengths, etc. It is very convenient to choose the basic variables such that they are independent, since dependence between basic variables usually adds some complexity to a reliability analysis.
- (ii) The probability distributions will then be assigned to the basic variables, depending on the knowledge that is available. If it can be assumed that past observations and experience for similar structure can be used, validly, for the structure under consideration the probability distributions might be inferred directly from such observed data. More generally, subjective information may be employed or some combination of techniques are required. Sometimes physical reasoning may be used to suggest an appropriate probability distribution.
- (iii) The parameters of the distribution are estimated from the data by using one of the usual methods viz: methods of moments, maximum likelihood, or other statistics.
- (iv) - Finally, when the model parameters have been selected, the model would be compared with the data. A graphical plot on

appropriate probability paper is often very revealing, but analytical "goodness of fit" tests could be used also.

- (b) When or after the basic variables and their probability distributions have been established, the next step is to replace the simple $R - S$ form of limit state function with a generalized version expressed directly in terms of basic variables.

The vector X will be used to represent all the basic variables involved in the problem. Then the resistance R will be expressed as $R = G_R(X)$ and the loading or load effect as $S = G_S(X)$. The limit state function $G(R, S)$ in (4.6) can be generalized. When the functions $G_R(X)$ and $G_S(X)$ are used in $G(R, S)$, the resulting limit state function will be written simply as $G(X)$, where X is the vector of all relevant basic variables and $G(\)$ is some function expressing the relationship between the limit state and the basic variables.

The limit state equation $G(x) = 0$ now defines the boundary between the satisfactory or 'safe' domain $G > 0$ and the unsatisfactory or 'unsafe' domain $G \leq 0$ in n -dimensional basic variable space.

- (c) With the limit state function expressed as $G(x)$, the generalization of

$$P_f = P(R - S \leq 0) = \iiint_D f_{RS}(r, s) dr ds \dots \dots \dots (4.8)$$

becomes

$$P_f = P(G(x) \leq 0) = \int \dots \int_{G(x) \leq 0} f_x(x) dx \dots \dots (4.9)$$

Sidestepping the integration process completely by transforming $f_x(x)$ in (4.9) to a multi-normal probability density function and using some remarkable properties which may then

be used to determine, approximately, the probability of failure – the so-called 'First Order Second Moment' methods.

4.2 Second – Moment Concepts (FOSM)

Rather than use approximate (and numerical) methods to perform the integration required in the reliability integral:

$$P_f = P(G(x) \leq 0) = \int \dots \int_{G(x) \leq 0} f_x(x) dx \quad (4.10)$$

the probability density function $f_x(\cdot)$ in the integrand is simplified. In this case the reliability estimation with each of its variable is represented only by its first two moments i.e. by its mean and standard deviation. This is known as the "Second – Moment" level of representation. A convenient way in which the second-moment representation might be interpreted is that each random variable is represented by the normal distribution. This continuous probability distribution is described completely by its first two moments.

Because of their inherent simplicity, the so-called "Second-moment" methods have become very popular. Early works by Mayer (1926), Freudenthal (1956), Rzhantitsyn (1957) and Basler (1961) contained second moment concepts. Not until the late 1960s, however, was the time ripe for the ideas to gain a measure of acceptance, prompted by the work of Cornell (1969) as reported by Melchers (2002).

When resistance R and load effects are each Second-moment random variables (i.e. such as having normal distributions), the limit state equation is the "safety margin" $Z = R - R$ and the probability of failure P_f is

$$P_f = \phi(-\beta) = \beta = \frac{U_z}{\sigma_z} \quad \text{--- -- -- -- --} \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.11)$$

where β is the (simple) 'safety' (or 'reliability') index and $\phi(\quad)$ the standard normal distribution function. Equation (4.2) yields the exact probability of failure P_f when both R and S are normally distributed. However, P_f defined in this way is only a 'nominal' failure probability when R and S have distributions other than normal. Conceptually it is probably better in this case not to refer to probabilities at all, but simply to β the 'safety' or 'reliability' index.

So far, only the first two moments of each random variable have been considered in the calculation of the failure probability. However, if information about probability distributions is available for some or all of the basic variables it would be sensible to incorporate it in the reliability analysis if possible. One way of doing this is to transform non-normal distributions into equivalent normal distributions (Paloheimo and Hannus, 1974).

The transformation from non-normally distributed variables to equivalent standardized normal variable is not as elementary as suggested so far. However, once it has been done the resulting normal equivalents can be used directly in second-moment calculation procedures.

The transformation approach was developed as an extension of the FOSM methods, the term First Order Reliability (FOR) method was adopted for it, and since the limit state functions are still linearized (hence of 'first order') but probability distributions are no longer approximated only by their first and second moments.

4.3 First Order Reliability Method (FORM)

FORM has been designed for the approximate computation of general probability integrals over given domains with locally smooth boundaries but especially for probability integrals occurring in structural reliability. Similar integrals are also found in many other areas, for example, in hydrology, mathematical statistics, control theory, classical hardware reliability and

econometrics which are the areas where FORM has already been applied.
The concept of FORM is, essentially, based on

$$P_F = P(X \in F) = P(g(X) \leq 0) = \int_{G(x) \leq 0} dF_X(x) \dots \dots \dots (4.12)$$

which, in turn, is based on

$$P_f \approx \Phi(-\beta) \quad \dots \quad \dots \quad \dots \quad \dots \quad \dots \quad \dots \quad \dots \quad (4.13)$$

$$\beta = \min \{ \|u\| \text{ for } \{u : q(u) \leq 0\} \} \quad \dots \quad (4.14)$$

The present version of FORM, FORM5, is an update to earlier versions described in

$$\beta = \alpha^T u^* = \|u^*\| \quad \dots \dots \dots (4.15)$$

The general problem to which FORM provides an approximate solution is as follows. The state of a system is a function of many variables some of which are uncertain. These uncertain variables are random with joint distribution function $F_X(x) = P(\bigcap_{i=1}^n \{X_i \leq x_i\})$ defining the stochastic model. For FORM, it is required that $F_X(x)$ is at least locally continuously differentiable, i.e. that probability densities exist. The random variables $X = (X_1, \dots, X_n)^T$ are called basic variables. The locally sufficiently smooth (at least once differentiable) state function is denoted by $g(X)$. It is defined such that $g(X) > 0$ corresponds to favorable (safe, intact, acceptable, ...) state. $g(X) = 0$ denotes the so-called limit state or the failure boundary. Therefore, $g(X) < 0$ (sometimes also $g(X) \leq 0$) defines the failure (unacceptable, adverse,...) domain F . $g(X)$ can be defined as an analytic function or an algorithm (e.g. a finite element code). In the context of FORM it is convenient but necessary only locally that $g(X)$ is a monotonic function in each component of X . Among other useful information FORM produces an approximation to Equation (4.12).

This is achieved by transforming non-gaussian (non-normal) variables into independent standard normal variables, by locating the so-called β point (checking point, most likely failure point, design point) through an optimization procedure, by linearising the limit state function in that point and by estimating the failure probability using the standard normal integral.

In Table 5.1, the statistical and probabilistic descriptions of the variables in the functional relations are presented.

Table 5.1 – Stochastic model for concrete piling in cohesive soils.

i	x_i	pdf	$\bar{X}$	v	$\alpha x = \bar{X} \cdot v$
1	f_{cu}	LN	40×10^3	0.15	6.0×10^3
2	D_1	N	$3 \times 10^{-1}m$	0.06	1.8×10^{-2}
3	f_y	LN	$460 \times 10^3KN/m^2$	0.15	6.9×10^4
4	D_2	N	$25 \times 10^{-3}m$	0.06	1.5×10^{-3}
5	S_u	LN	$12KN/m^2$	0.15	1.8
6	α	LN	1.00	0.15	1.5×10^{-1}
7	L_b	N	23.0m	0.06	1.38
8	SF	LN	3.0	0.15	4.5×10^{-1}

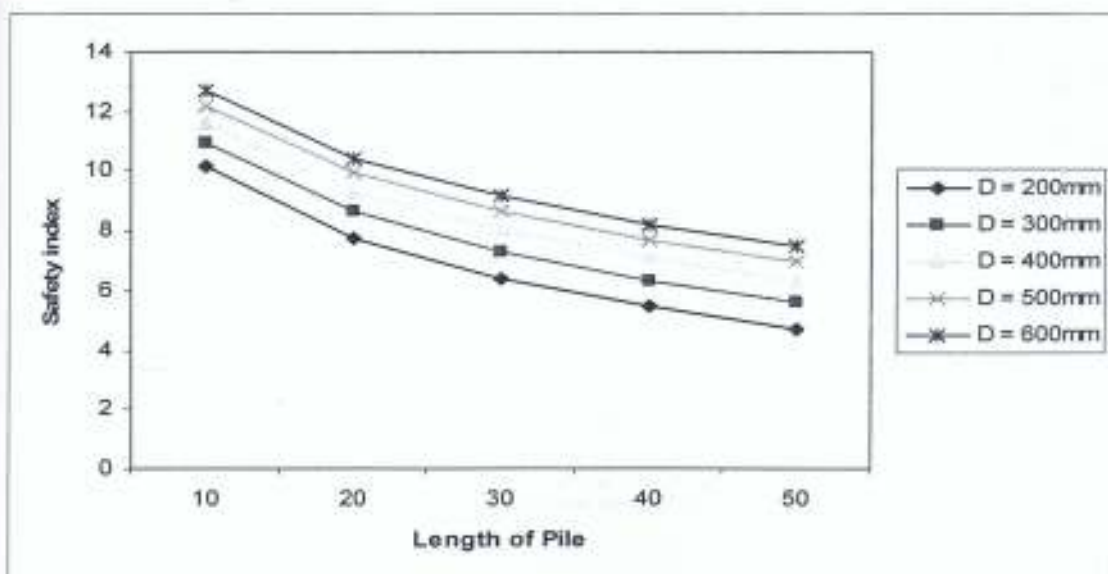


Fig. 5.1- Safety Index (β_R) against Length of Pile with varying Diameters in Cohesive Soils

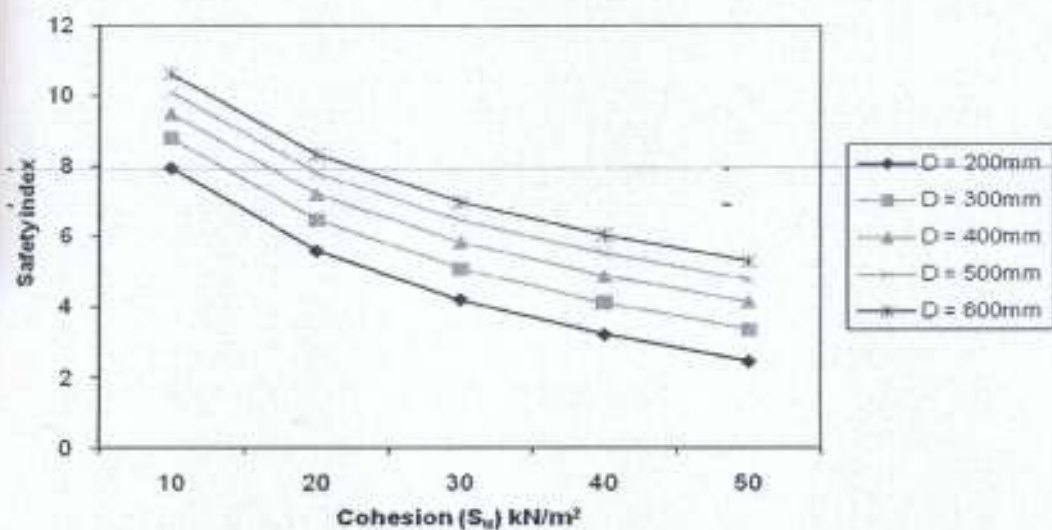


Fig. 5.2- Safety index (β_R) against Cohesion with varying Diameters in Cohesive Soils

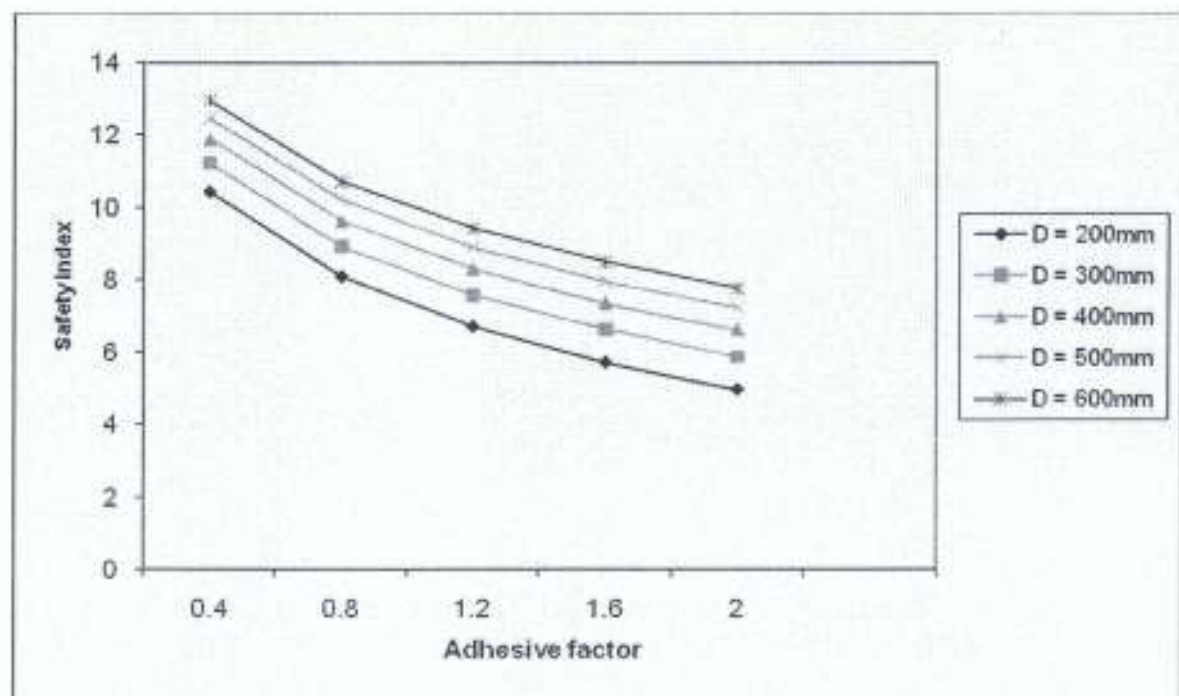


Fig. 5.3 - Safety index (β_R) against Adhesive factor with varying Diameters in Cohesive Soils

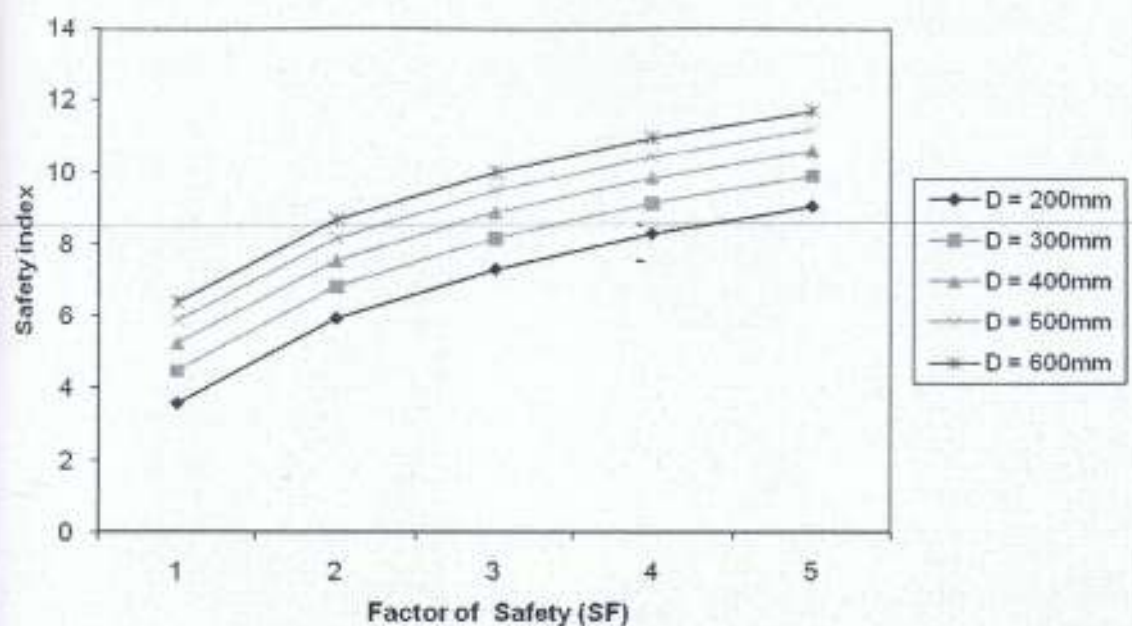


Fig. 5.4 - Safety Index (β_R) against Factor of Safety with varying Diameters in Cohesive Soils

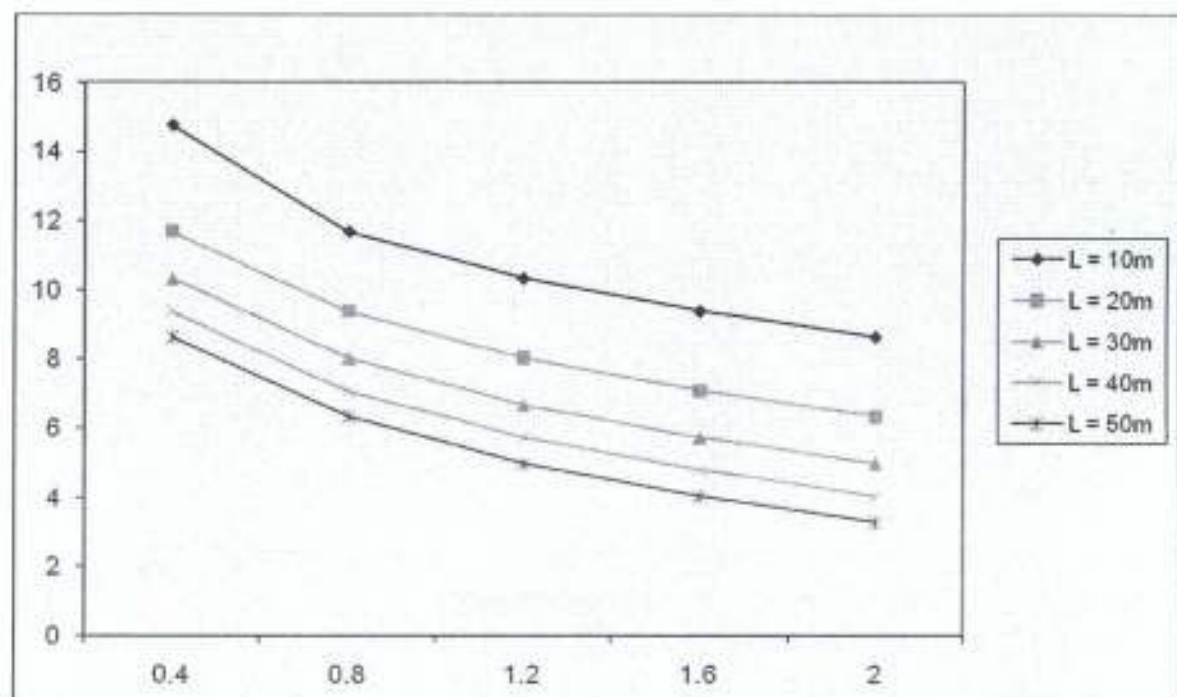


Fig. 5.5 - Safety Index (β_R) against Adhesion with varying Lengths

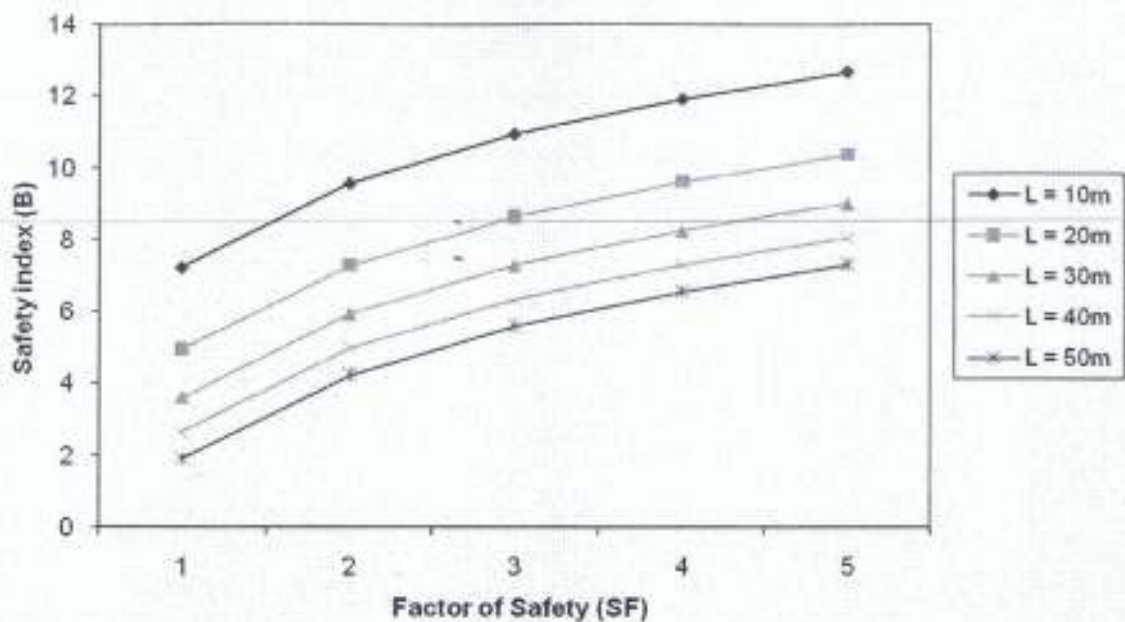


Fig. 5.6 - Safety Index (β_R) against Factor of Safety with varying Lengths

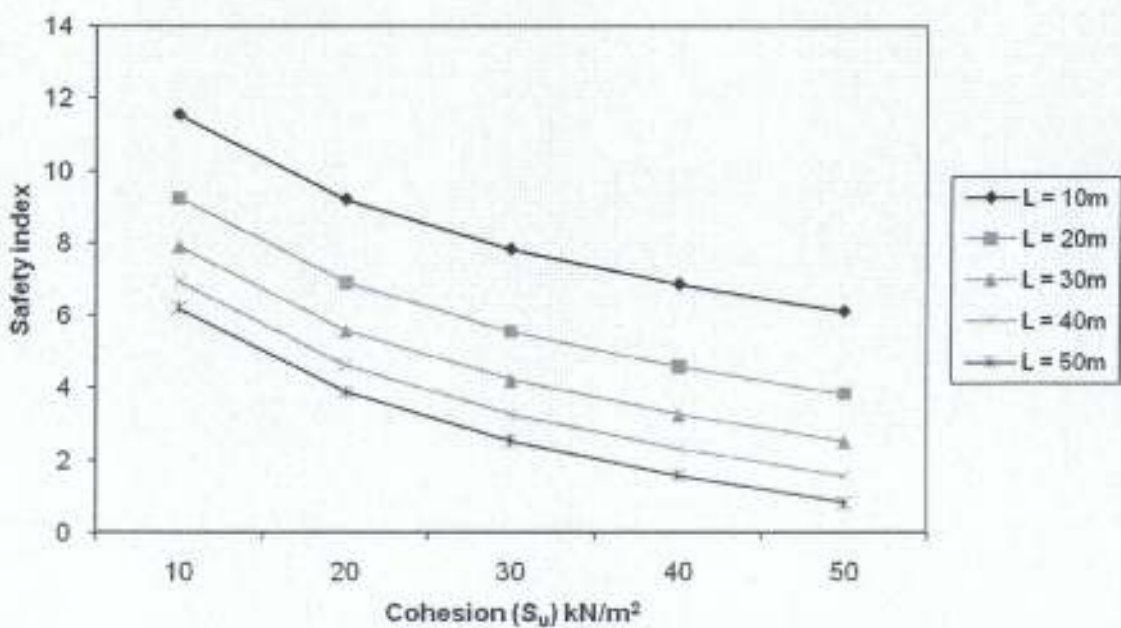


Fig. 5.7 - Safety Index (β_R) against Cohesion with varying Lengths

In cohesive soils, the safety level associated with piling capacity shows a consistent conservation. At the same time, safety level decreases with increasing length of pile

(Fig. 5.1), the undrained shear strength of the soil (Fig. 5.2), and adhesion factor of pile (Fig. 5.3). The decrease however depends on the diameter of the pile. Also it can be seen in Fig. 5.1 that the safety level increases with the diameter of the pile, though it decreases with pile length. At any rate, the safety level is generally high for all the range of values for pile length, implying that the estimated pile capacity is highly conservative.

