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Ana Catarina Martins Teixeira Reliability and Cost Models of Axial Pile Foundations

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ANA

Abstract

Reliability and cost models of pile foundations

Pile foundations are often used for important structures, and thus, reliability evaluation is an important aspect of the design. Unlike the approach to reliability evaluation used in structural engineering, the traditional procedure used in geotechnical designs addresses uncertainties through high global or partial safety factors, mostly based on past experience. However, this approach to addressing uncertainties does not provide a rational basis for understanding their influence on design. For this reason, and because of regulation codes and social concerns (such as sustainability), geotechnical engineers need to improve their ability to deal with uncertainties and probabilities to help with decision-making. Reliability methods have become increasingly important as decision support tools. The main benefit of reliability analysis is that it provides quantitative information about the parameters (uncertainties) that most significantly influence the behaviour under study. This makes risk control, the determination of the potential causes of adverse effects on the structure, possible. In particular, the design of pile foundations still involves many limitations and uncertainties, particularly when there is not enough investment in soil characterisation and/or pile load tests. In addition to the uncertainties associated with soil characterisation, physical, statistical, spatial and human uncertainties exist. Hence, because it is technically and economically impossible to produce designs of pile foundations in the most unfavourable of cases, it is the engineer's goal to minimise the risk and limit it to an acceptable level in the most economical manner possible. Towards this, reliability theory needs to be adapted to the needs and objectives of geotechnical engineering. In this subject, the primary purpose of this dissertation is to demonstrate the application of reliability methods to geotechnical design and more particularly to two distinct case studies of vertical single pile foundations under axial loading. This dissertation also presents a simple and practical approach to performing reliability-based design, obtaining valuable information from it. For that purpose, sensitivity and cost analyses were conducted to study the influence of each uncertainty type. Two well-known reliability methods, the first-order reliability method (FORM) and Monte Carlo simulations (MCS) were applied to the case studies for comparison. In addition, reliability-based safety factors were evaluated and discussed. Another purpose of this dissertation is to demonstrate the advantages of employing reliability tools in the decision-making process for pile foundation design. The decision-making related to the economic and research investments required for gathering the information necessary to characterise the uncertainties associated with important random variables, in both pile design and its reliability, is facilitated by the type of balanced analyses presented in this dissertation. It is concluded that, even though the extent to which this can be accomplished depends on the engineer's knowledge and the project's budget for investigation, geotechnical engineering definitely benefits from the consideration of reliability in design. It is finally intended to provide knowledge and tools for code harmonisation between structural and geotechnical designs, and also encourage the development of such in geotechnical practice, international standards and conformity in assessment systems.

Key-words: Costs; CPT; Design codes; FORM; Geotechnical reliability; *In situ* soil investigation; MCS; Partial safety factors; Pile foundation; PMT; Probability of failure; Reliability-based design; Reliability index; Risk; SPT; Uncertainty.

Resumo

Modelos de fiabilidade e custo de fundações por estaca

As fundações por estaca são utilizadas em obras de grande importância, e por esse motivo a fiabilidade na avaliação da segurança é um ponto essencial no seu dimensionamento. Ao contrário do que acontece em engenharia estrutural, a fiabilidade geotécnica é ainda obtida através de elevados coeficientes de segurança, globais ou parciais, na sua maioria com base empírica. No entanto, esta forma de tratar as incertezas não apresenta uma base racional para compreender a sua influência no dimensionamento e no projeto. Por estas razões, por questões de preocupação sociais (como a sustentabilidade) e também para obedecer às novas regulamentações, os engenheiros geotécnicos devem melhorar a sua capacidade para tratar as incertezas e gerir probabilidades, para que com isto possam ter ajuda nas tomadas de decisão. Os métodos de fiabilidade têm ganho uma importância crescente como ferramentas de ajuda e suporte a tomadas de decisão. As principais vantagens são a quantificação da probabilidade de ocorrência do comportamento da estrutura em estudo e a obtenção de informação sobre os parâmetros (incertezas) que mais o influenciam. Isto melhora o controlo do risco e a determinação das potenciais causas de efeitos adversos sobre a estrutura. Em particular, o dimensionamento de fundações por estaca ainda tem várias limitações e diversas incertezas, especialmente quando não existe investimento suficiente na caracterização do solo e/ou na realização de ensaios de carga. A esta incerteza no solo e seu comportamento, acrescentam-se ainda as incertezas físicas, estatísticas, espaciais e erros humanos. Assim, sendo tecnicamente e economicamente impossível fazer dimensionamentos considerando os casos mais desfavoráveis, é objetivo de um engenheiro minimizar e controlar os riscos a um nível aceitável da forma mais económica possível. Para tal a teoria da fiabilidade deve ser adaptada às necessidades e objetivos da engenharia geotécnica. Neste contexto, este trabalho pretende demonstrar como realizar análises de fiabilidade, introduzindo as incertezas no dimensionamento do ponto de vista geotécnico. Dois casos de estudo de duas fundações por estaca submetidas a carga axial são apresentados, explicando metodologias simples e práticas para realizar análises de fiabilidade, das quais um engenheiro pode obter informações valiosas e importantes. Para tal, análises de sensibilidade envolvendo as técnicas de fiabilidade e custos foram realizadas a fim de investigar a influência de cada parâmetro (incerteza) considerada. Dois métodos tradicionais de fiabilidade, o método de fiabilidade de primeira ordem (FORM) e o método de simulação de Monte Carlo (MCS), foram aplicados aos casos de estudo para comparação entre si. Além disso, coeficientes de segurança baseados nas técnicas de fiabilidade foram também avaliados e discutidos. Outro objetivo deste trabalho é demonstrar as vantagens da utilização das ferramentas de fiabilidade no processo de tomada de decisão no projeto e dimensionamento de fundações por estaca. As tomadas de decisão relativas a investimentos económicos e em investigação, necessários para a recolha de informação essencial para caracterizar as incertezas mais influentes, é facilitada com o tipo de análises apresentadas neste trabalho. Conclui-se portanto que, embora este tipo de investimento depende consideravelmente do conhecimento do engenheiro responsável e do orçamento disponível para a obra em questão, o projeto e dimensionamento iriam beneficiar notavelmente com este tipo de análises baseadas nas técnicas de fiabilidade. Finalmente, este trabalho é destinado a fornecer conhecimentos e ferramentas para a harmonização entre os dimensionamentos estrutural e geotécnico, e também incentivar o desenvolvimento destas técnicas na prática de geotecnia, na normalização internacional e na conformidade dos sistemas de avaliação.

Palavras-chave: Custos; CPT; Normas de dimensionamento; FORM; Fiabilidade geotécnica; Ensaios de campo; MCS; Coeficientes parciais de segurança; Fundações por estaca; PMT; Probabilidade de rotura; Dimensionamento baseado em técnicas de fiabilidade; Índice de fiabilidade; Risco; SPT; Incertezas.