5.3 Static Pile Capacity of Concrete piling in Cohesionless soils

Similar to the functional relationship between allowable design load and the allowable pile capacity expressed for cohesive soils, we also have:

$G(x)$ = Allowable Design Load – Allowable Pile Capacity

$$G(X) = \left[\frac{0.259 f_{cu} D_1^2 + 0.314 f_y D_2^2 - k_f}{SF} \right] \quad (5.10)$$

in which

$$k_f = 0.785 \gamma L D_1^2 \left(\frac{[e^{(2.36+0.5\phi)\tan\phi}]^2}{2 \cos^2(45-0.5\phi)} \right) + 3.142 D_1 L_b + \gamma L (1 - \sin \phi') \tan \delta$$

as the expression for assessing the performance of concrete piling in cohesionless soils. In equation (5.10), the additional variables not in equation (5.9) are: γ = unit weight of soil, ϕ = drained angle, ϕ' = effective stress angle, and δ = effective friction angle. The assumed statistical values and their corresponding probability distributions are shown in Table 5.2.

Table 5.2- Stochastic model for concrete piling in cohesionless soils

i	x_i	pdf	$\bar{x}$	v	$\sigma x = \bar{x} \cdot v$
1	f_{cu}	LN	$40 \times 10^3 \text{KN/m}^2$	0.15	6.0×10^3
2	D_1	N	$4.5 \times 10^{-1} \text{m}$	0.06	2.7×10^{-2}
3	f_y	LN	$460 \times 10^3 \text{KN/m}^2$	0.15	6.9×10^4
4	D_2	N	$25 \times 10^{-3} \text{m}$	0.06	1.5×10^{-3}
5	γ	LN	17.3KN/m^3	0.15	2.60
6	L	N	12m	0.06	7.2×10^{-1}
7	ϕ	LN	34°	0.15	5.1
8	ϕ'	LN	34°	0.15	5.1
9	δ	LN	22°	0.15	3.3
10	SF	LN	3.0	0.15	4.5×10^{-1}

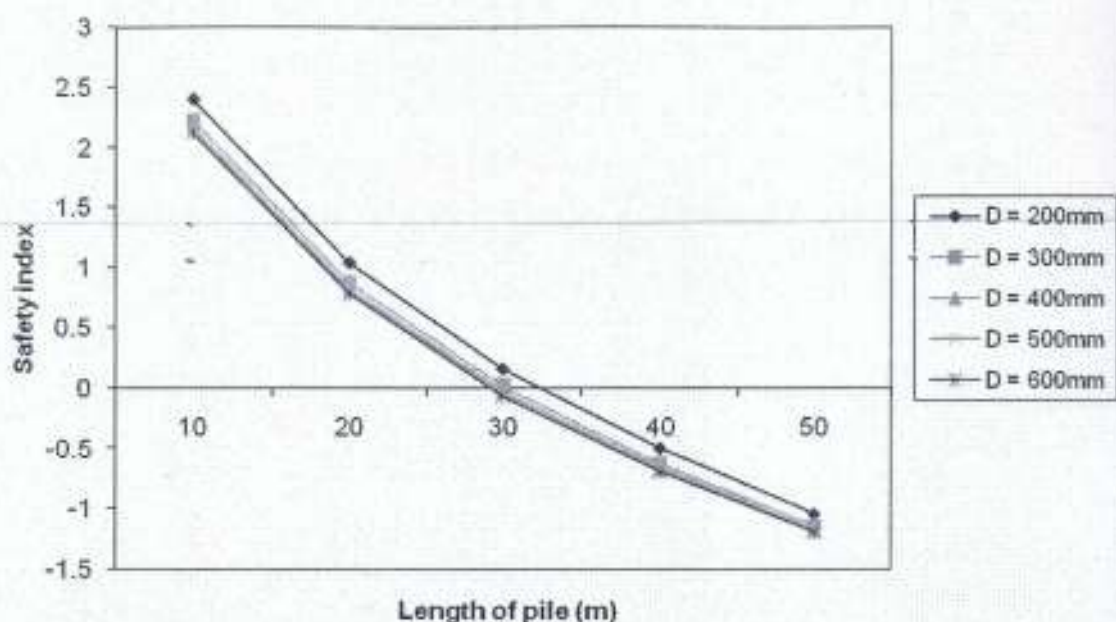


Fig. 5.8 - Safety index (β_R) against Length of Pile with varying Diameters

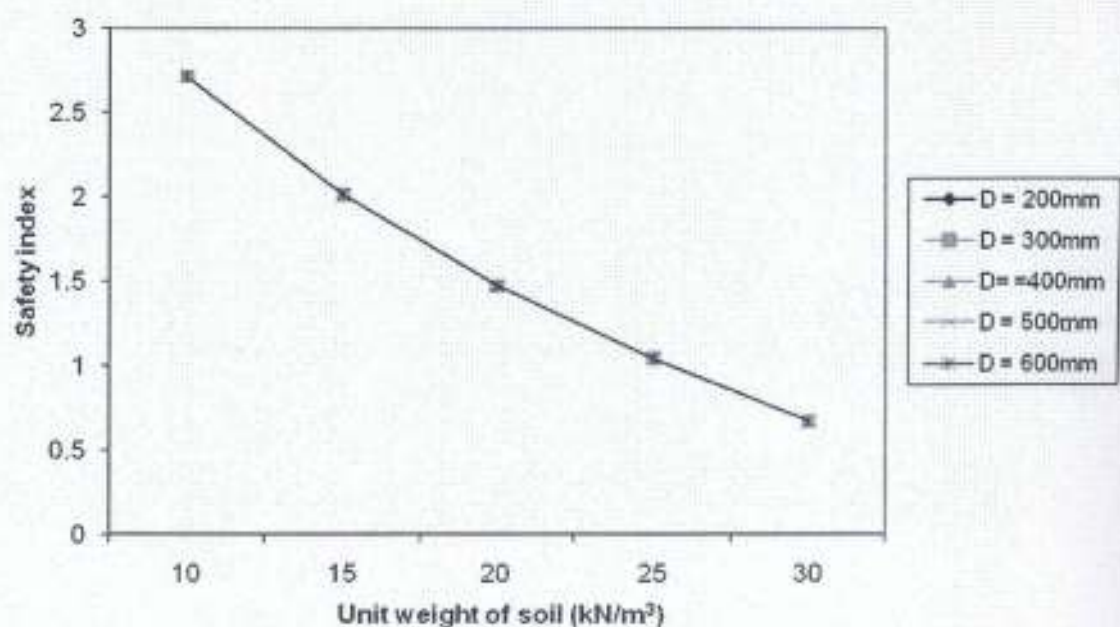


Fig. 5.9 - Safety index (β_R) against Unit weight of soil with varying Diameters

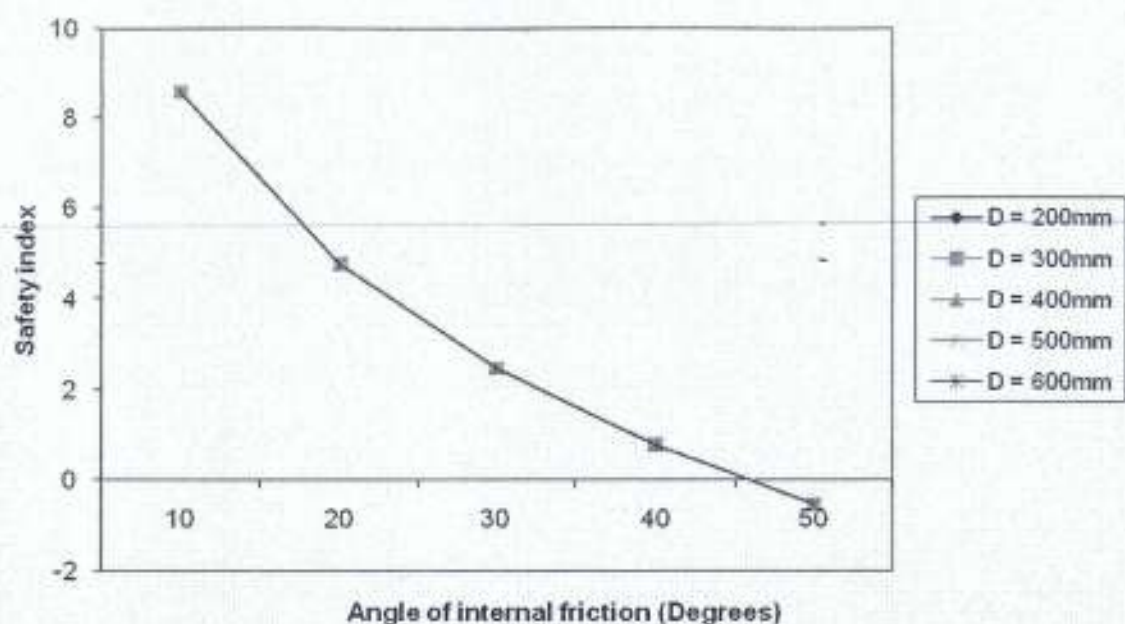


Fig. 5.10 - Safety index (β_R) against Angle of internal friction with varying Diameters

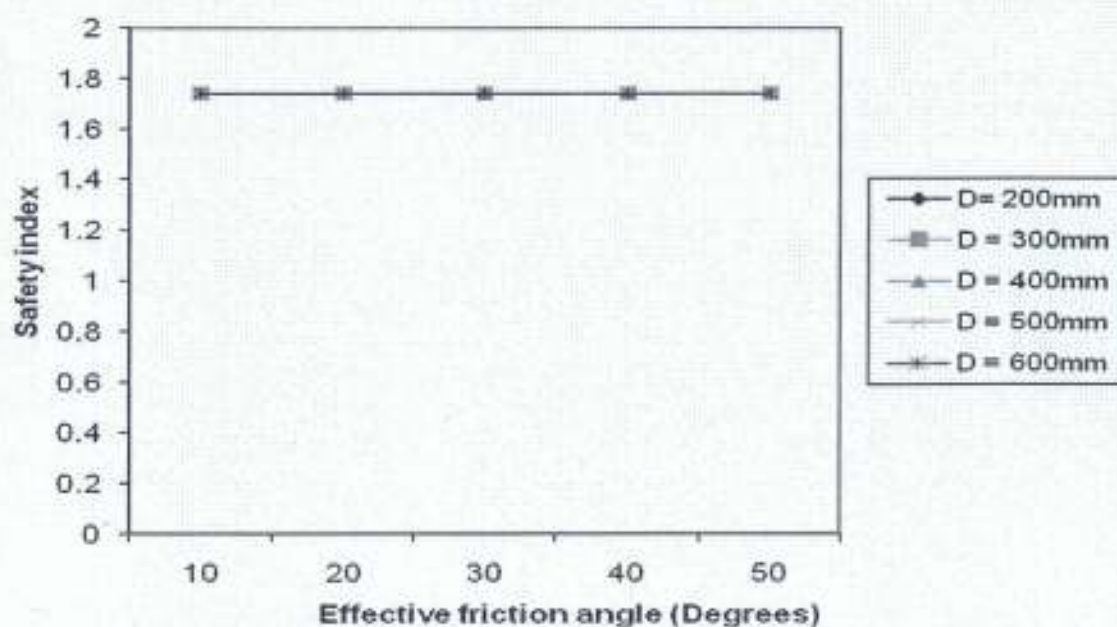


Fig. 5.11 - Safety index (β_R) against Effective friction angle with varying Diameters

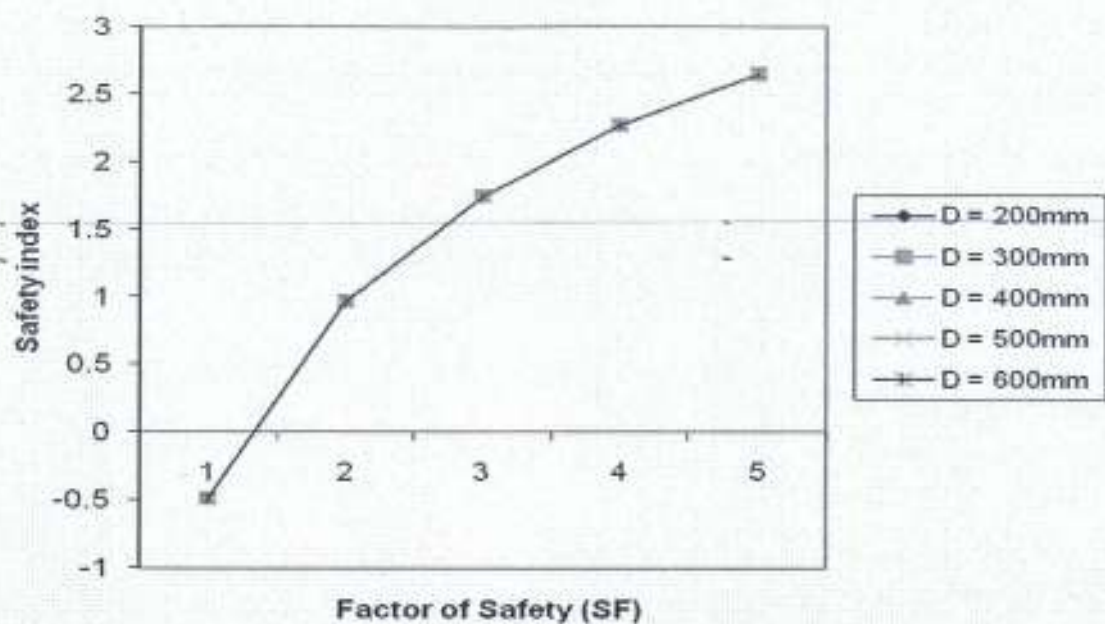


Fig. 5.12 - Safety Index (β_R) against Factor of Safety with varying Diameters

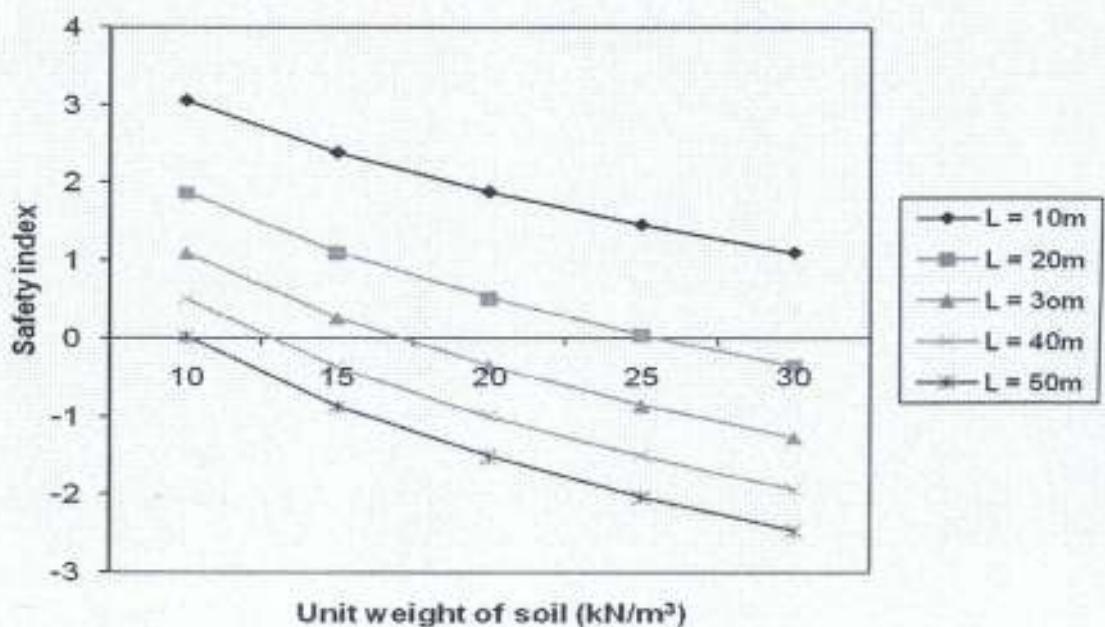


Fig. 5.13 - Safety Index (β_R) against Unit weight of soil with varying Lengths

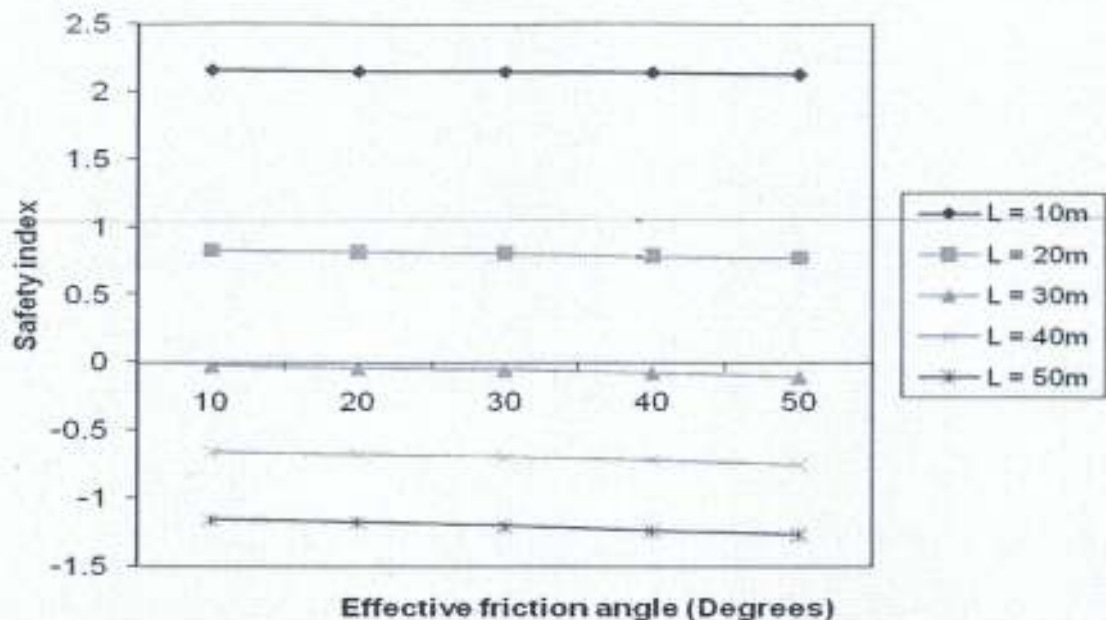


Fig. 5.14 - Safety index (β_R) against Effective friction angle with varying Lengths

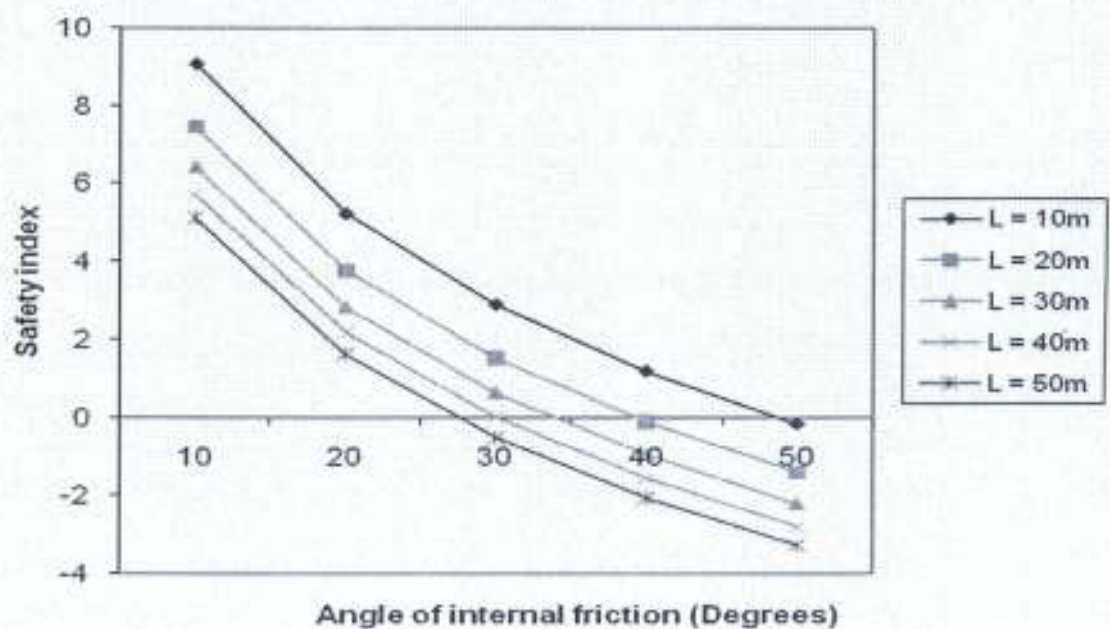


Fig. 5.15 - Safety index (β_R) against Angle of internal friction with varying Lengths

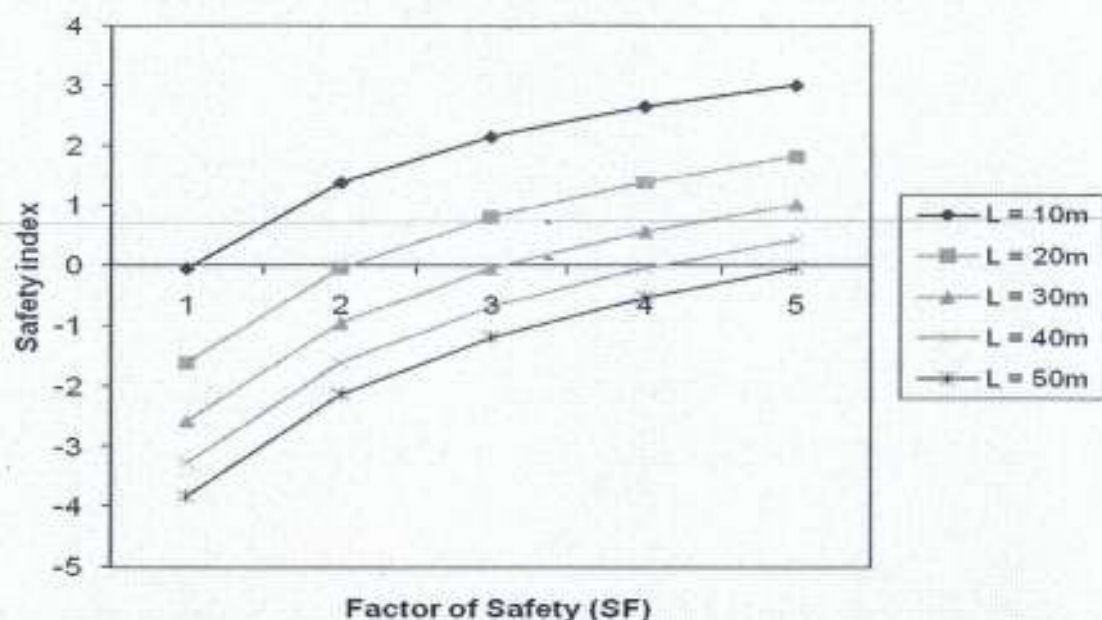


Fig. 5.16 - Safety Index (β_R) against Factor of Safety with varying

lengths

The predicted capacity in cohesionless soils leads to a more rapid degeneration of the safety level than in cohesive soils. For instance, it is noted that when piling length is between 10 m and 20 m, there is some margin of safety. At a length of 30 m, the safety level has reduced to zero, implying a total violation of the predicted capacity. This can be seen in Fig. 5.8. The unit weight of the soil plays a significant role in capacity prediction. The denser the soil, the lower the safety level, no matter the diameter of the pile (see Fig. 5.9). This similar trend is noted when the angle of internal friction increases (Fig. 5.10).