Résumé

Modèles de fiabilité e de coûts des pieux

Les pieux sont souvent utilisés dans des structures importantes ce qui rend l'évaluation de sa fiabilité un aspect important à tenir en compte dans le dimensionnement. Au contraire de l'approche d'évaluation de la fiabilité utilisée dans le domaine des structures, la procédure traditionnelle en géotechnique s'appuie sur des coefficients de sécurité global ou partiels élevés, basés sur des méthodes empiriques. Cependant, cette méthode ne fournit pas une base rationnelle pour adresser l'influence des incertitudes dans le dimensionnement. C'est pour cela que, de façon à obéir aux nouveaux règlements et aussi pour des raisons sociales, que les ingénieurs géotechniciens doivent améliorer ses capacités de prendre en compte des incertitudes et de gérer les probabilités, qui seraient ensuite utiles dans la procédure de prise de décision. Les méthodes de fiabilité ont de plus en plus été appliqués comme un outil d'aide à la décision dans les dernières années. Son avantage principale s'appuie sur l'évaluation de l'influence des paramètres (incertitudes) sur le comportement de la structure, ce qui rend la gestion des risques possible, bien comme l'évaluation des causes potentielles des effets négatifs pour la structure. Plus particulièrement, le dimensionnement des fondation profondes par pieux a toujours quelques limitations et entraîne plusieurs incertitudes, en spécial quand il n'y a pas un grand investissement dans l'évaluation de la nature du sol et quand les essais de charge ne sont pas réalisés. À cette incertitude viennent s'ajouter les incertitudes physiques, statistiques, spatiales et les erreurs humains. Cependant, et parce que ça reste techniquement et économiquement impossible de procéder à un calcul de dimensionnement en tenant compte des cas les plus défavorables, l'objectif de l'ingénieur est bien de minimiser et de contrôler les risques à un niveau qui soit considéré comme acceptable, de la façon la plus rentable possible. Pour tout ce qui a été exposé dans ce résumé, la théorie de la fiabilité doit être adapté aux besoins et objectifs de l'ingénierie géotechnique. Ce travail se développe dans ce cadre avec le but de démontrer comment réaliser des analyses de fiabilité, en introduisant les incertitudes dans le dimensionnement du point de vue géotechnique. Pour cela, deux cas d'étude de fondation par pieux soumis à chargement axial sont présentés. Ce travail présente aussi une méthodologie simples et pratique pour réaliser des analyses de fiabilité d'où l'ingénieur pourra obtenir des informations utiles et importantes. Des analyses de sensibilité qui englobent les techniques de fiabilité et coûts ont été réalisées avec l'objectif d'évaluer l'influence de chaque incertitude considérée. De plus, deux méthodes traditionnelles de fiabilité (FORM et MCS) ont été appliqués aux cas d'études et les résultats ont été comparés. Les coefficients de sécurité basés sur des techniques de fiabilité ont été évalués et discutés. Un autre objectif de ce travail est de démontrer les avantages de l'utilisation des outils de fiabilité dans la procédure de prise de décision dans un projet et dimensionnement de fondations par pieux. Les prises de décision concernant les investissements économiques et recherche, essentiaux pour le recueil d'information utile à la caractérisation des incertitudes liées aux variables influant le dimensionnement des pieux et de sa fiabilité, devient plus facile avec l'application de l'analyse présentée dans ce travail. Il peut donc être conclu que malgré que ce type d'investissement soit considérablement dépendant de l'ingénieur responsable et du budget disponible pour les travaux en cause, le projet de dimensionnement serait bénéficié en grande mesure par ce type d'analyse basée dans des techniques de fiabilité. Finalement, ce travail est destiné à fournir des connaissances et outils pour relier les dimensionnement structurels et géotechniques, bien comme de motiver le développement de ces techniques dans la domaine géotechnique, dans la normalisation internationale et dans la conformité des systèmes d'évaluation.

Mots-clés: Coûts; CPT; Codes de dimensionnement; FORM; Fiabilité géotechnique; Investigation du sol *in situ*; MCS; Coefficients de sécurité partiels; Pieux de fondation; PMT; Probabilité de faille; Dimensionnement avec des méthodes de fiabilité; Indice de fiabilité; Risque; SPT; Incertitude.

Glossary

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AFOSM	Advanced First-Order Second Moment
AI	Artificial Intelligence
AIJ	Architectural Institute of Japan
ANN	Artificial Neural Network(s), same as NN
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
A&V	Aoki & Velloso model for prediction of the pile vertical bearing capacity (resistance)
BN	Bayesian network(s)
BS	British Standard(s)
CDF	Cumulative Density Function (also denoted as F)
CEN	Comité Européen de Normalisation (European Committee for Standardisation)
COV	Coefficient of Variation (standard deviation divided by mean value)
CPT	Cone Penetration Test
CS	Construction Strategy
DVM	Design Value Method
DGF	Dansk Geoteknisk Forening (Danish Geotechnical Society)
DGI	Danish Geotechnical Institute

DIN	Deutsche Industrie Norm (German Industrial Standard)
ETC10	European Technical Committee 10 (by ISSMGE)
FEM	Finite Elements Method
FHWA	Federal Highway Administration
FORM	First-Order Reliability Method
FOSM	First-Order Second Moment method
GS	Geotechnical State
GWL	Ground Water Level
i.i.d.	Independent and Identically Distributed
ISRM	International Society of Rock Mechanics
ISSMGE	International Society of Soil Mechanics and Geotechnical Engineering
JGS	Japanese Geotechnical Society
LC	Load Combination(s)
LRFD	Load and Resistance Factor Design
LSD	Limit State Design
LT	Pile Load Test(s)
MCS	Monte Carlo Simulation(s)
MFA	Material Factor Approach
MRFA	Multiple Resistance Factor Approach
MU	Monetary Units
NEN	Nederlandse Norm (Dutch Standard)
NF	Norme Francaise (French Standard)
NN	Neural network(s), same as ANN
PDA	Pile Driving Analyser
PDF	Probability density Function (also denoted as f)
PFA	Partial Factor Approach
PFEM	Probability Finite Elements Method

PMT Menard Pressuremeter Test

RA Reliability Analysis/Analyses

RBD Reliability-Based Design

RF Random Field(s)

RFA Resistance Factor Approach

RFEM Random Finite Elements Method

RS Response Surface(s)

RV Random Variable(s) (also denoted as X)

SF Safety Factor(s) (also denoted as γ)

SFEM Stochastic Finite Elements Method

SHB Specifications for Highway Bridges

Also referred as the model for prediction of the pile vertical bearing capacity (resistance) recommended by SHB (Japan)

SLS Serviceability Limit State

SORM Second-Order Reliability Method

SOSM Second-Order Second Moment method

SPT Standard Penetration Test

SS Svensk Standard (Swedish Standard)

S1,S2,S3 Case Scenarios for cost-reliability-risk analyses

ULS Ultimate Limit State

WL Water Line

WSD Working Stress Design

Roman letters

A,B,C Scalar, Vector or Matrix of a calculation model

a,b,c Constants or parameters of a calculation model

A Area of the pile tip ($\pi.B^2$) [m^2]

B Diameter of the pile [m]

C	Consequences or Costs
C_X	Covariance matrix
Cov	Covariance between two variables
C$	Construction costs [MU]
D	Length of the pile [m]
E	Load(s) or action(s) or its effects [kN]
E[X]	Mean value of X
Ev	Undesirable event
F	Cumulative density function (CDF)
F$	Failure costs [MU]
F_{side}	Predicted side resistance [kN]
f	Probability density function (PDF)
fs	Cone sleeve friction by CPT [kPa]
f_{side}	Predicted unit side resistance [kPa]
G	Permanent action(s) [kN]
G_k	Characteristic value of the permanent action(s) [kN]
$g(\,)$	Performance function
H	Normalised sample line length [m]
I	Failure indicator, 0 or 1
I$	Investment costs [MU]
$\sim LN$	Follows a Lognormal distribution (Lognormal PDF)
M	Safety margin
mr	Measurement variability
$\sim N$	Follows a Normal distribution (Normal PDF)
N	Number of blows by SPT
n	Number of simulations

$P[\ ]$ Probability

pf Probability of failure

pl Limit pressure by PMT [MPa]

Q Variable action(s) [kN]

Q_k Characteristic value of the variable action(s) [kN]

qc Cone tip resistance by CPT [MPa]

q_{tip} Predicted unit tip resistance [kPa]

R Resistance(s) [kN]

$T(\ )$ Transformation function

t Deterministic trend

tr Zero mean transformation variability

U Area of the pile side in contact with soil ($\pi.B.D$) [m^2]

Also referred as the utility in risk analyses

w Inherent variability

X Random variable or vector of random variables (RV)

X^* Design point

X_k Characteristic value of the random variable

Y Random variable or vector of random variables (RV)

Z Normalised random variable(s) (μ_z = 0, σ_z^2 = 1)

Z^* Design point in normalised space

Greek letters

α Sensitivity factors

β Reliability index

β_T Target reliability index

γ Multiplying safety factor

Δ Variation of a parameter or variable

δ Error or bias that defines an uncertainty

- δ_f side resistance component
- δ_G permanent action component
- δ_m model error component
- δ_Q variable action component
- δ_t tip resistance component

θ Correlation length/distance or scale of fluctuation or autocorrelation value [m]

μ Mean value

σ Standard deviation

σ^2 Variance

Φ Normal CDF with μ = 0 and σ^2 = 1

Contents

Chapter 1

INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Absolute safety of a structure can never be guaranteed because of our inability to predict the future conditions. Also, we have a limited, or inaccurate, knowledge about actions, properties and behaviour of materials and structures. Therefore, there is always a certain level of risk involved in the design of any element or structure. It is the engineer's goal to minimise or control this risk in the most economical way.

The Eurocodes' mandatory implementation in Europe started in 2010. Alike design codes from other countries, the Eurocodes changed some design methodologies to the designated reliability-based design. Reliability-based designs introduce new safety concepts that require probabilistic and statistical knowledge. The main goal is to take into account in a rational way the major uncertainties of a project or design. Already in 1970, Meyerhof highlighted the need for consideration of variability of loads and soils' strength parameters in foundation design (Meyerhof, 1970).

Reliability-based designs are being introduced in European structural and geotechnical designs through Eurocodes. These new guidelines to design brings a different approach from the traditionally one (global safety factor). Therefore, there is the need to answer to questions like (Baecher & Christian, 2003):

- how much confidence should the engineer place on a safety factor?

- how can the engineer demonstrate that a design based on more data and more consistent information is more robust than one based on partial information, and therefore worth the extra cost of obtaining those data?
- how can the engineer distinguish between different consequences of failure or separate cases in which progressive failure occurs from those in which an average behaviour is to be expected?

The reliability analyses have the intention of knowing the probability of a particular behaviour (e.g.: the probability of failure of a structure). For this type of analysis, as input, we need the geometrics', constitutive properties' and actions' randomness. One of the biggest advantages is that it quantifies and gives information about the parameters that mostly influence the behaviour under study. Furthermore, it allows the determination of the possible responsible causes for adverse effects on the structure (risk) and the quantification of the frequency of occurrence associated (Duncan, 2001). In summary, the reliability-based design allows the assessment and control of the probability of failure, and as a consequence, the control of risks.

The study of the sensitivity of the probability of failure to each type of uncertainty can support some decisions in geotechnical design. By determining the uncertainty that mostly influence the probability of failure of a geotechnical structure, the engineer can guide the investments in pile design in order to increase reliability.

In areas like concrete or steel structures this kind of reliability-based design approach is commonly used and accepted. These areas deal with manufactured materials, with quality control, minimising part of the uncertainties. Also, they can perform low cost trial tests to study the behaviour of the materials. Unlike this is the area of geotechnical engineering. In geotechnical problems we deal with natural materials, where the soil behaviour and interaction soil-structure is not so linear. Soil has a vast variation and geotechnical structures are extensive and sometimes difficult to test. Thus, there is a need for greater efforts from geotechnical community to carry out reliability analysis and quantify the uncertainties of all types of soils and structures.

As referred by Baecher & Christian (2003): "the uncertainties in structural engineering are largely deductive but the uncertainties in geotechnical engineering are largely inductive: starting from limited observations, judgment, knowledge of geology, and statistical reasoning are employed to infer the behaviour of a poorly-defined universe".

In the specific area of pile foundations, the design methodology still has many doubts and uncertainties, especially when there is a lack on good soil characterisation (*in situ* or laboratory tests) or/and when there are no load tests (static or dynamic) to compare or confirm the analyses results. In addition to the uncertainties of pile design based on insufficient data, we have the physical, statistical, modelling and spatial uncertainties and also uncertainties due to human factors (human errors).

Gathering the necessary information to characterise the random variables in geotechnical problems is a difficult task. There is literature published with such information and recommendations. Nonetheless, the values recommended in the literature often cannot be applied to a particular case under study due to high soil variability.

For these and other reasons, and although successfully applied to structural engineering, reliability analyses have been controversial in geotechnical structures, and especially in foundations engineering (Phoon *et al.*, 2003a; Christian, 2004). Nevertheless, several studies on this subject have been carried out.

Some comparative studies and discussions between the pile design using reliability analyses and using the empirical partial safety factors (SF) recommended by the geotechnical Eurocode (CEN, 2007) were already published (Orr and Breysse in *Chapter 8* in Phoon, 2008a; Ching, 2009; Wang *et al.*, 2011a; Hara *et al.*, 2011; Takács, 2011). These studies concluded that in many cases the SF recommended do not guarantee automatic fulfilment of the target reliability. Concerning the reliability-based design, among the most recently referred works applied to pile foundations, are Honjo's research team works, especially in Japanese codes calibration (Honjo *et al.*, 2002a,b, 2003a; Honjo, 2003; Honjo & Amatya, 2005; Honjo *et al.*, 2009; Honjo *et al.*, 2010d, etc.), Phoon studies of reliability methodologies and values for coefficients of variation (Phoon *et al.*, 1990, 1995; Phoon & Kulhawy, 1999a,b, 2005; Phoon *et al.*, 2000, 2003b; Phoon, 2008a,b) and also Fenton, Griffiths, Zhang, Najjar and Haldar researches (e.g.: Fenton & Giffiths, 2000, 2002; Zhang *et al.*, 2001, 2005; Zhang, 2004; Najjar, 2005; Fenton & Giffiths, 2007; Haldar, 2008; Zhang *et al.*, 2009a,b; Najjar *et al.*, 2009; Zhang & Dasaka, 2010). All these authors believe that the reliability analyses are an important tool for geotechnical engineering, and their works support their belief. Structural engineering is not the only that can obtain valuable and more rational information. Also, geotechnical design can be improved and based on probability theory.

Though, the main problem comes from the high variability and the different materials one engineer can find in a geotechnical project. For that matter, some of the referred studies also discuss about what it is important to be investigated and what influences the probability of failure. Moreover, some studies provide recommendations and calibration of the new trend of reliability-based design codes, such as Kulhawy & Phoon (2002), Aoki *et al.* (2002b), Foye *et al.* (2004, 2006a,b, 2011), Paikowsky (2004), Allen (2005a,b), Honjo (2003, 2004), Honjo & Kusakabe (2002), Honjo & Nagao (2007), Honjo *et al.* (2000b, 2002a,b, 2003a,b, 2009, 2010d), Haldar & Babu (2008b), Ching *et al.* (2009), Ching & Phoon (2011) are some of the attempts published to demonstrate to geotechnical engineers the advantages of reliability analyses and reliability-based SF for foundations design. Other studies demonstrate and compare the new reliability methodologies with well-known methods used in practice, this is the case of studies of Yamamoto & Karkee (2004), Yang (2006), Cherubini & Vessia (2007), Juang *et al.* (2009) and Huang *et al.* (2010). In spite of the great deal of research, there have been several discussions around this subject.

Finally, during the last decade there has been an increasing social concern on sustainable developments (conservation of the environment, wellbeing and safety of the individual and the optimal allocation of the available natural and financial resources). As a consequence, the methods of risk and reliability analyses, mainly developed during the last three decades, are increasingly gaining importance as decision support tools in civil engineering applications (Faber & Stewart, 2003).

1.2 APPROACH AND SCOPE OF THE WORK

This dissertation focus on the improvement of axial pile design using reliability-based analyses and on a sensitivity study of the uncertainties involved. This is an important area of study, not only because of legal pressures (new regulation design codes) but also because of social concerns (sustainability). Geotechnical engineers need to increase their ability to deal with uncertainty, and learn how to update the design methods, including the reliability tools, while still meeting design costs and performance targets.

While other fields of civil engineering have made major commitments to reliability-based design, practical geotechnical engineers are held with the traditional ways of treating uncertainties (empirical SF). Therefore, because of the lack of enthusiasm to adopt the more formal and rational approaches using reliability theory in geotechnical engineering, the goal of this dissertation is to contribute for axial pile design based on reliability analyses. Reliability-based analyses, comparing different solutions, could help decision-making due to an increased understanding of (1) the design, (2) its random variables and (3) the uncertainties that mostly influence pile behaviour.

This dissertation seeks to provide tools for code harmonisation between structural and geotechnical designs, and also encourage the development of such international standards and conformity in assessment systems. Accordingly, literature about the subject is reviewed, explaining the current methods of reliability theory suitable for geotechnical problems with emphasis to the difficulties that arise from its applications.

Then, different types of practical reliability-based methodologies and approaches are presented and a sensitivity study is performed concerning the influence of each uncertainty type in the probability of failure. Reliability-based applications such as (1) first-order reliability method (FORM), (2) Monte Carlo simulations (MCS), (3) approaches to determine the minimum length or maximum load to achieve a pre-selected target reliability index, and (4) reliability-based SF for actions and resistances are demonstrated through examples of real life pile foundations (case studies). By explaining these applications step by step, and applying them to some examples, it is intended to contribute to preventing the loss of intuitive understanding when applying reliability tools to axial pile design problems, which is an important issue in geotechnical engineering.

This dissertation is presented also as an aid to pile design decision makers in assessing the uncertainties associated with the random variables that most influence both the probability of failure and pile behaviour. Different approaches can be adopted and all provide very useful tools for modelling the uncertainties and for quantifying their influence on the behaviour under study, also helping achieve a more rational design. However, achieving economy is also a very important aspect of the designs and construction processes, especially nowadays. It is important to invest in quality and cost optimisation; thus, studies on the behaviour of costs are of relevant importance. For this purpose, a study is presented about the relationship between the reliability of a design procedure adopted and the corresponding costs. Such information would support cost-benefit decisions, helping and guiding the investigation of a pile foundation project/design, avoiding spending or investing where it is not appropriate for the reliability of the pile.

1.3 OUTLINE OF THE DISSERTATION

The main goal of this dissertation is to create a document to guide geotechnical engineers that sustain reliability-based analyses and cost-efficient decisions. It provides a support for reliability-based analyses of general geotechnical problems, but more particularly for axial pile design (Teixeira *et al.*, 2002a). To achieve that goal, this dissertation is divided in seven chapters, including this first chapter. All chapters are organised as schematised in Figure 1.1., which also includes a brief description of each chapter.