5.4 Static pile capacity of steel piling in cohesive soils

The functional relationship between allowable design load and allowable pile capacity for steel piling in cohesive soils is:

$$G(x) = \text{Allowable Design Load} - \text{Allowable Pile Capacity}$$

$$= 0.35f_y A_p - \{A_p(qS_u) + \sum(A_s \alpha S_u)\} / SF$$

$$= 0.35f_y A_p - \{S_u(qA_p + \alpha A_s)\} / SF \quad (5.11)$$

where A_p = area of pile, A_s = area of shaft

The assumed statistical values and their corresponding probability distributions are shown in Table 5.3

Table 5.3 - Stochastic model for steel piling in cohesive soils

i	x_i	pdf	$\bar{x}$	v	$\sigma x = \bar{x} \cdot v$
1	F_y	LN	$460 \times 10^3 \text{ KN/m}^2$	0.15	6.9×10^4
2	A_p	N	$1.4 \times 10^{-1} \text{ m}^2$	0.06	8.4×10^{-3}
3	S_u	LN	28 KN/m^2	0.15	4.2
4	α	LN	1.00	0.15	1.5×10^{-1}
5	A_s	N	48.84 m^2	0.06	2.93
6	SF	LN	2.0	0.15	3.0×10^{-1}

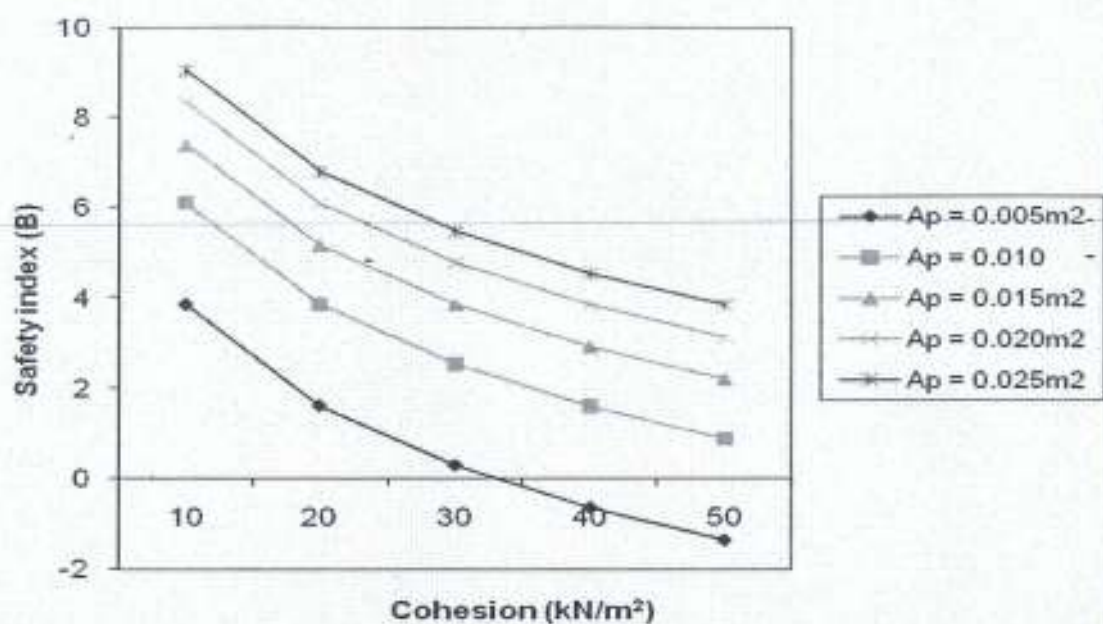


Fig. 5.17 - Safety index (β_R) against Cohesion with varying Pile areas

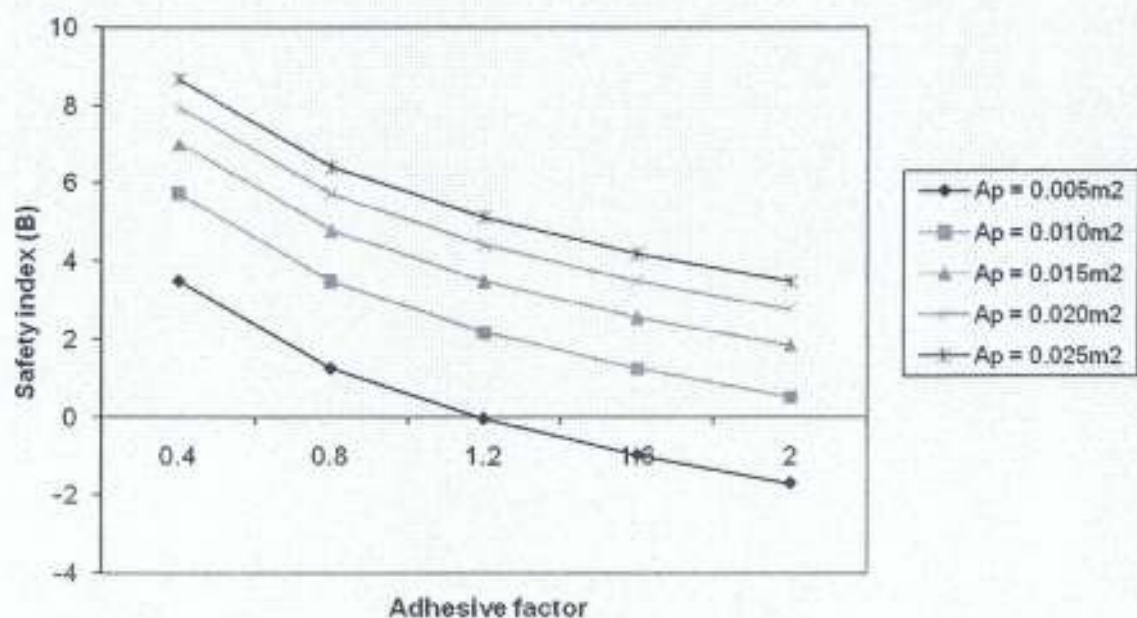


Fig. 5.18 - Safety index (β_R) against Adhesive factor with varying Pile areas

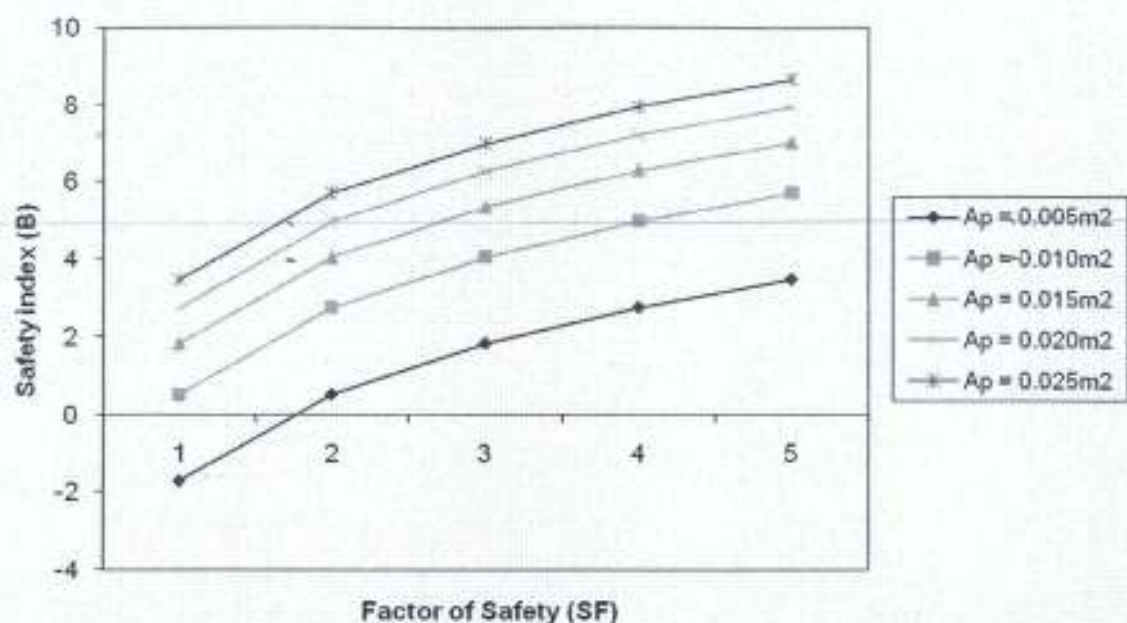


Fig. 5.19 - Safety Index (β_R) against Factor of Safety with varying Pile areas

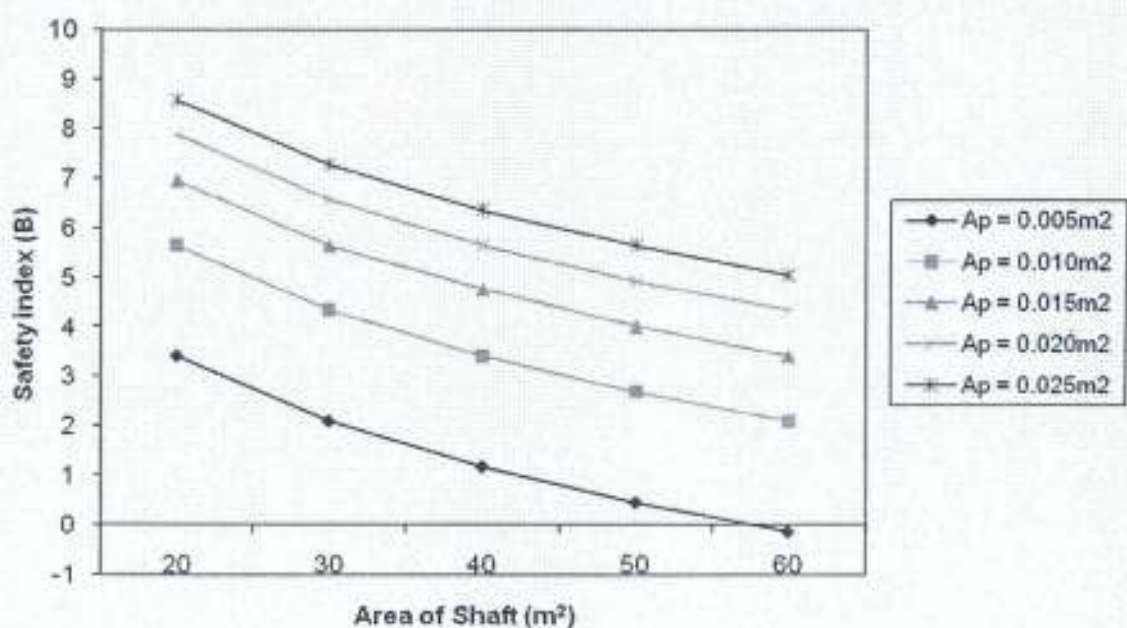


Fig. 5.20 - Safety Index (β_R) against Area of shaft with varying Pile areas

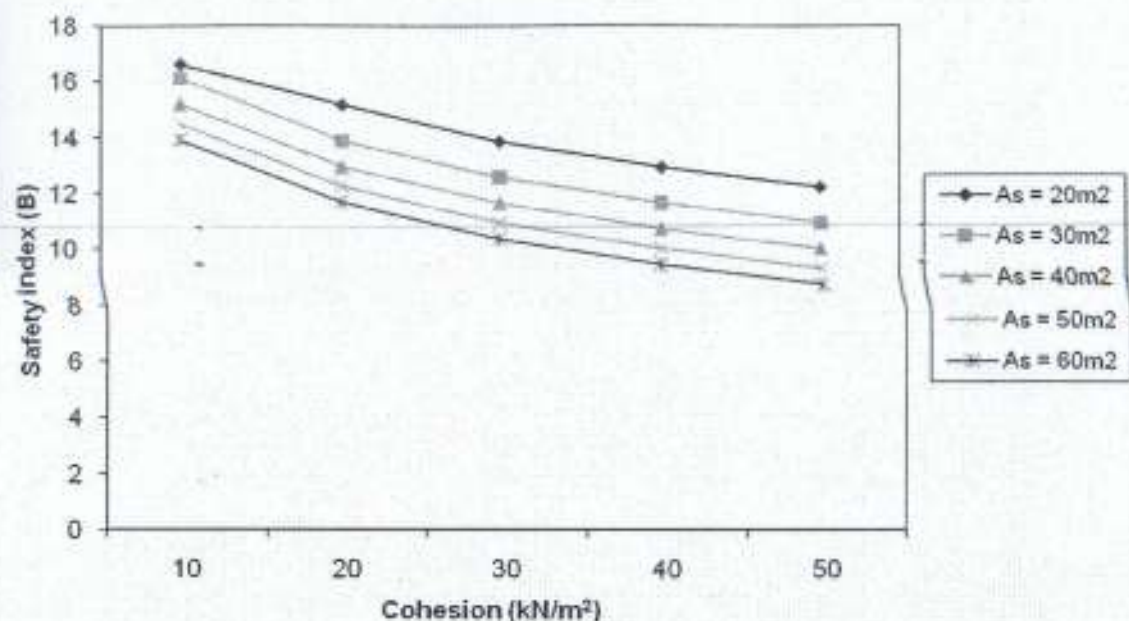


Fig. 5.21 - Safety index (β_R) against Cohesion with varying Shaft areas

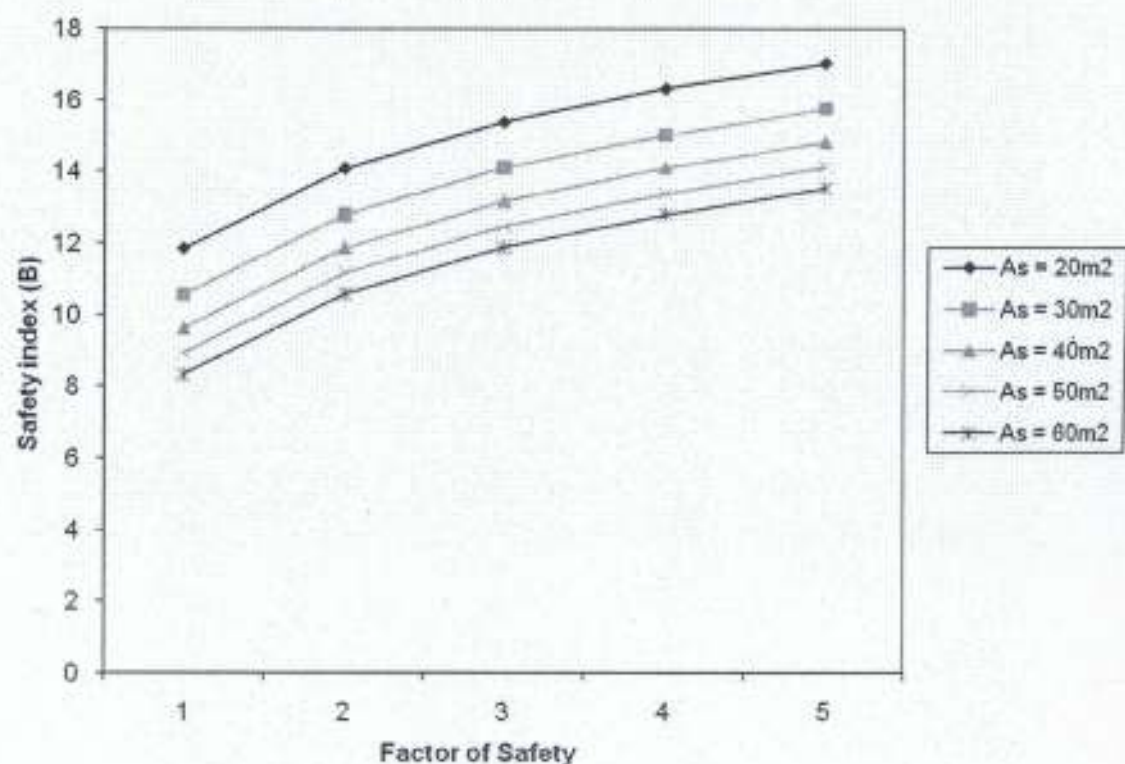


Fig. 5.22 - Safety index (β_R) against Factor of Safety with varying Shaft areas

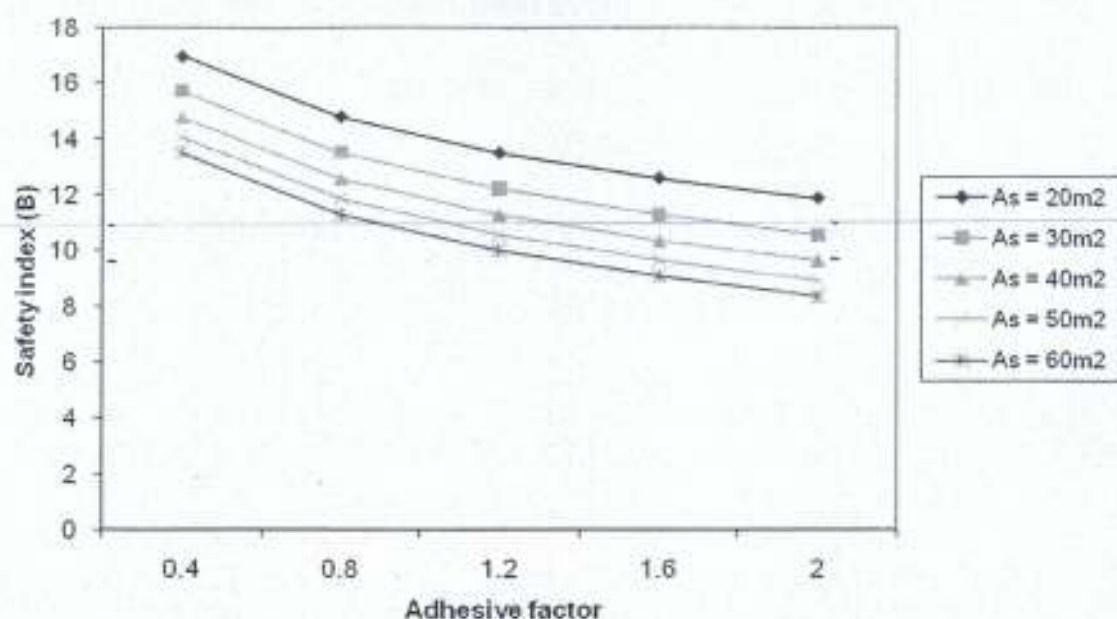


Fig. 5.23 - Safety Index (β_R) against Adhesive factor with varying Shaft

areas

The implied safety level associated with piling capacity of steel in cohesive soils shows a consistent conservation. Although, it degenerates with cohesion, adhesion factor and the area of shaft as shown in Figs. 5.17, 5.18 and 5.20 respectively. However, this decrease depends on the area of pile. While the safety level of the piling capacity follows the same trend based on the area of shaft of the pile, but, degenerates less rapidly and highly consistently conservative, see Figs. 5.21 and 5.23. It is also noted here that the higher the adhesion factor and the cohesion, the less conservative the static pile capacity.

The functional relationship between allowable design load and the allowable pile capacity can be expressed as follows:

$$G(x) = \text{Allowable Design Load} - \text{Allowable Pile Capacity}$$

So that, $G(x) = 0.35f_y A_p - \left\{ A_p \bar{q} N_q + \sum (A_s + \bar{q} K \tan \sigma) \right\} / SF$

$$= 0.35f_y A_p - \left\{ A_p \gamma L \left(\frac{e^{(2.356 + 0.5\phi) \tan \phi}}{2 \cos^2 (45 - 0.5\phi)} \right) + \right. \\ \left. A_s + \gamma L (1 - \sin \phi') \tan \sigma \right\} / SF \quad (5.12)$$

In Table 5.4, the statistical and probabilistic descriptions of the variables in the functional relations are presented.