Chapter 1: INTRODUCTION

Chapter 2: REVIEW ON RELIABILITY FOR GEOTECHNICAL DESIGN

Firstly, this dissertation reviews the literature about the main subject, reliability theory, and the existing methods are explained and its merits described, comparing to the traditional methodologies. Comments are presented regarding the most suitable methods and type of uncertainties for geotechnical applications, with emphasis to the difficulties that arise from its applications in design of pile foundations. A state-of-art is presented in the final part of this chapter concerning reliability applied to pile foundations.

Chapter 3: RELIABILITY-BASED METHODS FOR DESIGN AND SAFETY OF AXIAL PILES

With the theoretical background of Chapter 2, different types of reliability-based analyses are explained in detail. A simple application example is presented in order to better explain all the methodologies proposed. Furthermore, a discussion of some pile reliability-based methodologies is done, namely FORM, MCS, reliability-based design, safety evaluation, reliability-based safety factors and reliability-based sensitivity analyses. These methodologies are later applied to two case studies of pile foundations:

Application examples	
Case study 1	**Case study 2**
Chapter 4: APPLICATION OF RELIABILITY TO AN EXPERIMENTAL SITE	**Chapter 5: APPLICATION OF RELIABILITY TO A BRIDGE PILE FOUNDATION**

All previously described reliability-based methodologies are applied and compared. An evaluation of reliability-based safety factors is done and the results compared with the values recommended by some design codes. The importance of reliability-based sensitivity analyses in pile design is highlighted for different types of uncertainties associated to axial pile design.

Chapter 6: COSTS AND RISKS ASSOCIATED TO AXIAL PILES

In this chapter a study is presented concerning the relationship between investment costs (soil characterisation, performance or not of load tests and construction), the reliability of the pile(s) and the risk costs (different risk levels). Such analyses and derived information would support cost-benefit decisions and allow the engineer to understand where it is advantageous to spend time, money and resources.

Chapter 7: CONCLUSIONS

Finally, the relevant conclusions of this dissertation are presented, as well as recommendations and notes for future research based on the outcomes of this work.

Figure 1.1 – Outline of the dissertation “Reliability and Cost Models of Axial Pile Foundations”

Chapter 2

REVIEW ON RELIABILITY FOR GEOTECHNICAL DESIGN

2.1 INTRODUCTION

Due to the booming in construction after World War II (1939-1945) large advances in civil engineering were made, and right after, Freudenthal (1945, 1956) introduced classic reliability-based tools. These methodologies have been gradually implemented in design codes (since the 70's until nowadays), transforming the traditional deterministic designs (WSD/ASD[1]) into reliability-based designs that provide a more consistent assurance of safety. With reliability tools, the structure under study achieves safety, functional and performance requirements for a target reliability level (β_T) and a specific period of life, based on the uncertainties involved and on a probabilistic analysis.

In conventional practice, the geotechnical design consists in a simple application of partial or global safety factors (SF). These SF are based on past experience of the design engineers (Casagrande, 1965). This is the way that engineers are used to insert the uncertainties in the design of geotechnical structures. For many reasons, these methodologies need to be adapted to incorporate reliability analyses and probabilistic bases.

Codes, such as Eurocodes in Europe (CEN, 2002a), Load and Resistance Factor Design (LRFD) specifications in United States of America (Paikowsky, 2003, 2004), Limit State Design (LSD) codes in Canada (Green & Becker, 2001; Becker, 1996a,b, 2003; NRC, 2006) and "Geo-code 21" by JGS[2] in Japan (Honjo & Kusakabe, 2000, 2002; Honjo, 2003; Honjo & Nagao, 2007; Honjo

[1] Working Stress Design / Allowable Stress Design

[2] Japanese Geotechnical Society

et al. 2000a, 2003b, 2005, 2009, 2010d), or others (Zhang *et al.*, 2003) have already changed the design methodology, introducing new safety concepts that require probabilistic and statistical knowledge. This modifications aim to take into account the major uncertainties present in a project or design, allowing the knowledge and control of the probability of failure.

Reliability theory was transferred from other areas, such as structures or aerospace, and it requires special adaptation to deal with the geological environment (Allen, 1975; Honjo, 2011). In this chapter the basis of reliability, such as the tools that are relevant for reliability in geotechnical engineering and the difficulties in practical application are presented. The geotechnical engineering problems have a number of unique aspects, as referred by the document of US Army Corps of Engineers (1999):

- coefficients of variation (COV, ratio of the standard deviation to the mean) are related to the variability of natural materials, which may need to be assessed on a site specific basis;
- geotechnical parameters may have relatively high COV (some may exceed 100%) and may be correlated;
- either a total stress analysis or an effective stress analysis can be performed to define the soil strength. In the former, the uncertainties in strength and pore pressure are lumped; in the latter, they are treated separately;
- in soils, properties vary from point to point, requiring consideration of spatial correlation.

Another of the main problems encountered is to adequately quantify the value of standard deviation and mean of the geotechnical parameters. These values are influenced by the amount of samples analysed, which depend on the project budget, that most of the times is very limited. Due to this limitations, the standard deviation value used in analysis is generally higher than the actual value, and most of the times, the effect of a spatial correlation of soil properties is not taken into account. Besides, in geotechnical engineering, a reliability analysis may involve more than one performance function (e.g.: a pad foundation with inclined loading would have the load capacity function and slip function).

These kinds of complexities have slowed the adoption of probabilistic methods in geotechnical area, but geotechnical engineers need to get familiar with the terms "random variables", "uncertainties", "reliability index", "performance function", "probability of failure" among others. This chapter will provide the definitions and explanations of these basic terms and their application in practical design.

2.2 UNCERTAINTIES

The random variables (RV) in reliability analyses are the basic variables of the problem, the fundamental variables that have some uncertainties associated. These variables characterise the behaviour and safety of a structure or system (e.g.: load, unit weight, strength, etc.).

2.2.1 Classification of uncertainties

There are some classifications systems for uncertainties, depending on what we want to know or do with them and depending on the area we work. But, in general, the uncertainties can be:

Aleatory or **statistical** (non-cognitive): when talking about random events that differ in each experiment. These uncertainties are treated as random variables with probability distributions (PDF) based on statistical data, and cannot be eliminated by more accurate measurements,

Epistemic or **systematic** (cognitive): these uncertainties are due to things we could know but we do not control entirely (e.g.: the level of accuracy of measurements or use of too simplified calculation models). Axioms of probability and statistics are limited to treat this type of uncertainty, and sometimes it is necessary to use other tools, like Fuzzy arithmetic approaches (Dodagoudar & Venkarachalam, 2000; Kunitaki *et al.*, 2008; Tumay *et al.*, 2008) or Bayesian update (Tang, 1971; Ditlevsen *et al.*, 2000; Zhang *et al.*, 2004; Phoon & Kulhawy, 2005; Miranda, 2007; Zhang *et al.*, 2009a).

Table 2.1 presents the terms used in literature to describe this dual meaning of uncertainty.

Table 2.1 – Terms used in literature to describe dual meaning of uncertainty
(adapted from Baecher & Christian, 2003)

Uncertainty due to naturally variable phenomena in time or space	Uncertainty due to lack of knowledge or understanding of nature
Aleatory uncertainty	Epistemic uncertainty
Natural variability	Knowledge uncertainty
Random or stochastic variability	Functional uncertainty
Objective uncertainty	Subjective uncertainty
External uncertainty	Internal uncertainty
Statistical uncertainty	Inductive probability
Chance	Probability

The problem facing the geotechnical or geological engineer is epistemic rather than aleatory (Christian, 2004). In civil engineering problems and reliability analyses, the uncertainty is normally divided in the following groups (Der Kiureghian, 1989):

Physical uncertainties: associated with the inherent uncertain nature of material's properties, geometry and the variability and simultaneity of different actions. These uncertainties are generally not known at first, but can be estimated through observations or past experience, and controlled through a large database or quality control.

Modelling uncertainties: these uncertainties come from the theoretical approaches to the actual behaviour of materials and its simplifications (models). They can be considered through a coefficient that represents the relation between the real and predicted response. The tools of geotechnical analysis are countless, and while some are well founded and with little model error, others have large, and largely unknown errors.

Statistical uncertainties: this group includes the uncertainty associated with the finite size and fluctuations in the samples used in estimation of relevant statistical parameters. This type of uncertainty is impossible to reduce or eliminate (refer to Honjo *et al.*, 2006).

Human errors: this type of uncertainty is due, not only to natural variation in the execution of multiple tasks, but also due to interventions and errors in the processes of documentation, design, construction and use of the structure. Knowledge of these uncertainties is limited. It is, nevertheless, clear that it causes an increased uncertainty. An adequate margin against human error is important, because this type of uncertainty is not considered in reliability-based design methodologies, which usually lacks a way of taking account these uncertainties (Simpson, 2011).