Table 5.4 Stochastic model for steel piling in cohesionless soils.

<i>i</i>	<i>x_i</i>	pdf	$\bar{X}$	<i>v</i>	$\sigma_X = \bar{X} \cdot v$
1	<i>F_y</i>	LN	460 x 10 ³ KN/m ²	0.15	6.9 x 10 ⁴
2	<i>A_p</i>	N	1.60 x 10 ⁻¹ m ²	0.06	9.6 x 10 ⁻³
3	<i>γ</i>	LN	17.3KN/m ³	0.15	2.60
4	<i>L</i>	N	12m	0.06	7.2 x 10 ⁻¹
5	<i>φ</i>	LN	34°	0.15	5.1
6	<i>A_s</i>	N	16.96 m ²	0.06	1.02
7	<i>φ'</i>	LN	34°	0.15	5.1
8	<i>δ</i>	LN	22°	0.15	3.3
9	<i>SF</i>	LN	3.0	0.15	4.5 x 10 ⁻¹

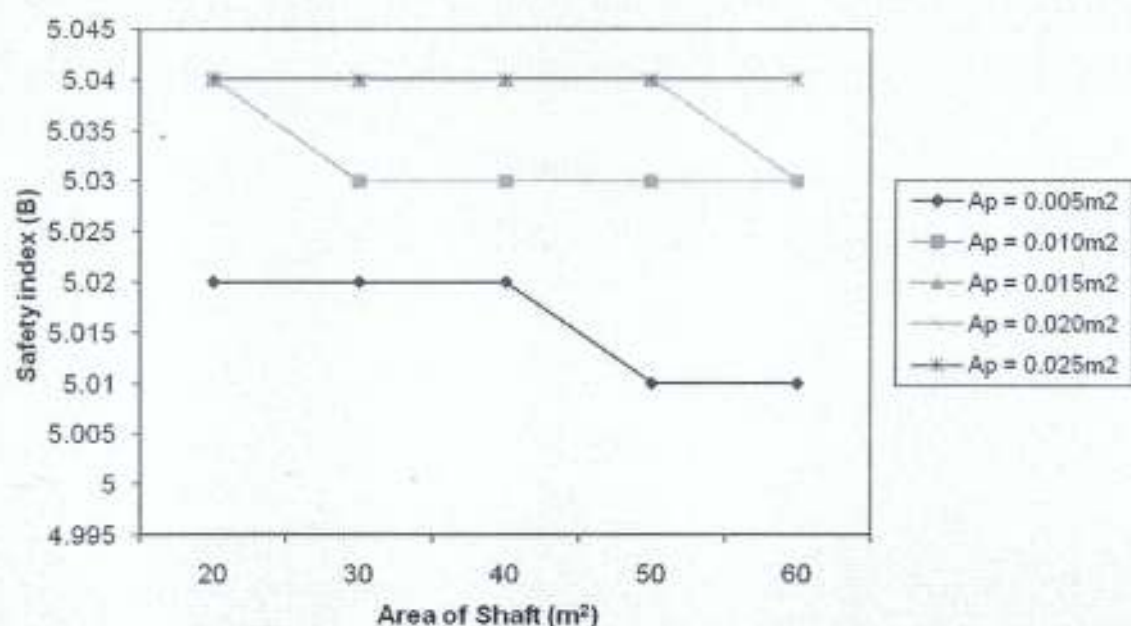


Fig. 5.24 - Safety Index (β_R) against Area of Shaft with varying Pile areas

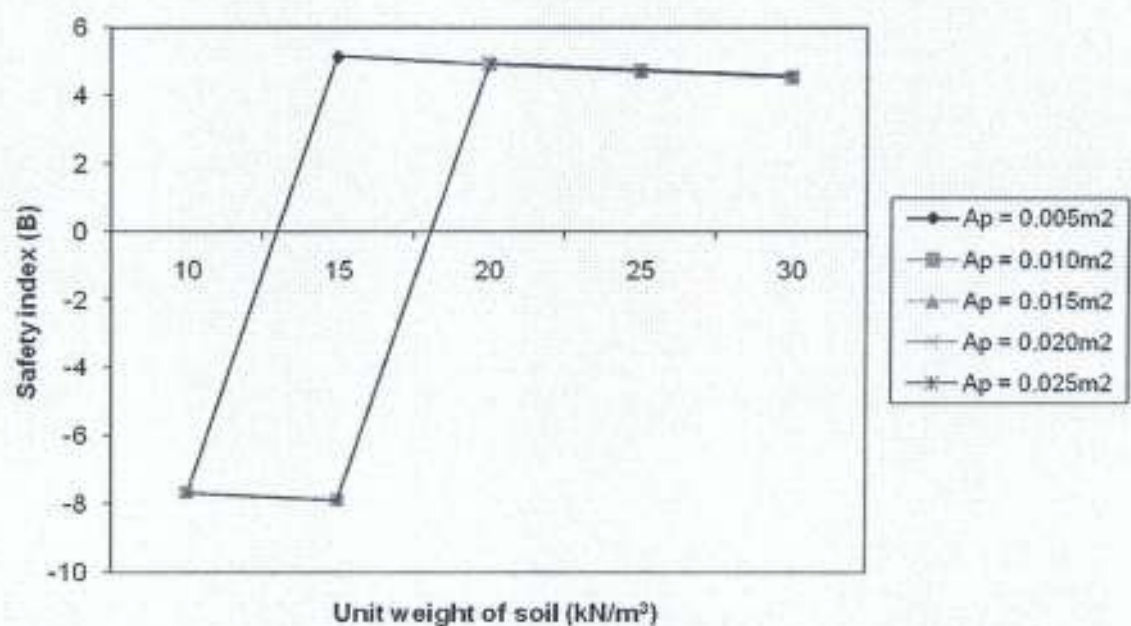


Fig. 5.25 - Safety Index (β_R) against Unit weight of soil with varying Pile areas

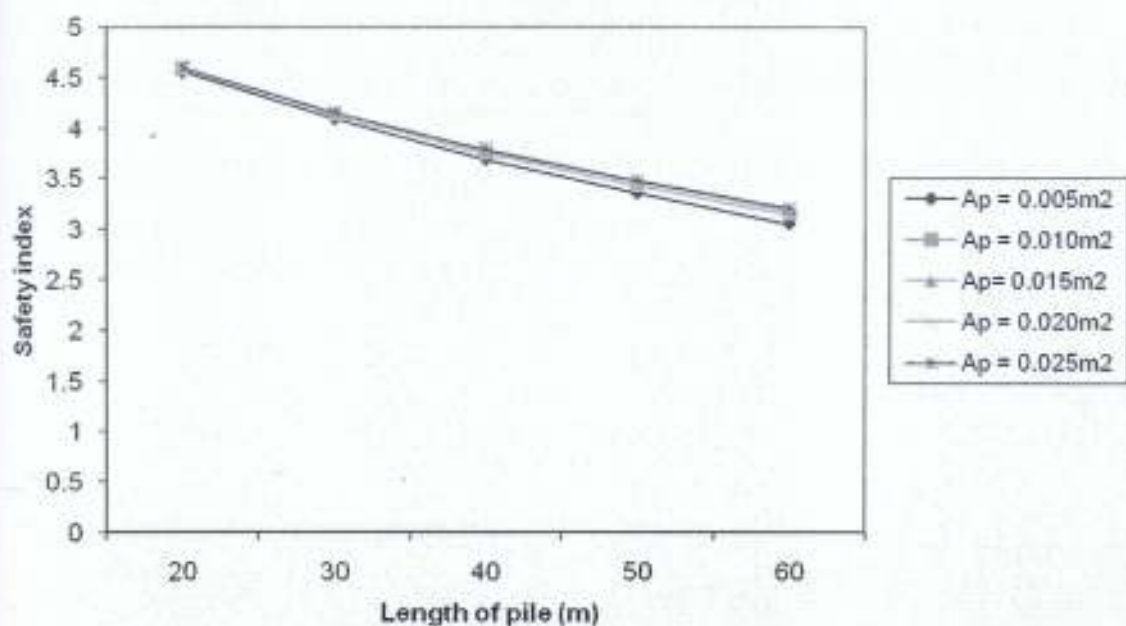


Fig. 5.26 - Safety Index (β_R) against Length of Pile with varying Pile areas

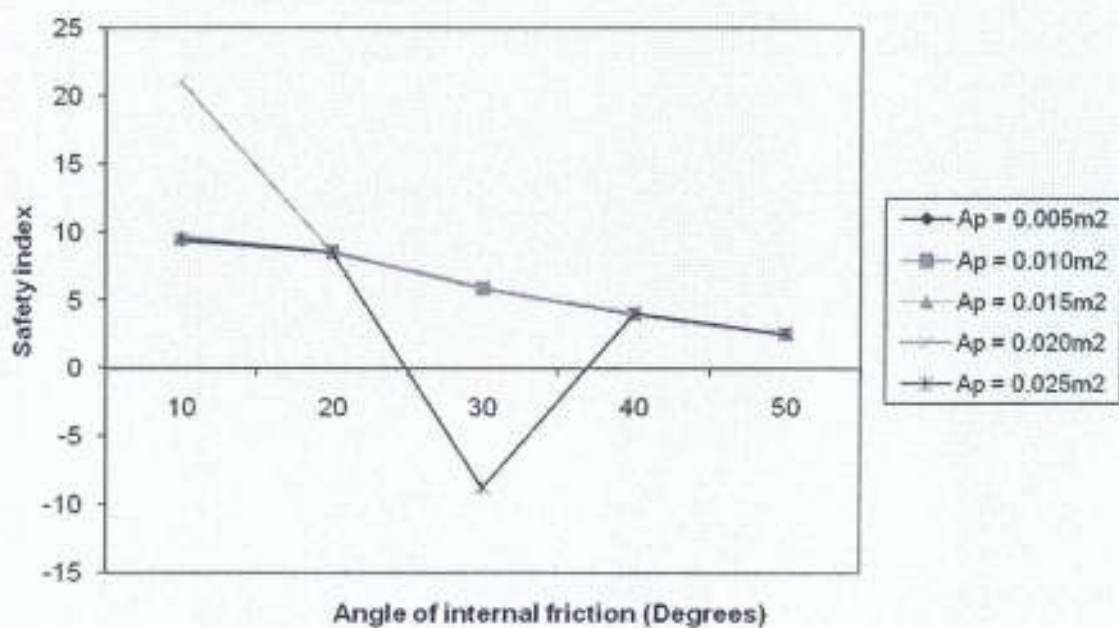


Fig. 5.27 - Safety Index (β_R) against Angle of internal friction with varying Pile areas

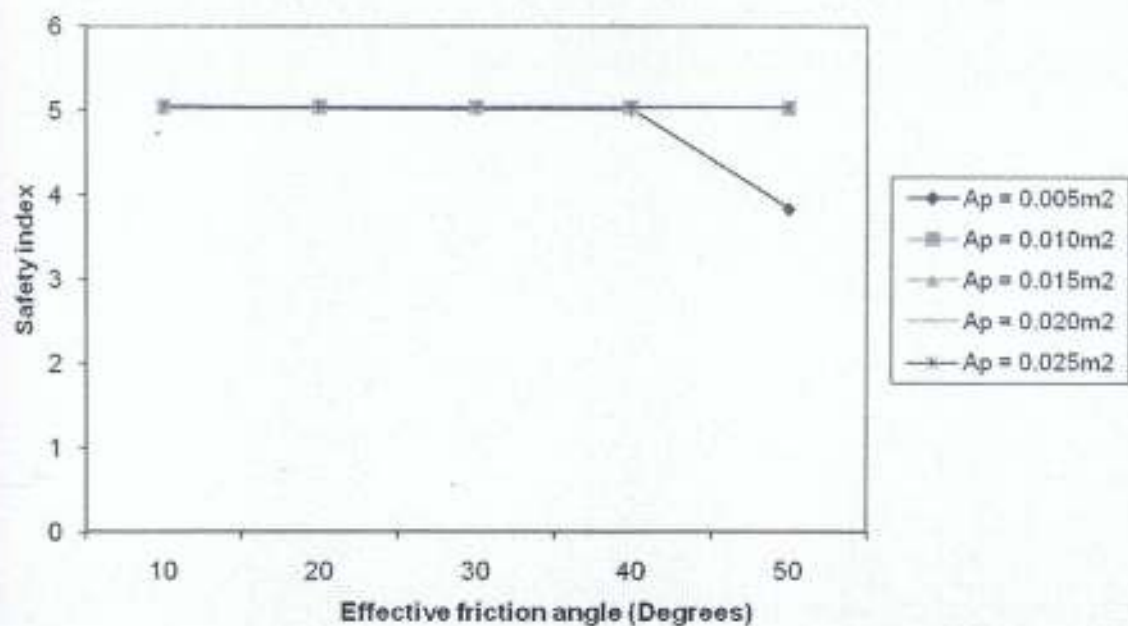


Fig. 5.28 - Safety index (β_R) against Effective friction angle with varying Pile areas