2.2.2 Characterisation of uncertainties

Routinely, the variables considered in geotechnical reliability-based analyses are continuous RV that assume a continuous range of values over a domain, with a probability or frequency associated (PDF). These variables are characterised by their statistical moments: mean and variance.

Some examples of continuous RV in geotechnical reliability are the friction angle, the parameter of a test (e.g.: SPT N value) and error of a predictive model (Honjo & Kuroda, 1991; Fenton, 1999a). Even though there is a great number of possible RV, only the variability of the most important and influent ones are worth considering (Baecher & Christian, 2003). For this, one can perform a sensitivity analysis study to select the most influent RV.

Commonly the probability distributions of RV are assumed as Normal ($\sim N$) or Lognormal ($\sim LN$), although there are many other types of distributions, such as Binomial, Geometric, Poisson for discrete variables or Exponential, Gama, Beta, Gumbel for continuous variables, among others. The selection of the type, or shape, of the distribution is sometimes made because it simplifies computations (Li *et al.*, 2012). Nevertheless, the goodness of the fitting between the data set and the candidate distributions can be assessed by some standard statistical tests, such as the Chi-squared or Kolmogorv-Smirnov tests, found in most statistical textbooks. In Annex A one can consult additional basic concepts of probability theory and statistics of RV. For additional information on this subject (fit suitability) please refer to Matsuo & Kuroda (1974), Lacasse & Nadim (1996), Liang *et al.* (1999) and Low (2005).

Uncertainties' distributions are researched and discussed in Kamien (1995), Gilbert (1996), Baecher & Christian (2003) and Phoon (2006a). As example, Najjar & Gilbert (2009) studied the effects of having a lower-bound in the distribution of the resistance. Their study indicate that the incorporation of a lower-bound capacity for design of pile foundations is expected to provide a significant increase in the calculated reliability and provide a more realistic and rational basis for design (see also Najjar, 2005).

The studies of Kulhawy & Mayne (1990) and Phoon & Kulhawy (1999a,b) present a literature review for the COV of inherent variability, correlation length[3], and COV of measurement error. When data from the specific site in study is not available, or is not sufficient to estimate variability of RV, uncertainty can be characterised by a COV observed at a similar site. Typical values of COV for soil properties and *in situ* test results have been compiled and reported by Phoon *et al.* (1995), Jones *et al.* (2002), and more recently by Phoon (2008a) or Uzielli *et al.* (2005, 2007) – consult in Annex B a summary of these recommendations. Traditionally, the COV below 15% are considered low COV, if between 15% and 40% they are considered medium COV and if higher than 40% they are considered high COV.

2.2.3 Soil and spatial variability

In geotechnical engineering the uncertainty depends on many variables, such as the site conditions, the degree of equipment and procedural control, and the precision of the correlation model used (Einstein, 2001). As such, soil parameters' statistics that are determined from total uncertainty

[3] also called scale of fluctuation or autocorrelation value

analyses can only be applied to the specific set of circumstances for which the design soil properties were derived (Phoon & Kulhawy, 1999a,b; Kieu Le, 2008).

The following approach referred in Haldar (2008) can be adopted for the specific site conditions. The soil property under study (X) can be quantified for use in reliability analyses from the measured *in situ* or laboratory soil property (X_m) using eq.(2.1).

$$X = T(X_m, tr) = T(t + w + mr, tr) \tag{2.1}$$

Where $T(\ )$ is the transformation function, tr is the zero mean transformation variability, t is the deterministic trend, w is the inherent variability and mr is the measurement variability.

Based on this and using first-order Taylor series expression (Phoon & Kulhawy, 1999b) and second moment probabilistic method, the mean (μ) and standard deviation (σ) can be written as:

$$\mu_X \simeq T(t, 0) \tag{2.2}$$

$$\sigma_X = \sqrt{\left(\left.\frac{\partial T}{\partial w}\right|_{(t,0)}\right)^2 \cdot {\sigma_w}^2 + \left(\left.\frac{\partial T}{\partial mr}\right|_{(t,0)}\right)^2 \cdot {\sigma_{mr}}^2 + \left(\left.\frac{\partial T}{\partial tr}\right|_{(t,0)}\right)^2 \cdot {\sigma_{tr}}^2} \tag{2.3}$$

This formulation is used when the parameter (like soil cohesion or friction angle) is derived from a test (laboratory or *in situ*). Then, that same parameter will be introduced in a model (resistance model) that has its bias associated with the model prediction. This formulation (eq.(2.3)) is not required when the test values (laboratory or *in situ*) are used directly in the resistance model. These are two different uncertainties to be considered called the transformation uncertainty (transformation of a parameter in other type of parameters) and model uncertainty (transformation of a parameter in a resistance value).

Furthermore, in geotechnical engineering many RV vary continuously over space and/or time (Rungbanaphan *et al.*, 2010; Kim, 2011). These variables are referred to as random fields, where an autocorrelation between the values of that variable exists. Normally the parameters measured at considerable distances are independent, but, if one measures the value of a parameter, the uncertainty in the value at a nearby point, becomes less uncertain, because it is highly correlated to the value of the first point (Vanmarcke, 1977; Vanmarcke, 1983; Fenton & Vanmarcke, 1990; Fenton, 1994, 1999b).

To characterise a random field, the mean and standard deviation (or variance) are required, plus some quantification of the autocorrelation function and the correlation length/distance. The autocorrelation function describes the reduction in correlation between parameters with distance.

As referred in *Chapter 6* of Phoon (2008a), the fluctuations of a soil property around their spatial trends exhibit in general some degree of coherence/correlation as a function of depth. The similarity between the fluctuations recorded at two points as a function of the distance between those two points is quantified by the correlation structure. The correlation between values of the same material property measured at different locations is described by the autocorrelation function. A significant parameter associated with the autocorrelation function is called the autocorrelation value (or correlation length) and represents a length over which significant coherence is still manifested. The variance reduction function, both vertical and horizontal directions, is based on the relative position of the pile and the location where the parameter was measured (Honjo, 2009).

The initial standard deviation can be reduced using estimation variance functions that depend on parameters such as the number of sampling points, normalised sample line length and the autocorrelation distance (Wang & Wang, 2007).

The autocorrelation value (correlation length), denoted θ, is determined by fitting analytical expressions to the sample autocorrelations (see Annex C). Typically the exponential or squared exponential autocorrelation functions are used to fit the data using the least square error approach. Some models are presented in Table 2.2.

Table 2.2 – Theoretical models for autocorrelation functions

	Model	Autocorrelation function	θ
1	Triangular	$\begin{cases} 1-\dfrac{\lvert\Delta z\rvert}{a} & for\ \lvert\Delta z\rvert \le a \\ 0 & for\ \lvert\Delta z\rvert \ge a \end{cases}$	a
2	Single exponential	$\exp\left(-\dfrac{\lvert\Delta z\rvert}{b}\right)$	b
3	Double exponential	$\exp\left(-\left[\dfrac{\lvert\Delta z\rvert}{c}\right]^2\right)$	c
4	Second-order Markov	$\exp\left(-\dfrac{\lvert\Delta z\rvert}{d}\right)*\left(1+\dfrac{\lvert\Delta z\rvert}{d}\right)$	d
5	Cosine exponential	$\exp\left(-\dfrac{\lvert\Delta z\rvert}{e}\right)*\cos\left(\dfrac{\lvert\Delta z\rvert}{e}\right)$	e

Thus, when one wants to consider the spatial variability, it is possible to the reduce variance of a soil parameter. The Table 2.3 presents the variance reduction functions for triangular, exponential and squared exponential autocorrelation functions. Figure 2.1 presents the value of the reduction coefficient (single exponential) as function of the autocorrelation value (θ) and thickness averaging (H - normalised sample line length).