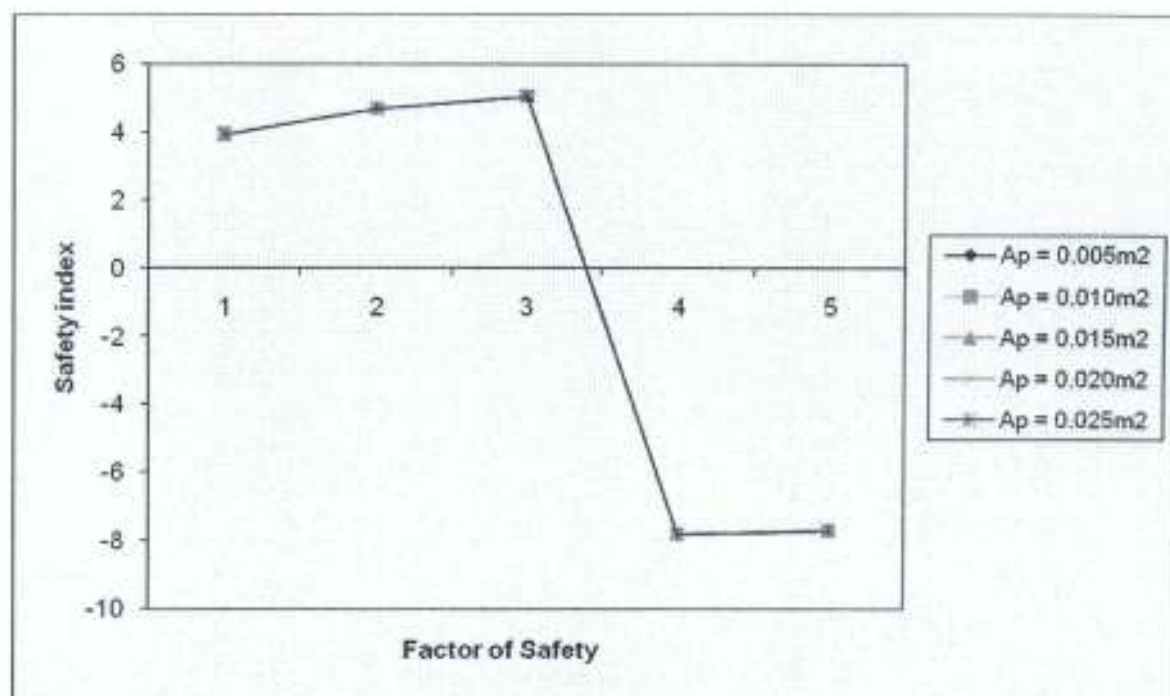


Fig. 5.29 - Safety index (β_R) against Factor of Safety with varying

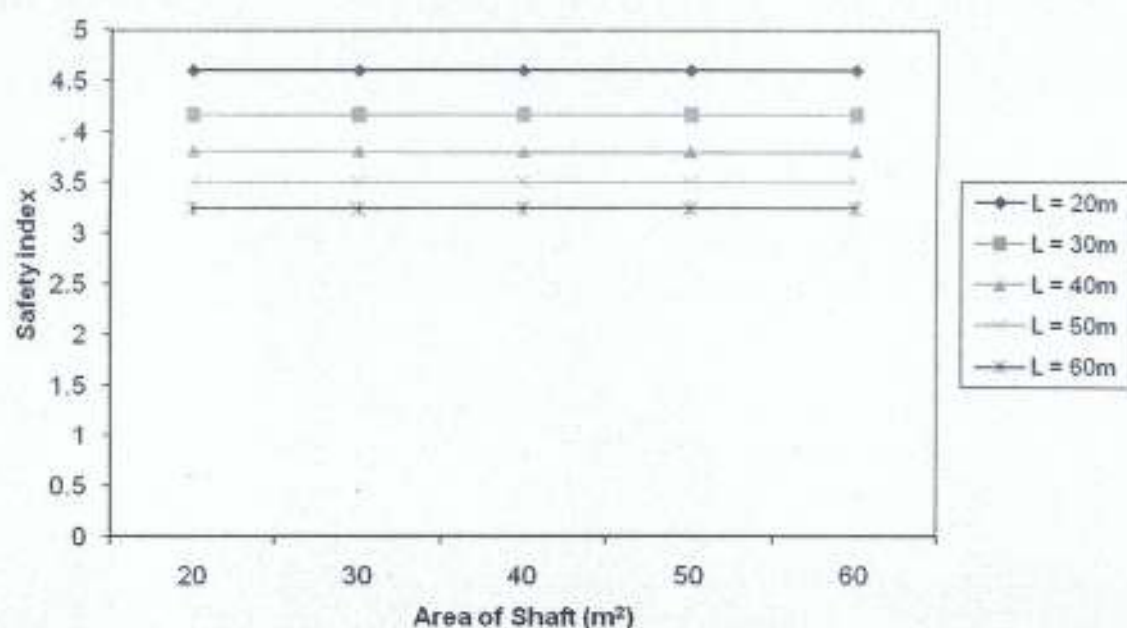


Fig. 5.30 - Safety index (β_R) against Area of shaft with varying Lengths

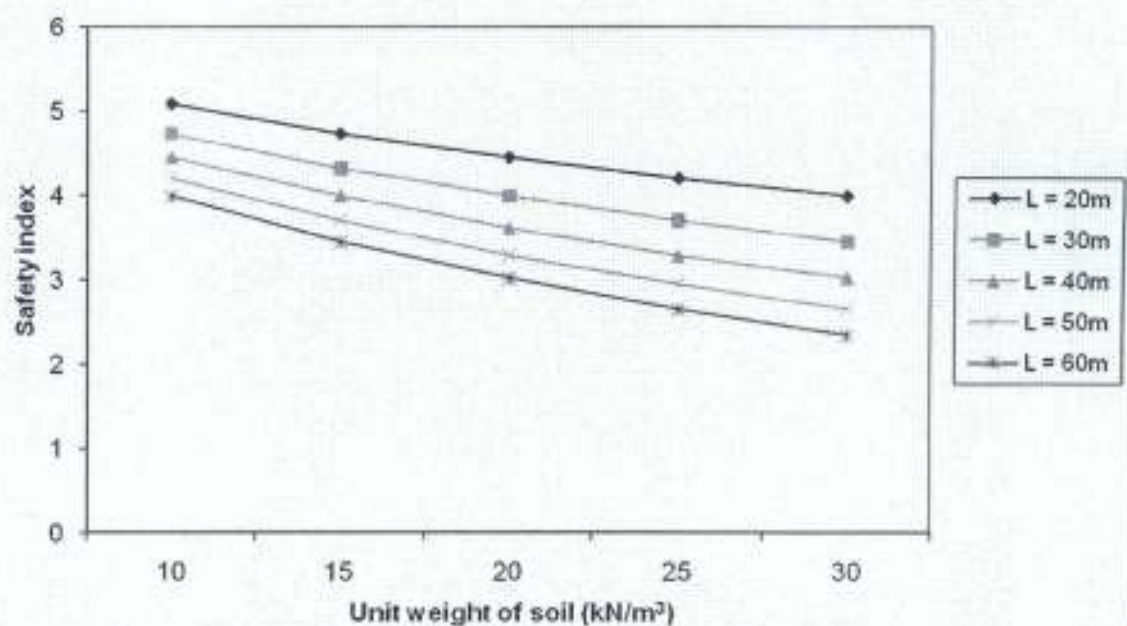


Fig. 5.31 - Safety index (β_R) against Unit weight of soil with varying Lengths

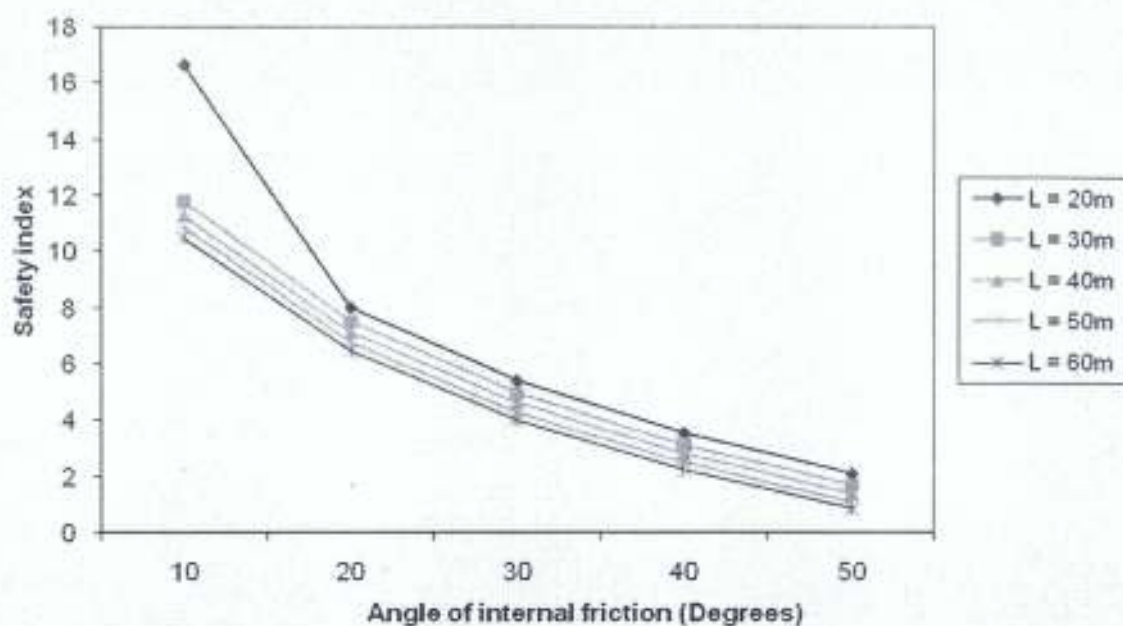


Fig. 5.32 - Safety Index (β_R) against Angle of internal friction with varying

lengths

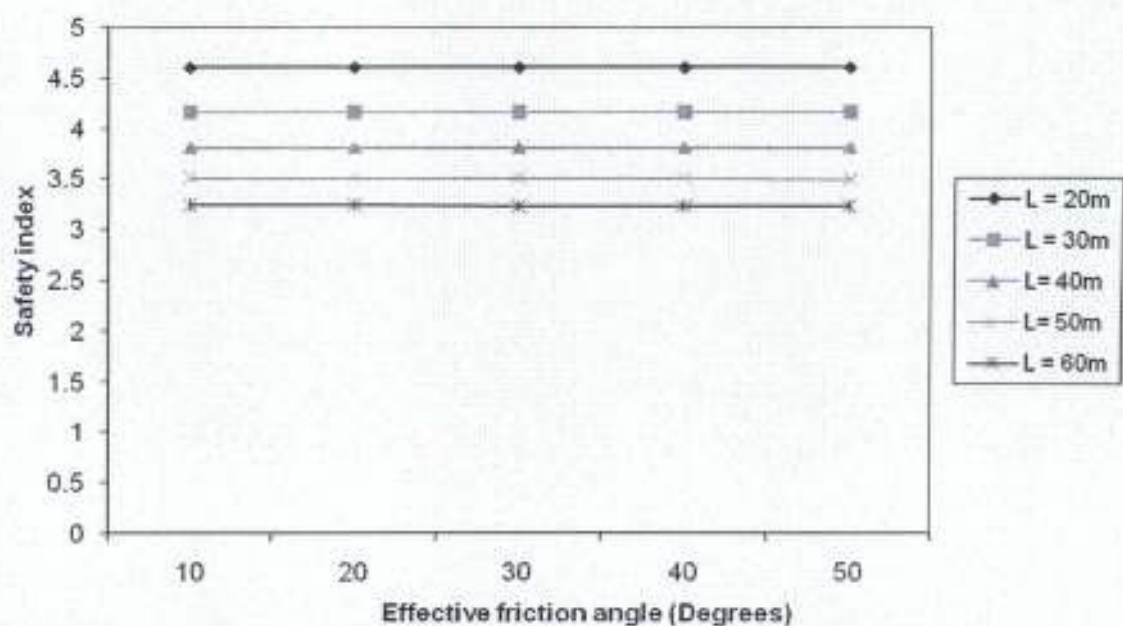


Fig. 5.33 - Safety Index (β_R) against Effective friction angle with varying

lengths

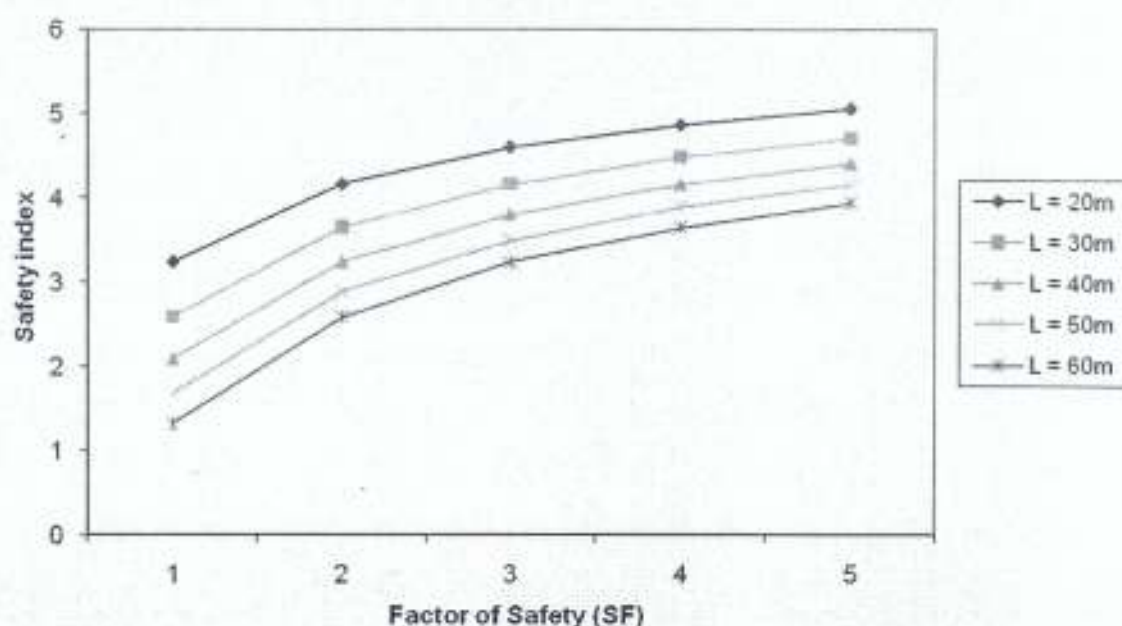


Fig. 5.34 - Safety Index (β_R) against Factor of safety with varying Lengths

Piling reliability/safety level decreases as pile length, unit weight of soil and the angle of internal friction increases (Figs.5.31 and 5.32) but the rate of decrease is more rapid for precast concrete than steel (Figs. 5.26 and 5.8). For concrete piling, pile length greater than 30 m will result in catastrophe while much longer steel piles that economy will permit are admissible.

5.6 Dynamic pile capacity using Hiley formula

The functional relationship between allowable design load and the allowable dynamic pile capacity using Hiley formula is expressed as:

$$G(x) = \text{Allowable Design Load} - \text{Allowable Pile Capacity}$$

So that,

$$G(x) = 0.35f_y A_p - \left\{ \frac{eh Eh}{S + \frac{1}{2}(K_1 + K_2 + K_3)} + \frac{W + n^2 W_p}{W + W_p} \right\} / SF$$

$$= 0.35f_y A_p - \left\{ \frac{eh Eh}{S + 0.5 \left(K_1 + \frac{P_u L}{AE} + K_3 \right)} + \frac{W + n^2 W_p}{W + W_p} \right\} / SF$$

(5.13)

Where:

A = Pile cross-sectional area, E = Modulus of Elasticity, e_h = Hammer efficiency, E_h = Manufacturers' hammer-energy rating, K_1 = elastic compression of capblock and pile cap and is a form of $P_u L / AE$, K_3 = elastic compression of soil, also termed quake for wave-equation, L = Pile length, n = Co-efficient of restitution, P_u = Ultimate pile capacity, s = amount of point penetration per blow, W_p = Weight of pile including weight of pile, cap, driving shoe, and capblock (also includes anvil for double-acting steam hammers), W_r = Weight of ram (for double-acting hammers include weight of casing).

Table 5.5 shows the assumed statistical values and their corresponding probability distributions.

Table 5.5 Stochastic model for dynamic pile capacity using Hilley formula

I	x_i	pdf	$\bar{X}$	v	$\sigma_X = \bar{X} \cdot v$
1	F_y	LN	$460 \times 10^3 \text{KN/m}^2$	0.15	6.9×10^4
2	A_p	N	$1.60 \times 10^{-2} \text{m}^2$	0.06	9.6×10^{-4}
3	e_h	N	0.84	0.06	5.04×10^{-2}
4	E_h	LN	33.12 KN/m	0.15	4.97
5	S	LN	$1.79 \times 10^{-2} \text{m}$	0.15	2.69×10^{-3}
6	K_1	LN	$4.06 \times 10^{-3} \text{m}$	0.15	6.09×10^{-4}
7	P_u	LN	950KN	0.15	1.425×10^2
8	L	N	12.18m	0.06	7.31×10^{-1}
9	E	LN	$209 \times 10^6 \text{KN/m}^2$	0.15	3.14×10^7
10	K_3	LN	$2.54 \times 10^{-3} \text{m}$	0.15	3.81×10^{-4}
11	W	Gumbel	80 KN	0.30	24
12	n	LN	0.5	0.15	7.5×10^{-2}
13	W_p	LN	18.5 KN	0.15	2.775
14	SF	LN	4.0	0.15	6.0×10^{-1}

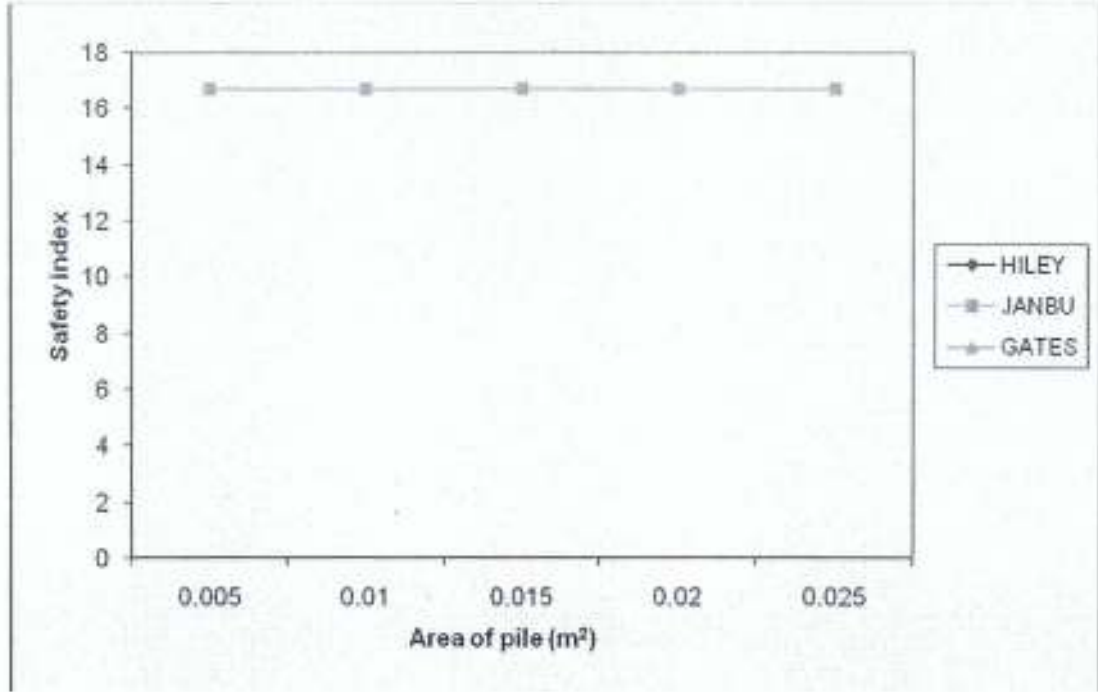


Fig. 5.35 - Safety index (β_R) against Area of pile using Hiley, Janbu and Gates formulae

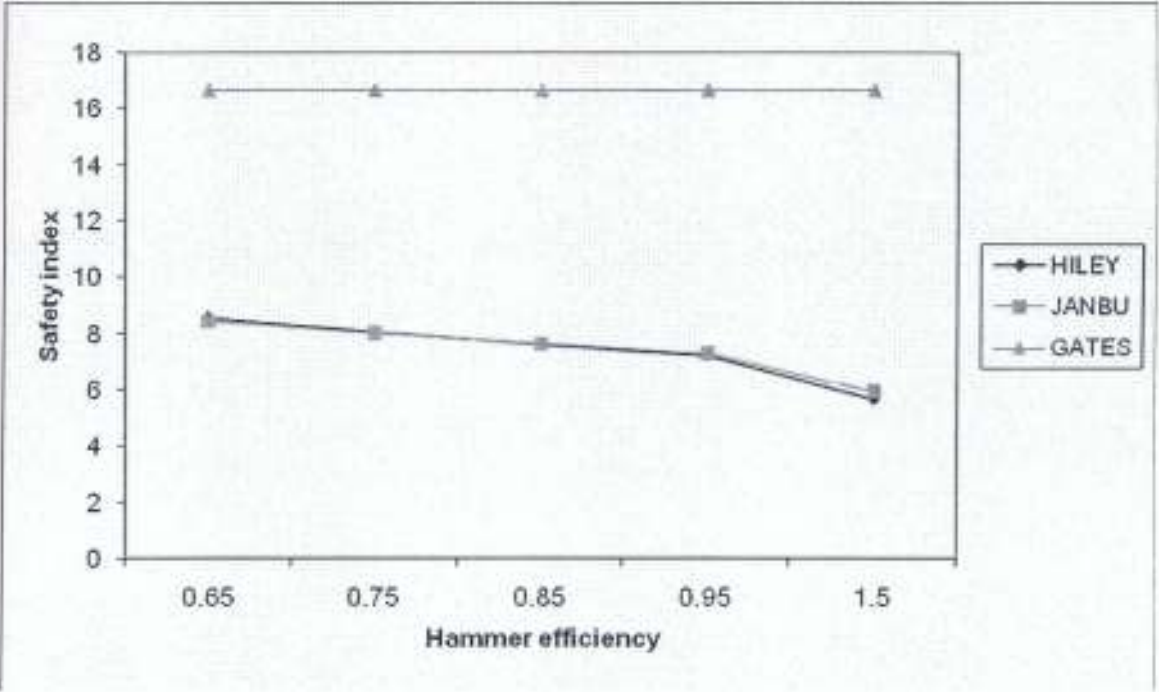


Fig. 5.36 - Safety index (β_R) against Hammer efficiency using Hiley, Janbu and Gates formulae

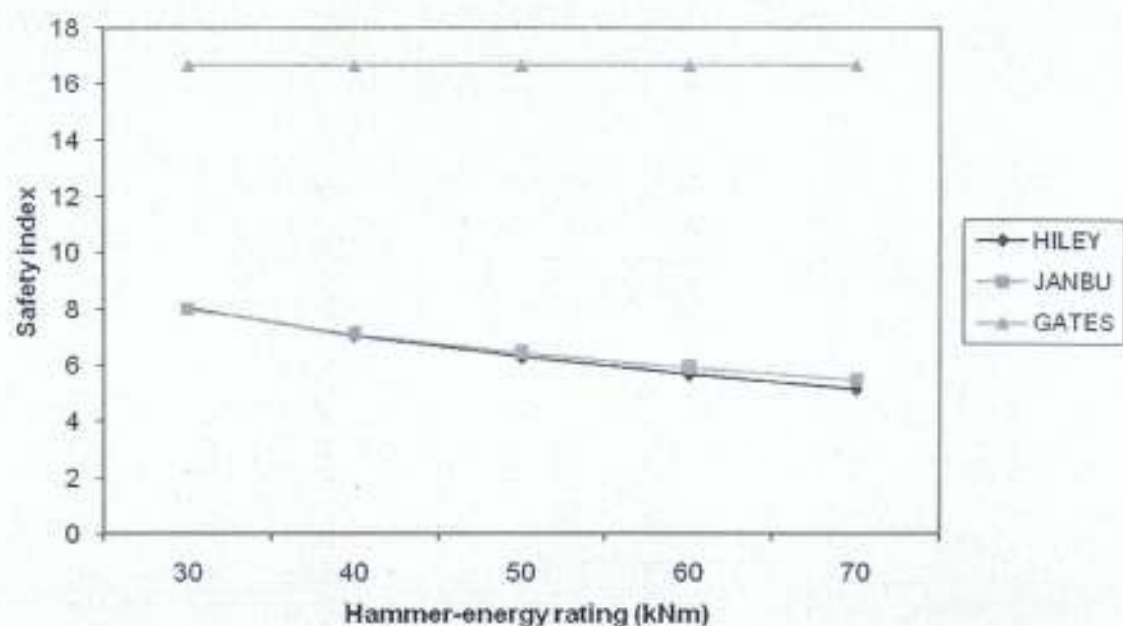


Fig. 5.37 - Safety index (β_R) against Hammer- energy rating using Hiley, Janbu and Gates formulae

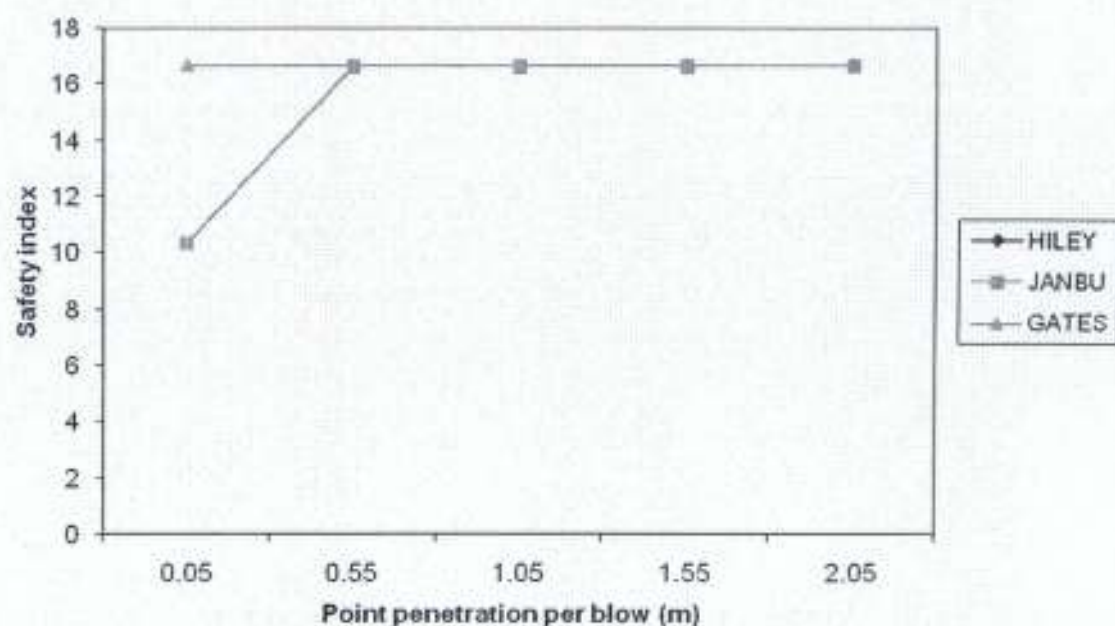


Fig. 5.38 - Safety index (β_R) against Point penetration per blow using Hiley, Janbu and Gates formulae

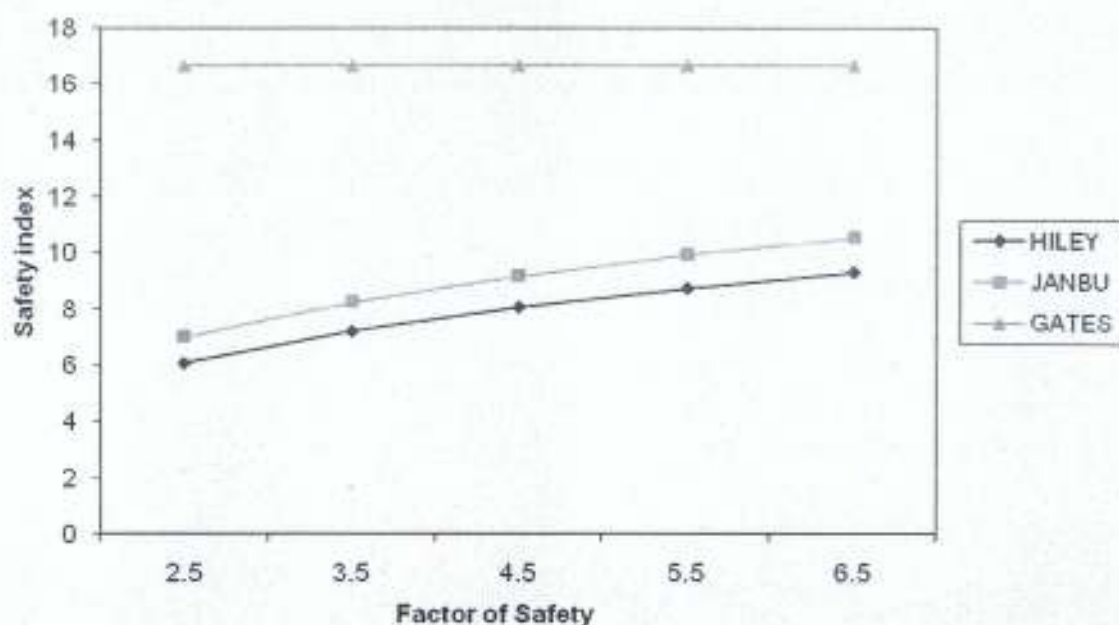


Fig. 5.39 - Safety Index (β_R) against Factor of safety using Hiley, Janbu and Gates formulae

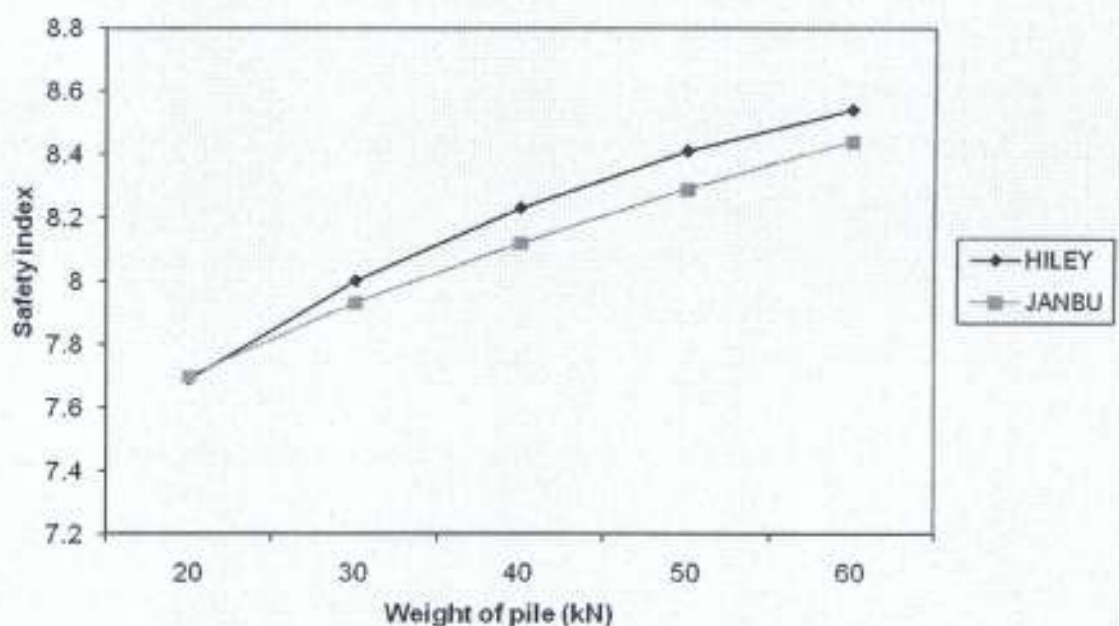


Fig. 5.40 - Safety Index (β_R) against Weight of pile using Hiley and Janbu formulae

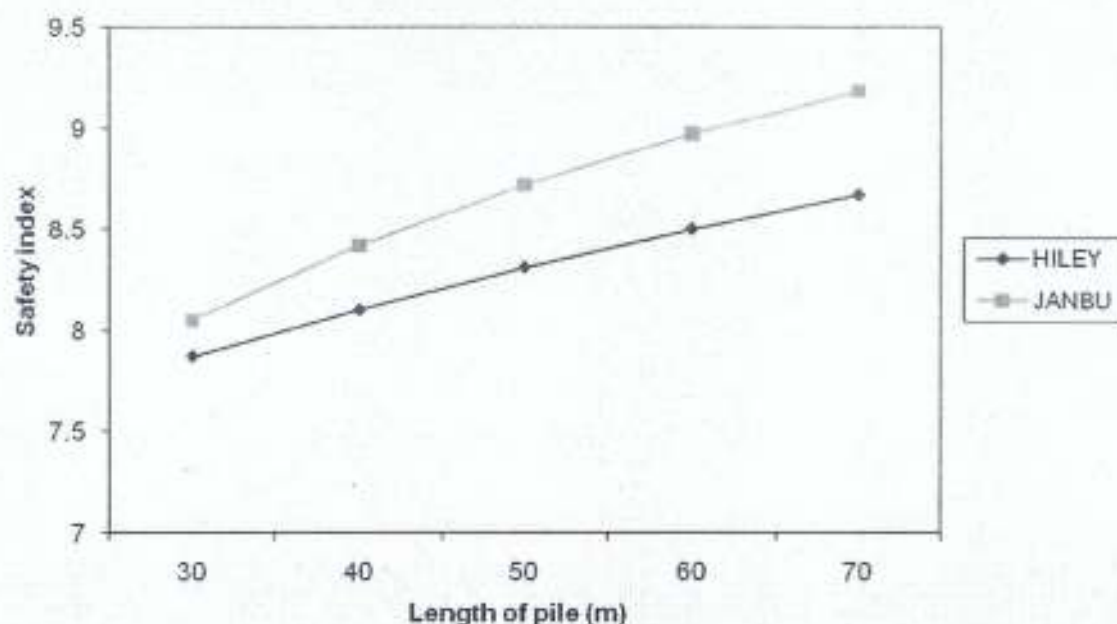


Fig. 5.41 - Safety Index (β_R) against Length of pile using Hiley and Janbu formulae

Hiley formula generally and grossly provides a very conservative pile capacity as seen in Fig. 5.35 to 5.41. Nevertheless, the safety level does not change with area of pile (Fig. 5.35) and the point penetration per blow (Fig. 5.38). As hammer efficiency and hammer energy rating increase, the safety level reduces significantly as in Fig. 5.36 and Fig. 5.37 respectively. On the other hand, safety level increases with increasing factor of safety as normally expected (Fig. 5.39), weight of pile (Fig. 5.40) and length of pile (Fig. 5.41).

5.7 Dynamic pile capacity using Janbu Formula

The functional relationship between the allowable design load and the allowable dynamic pile capacity using Janbu formula is expressed as follows:

$$G(x) = \text{Allowable Design Load} - \text{Allowable Pile Capacity}$$

So that,

$$G(x) = 0.35f_y A_p \left\{ \frac{eh E_h}{K_u S} \right\} / SF$$

$$= 0.35f_y A_p \left\{ eh E_h / \left(0.75 + 0.15 \frac{W_p}{W_r} \right) \left(1 + \sqrt{1 + \frac{eh E_h L / A E S^2}{0.75 + 0.15 \frac{W_p}{W_r}}} \right) S \right\} / SF \quad (5.14)$$

All the variables in Equation (5.14) are as defined in Equation (5.13)

In Table 5.6, the statistical and probabilistic descriptions of the variables in the functional relations are presented

Table 5.6 Stochastic model for dynamic using Janbu formula

<i>I</i>	<i>x_i</i>	pdf	$\bar{x}$	<i>v</i>	$\sigma x = \bar{x} \cdot v$
1	<i>F_y</i>	LN	460 x 10 ³ KN/m ²	0.15	6.9 x 10 ⁴
2	<i>A_p</i>	N	1.60 x 10 ⁻² m ²	0.06	9.6 x 10 ⁻⁴
3	<i>e_h</i>	N	0.84	0.06	5.04 x 10 ⁻²
4	<i>E_h</i>	LN	33.12 KN/m	0.15	4.97
5	<i>W_p</i>	LN	18.5 KN	0.15	2.78
6	<i>W_r</i>	Gumbel	35.58 KN	0.30	1.07 x 10 ¹
7	<i>L</i>	N	12.18m	0.15	7.31 x 10 ⁻¹
8	<i>E</i>	LN	209 x 10 ⁶ KN/m ²	0.06	3.14 x 10 ⁷
9	<i>S</i>	LN	1.79 x 10 ⁻² m	0.15	2.69 x 10 ⁻³
10	<i>SF</i>	LN	6.0	0.15	9.0 x 10 ⁻¹

Just like the Hiley formula, Janbu formula shows generally and grossly conservative pile capacity (Figs. 5.35 to 5.41). Even though, the safety level does not change with area of pile and the point penetration per blow (Figs.

5.35 and 5.38), reduces as hammer efficiency and hammer rating increase (Figs. 5.36 and 5.37), increases as factor of safety, weight of pile and length of pile increases, but; Janbu's results provides lesser piling conservation compared to Hiley's with respect to weight of pile (Fig. 5.40), and higher in the cases of factor of safety and length of pile (Figs. 5.39 and 5.41)

5.8 Dynamic pile capacity using Gates Formula

The functional relationship between the allowable design load and the allowable dynamic pile capacity using Gates formula is expressed as follows:

$G(x)$ = Allowable Design Load – Allowable Pile Capacity

So that, $G(x)$ = Allowable Design Load – Allowable Pile Capacity

$$\begin{aligned}
 &= 0.35f_y A_p - \left\{ a \sqrt{eh Eh (b - \log S)} \right\} / SF \\
 &= 0.35f_y A_p - \left\{ a (eh Eh)^{1/2} (b - \log S) \right\} / SF
 \end{aligned}
 \tag{5.15}$$

In Table 5.7, the statistical and probabilistic descriptions of the variables in the functional relations are presented

Table 5.7 Stochastic model for dynamic using Gate formula

i	x_i	pdf	$\bar{X}$	v	$\sigma_x = \bar{X} \cdot v$
1	F_y	LN	$460 \times 10^3 \text{KN/m}^2$	0.15	6.9×10^4
2	A_p	N	$1.60 \times 10^{-2} \text{m}^2$	0.06	9.6×10^{-4}
3	α	N	1.05×10^{-1}	0.06	6.3×10^{-3}
4	e_h	N	0.85	0.06	5.1×10^{-2}
5	E_h	LN	33.12 KNm	0.15	4.97
6	b	N	$2.4 \times 10^{-3} \text{m}$	0.06	1.44×10^{-4}
7	S	LN	$1.79 \times 10^{-2} \text{m}$	0.15	2.69×10^{-3}
8	SF	LN	3.0	0.15	4.5×10^{-1}

The implied safety level associated with piling capacity using Gates' formula is grossly conservative, even much more than both Hiley and Janbu formulae. The safety level does not change with the area of pile, hammer efficiency, hammer-energy rating and point penetration per blow (Figs. 5.35 to 5.38).

5.9 Discussion of results- Comparison of Piling Options

5.9.1 Static Pile Capacity of Concrete in Cohesive and Cohesionless Soils

It is interesting to note that it is unsafe to use concrete pile in cohesionless soils as can be seen in Fig. 5.8 compared to Fig. 5.1. The high safety levels indicate very conservative prediction of pile capacity, the consequence of this is an unnecessary economic involvement.

5.9.2 Static Pile Capacity of Steel in Cohesive and Cohesionless Soils.

The bigger the area of pile, the more conservative the static pile capacity for steel in cohesive soils (Figs. 5.17- 5.23). This is not the case where only sensitivity of the safety level to pile length is noted (Figs. 5.30-5.34).

5.9.3 Static Pile Capacity for Concrete and Steel in Cohesive Soils.

For Steel piling in cohesive soils the safety level is higher than for concrete piling and longer lengths of piles can be used for steel piling than for concrete piling (Figs. 5.7 and 5.21).

5.9.4 Static Pile Capacity for Concrete and Steel in Cohesionless Soils.

Piling capacity is grossly conservative for steel in cohesionless soils (Figs. 5.26 and 5.8). It is better to use steel piles in cohesionless soils than concrete piles.

5.9.5 Dynamic Pile Capacity using Hiley and Janbu Formulae

There is a correlation between the implied safety levels in Hiley and Janbu formulae. The safety levels are the same for both for a given area of pile, hammer efficiency, point penetration per blow, slightly different with hammer-energy rating, and clearly different with factor of safety, weight of pile and length of pile (Figs. 5.35- 5.41). Janbu formula leads to higher safety level than Hiley's for increasing pile length and safety factor.

5.9.6 Dynamic Pile Capacity using Hiley, Janbu and Gates Formulae.

The dynamic predictions of Hiley and Janbu lead to similar safety level while Gates' results in totally different implied safety levels. These observations are clearly noted in Figs. 5.35 to 5.41.

CONCLUSIONS AND RECOMMENDATIONS



6.1 Conclusions

From the results of this study, the following conclusions are hereby drawn:

- Precast concrete piling should be discouraged in cohesionless soils, and if it must be used at all, the pile length should not exceed 20 m
- Precast concrete piles in cohesive soils show piling safety that is grossly conservative, but the static pile capacity equations are very expensive. In cohesive soils, pile length that is as much as 50 m is still theoretically, if not economically admissible.
- Longer lengths of piles can be used for steel piling than for concrete piling, particularly in cohesive soils.
- It is better to use steel piles than concrete piles in cohesionless soils.
- There is a correlation between the implied safety levels in Hiley and Janbu formulae, while Gates' formula results in totally different implied safety levels.
- The pile capacity is an interactive function of the properties of the soil and the pile, both governed by pile length.
- There is an unsatisfactory prediction in pile capacity by dynamic formulae.

6.2 Recommendations

The following recommendations are hereby made:

- It is imperative that in choosing a design approach, the designer consider the functional significance of the project, the degree of uncertainty in the design parameters and the analytical approach, and probability of failure due to both collapse and functional inadequacy.
- Pile capacities should be computed by experienced designers thoroughly familiar with the various types of piles, how piles

behave when loaded, and the soil conditions that exist at the site.

- Except where well supported empirical correlations under a given set of physical and geological conditions are available, dynamic formulae should not be used.
- In safety classification, piling attracts a high degree of safety and as such, any design procedure should be able to afford safety index not less than 4.2 (see for example, NKB, 1978).
- Deterministic method is not a very cost-effective way to produce reliable structures, therefore, reliability-based design should be adopted to quantify reliability, and thus used to develop balance designs that are both more reliable and less expensive.

- Afolayan, J.O. (1999) "Economic Efficiency of Glued Joints in Timber Truss Systems", *Building and Environment: The International of Building Science and its Application*, Vol. 34, No. 2, pp. 101 – 107.
- Afolayan, J.O. and Abubakar I. (2003) "Reliability Analysis of Reinforced Concrete One-Way slabs; The Ultimate Conditions", *Nigerian Journal of Engineering*, Vol. 11, No. 1, pp. 29 – 37.
- Afolayan J.O. (2003) "Improved Design Format for Doubly Symmetric Thin-Walled Structural Steel Beam-Columns", *Botswana Journal of Technology* Vol. 12, No. 1, pp. 36 – 43.
- Afolayan J.O. and Opeyemi D.A. (2008a) "Reliability Analysis of Static Pile Capacity of Concrete in Cohesive and Cohesionless Soils", *Research Journal of Applied Sciences*, Vol.3, No. 5, pp. 407-411.
- Afolayan J.O. and Opeyemi D.A. (2008b) "Reliability Analysis of Static Pile Capacity for Concrete and Steel in Cohesionless Soils" *Proceedings of the 3rd International GEO-CHIANGMAI Conference on Geotechnical and Geoenvironmental Engineering, Rock Mechanics and Engineering Geology, Chiangmai, Thailand.* (Reviewed and accepted for publishing in the *Proceedings and inclusion as a full paper*).
- Ang, A.H.S., and Tang, W.H. (1990) *Probability Concepts in Engineering Planning and Design: Vol. II – Decision, risk, and reliability.* John Wiley, New York.
- API (1984), *API Recommended Practice for planning, Designing and Constructing Fixed Offshore Platforms*, 15/e, American Petroleum Institute, API RP2A, 115pp.
- ASCE (1941), "Pile driving Formulas" *Proc. ASCE*, Vol. 67, No. 5, May, pp. 853-866.
- Ayyub, B.M., and Haldar, A. (1984) "Practical Structural Reliability Techniques", *Journal of Structural Engineering*, ASCE, Vol. 110, No. 8, pp. 1707 – 1724.
- Ayyub, B.M., and Patev, C.R. (1998) "Reliability & Stability Assessment of Concrete Gravity Structures." (RCSLIDE): Theoretical Manual, US Army Corps of Engineers Technical Report ITL-98-6.
- Ayyub, B.M., and McCuen, R.H. (1997). *Numerical Methods for Engineers.* Prentice Hall, Upper Saddle River, N.J.
- Basler, B. (1961) *Untersuchungen uber den sicherheit-sbergreif von bauwerken*, *Schweiz. Arch.*, 27 (4) 133-160.
- Beringen F.L., Hooydonk Van W.R. and Schaap L.H.J. (1980) *Dynamic Pile Testing: An aid in a driving behavior.* H. Bredenberg (ed.). *Proceedings of the International Seminar on the Application of Wave Theory on Piles.* A.A. Balkema, Rotterdam, 77 – 98.

- Bowles, J.E. (1988) Foundation Analysis and Design. 4th ed., Mc Graw-Hill Book Company, Singapore.
- Briaud J.L. and Tucker L.M. Measured and Predicted Axial Response of 98 Piles. Journal of Geotechnical Engineering, ASCE, 1988, Vol. 114, No. 9, 984 – 1001.
- Bhushan, K. (1982), "Discussion: New Design Correlations for Pile in Sands". JGED, ASCE, GT11, Nov., pp. 1508 – 1510
- Burland, J.B. (1973), "Shaft Friction Piles in Clay – A Simple Fundamental Approach". Ground Engineering, Vol. 6 No. 3, pp. 30-42
- Chellis, R.D. (1961) Pile Foundations, 2nd ed., McGraw Hill, New York.
- Coduto, D.P. (2001) Foundation Design: Principles and practices. 2nd ed. Prentice – Hall Inc., New Jersey.
- Cornell, C.A. (1969) A Probability based Structural Code. Journal of American Concrete Institute, Vol. 66, No. 12, pp. 974 – 985.
- Elhewy, A.H., Mesbahi, E., and Pu, Y. (2006) "Reliability Analysis of Structure Using Neural Network Method". Journal of Probabilistic Engineering Mechanics, ELSEVIER, Vol. 21, pp. 44 – 53.
- ENR (1965), "Michigan Pile Test Program Test Results are Released," Eng. News – Record, May 20, pp. 26-28, 33-34.
- Esrig, M. L. and Kirby, R.C., (1979). "Soil Capacity for Supporting Deep Foundation Members in Clay," ASTM STP No. 670, pp. 27-63
- Farid Uddin, A.K.M. (2000) "Risk & Reliability Based Structural and Foundation Design of a Water Reservoir (capacity: 10 million gallon) on the top of Bukit Permatha Hill in Malaysia" 8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability.
- Flaate, K. and Selnes, P. (1977), "Side Friction of Piles in Clay," 9th ICSMFE, Vol. 1, pp. 517-522.
- Fox E.N. (1932) Stresses Phenomena Occurring in Pile Driving. Engineering, September, 263 – 265.
- Frendenthal, A.M. (1956) Safety and the Probability of Structural failure. Trans. ASCE, 121, 1337 – 1397.
- Gates, M. (1957), "Empirical Formula for Predicting Pile Bearing Capacity," Civil Engineering, ASCE, Vol.27, No. 3, March, pp. 65 – 66.
- Granville W.H., Grime G., Fox E.N. and Davies W.W. (1938) An investigation of the stresses in reinforced concrete piles during driving. British Building Research, Technical Paper No. 20

- Goble G.G. and Rausche F. (1976) Wave Program Document. National Information Service, Washington D.C.
- Goble G.G., Likins G.E. and Rausche F. (1975) Bearing Capacity of Piles from Dynamic Measurement Report, Department of Civil Engineering, Case Western Reserve University, Cleveland, Ohio.
- Goble G.G., Rausche F., and Likins G. (1980) The analysis of Pile Driving – A State-of-the-Art. Brendenberg H. (ed.) Proceedings of the First International Conference on the Application of Stress-Wave Theory . Stockholm, A.A. Balkema, 131 – 161.
- Hannigan P.J. and Webster S.D. (1987) Comparison of Static Load Test and Dynamic Pile Testing Results. Proceedings of the 2nd International Symposium, DFI, Luxembourg, May.
- Hannigan P.J. (1990) Dynamic monitoring and analysis of pile foundation installations. DFI Short Course Text.
- Hannigan P.L.J, Goble G.G., Thendean G., Likins G.E. and Rausche F. (1996) Design and construction of driven pile foundations. Workshop manual, Publication No. FHWA-HI-97-014.
- Hiley, A. (1930), "Pile-Driving Calculations with Notes on Driving Forces and Ground Resistances", The Structural Engineer, London, Vol. 8, pp. 246 – 259, 278-288.
- Holeyman A.E., (1992) Keynote lecture: "Technology of Pile Dynamic Testing". F. Barends (ed.), Procedure of the Fourth International Conference on the Application of Stress- Wave Theory to Piles, A.A. P. Rotterdam, 195-215.
- Holloway D.M., Clough G.W. and Vesic A.S. (1979) A Rational Procedure for Evaluating the Behaviour of Impact-driven Piles. Special Technical Publication 670, ASTM, 335-357. Ireland, H.O. (1957), "Pulling Tests on Piles in Sand," 4th ICSMFE, Vol.2, pp. 43-45. Isaacs D.V. (1939) Reinforced Concrete Pile Formulae. Trans. of the Inst. Of Engineers, Australia, Paper No. 370, 312-323.
- Lacasse, S., and Nadim, F. (2007) "Probabilistic Geo-technical Analyses for Offshore Facilities" GEORISK: Assessment and Management of Risk for Engineered System and Geohazards, Vol. 1, No. 1, pp. 21 – 42.
- Lee W., Lee I.M. Yoon S.J., Choi Y.J. and Kwon L.H.(1996). "Bearing Capacity Evaluation of the Injected Pile using CAPWAP", F. Townsend, M. Hussein & M. Mc Vay (eds.), Proceedings of the Fifth International Conference on the Application of Stress-Wave Theory to Piles, Orlando, University of Florida, 409 – 419.
- Liang R.Y. and Zhou J. (1997) "Probability Method Applied to Dynamic Pile-Driving Control". Journal of Geotechnical Engineering, ASCE, Vol.123, No. 2, 137-144.
- Liu C., Lin Q., and Shi F.(). "Determining the Bearing Capacity of Large-diameter bored cast-in-situ-high-strain Dynamic Pile Testing". F. Townsend, M.

- Hussein & M. Mc Vay (eds.), Proceedings of the Fifth International Conference on the Application of Stress-Wave Theory to Piles, Orlando, University of Florida 797 - 804.
- Mansur, C.L. and Hunter, A.H. (1970), "Pile Tests – Arkansas River Project," JSMFD, Vol. 96, SM5, Sept., pp. 142 – 146.
- Mayer, H. (1926) "Die Sicherheit-der Bauwerker", Springer, Berlin.
- Melchers, R.E. (2002) Structural Reliability, and Prediction. 2nd ed. John Wiley, England.
- NKB (1978)"Recommendation for Loading and Safety Regulations for Structural Design", NKB-Report No. 36, The Nordic Committee on Building Regulations.
- Olson, R.E. and Flaate, K.S. (1967), "Pile Driving Formulas for Friction Piles in Sand," JSMFD, ASCE, Vol. 93, SM6, Nov., pp. 279 – 296.
- Paikowsky S.G. and Chernauskas L.R.(1992). "Energy approach for capacity evaluation of driven piles". F. Barends (ed.), Proceedings of Fourth International Conference on the Application of Stress-Wave Theory to Piles, A.A. Balkema, The Hague, 595-601.
- Paikowsky S.G., Regan J.E. and McDonnell J.J. (1994). "A simplified field method for capacity evaluation of driven piles". Publication No. FHWA-RD-94-042.
- Paloheimo, E., and Hannus, M. (1974) "Structural Design based on Weighted Fractiles". Journal of Structural Division, ASCE Vol.100, No. 7, pp. 1367-1378.
- Patev, R.C. (1995) "Reliability Assessment of Pile-Founded Navigation Structures" Proceedings of the International Symposium of Uncertainty Modeling and Analysis and NAFIPS '95 (ISUMA '95)
- Rausche F., Thendean G., Abou-Matar H., Linkns G.E. and Goble, G.G (1996), "Determining Pile Driveability and Capacity from Penetration Tests". Final Report, FHWA Contract No. DTFH61-91 – C – 00047.
- Rausche F., Goble, G.G. and Likins, G. Dynamic Determination of Pile Capacity. Journal on Geotechnical Engineering, ASCE, 1985, Vol. 111 (3), 367-383.
- Rzhanitzyn, R. (1957) It is Necessary to Improve the Standard of Design of Building Structures, A Statistical Method of Design of Building Structures, Allan, D.E. (translation). Technical Translation No. 1368, National Research Council of Canada, Ottawa.
- Smith E.A.L. (1955) "Impact and Longitudinal wave transmission". Transaction ASCE, August, 963 - 973

- Smith E.A.L. (1960) "Pile Driving Analyses by the Wave Equation". Journal of the Soil Mechanics and Foundation Division, ASCE, vol. 86, 35-61.
- Svinkin M.R.(1997). "Time-Dependent Capacity of Piles in Clayey Soils by Dynamic Methods". Proceedings of the XIVth International Conference on Soil Mechanics and Foundation Engineering, Hamburg, Germany, September, Vol. 2, 1045- 1048.
- Tavenas, F.A. (1971), "Load Test Results on Friction Piles in Sand", CGJ, Ottawa, Vol. 8, No. 1, Feb., pp. 7 - 22
- Timoshenko S. and Goodier J.N. (1951) Theory of Elasticity, Second ed., McGraw-Hill Book Co. New York.
- Tomlinson, M.J. (1971), "Some Effects of Pile Driving on Skin Friction" Proceedings of the Conference on Behaviour of Piles, ICE London, pp 107 – 114.
- Vijayvergia, V.N., and Focht, J.A. (1972) "A New Way to Predict Capacity of Piles in Clay", OTC Paper 1718, 4th Offshore Technology Conference, Houston, TX.
- Whitaker, T. (1970) The Design of Piled Foundations. Pergamon, Oxford.
- White, G.J., and Ayyub, B.M. (1985) "Reliability Methods for Ship Structures", Naval Engineers Journal pp. 86 – 96.
- Zeitlen, J.G. and Paikowsky, S. (1982), "Discussion: New Design Correlations for Piles in Sands", JGED, ASCE, Vol. 108, GT 11, Nov., pp. 1515 – 1518.
- Zhang, L.M., and Ng, A.M.Y. (2005) "Probabilistic Limiting tolerable Displacements for Serviceability Limit State Design of Foundation". Thomas Telford Geotechnical Journals, Vol. 55, No. 2, pp. 151-161.