Table 2.3 – Values of the correlation and variance reduction functions

	Model	Variance reduction function	θ
1	Triangular	$\begin{cases} 1-\dfrac{H}{3\theta} & for\ \lvert\Delta z\rvert \le a \\ \dfrac{H}{\theta}\cdot\left(1-\dfrac{H}{3\theta}\right) & for\ \lvert\Delta z\rvert \ge a \end{cases}$	a
2	Single exponential	$2\cdot\left(\dfrac{\theta}{H}\right)^2\cdot\left(\dfrac{H}{\theta}-1+e^{\left[-\frac{H}{\theta}\right]}\right)$	b
3	Double exponential	$\left(\dfrac{\theta}{H}\right)^2\cdot\left(\sqrt{\pi}\cdot\dfrac{H}{\theta}\cdot E\left(\dfrac{H}{\theta}\right)-1+e^{\left[-\frac{H}{\theta}\right]^2}\right)$	c

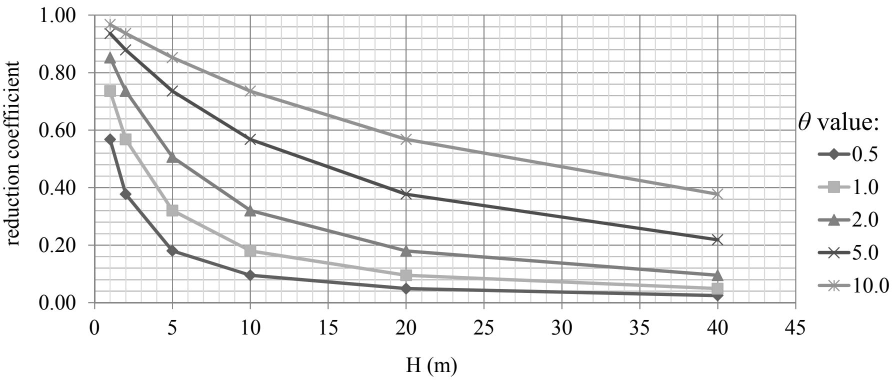

Figure 2.1 – Reduction coefficient (single exponential) as function of the autocorrelation and normalised sample line length

These reductions are seldom contemplated, mainly due to the following reasons: there are some difficulties in practical application, reliability is adapted from areas where COV alone are sufficient and this consideration becomes many times a complex consideration, turning reliability-based analyses into a slower methodology. Furthermore, since the consideration of autocorrelation reduces the variances, it could be said that it is consistent and conservative (although technically and economically incorrect) to perform probabilistic analyses without considering the spatial autocorrelation in computations (DeGroot & Baecher, 1993; Lacasse & Nadim, 1996; Kulhawy & Phoon, 1996; Honjo *et al.*, 2007; Oka & Tanaka, 2009; Papaioannou & Straub, 2012). The Annex B shows examples of autocorrelation values for some geotechnical parameters.

The statistical estimation error has also influence in the standard deviation of a parameter. The final standard deviation value can be calculated as explained in eq.(2.4).

$$\sigma_{final} = \sqrt{(\sigma_{corr})^2 + (\sigma_{stat})^2} \tag{2.4}$$

Where σ_{final} is the final standard deviation, based on statistical estimation error (σ_{corr}) and spatial variability (σ_{stat}).

Furthermore, it is possible to consider two types of situations for soil uncertainties consideration. General and local designs are common situations encountered by geotechnical engineers (Honjo & Setiawan, 2007a; Honjo, 2009). As stated in Honjo (2009), when general estimation is taken into account the uncertainty is considered in design without the concern of the relative position of the investigations and the structures' location(s). On the other hand, when local estimation is taken into, the relative position of the investigations and structure(s) to be designed, leading to a considerable reduction of the uncertainties. An example of this local estimation consideration for determining local average is when one wants to design a foundation for a building and makes a detailed soil investigation at the spot; in this case the uncertainties to be considered in ground conditions are low.

Engineers have been treating these conditions in implicit ways. These treatments are a part of the so-called *engineering judgement* in traditional engineering. With reliability techniques these situations are taken into account in a more explicit way when quantifying the uncertainties (Beacher & Christian, 2003; Honjo, 2009; Honjo *et al.*, 2011). The research of Kim (2011) presents the study four components concerning soil investigations/characterisation (statistical characteristics, data measurement, simulation, and educational training) and focus on the improvement of site investigation performance in geotechnical engineering, thereby improving reliability analysis in geotechnical practice.

2.3 Levels of reliability analyses

The reliability of a structure, or more generalised, a system, can be evaluated by different methods, each one with its level of accuracy. The different reliability analysis (RA) levels depend on the way that uncertainties interfere in the design (Madsen *et al.*, 1986; Nowak & Collins, 2000). The RA levels are summarised in Table 2.4 and described in the following paragraphs. More details about each RA level can be consulted in next sections.

RA level zero: corresponds to deterministic analyses, *i.e.* the traditional way of design. Here, the RV involved are taken as deterministic values and the uncertainties are taken into account by a global SF based on past experience (empirical) – section 2.4.

RA level I: is referred to the semi-probabilistic methods. Deterministic formulas are applied to the representative values of RV, called nominal or characteristic values, and they are then multiplied by partial SF. The characteristic values are established by statistical data, while the partial SF are based on RA level II or level III. This is the most used level and it is the one proposed by the design codes for ordinary structures – subsection 2.5.1.

RA level II: uses approximate probabilistic methods. The RV are characterised by their distribution and statistical parameters (mean and standard deviation) to evaluate the reliability of the limit state considered. The probabilistic evaluation of safety is done by approximated numerical techniques, a simplified hypothesis (one of the most mentioned methods is FORM, a reliability index method) – subsection 2.5.2.

RA level III: corresponds to full, or pure, probabilistic analysis. It is based on techniques that take into account all the probabilistic characteristics of the RV, and the probability of failure is analytically calculated. The analytical calculation is only possible when the problem is simple. In more complex problems (not linear) one needs to carry out simulations methods (e.g.: Monte Carlo simulations - MCS) – subsection 2.5.2.

RA level IV: is called risk analysis, where the consequences (cost) of failure are taken into account and the risk (consequences or costs multiplied by the probability of failure) is used as a measure of the reliability. In this way different designs can be compared on an economic basis taking into account uncertainty, costs and benefits – section 2.6.

Table 2.4 – Levels of reliability analyses

Information	RA level				
	zero	I	II	III	IV
Geotechnical parameters	✓	✓	✓	✓	✓
Calculation method (deterministic)	✓	✓	✓	✓	✓
Design parameter (statistical basis)	×	✓	✓	✓	✓
Variability of parameters	×	×	✓	✓	✓
- mean and standard deviation	×	×	✓	✓	✓
- PDF	×	×	×	✓	✓
Costs	×	×	×	×	✓
Type of analysis:	Global SF	Partial SF	e.g. FORM	e.g. MCS	Risk

✓ Considered

× Not considered

2.4 TRADITIONAL DESIGN PRACTICE (RA LEVEL ZERO)

The traditional way of design, called WSD or ASD, was used for many years, assuming that all loads and strengths were deterministic. The actions or its effects (E) applied to the structure under study are compared to the resistances (R) though a global SF:

$$E_{allowed} \leq \frac{R}{SF} \tag{2.5}$$

This SF is introduced as a safety margin, to reduce the risks of rupture (failure or collapse) and excessive deformation. The values adopted for the SF are always bigger than 1 and have a big range because they depend on many factors. For example, they depend on the basis for design, such as the use of empirical methods (bigger SF) or based on load tests (lower SF). SF can vary between 2 and 3 but it is also very common in geotechnical engineering to reach SF of 4.

The advantages of the traditional design are the simplicity and the fact that the structures that are designed by this method have generally good performance. On the other hand, as a disadvantage, is the indirect consideration of the uncertainties involved. Consequently, there is no consideration of the levels of uncertainty given by each variable (more or least accurate). Also, the SF depends mostly on the engineer experience and judgment, and it is not directly related to the level of risk.

2.5 RELIABILITY ASSESSMENT (RA LEVELS I, II AND III)

Reliability tools have been gradually implemented in structural design codes, transforming the traditional deterministic design into reliability-based design. In spite of all the qualities, such as the rational design, knowledge of the uncertainty and probability, information of the most important behaviours and parameters that influence them; the reliability-based methodologies have disadvantages, especially in geotechnical environment. Among these disadvantages are:

- difficulty in evaluating the PDF (f) of the RV, because of a lack or insufficient amount of data, especially the approximation at the tail (the most important part of the distribution – see Figure 2.2);
- difficult quantification of modelling and transformation uncertainties;

- and even with all the information above, it is many times difficult or impossible to calculate the double integral that gives the probability of failure value, given by eq.(2.8), that represents the shaded area in Figure 2.2 – failure domain.

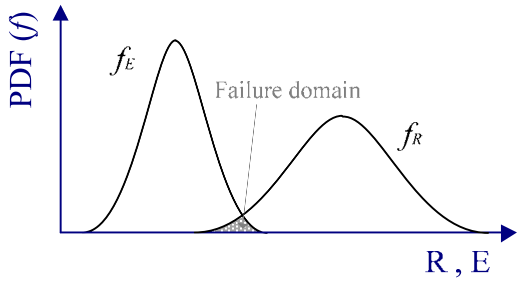

Figure 2.2 – Probability density function for Resistances (R) and Actions/Effects (E)

Nevertheless, researchers worked to overcome these difficulties. One of the first attempts was the First-Order Second Moment (FOSM) proposed by Cornell in 1969 that first introduced the concept of reliability index. FOSM ignores the shape of the PDF, using only the mean, variance and covariance of the RV, and the calculation model is linearized using Taylor's expansion. Next, Ditlevsen (1973), Hasofer & Lind (1974) and Rackwitz & Fiessler (1978) continued to develop FOSM, and the calculation method First-Order Reliability Method (FORM) was proposed, solving the invariance problem of FOSM and being nowadays one of the basic tools for RA level II. A full account of the reliability methods development and evolution can be found in Manohar & Gupta (2005).