APPENDIX
RESULTS OF RELIABILITY ANALYSIS

Table 1: Static pile capacity of concrete piling in cohesive soils.

D (mm)	L (m)	β	P_t
200	10	10.13	2.15×10^{-28}
	20	7.78	3.53×10^{-15}
	30	6.41	7.18×10^{-11}
	40	5.44	2.68×10^{-6}
	50	4.69	1.40×10^{-4}
300	10	10.93	4.83×10^{-28}
	20	8.63	3.11×10^{-18}
	30	7.29	1.63×10^{-13}
	40	6.33	1.24×10^{-10}
	50	5.59	1.15×10^{-8}
400	10	11.61	1.92×10^{-31}
	20	9.34	4.72×10^{-21}
	30	8.02	5.53×10^{-16}
	40	7.07	7.64×10^{-13}
	50	6.34	1.15×10^{-10}
500	10	12.18	1.97×10^{-34}
	20	9.94	1.38×10^{-23}
	30	8.63	3.19×10^{-18}
	40	7.69	7.31×10^{-12}
	50	6.97	1.64×10^{-12}
600	10	12.68	4.20×10^{-37}
	20	10.45	7.22×10^{-26}
	30	9.15	2.98×10^{-20}
	40	8.22	1.05×10^{-16}
	50	7.50	3.33×10^{-14}

D (mm)	S_u (kN/m ²)	β	P_f
200	10	7.94	1.06×10^{-15}
	20	5.57	1.30×10^{-6}
	30	4.19	1.43×10^{-5}
	40	3.21	6.75×10^{-4}
	50	2.45	7.23×10^{-3}
300	10	8.78	8.30×10^{-19}
	20	6.45	5.66×10^{-11}
	30	5.09	1.81×10^{-7}
	40	4.12	1.86×10^{-6}
	50	3.38	3.66×10^{-4}
400	10	9.94	1.13×10^{-21}
	20	7.19	3.36×10^{-13}
	30	5.84	2.65×10^{-9}
	40	4.88	5.23×10^{-7}
	50	4.14	1.72×10^{-5}
500	10	10.09	3.02×10^{-24}
	20	7.80	3.13×10^{-15}
	30	6.46	5.29×10^{-11}
	40	5.51	1.81×10^{-8}
	50	4.77	9.15×10^{-7}
600	10	10.60	1.45×10^{-26}
	20	8.32	4.45×10^{-17}
	30	6.98	1.44×10^{-12}
	40	6.04	7.91×10^{-10}
	50	5.30	5.77×10^{-8}

00	α	β	P_f
	0.4	10.41	1.17×10^{-26}
	0.8	8.07	3.66×10^{-18}
	1.2	6.70	1.09×10^{-11}
	1.6	5.72	5.03×10^{-9}
	2.0	4.97	3.41×10^{-7}
00	0.4	11.20	2.02×10^{-26}
	0.8	8.91	2.67×10^{-18}
	1.2	7.56	7.56×10^{-14}
	1.6	6.61	1.99×10^{-11}
	2.0	5.87	2.25×10^{-9}
00	0.4	11.88	7.78×10^{-26}
	0.8	9.62	3.45×10^{-18}
	1.2	8.29	5.77×10^{-17}
	1.6	7.35	1.03×10^{-18}
	2.0	6.62	1.87×10^{-11}

200	SF	β	P_i
	1	3.56	1.83×10^{-4}
	2	5.93	1.55×10^{-9}
	3	7.31	1.33×10^{-13}
	4	8.30	5.50×10^{-17}
	5	9.06	6.73×10^{-20}
300	1	4.48	3.78×10^{-4}
	2	6.80	5.16×10^{-12}
	3	8.17	1.61×10^{-17}
	4	9.14	3.33×10^{-20}
	5	9.89	2.42×10^{-23}
400	1	5.23	8.37×10^{-8}
	2	7.54	2.43×10^{-14}
	3	8.89	3.20×10^{-19}
	4	9.85	3.67×10^{-23}
	5	10.59	1.71×10^{-26}
500	1	5.86	2.37×10^{-9}
	2	8.15	1.87×10^{-14}
	3	9.49	1.18×10^{-21}
	4	10.44	8.13×10^{-26}
	5	11.18	2.58×10^{-29}
600	1	6.38	8.70×10^{-11}
	2	8.67	2.25×10^{-18}
	3	10.00	7.51×10^{-24}
	4	10.95	3.34×10^{-28}
	5	11.69	7.57×10^{-32}

L (m)	α	β	P_r
10	0.4	14.74	1.46×10^{-49}
	0.8	11.67	9.94×10^{-32}
	1.2	10.32	2.81×10^{-25}
	1.6	9.37	3.68×10^{-21}
	2.0	8.63	3.11×10^{-18}
20	0.4	11.67	9.94×10^{-32}
	0.8	9.37	3.68×10^{-21}
	1.2	8.03	5.14×10^{-16}
	1.6	7.07	7.81×10^{-13}
	2.0	6.33	1.24×10^{-10}
30	0.4	10.32	2.81×10^{-25}
	0.8	8.03	5.14×10^{-16}
	1.2	6.68	1.21×10^{-11}
	1.6	5.72	5.22×10^{-9}
	2.0	4.98	3.134×10^{-7}
40	0.4	9.37	3.68×10^{-21}
	0.8	7.07	7.81×10^{-13}
	1.2	5.72	5.22×10^{-9}
	1.6	4.77	9.24×10^{-7}
	2.0	4.03	2.79×10^{-5}
50	0.4	8.63	3.11×10^{-18}
	0.8	6.33	1.24×10^{-10}
	1.2	4.98	3.13×10^{-7}
	1.6	4.03	2.79×10^{-5}
	2.0	3.29	5.00×10^{-4}

(m)	SF	β	P_i
10	1	7.21	2.86×10^{-13}
	2	9.55	6.39×10^{-22}
	3	10.93	4.38×10^{-28}
	4	11.90	5.91×10^{-38}
	5	12.66	5.11×10^{-47}
20	1	4.94	3.98×10^{-7}
	2	7.27	1.88×10^{-19}
	3	8.63	3.11×10^{-17}
	4	9.60	4.08×10^{-22}
	5	10.35	2.09×10^{-26}
30	1	3.60	1.57×10^{-4}
	2	5.92	1.58×10^{-9}
	3	7.28	1.63×10^{-13}
	4	8.25	7.96×10^{-17}
	5	9.00	1.13×10^{-19}
40	1	2.66	3.94×10^{-3}
	2	4.97	3.32×10^{-7}
	3	6.33	1.24×10^{-10}
	4	7.29	1.25×10^{-13}
	5	8.04	4.44×10^{-15}
50	1	1.92	2.72×10^{-2}
	2	4.23	1.15×10^{-6}
	3	5.59	1.15×10^{-8}
	4	6.55	2.86×10^{-11}
	5	7.30	1.46×10^{-13}

10	S_v (KN/m ²)	β	P_r
	10	11,55	$4,01 \times 10^{-21}$
	20	9,20	$1,88 \times 10^{-20}$
	30	7,82	$2,57 \times 10^{-18}$
	40	6,85	$3,67 \times 10^{-12}$
	50	6,10	$5,41 \times 10^{-10}$
20	10	9,24	$1,20 \times 10^{-20}$
	20	6,91	$2,43 \times 10^{-12}$
	30	5,55	$1,45 \times 10^{-8}$
	40	4,58	$2,29 \times 10^{-4}$
	50	3,84	$6,28 \times 10^{-2}$
30	10	7,90	$1,45 \times 10^{-18}$
	20	5,57	$1,27 \times 10^{-8}$
	30	4,21	$1,26 \times 10^{-4}$
	40	3,25	$5,74 \times 10^{-4}$
	50	2,51	$6,09 \times 10^{-3}$
40	10	6,94	$1,97 \times 10^{-12}$
	20	4,62	$1,92 \times 10^{-6}$
	30	3,27	$5,46 \times 10^{-4}$
	40	2,31	$1,05 \times 10^{-2}$
	50	1,57	$5,79 \times 10^{-2}$
50	10	6,20	$2,86 \times 10^{-10}$
	20	3,88	$4,18 \times 10^{-6}$
	30	2,53	$5,70 \times 10^{-4}$
	40	1,57	$5,79 \times 10^{-2}$
	50	0,83	$2,03 \times 10^{-1}$

Table 2: Static pile capacity of concrete piling in cohesionless soils.

D (mm)	L (m)	β	P_v
30	10	2.39	8.49×10^{-2}
	20	1.04	1.49×10^{-1}
	30	0.16	4.37×10^{-1}
	40	-0.50	6.93×10^{-1}
	50	-1.04	8.50×10^{-1}
30	10	2.22	1.31×10^{-2}
	20	0.88	1.89×10^{-1}
	30	0.02	4.93×10^{-1}
	40	-0.63	7.35×10^{-1}
	50	-1.14	8.73×10^{-1}
40	10	2.16	1.54×10^{-2}
	20	0.82	2.05×10^{-1}
	30	-0.03	5.14×10^{-1}
	40	-0.67	7.49×10^{-1}
	50	-1.18	8.81×10^{-1}
50	10	2.13	1.66×10^{-2}
	20	0.80	2.13×10^{-1}
	30	-0.06	5.23×10^{-1}
	40	-0.69	7.55×10^{-1}
	50	-1.19	8.84×10^{-1}
60	10	2.11	1.73×10^{-2}
	20	0.78	2.18×10^{-1}
	30	-0.07	5.28×10^{-1}
	40	-0.70	7.58×10^{-1}
	50	-1.20	8.85×10^{-1}

D (mm)	α (kN/m ²)	β	P_i
200	10	2.71	3.41×10^{-3}
	15	2.01	2.25×10^{-2}
	20	1.47	7.0737×10^{-2}
	25	1.04	1.50×10^{-1}
	30	0.67	2.52×10^{-1}
300	10	2.71	3.41×10^{-3}
	15	2.01	2.25×10^{-2}
	20	1.47	7.0737×10^{-2}
	25	1.04	1.50×10^{-1}
	30	0.67	2.52×10^{-1}
400	10	2.71	3.41×10^{-3}
	15	2.01	2.25×10^{-2}
	20	1.47	7.0737×10^{-2}
	25	1.04	1.50×10^{-1}
	30	0.67	2.52×10^{-1}
500	10	2.71	3.41×10^{-3}
	15	2.01	2.25×10^{-2}
	20	1.47	7.0737×10^{-2}
	25	1.04	1.50×10^{-1}
	30	0.67	2.52×10^{-1}
600	10	2.71	3.41×10^{-3}
	15	2.01	2.25×10^{-2}
	20	1.47	7.0737×10^{-2}
	25	1.04	1.50×10^{-1}
	30	0.67	2.52×10^{-1}

D (mm)	Φ (°)	β	P_i
20	10	8.58	4.68×10^{-18}
	20	4.78	8.60×10^{-7}
	30	2.47	6.74×10^{-3}
	40	0.79	2.14×10^{-1}
	50	-0.53	7.02×10^{-1}
100	10	8.58	4.68×10^{-18}
	20	4.78	8.60×10^{-7}
	30	2.47	6.74×10^{-3}
	40	0.79	2.14×10^{-1}
	50	-0.53	7.02×10^{-1}
400	10	8.58	4.68×10^{-18}
	20	4.78	8.60×10^{-7}
	30	2.47	6.74×10^{-3}
	40	0.79	2.14×10^{-1}
	50	-0.53	7.02×10^{-1}
500	10	8.58	4.68×10^{-18}
	20	4.78	8.60×10^{-7}
	30	2.47	6.74×10^{-3}
	40	0.79	2.14×10^{-1}
	50	-0.53	7.02×10^{-1}
600	10	8.58	4.68×10^{-18}
	20	4.78	8.60×10^{-7}
	30	2.47	6.74×10^{-3}
	40	0.79	2.14×10^{-1}
	50	-0.53	7.02×10^{-1}

Q (mm)	δ (°)	β	Pr
200	10	1,74	$4,06 \times 10^{-2}$
	20	1,74	$4,06 \times 10^{-2}$
	30	1,74	$4,06 \times 10^{-2}$
	40	1,74	$4,06 \times 10^{-2}$
	50	1,74	$4,06 \times 10^{-2}$
300	10	1,74	$4,06 \times 10^{-2}$
	20	1,74	$4,06 \times 10^{-2}$
	30	1,74	$4,06 \times 10^{-2}$
	40	1,74	$4,06 \times 10^{-2}$
	50	1,74	$4,06 \times 10^{-2}$
400	10	1,74	$4,06 \times 10^{-2}$
	20	1,74	$4,06 \times 10^{-2}$
	30	1,74	$4,06 \times 10^{-2}$
	40	1,74	$4,06 \times 10^{-2}$
	50	1,74	$4,06 \times 10^{-2}$
500	10	1,74	$4,06 \times 10^{-2}$
	20	1,74	$4,06 \times 10^{-2}$
	30	1,74	$4,06 \times 10^{-2}$
	40	1,74	$4,06 \times 10^{-2}$
	50	1,74	$4,06 \times 10^{-2}$
600	10	1,74	$4,06 \times 10^{-2}$
	20	1,74	$4,06 \times 10^{-2}$
	30	1,74	$4,06 \times 10^{-2}$
	40	1,74	$4,06 \times 10^{-2}$
	50	1,74	$4,06 \times 10^{-2}$

200	D (mm)	SF	β	P_i
		1	-0.49	6.86×10^{-1}
		2	0.96	1.68×10^{-1}
		3	1.74	4.06×10^{-2}
		4	2.26	1.18×10^{-2}
300		5	2.64	4.10×10^{-3}
		1	-0.49	6.86×10^{-1}
		2	0.96	1.68×10^{-1}
		3	1.74	4.06×10^{-2}
		4	2.26	1.18×10^{-2}
400		5	2.64	4.10×10^{-3}
		1	-0.49	6.86×10^{-1}
		2	0.96	1.68×10^{-1}
		3	1.74	4.06×10^{-2}
		4	2.26	1.18×10^{-2}
500		5	2.64	4.10×10^{-3}
		1	-0.49	6.86×10^{-1}
		2	0.96	1.68×10^{-1}
		3	1.74	4.06×10^{-2}
		4	2.26	1.18×10^{-2}
600		5	2.64	4.10×10^{-3}

L (m)	α (kN/m ²)	β	P_1
10	10	3.06	1.11×10^{-2}
	15	2.39	8.34×10^{-3}
	20	1.88	3.02×10^{-2}
	25	1.46	7.26×10^{-3}
	30	1.10	1.36×10^{-1}
20	10	1.87	3.05×10^{-2}
	15	1.10	1.37×10^{-1}
	20	0.51	3.06×10^{-1}
	25	01.04	4.85×10^{-1}
	30	-0.36	6.40×10^{-1}
30	10	1.10	1.38×10^{-1}
	15	0.26	3.99×10^{-1}
	20	-0.36	6.42×10^{-1}
	25	-0.86	8.04×10^{-1}
	30	-1.27	8.98×10^{-1}
40	10	0.50	3.09×10^{-1}
	15	-0.37	6.43×10^{-1}
	20	-1.01	8.43×10^{-1}
	25	-1.51	9.35×10^{-1}
	30	-1.94	9.74×10^{-1}
50	10	0.02	4.91×10^{-1}
	15	-0.87	8.07×10^{-1}
	20	-1.52	9.36×10^{-1}
	25	-2.04	9.79×10^{-1}
	30	-2.47	9.93×10^{-1}

L (m)	δ (°)	β	P_f
10	10	2.15	1.58×10^{-2}
	20	2.14	1.60×10^{-2}
	30	2.14	1.63×10^{-2}
	40	2.13	1.66×10^{-2}
	50	2.12	1.70×10^{-2}
20	10	0.82	2.06×10^{-1}
	20	0.81	2.09×10^{-1}
	30	0.80	2.13×10^{-1}
	40	0.78	2.17×10^{-1}
	50	0.77	2.22×10^{-1}
30	10	-0.03	5.12×10^{-1}
	20	-0.05	5.18×10^{-1}
	30	-0.06	5.24×10^{-1}
	40	-0.08	5.32×10^{-1}
	50	-0.11	5.43×10^{-1}
40	10	-0.66	7.46×10^{-1}
	20	-0.68	7.51×10^{-1}
	30	-0.70	7.58×10^{-1}
	40	-0.72	7.65×10^{-1}
	50	-0.76	7.75×10^{-1}
50	10	-1.16	8.77×10^{-1}
	20	-1.18	8.82×10^{-1}
	30	-1.21	8.86×10^{-1}
	40	-1.24	8.92×10^{-1}
	50	-1.27	8.99×10^{-1}



L (m)	Φ [°]	β	P_i
10	10	9.08	5.59×10^{-20}
	20	5.23	8.51×10^{-8}
	30	2.88	1.99×10^{-3}
	40	1.18	1.20×10^{-1}
	50	-0.17	5.65×10^{-1}
20	10	7.44	5.20×10^{-14}
	20	3.76	8.65×10^{-5}
	30	1.51	6.52×10^{-2}
	40	-0.12	5.47×10^{-1}
	50	-1.40	9.20×10^{-1}
30	10	6.43	6.38×10^{-11}
	20	2.83	2.33×10^{-3}
	30	0.64	2.61×10^{-1}
	40	-0.95	8.29×10^{-1}
	50	-2.21	9.86×10^{-1}
40	10	5.70	600×10^{-18}
	20	2.15	1.58×10^{-7}
	30	-0.01	5.02×10^{-3}
	40	-1.57	9.42×10^{-1}
	50	-2.81	9.97×10^{-1}
50	10	5.12	1.49×10^{-7}
	20	1.61	5.34×10^{-2}
	30	-0.52	6.98×10^{-1}
	40	-2.07	9.81×10^{-1}
	50	-3.29	9.99×10^{-1}

L (m)	SF	β	P_f
10	1	-0.05	5.19×10^{-1}
	2	1.38	8.34×10^{-1}
	3	2.14	1.61×10^{-2}
	4	2.64	4.13×10^{-3}
	5	3.00	1.34×10^{-3}
20	1	-1.61	9.46×10^{-1}
	2	-0.05	5.19×10^{-1}
	3	0.81	2.10×10^{-1}
	4	1.38	8.34×10^{-2}
	5	1.81	3.53×10^{-2}
30	1	-2.57	9.95×10^{-1}
	2	-0.95	8.28×10^{-1}
	3	-0.05	5.19×10^{-1}
	4	0.56	2.86×10^{-1}
	5	1.02	1.54×10^{-1}
40	1	-3.27	9.99×10^{-1}
	2	-1.61	9.46×10^{-1}
	3	-0.68	7.52×10^{-1}
	4	-0.05	5.19×10^{-1}
	5	0.43	3.34×10^{-1}
50	1	-3.82	1.00
	2	-2.13	9.84×10^{-1}
	3	-1.19	8.82×10^{-1}
	4	-0.54	7.05×10^{-1}
	5	-0.05	5.19×10^{-1}

Table 3: Static pile capacity of steel piling in cohesive soils.