2.5.1 Partial safety factor methodology

RA level I is the lowest level to take into account the uncertainties. The uncertainties are inserted in the design by SF. Partial SF are evaluated by qualified engineers in code calibration, with sufficient safety margin, that already take into account the uncertainties involved in the design. Therefore, to use RA level I methods, the designer does not need to understand the probability theory or know the statistics of the problem.

The partial SF methodology can be divided in two categories:

- Material Factor Approach (MFA), originated from Europe and also known as Partial Factor Approach (PFM);

- Resistance Factor Approach (RFA) or multiple resistance factor approach (MRFA), originated from North America (named LRFD[4] in USA and LSD[5] in Canada).

In both approaches, the design is carried out in a deterministic manner without requiring explicit use of the probabilistic description of the variables. The SF, as referred, are calculated based on RA level II or III. Until now there are still many debates about these two methodologies (Simpson, 2000): Should the uncertainties be treated on their sources (MFA) or at actions effects and resistances (RFA)? The following paragraphs present more information about these two approaches.

MFA treats the uncertainties in their origin, using general design verification formula presented in eq.(2.6). A partial SF is applied to the characteristic values (e.g.: unit weights) before it is inserted in a design equation to calculate actions or resistance. Although MFA looks theoretically correct, it encounters many difficulties in practical application.

In **RFA**, the loads/actions and resistances are first calculated based on the characteristic values and then partial SF are applied using eq.(2.7). In some cases resistance can be divided in different terms, each resistance component would have a partial SF (like pile tip and side resistances).

$$\gamma_{E_1} \cdot \sum_j \gamma_{E_{2j}} \cdot E_j\left(x_{k_1}, \dots, x_{k_n}\right) \leq R\left(\gamma_{k_1} \cdot x_{k_1}, \dots, \gamma_{k_m} \cdot x_{k_m}, \gamma_{R_1}, \dots, \gamma_{R_m}\right) \tag{2.6}$$

$$\sum_j \gamma_{E_j} \cdot E_j\left(x_{k_1}, \dots, x_{k_n}\right) \leq \gamma_R \cdot R\left(x_{k_1}, \dots, x_{k_m}\right) \tag{2.7}$$

Where, γ_{E_1} is the load factor, $\gamma_{E_{2j}}$ (j = 1,...,n) are the partial SF multiplying to load components E_j, R is the resistance, γ_{k_i} (i = 1,...,m) are the partial SF for characteristic values of the material properties (x_{k_i}) and γ_{R_i} are the resistance factors.

RFA is much simpler to apply than MFA. It has been traditionally used for ultimate limit state (ULS) checks. More recently also serviceability limit states (SLS) have been brought into the RFA (LRFD) framework.

It is considered that MFA calculation may not be favourable, especially in geotechnical designs, where the engineering judgement plays an important role. In geotechnical designs there should be a philosophy that "a designer should keep track of the most likely behaviour of the structure towards the end of the design calculation as much as possible" (Honjo & Amatya, 2005).

[4] Load and Resistance Factor Design
[5] Limit State Design

This idea is more coincident with RFA. RFA is the simplest format. Here, all uncertainties are covered by one system factor, while MFA involves many more steps. Also, when one investigates the uncertainties in a design calculation, it happens that only total results are comparable (e.g: predicted pile capacity and load test result). In these cases only overall uncertainty can be quantified for use in a reliability analyses. This fact implies that is more reasonable to carry out a calibration with resistance factor approach (RFA) than with material factor approach (MFA). In Annex D are presented in more detail the pros and cons of these two methodologies.

These methodologies represent attempts to apply probabilistic based methods to routine design procedures. They have been used successfully in structural engineering, but their application in geotechnical engineering, especially foundation engineering, has been controversial (Christian, 2004; *Chapter 8* of Phoon 2008a).

2.5.2 Reliability-based methods

As referred, the main goal is to evaluate the probability of the failure region - $domain$ (Figure 2.2). That value can be obtained solving the double integral in eq.(2.8).

$$\mathrm{P}[g(R,E) < 0] = \int_{domain} \int_{R,E} f_{R,E}(r,e)\, dr\, de \quad ; domain = \{R, E : g(R,E) < 0\} \tag{2.8}$$

Where $\mathrm{P}[\ \ ]$ is the probability of the event, $g(\ \ )$ is the performance function that describes the limit state considered, R is the resistance variable and E is the action or effects of action variable.

The two RV considered in eq.(2.8), R and E, are actually macro-variables that result from several other basic variables. For example, the resistance depends on variables such as physical and mechanical properties of the material(s). Furthermore these basic variables are not, in most problems, independent and normally distributed and the performance function is difficult to obtain (see subsection 2.5.3).

The performance function g describes the response of a system for a specific a limit state and it is a function of the basic variables (X_1, X_2, ..., X_n), as shown in eq.(2.9).

$$M = g(X) = g(X_1, X_2, \dots, X_n) \tag{2.9}$$

Where M is the safety margin, $g(X)$=0 is the limit state function, $g(X)$<0 is a failure region and $g(X)$>0 is a safety region. The goal is to calculate the value of the probability of the failure region, denominated probability of failure - pf = $\mathrm{P}[g(X)<0]$.

The performance function can be found in the form of displacements, strains or stresses and depends on the type of limit state. The limit states, according to the design codes, are divided in two groups: ultimate limit states (ULS) and serviceability limit states (SLS).

According to Eurocode (CEN, 2002a; Gulvanessian *et al.*, 2002), ULS are associated with collapse or with other similar forms of structural failure (generally corresponds to the maximum load-carrying resistance), and SLS corresponds to conditions beyond which specified service requirements for a structure or structural member are no longer met. SLS can be reversible or irreversible.

The most basic way to solve a reliability problem, and the double integral, is when the performance function is a linear combination of the basic variables R and E (eq.(2.10)), which are i.i.d.[6] RV and normally distributed ($\sim N$ (μ;σ^2)). In these conditions, the exact value of the reliability index and probability of failure is calculated directly according to eq.(2.11) and eq.(2.12).

$$M = g(R, E) = R - E \tag{2.10}$$

$$\beta = \frac{\mu_M}{\sigma_M} = \frac{\mu_R - \mu_E}{\sqrt{{\sigma_R}^2 + {\sigma_E}^2}} \tag{2.11}$$

$$pf = \Phi(-\beta) = 1 - \Phi(\beta) \tag{2.12}$$

Where M is the safety margin, g is the performance function, R the resistances, E the actions, β the reliability index, μ the mean value, σ the standard deviation, pf the probability of failure and Φ is the Normal CDF[7] with mean 0 (zero) and variance 1 (one) (see Table A.1 in Annex A).

When there are more than two RV (X_1, X_2,..., X_n), i.i.d., normally distributed and M is a linear combination of these, the same direct procedure can be used to evaluate the probability of failure, as shown in eq.(2.13) and eq.(2.14).

$$M = g(X) = a_0 + \sum_{i=1}^{n} a_i \cdot X_i \tag{2.13}$$

$$\beta = \frac{\mu_M}{\sigma_M} = \frac{a_0 + \sum_{i=1}^{n} a_i \cdot \bar{\bar{X}}_i}{\sqrt{\sum_{i=1}^{n} {a_i}^2 \cdot {\sigma_{X_i}}^2}} \tag{2.14}$$

Where a is a constant value, n is the number of RV and $\bar{X}$ is mean value.

[6] Independent and identically distributed random variables

[7] Cumulative density function

If the variables are non-normal and/or statistically dependent there is the need to transform the non-normal RV in normal equivalent RV (especially in the tail part – Figure 2.3) and to introduce in calculations the covariance matrix given by eq.(2.15).

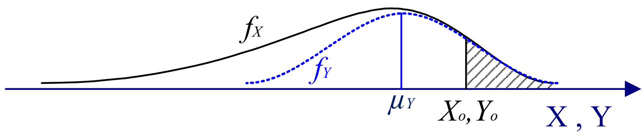

Figure 2.3 – Approximation of a Normal distribution to a non-normal RV (considering the most important region for the reliability problem, the tail)

$$C_X = \begin{bmatrix} \sigma_{X_1} & Cov(X_1, X_2) & \cdots & Cov(X_1, X_n) \\ \vdots & \vdots & \ddots & \vdots \\ Cov(X_n, X_1) & Cov(X_n, X_2) & \cdots & \sigma_{X_n} \end{bmatrix} \tag{2.15}$$

Where C_X is the covariance matrix of random variables X, Cov the covariance between variables given by $Cov(X_i, X_j)=E[(X_i-\mu_i)\cdot(X_j-\mu_j)]$, where E is the mean value.