A_p ($\times 10^{-2} \text{ m}^2$)	S_u (kN/m ²)	β	P_s
5	10	3.84	6.07×10^{-1}
	20	1.61	5.39×10^{-1}
	30	0.30	3.82×10^{-1}
	40	-0.63	7.35×10^{-1}
	50	-1.35	9.11×10^{-1}
10	10	6.08	6.21×10^{-10}
	20	3.84	6.12×10^{-8}
	30	2.53	5.64×10^{-7}
	40	1.61	5.41×10^{-6}
	50	0.89	1.88×10^{-1}
15	10	7.38	7.98×10^{-14}
	20	5.15	1.33×10^{-7}
	30	3.84	6.18×10^{-6}
	40	2.91	1.80×10^{-5}
	50	2.19	1.42×10^{-2}
20	10	8.31	5.05×10^{-12}
	20	6.07	6.34×10^{-10}
	30	4.77	9.46×10^{-7}
	40	3.84	6.23×10^{-6}
	50	3.12	9.13×10^{-4}
25	10	9.02	9.37×10^{-20}
	20	6.79	5.66×10^{-12}
	30	5.48	2.10×10^{-8}
	40	4.52	2.62×10^{-6}
	50	3.84	6.28×10^{-2}

5	λ_0 [X10 ⁻³ m ²]	α	β	P_z
		0.4	3.48	2.56×10^{-4}
		0.8	1.24	1.07×10^{-1}
		1.2	-0.06	5.26×10^{-1}
		1.6	-0.09	8.39×10^{-1}
		2.0	-1.71	9.56×10^{-1}
10		0.4	5.71	5.84×10^{-6}
		0.8	3.48	2.56×10^{-4}
		1.2	2.17	1.50×10^{-2}
		1.6	1.24	1.07×10^{-1}
		2.0	0.52	3.00×10^{-1}
15		0.4	7.01	1.22×10^{-12}
		0.8	4.78	8.79×10^{-7}
		1.2	3.48	2.56×10^{-4}
		1.6	2.55	5.41×10^{-3}
		2.0	1.83	3.36×10^{-2}
20		0.4	7.93	1.09×10^{-15}
		0.8	5.71	5.84×10^{-6}
		1.2	4.40	5.40×10^{-4}
		1.6	3.48	2.56×10^{-4}
		2.0	2.76	2.92×10^{-3}
25		0.4	8.65	2.67×10^{-18}
		0.8	6.42	6.75×10^{-11}
		1.2	5.12	1.54×10^{-7}
		1.6	4.19	1.38×10^{-5}
		2.0	3.48	2.56×10^{-4}

5	$\lambda_0(10^{-9}\text{m})$	SF	β	P_i
		1	-1.71	9.57×10^{-1}
		2	0.52	3.00×10^{-1}
		3	1.83	3.36×10^{-2}
		4	2.76	2.90×10^{-3}
		5	3.48	2.53×10^{-4}
10		1	0.52	3.01×10^{-1}
		2	2.76	2.92×10^{-2}
		3	4.06	2.42×10^{-3}
		4	4.99	3.01×10^{-7}
		5	5.71	5.66×10^{-9}
15		1	1.83	3.39×10^{-2}
		2	4.06	2.24×10^{-3}
		3	5.37	3.97×10^{-6}
		4	6.30	1.53×10^{-10}
		5	7.02	1.16×10^{-12}
20		1	2.75	2.96×10^{-2}
		2	4.99	3.07×10^{-7}
		3	6.29	1.55×10^{-10}
		4	7.22	2.59×10^{-13}
		5	7.94	1.02×10^{-15}
25		1	3.47	2.61×10^{-4}
		2	5.71	5.84×10^{-9}
		3	7.01	1.18×10^{-12}
		4	7.94	1.03×10^{-15}
		5	8.66	2.45×10^{-18}



$A_0 (X10^{-3}m^2)$	$A_v (m^2)$	β	P_t
5	20	3.40	3.38×10^{-4}
	30	2.09	1.82×10^{-2}
	40	1.17	1.22×10^{-1}
	50	0.45	3.27×10^{-1}
	60	-0.14	5.56×10^{-1}
10	20	5.63	9.06×10^{-9}
	30	4.33	7.63×10^{-8}
	40	3.40	3.38×10^{-4}
	50	2.68	3.67×10^{-3}
	60	2.09	1.82×10^{-2}
15	20	6.93	2.08×10^{-12}
	30	5.63	9.06×10^{-9}
	40	4.74	1.28×10^{-4}
	50	3.99	3.36×10^{-4}
	60	3.40	3.38×10^{-4}
20	20	7.86	2.00×10^{-16}
	30	6.55	2.81×10^{-11}
	40	5.63	9.06×10^{-9}
	50	4.91	4.52×10^{-7}
	60	4.33	7.63×10^{-6}
25	20	8.57	5.15×10^{-20}
	30	7.27	1.80×10^{-13}
	40	6.35	1.10×10^{-10}
	50	5.63	9.06×10^{-9}
	60	5.04	2.29×10^{-7}

A_s (m ²)	S_u (kN/m ²)	β	P_r
20	10	16.59	4.51×10^{-52}
	20	15.15	3.91×10^{-52}
	30	13.85	6.98×10^{-44}
	40	12.92	1.83×10^{-36}
	50	12.20	1.63×10^{-24}
30	10	16.09	1.60×10^{-50}
	20	13.86	5.44×10^{-44}
	30	12.56	1.83×10^{-36}
	40	11.63	1.46×10^{-31}
	50	10.91	5.19×10^{-26}
40	10	15.18	2.74×10^{-42}
	20	12.95	1.25×10^{-36}
	30	11.64	1.28×10^{-31}
	40	10.72	4.40×10^{-27}
	50	10.00	8.12×10^{-24}
50	10	14.46	1.07×10^{-47}
	20	12.24	1.03×10^{-47}
	30	10.93	4.21×10^{-38}
	40	10.00	7.50×10^{-24}
	50	9.28	8.32×10^{-21}
60	10	13.68	4.24×10^{-44}
	20	11.65	1.13×10^{-35}
	30	10.35	2.18×10^{-28}
	40	9.42	2.27×10^{-23}
	50	8.70	1.66×10^{-18}

A_s (m ²)	SF	β	P_r
20	1	11.83	1.37×10^{-32}
	2	14.07	3.09×10^{-45}
	3	15.37	1.31×10^{-52}
	4	16.30	5.53×10^{-60}
	5	17.01	3.58×10^{-68}
30	1	10.55	2.72×10^{-26}
	2	12.78	1.08×10^{-37}
	3	14.09	2.42×10^{-45}
	4	15.01	3.29×10^{-51}
	5	15.73	5.27×10^{-56}
40	1	9.63	3.04×10^{-22}
	2	11.87	9.25×10^{-35}
	3	13.17	6.77×10^{-40}
	4	14.10	2.14×10^{-48}
	5	14.81	6.51×10^{-50}
50	1	8.92	2.41×10^{-19}
	2	11.15	3.55×10^{-29}
	3	12.46	6.51×10^{-36}
	4	13.38	3.94×10^{-41}
	5	14.10	1.98×10^{-45}
60	1	8.34	3.91×10^{-17}
	2	10.57	2.09×10^{-26}
	3	11.88	8.12×10^{-33}
	4	12.80	36×10^{-38}
	5	13.52	36×10^{-42}

A_h (m ²)	α	β	P_f
20	0.4	16,95	9.63×10^{-46}
	0.8	14,78	1.12×10^{-46}
	1.2	13,49	9.27×10^{-47}
	1.6	12,58	1.48×10^{-46}
	2.0	11,87	9.25×10^{-48}
30	0.4	15,68	1.06×10^{-46}
	0.8	13,49	9.27×10^{-47}
	1.2	12,20	1.59×10^{-46}
	1.6	11,28	8.13×10^{-48}
	2.0	10,57	2.09×10^{-46}
40	0.4	14,78	1.12×10^{-47}
	0.8	12,58	1.48×10^{-46}
	1.2	11,28	8.13×10^{-48}
	1.6	10,36	1.84×10^{-48}
	2.0	9,65	2.50×10^{-47}
50	0.4	14,07	3.09×10^{-46}
	0.8	11,87	9.25×10^{-48}
	1.2	10,57	2.09×10^{-48}
	1.6	9,65	2.50×10^{-47}
	2.0	8,94	2.06×10^{-49}
60	0.4	13,49	9.27×10^{-47}
	0.8	11,28	8.13×10^{-48}
	1.2	9,99	8.82×10^{-48}
	1.6	9,07	6.25×10^{-48}
	2.0	8,35	3.43×10^{-47}

Table 4: Static pile capacity of steel piling in cohesionless soils.

A_s ($\times 10^{-2} \text{m}^2$)	A_s (kN/m^2)	β	P_t
5	20	5.02	2.54×10^7
	30	5.02	2.60×10^7
	40	5.02	2.66×10^7
	50	5.01	2.72×10^7
	60	5.01	2.78×10^7
10	20	5.04	2.38×10^7
	30	5.03	2.41×10^7
	40	5.03	2.43×10^7
	50	5.03	2.46×10^7
	60	5.03	2.49×10^7
15	20	5.04	2.33×10^7
	30	5.04	2.35×10^7
	40	5.04	2.37×10^7
	50	5.04	2.38×10^7
	60	5.03	2.40×10^7
20	20	5.04	2.31×10^7
	30	5.04	2.32×10^7
	40	5.04	2.33×10^7
	50	5.04	2.35×10^7
	60	5.04	2.36×10^7
25	20	5.04	2.29×10^7
	30	5.04	2.30×10^7
	40	5.04	2.31×10^7
	50	5.04	2.32×10^7
	60	5.04	2.33×10^7

A_0 ($\times 10^{-3} \text{m}^2$)	γ (kN/m ³)	β	P_i
5	10	-7.67	1.00
	15	5.14	1.41×10^{-7}
	20	4.90	4.73×10^{-7}
	25	4.70	1.32×10^{-6}
	30	4.51	3.22×10^{-6}
10	10	-7.67	1.00
	15	-7.89	1.00
	20	4.92	4.40×10^{-7}
	25	4.72	1.20×10^{-6}
	30	4.54	2.87×10^{-6}
15	10	-7.67	1.00
	15	-7.89	1.00
	20	4.92	4.29×10^{-7}
	25	4.72	1.17×10^{-6}
	30	4.54	2.76×10^{-6}
20	10	-7.67	1.00
	15	-7.89	1.00
	20	4.92	4.24×10^{-7}
	25	4.73	1.15×10^{-6}
	30	4.55	2.71×10^{-6}
25	10	-7.67	1.00
	15	-7.89	1.00
	20	4.93	4.21×10^{-7}
	25	4.73	1.14×10^{-6}
	30	4.55	2.68×10^{-6}

A_p ($\times 10^{-2} \text{m}^2$)	$L(\text{m})$	β	P_f
5	20	4.55	2.64×10^{-4}
	30	4.08	2.21×10^{-4}
	40	3.69	1.11×10^{-4}
	50	3.35	4.03×10^{-5}
	60	3.05	1.16×10^{-5}
10	20	4.58	2.37×10^{-4}
	30	4.13	1.85×10^{-4}
	40	3.75	8.75×10^{-5}
	50	3.43	2.99×10^{-5}
	60	3.15	8.18×10^{-6}
15	20	4.58	2.28×10^{-4}
	30	4.14	1.75×10^{-4}
	40	3.77	8.11×10^{-5}
	50	3.46	2.72×10^{-5}
	60	3.18	7.33×10^{-6}
20	20	4.59	2.24×10^{-4}
	30	4.14	1.70×10^{-4}
	40	3.78	7.81×10^{-5}
	50	3.47	2.60×10^{-5}
	60	3.20	6.94×10^{-6}
25	20	4.59	2.22×10^{-4}
	30	4.15	1.68×10^{-4}
	40	3.79	7.64×10^{-5}
	50	3.48	2.53×10^{-5}
	60	3.21	6.72×10^{-6}

A_0 ($\times 10^{-3} \text{m}^2$)	$\varnothing$ ($^\circ$)	β	P_i
5	10	9.33	5.35×10^{-21}
	20	8.49	1.07×10^{-17}
	30	5.84	2.57×10^{-9}
	40	3.96	3.75×10^{-6}
	50	2.50	6.29×10^{-3}
10	10	5.04	2.32×10^{-7}
	20	5.04	2.37×10^{-7}
	30	5.03	2.42×10^{-7}
	40	5.03	2.49×10^{-7}
	50	5.02	2.59×10^{-7}
15	10	5.04	2.19×10^{-7}
	20	5.04	2.32×10^{-7}
	30	5.04	2.36×10^{-7}
	40	5.03	2.40×10^{-7}
	50	5.03	2.46×10^{-7}
25	10	5.05	2.27×10^{-7}
	20	5.04	2.29×10^{-7}
	30	5.04	2.31×10^{-7}
	40	5.04	2.33×10^{-7}
	50	5.04	2.37×10^{-7}

A_p ($\times 10^{-3} \text{m}^2$)	SF	β	P_i
5	1	3.94	4.03×10^{-6}
	2	4.70	1.30×10^{-4}
	3	5.05	2.24×10^{-7}
	4	-7.80	1.00
	5	-7.69	1.00
10	1	3.94	4.03×10^{-6}
	2	4.70	1.30×10^{-4}
	3	5.05	2.24×10^{-7}
	4	-7.80	1.00
	5	-7.69	1.00
15	1	3.90	4.75×10^{-6}
	2	4.69	1.40×10^{-4}
	3	5.04	2.33×10^{-7}
	4	-7.81	1.00
	5	-7.69	1.00
20	1	3.94	4.03×10^{-6}
	2	4.69	1.37×10^{-4}
	3	5.04	2.31×10^{-7}
	4	-7.81	1.00
	5	-7.69	1.00
25	1	3.92	4.44×10^{-6}
	2	4.69	1.36×10^{-4}
	3	5.04	2.29×10^{-7}
	4	-7.81	1.00
	5	-7.69	1.00



L (m)	A _s (m ²)	β	P _r
20	20	4.60	2.14 X 10 ⁻⁴
	30	4.60	2.14 X 10 ⁻⁴
	40	4.60	2.14 X 10 ⁻⁴
	50	4.60	2.15 X 10 ⁻⁴
	60	4.60	2.15 X 10 ⁻⁴
30	20	4.16	1.59 X 10 ⁻⁵
	30	4.16	1.59 X 10 ⁻⁵
	40	4.16	1.59 X 10 ⁻⁵
	50	4.16	1.59 X 10 ⁻⁵
	60	4.16	1.59 X 10 ⁻⁵
40	20	3.81	7.10 X 10 ⁻⁵
	30	3.81	7.10 X 10 ⁻⁵
	40	3.80	7.11X 10 ⁻⁵
	50	3.80	7.11X 10 ⁻⁵
	60	3.80	7.12X 10 ⁻⁵
50	20	3.50	2.31 X 10 ⁻⁴
	30	3.50	2.31 X 10 ⁻⁴
	40	3.50	2.32 X 10 ⁻⁴
	50	3.50	2.32 X 10 ⁻⁴
	60	3.50	2.32 X 10 ⁻⁴
60	20	3.24	6.05 X 10 ⁻⁵
	30	3.24	6.06 X 10 ⁻⁵
	40	3.24	6.06 X 10 ⁻⁵
	50	3.24	6.07 X 10 ⁻⁵
	60	3.24	6.07 X 10 ⁻⁵

L (m)	γ (kN/m ²)	β	P_t
20	10	5.08	1.93×10^{-7}
	15	4.73	1.10×10^{-6}
	20	4.45	4.30×10^{-6}
	25	4.20	1.31×10^{-5}
	30	3.99	3.36×10^{-5}
30	10	4.73	1.10×10^{-6}
	15	4.32	7.70×10^{-6}
	20	3.99	3.36×10^{-5}
	25	3.70	1.09×10^{-4}
	30	3.45	2.85×10^{-4}
40	10	4.45	4.30×10^{-6}
	15	3.99	3.36×10^{-6}
	20	3.61	1.53×10^{-4}
	25	3.29	4.97×10^{-4}
	30	3.02	1.29×10^{-3}
50	10	4.20	1.31×10^{-5}
	15	3.70	1.09×10^{-4}
	20	3.29	4.97×10^{-4}
	25	2.95	1.59×10^{-3}
	30	2.65	3.97×10^{-3}
60	10	3.99	3.36×10^{-4}
	15	3.45	2.85×10^{-4}
	20	3.02	1.29×10^{-3}
	25	2.65	3.97×10^{-3}
	30	2.34	9.54×10^{-3}

L (m)	$\varnothing$ (°)	β	F_f
20	10	16.65	1.63×10^{-42}
	20	8.02	5.58×10^{-10}
	30	5.41	3.22×10^{-8}
	40	3.55	1.97×10^{-4}
	50	2.10	1.81×10^{-2}
30	10	11.79	2.39×10^{-30}
	20	7.52	2.82×10^{-14}
	30	4.96	3.58×10^{-7}
	40	3.12	8.95×10^{-4}
	50	1.69	4.53×10^{-2}
40	10	11.28	8.48×10^{-30}
	20	7.11	5.88×10^{-13}
	30	4.59	2.21×10^{-6}
	40	2.78	2.72×10^{-3}
	50	1.37	8.60×10^{-2}
50	10	10.86	9.69×10^{-28}
	20	6.76	6.88×10^{-12}
	30	4.28	9.44×10^{-4}
	40	2.49	6.42×10^{-2}
	50	1.09	1.38×10^{-1}
60	10	10.49	4.94×10^{-26}
	20	6.46	5.34×10^{-11}
	30	4.01	3.11×10^{-6}
	40	2.232	1.28×10^{-2}
	50	0.85	1.99×10^{-1}

L (m)	δ (°)	β	P_i
20	10	4,60	$2,13 \times 10^{-4}$
	20	4,60	$2,14 \times 10^{-4}$
	30	4,60	$2,15 \times 10^{-4}$
	40	4,60	$2,15 \times 10^{-4}$
	50	4,60	$2,16 \times 10^{-4}$
30	10	4,16	$1,58 \times 10^{-4}$
	20	4,16	$1,58 \times 10^{-4}$
	30	4,16	$1,59 \times 10^{-4}$
	40	4,16	$1,60 \times 10^{-4}$
	50	4,16	$1,61 \times 10^{-4}$
40	10	3,81	$7,05 \times 10^{-5}$
	20	3,81	$7,09 \times 10^{-5}$
	30	3,81	$7,13 \times 10^{-5}$
	40	3,81	$7,19 \times 10^{-5}$
	50	3,81	$7,26 \times 10^{-5}$
50	10	3,50	$2,29 \times 10^{-4}$
	20	3,50	$2,31 \times 10^{-4}$
	30	3,50	$2,33 \times 10^{-4}$
	40	3,50	$2,35 \times 10^{-4}$
	50	3,49	$2,38 \times 10^{-4}$
60	10	3,24	$5,99 \times 10^{-4}$
	20	3,24	$6,04 \times 10^{-4}$
	30	3,23	$6,10 \times 10^{-4}$
	40	3,23	$6,17 \times 10^{-4}$
	50	3,23	$6,26 \times 10^{-4}$

L (mm)	SF	β	Pr
20	1	3.24	6.07×10^{-4}
	2	4.16	1.59×10^{-5}
	3	4.60	2.14×10^{-6}
	4	4.86	5.77×10^{-7}
	5	5.05	2.25×10^{-7}
30	1	2.59	4.78×10^{-3}
	2	3.65	1.32×10^{-4}
	3	4.16	1.59×10^{-5}
	4	4.48	3.76×10^{-6}
	5	4.70	1.31×10^{-6}
40	1	2.09	1.83×10^{-2}
	2	3.25	6.05×10^{-4}
	3	3.81	7.09×10^{-5}
	4	4.16	1.59×10^{-5}
	5	4.41	5.16×10^{-6}
50	1	1.68	4.65×10^{-2}
	2	2.89	1.92×10^{-3}
	3	3.50	2.31×10^{-4}
	4	3.89	5.05×10^{-5}
	5	4.16	1.58×10^{-5}
60	1	1.33	9.17×10^{-2}
	2	2.59	4.77×10^{-3}
	3	3.24	6.05×10^{-4}
	4	3.65	1.32×10^{-4}
	5	3.94	4.08×10^{-5}

Table 5: Dynamic pile capacity using Hiley, Janbu and Gates formulae

	A_p ($\times 10^{-3} \text{m}^2$)	β	P_i
Hiley	5	16.67	1.25×10^{-42}
	10	16.67	1.24×10^{-42}
	15	16.67	1.23×10^{-42}
	20	16.67	1.23×10^{-42}
	25	16.67	1.22×10^{-42}
Janbu	5		
	10	16.67	1.24×10^{-42}
	15		
	20	16.67	1.23×10^{-42}
	25	16.67	1.23×10^{-42}
Gates	5	16.67	1.21×10^{-42}
	10	16.67	1.21×10^{-42}
	15	16.67	1.21×10^{-42}
	20	16.67	1.21×10^{-42}
	25	16.67	1.21×10^{-42}

	α_n	β	P_t
	0.65	8.53	7.52×10^{-18}
	0.75	8.04	4.46×10^{-16}
	0.85	7.62	1.31×10^{-14}
Hiley	0.95	7.24	2.26×10^{-13}
	1.50	5.69	6.28×10^{-6}
	0.65	8.44	1.55×10^{-17}
	0.75	8.02	5.56×10^{-16}
	0.85	7.64	1.09×10^{-14}
Janbu	0.95	7.31	1.36×10^{-13}
	1.50	5.95	1.32×10^{-6}
	16.67	8.44	1.25×10^{-42}
	16.67	8.02	1.26×10^{-42}
	16.67	7.64	1.26×10^{-42}
Gates	16.66	7.31	1.26×10^{-42}
	16.66	5.95	1.27×10^{-42}

	E_n (kN/m)	β	P_t
	30	7.99	6.64×10^{-16}
	40	7.02	1.14×10^{-12}
	50	6.26	1.92×10^{-10}
Hiley	60	5.64	8.34×10^{-9}
	70	5.12	1.51×10^{-7}
	30	7.97	7.95×10^{-16}
	40	7.11	5.72×10^{-13}
	50	6.45	5.62×10^{-11}
Janbu	60	5.91	1.71×10^{-9}
	70	5.46	2.43×10^{-8}

Hiley	S (m)	β	P_i
	0.05	10.35	2.24×10^{-25}
	0.55	16.65	1.70×10^{-42}
	1.05	16.66	1.49×10^{-42}
	1.55	16.66	1.39×10^{-42}
	2.05	16.66	1.35×10^{-42}
Janbu	0.05	10.35	2.17×10^{-25}
	0.55		
	1.05	16.63	2.21×10^{-42}
	1.55	16.64	1.81×10^{-42}
	2.05	16.65	1.64×10^{-42}
Gates	0.05	16.67	1.25×10^{-42}
	0.55	16.67	1.22×10^{-42}
	1.05	16.67	1.21×10^{-42}
	1.55	16.67	1.21×10^{-42}
	2.05	16.67	1.21×10^{-42}

	SF	β	P_1
Hiley	2.5	6.06	6.66×10^{-10}
	3.5	7.20	2.94×10^{-13}
	4.5	8.06	3.94×10^{-16}
	5.5	8.74	1.19×10^{-18}
	6.5	9.31	6.66×10^{-21}
Janbu	2.5	7.01	1.20×10^{-10}
	3.5	8.24	8.55×10^{-13}
	4.5	9.17	2.39×10^{-16}
	5.5	9.92	1.81×10^{-18}
	6.5	10.54	2.90×10^{-21}
Gates	2.5	16.66	1.27×10^{-40}
	3.5	16.67	1.25×10^{-40}
	4.5	16.67	1.24×10^{-40}
	5.5	16.67	1.24×10^{-40}
	6.5	16.67	1.23×10^{-40}

Table 6: Dynamic capacity using Hiley and Janbu formulae

	W_r (kN/m)	β	P_r
Hiley	30	7.69	7.68×10^{-16}
	40	8.00	6.36×10^{-16}
	50	8.23	9.52×10^{-17}
	60	8.41	2.16×10^{-17}
	70	8.54	7.10×10^{-18}
Janbu	30	7.70	7.13×10^{-16}
	40	7.93	1.16×10^{-16}
	50	8.12	2.40×10^{-16}
	60	8.29	5.90×10^{-17}
	70	8.44	1.65×10^{-17}

	L (m)	β	P_l
Hiley	30	7.67	1.85×10^{-16}
	40	8.10	2.73×10^{-16}
	50	8.31	4.79×10^{-17}
	60	8.50	9.61×10^{-18}
	70	8.67	2.16×10^{-18}
Janbu	30	8.05	4.10×10^{-16}
	40	8.42	1.86×10^{-17}
	50	8.72	1.42×10^{-18}
	60	8.97	1.54×10^{-19}
	70	9.18	2.14×10^{-20}