However, many times, the problems/designs that the engineer may encounter are not that simple. Thus, it is very difficult, and sometimes impossible, to solve the double integration in eq.(2.8). So, to overcome this problem, many authors/researchers studied alternative methods to approximate the value of the probability of failure. These methods are presented next.

First- and **Second-order methods** (RA level II), they are approximated probabilistic methods.

- FOSM: this method uses a linear approximation to the non-linear function, with statistically independent and normally distributed RV, where only first and second moment of the RV are used to assess reliability (Cornell, 1969; Ang & Cornell, 1974).
- FORM and SORM[8]: use linear and non-linear approximations to the failure surface, respectively, with statistically dependent and/or non-normally distributed RV. Both methods use standard normal space for calculations (Hasofer & Lind, 1974; Ang & Tang, 1984; Der Kiueghian *et al.*, 1987; Zhao & Ono, 1999a,b; Phoon, 2004; Low & Tang, 2007).

[8] First-Order Reliability method and Second-Order Reliability Method

In the case of a non-linear performance function the response can be non-normal, even if the RV are normally distributed. In FOSM, an approximation by Taylor series (linear approximations) is done near the design point of the problem (X^*) calculated by the maximum likelihood or sometimes using the mean values. With FOSM, the reliability index can be estimated by the generalised version, eq.(2.14). The dependence of this method in the point X^* and its invariance problem was solved by Hasofer & Lind (1974), as referred earlier.

FORM, sometimes called AFOSM (Advanced First-order Second Moment), was the solution. In FORM, the first step is to transform all RV in standard normalised RV (see eq.(A.10) in Annex A). After, the performance function is written with the normalised RV, $g(Z)$, and in the normalised space we select the design point (Z^*) that is the point closest to the origin (*i.e.* the mean values of initial RV, X). Finally the reliability index (β) can be evaluated as the distance between origin and the design point Z^*. Also sensitivity factors (α) can be calculated. With sensitivity factors one can assess the influence of each RV, this can help choosing the necessary number of basic variables of a problem. Based on this factors the RV taken into account can be reduced, without compromising the accuracy of the reliability calculation. See explanation in Figure 2.4.

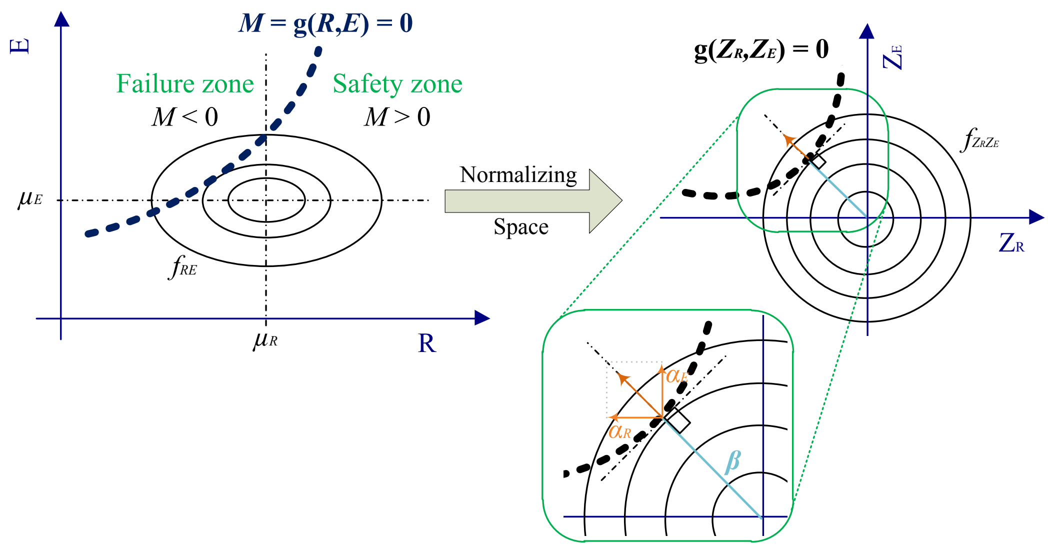

Figure 2.4 – Normalisation of the space and representation of the design point (Z^*), reliability index (β) and sensitivity factors (α)

An alternative version of FORM is SORM, sometimes called SOSM (Second-Order Second Moment). This method is used when the linear approximation (FORM) does not give satisfactory results. Therefore, a 2nd order approximation is done near the design point. This method assumes two ways of calculation, curvature fitting or point fitting (Der Kiureghian *et al.*, 1987,1991; Zhao & Ono, 1999a,b), since for the former the performance function needs to be continuous and twice differentiable.

Simulation methods (RA level III): these methods can be applied to RV with non-normal distribution and complex performance functions (e.g.: nonlinear, FEM[9]). They use all the statistical information such as mean, standard deviation and PDF. The reference method is the ordinary Monte Carlo Simulations (MCS), some techniques can be also applied. These techniques include importance sampling, stratified sampling, Latin hypercube and Markov chain Monte Carlo.

Ordinary MCS is the most accurate method. All subsequent methods are enhancements for the time consumption of MCS when the calculation for each simulation is very time consuming. This way, instead of doing, for example, 10,000,000 simulations, a group of significant points are selected, based on different techniques, enabling the determination of the probability of failure with less time consumption for calculation (like selecting only points near failure, or selecting groups and ensuring that all groups of the solutions are represented in calculations - refer to Annex E for more information about these techniques).

The previous described methods (FOSM, FORM and SORM) are RA level II and have some limitations (Honjo & Amatya, 2005; Teixeira *et al.*, 2012d). The reference method is the MCS. This is a powerful tool to determine the solution of an integral. This method can be used as a validation method of the previous ones or to evaluate the solution of large and complex problems. The methodology for ordinary MCS is:

- Generation of the basic RV (X_1, X_2,...,X_n), considering their distributions (n simulations);
- calculate the performance function $g(X)$ for each n generation;
- and estimate the probability of failure as the sum of the simulations that fail divided by the total number of simulations.

Although this is a very simple and powerful method, it has the disadvantage of being time consuming when the probability of failure is too low or when the problem is too complex. But, with reduction of variance techniques this problem can be solved (Phoon & Honjo, 2005; Phoon, 2008a; Zhang *et al.*, 2010, 2011a, 2012).

MCS allow the determination of the shape of the performance function distribution, permitting more accurate estimation of the probability values. In spite of the simplicity and range of

[9] Finite Elements Method

application, MCS needs the shapes of the basic RV distributions. Hence, and very important, the distribution obtained for the performance function is only accurate if the shapes assumed for the RV are accurate. Regardless of these disadvantages, MCS technique is likely to become increasingly common as computing capabilities continue to improve (Foschi *et al.*, 2002).

2.5.3 Complex performance functions

The performance function is a function of the basic RV and can be very simple. For instance, when the resistance is estimated based on resistance parameters or empirical formulas. But also it can be difficult to estimate, for example when FEM analysis is necessary.

The development of structural analysis that combines FEM with the probability theory began in the 70's, allowing Stochastic or Probabilistic FEM (SFEM or PFEM), or ever Random Finite Elements Method (RFEM) that were developed more recently (Fenton & Griffiths, 2008). These methods take into account the uncertainties in geometry and material properties of a structure, as well as the uncertainties in actions (Smith & Griffiths, 2004; Veiga, 2008; Otake *et al.*, 2011).

When dealing with a complex problem, such a complex formula or necessity to use software, the performance function or software can be replaced by a simpler function. This can be achieved with Response Surface (RS) method or Artificial Intelligence (AI) techniques.

In RS method the performance function is replaced by a formula given by eq.(2.16). An application of this technique can be seen in Bucher & Bourgund (1990) and Honjo *et al.* (2010a,b,c).

$$\bar{g} = A + X^T \cdot B + X^T \cdot C \cdot X \tag{2.16}$$

Where A is a scalar, B a $n\times1$ vector, C a $n\times n$ matrix, and X the RV vector, with n being the number of variables of the problem.

One example of AI technique is the Neural Networks (NN or ANN). These NN are the most used ones. The NN are a mathematical model to simulate the neuron behaviour, and includes an input layer containing the components of the input vector, one or several hidden layers and an output layer (Figure 2.5). Each layer has its corresponding neurons (processing elements) linked by connections with associated weights. The number of neurons and their weights are adjusted so that the prediction error is minimised. After training (or learning), tests and validation, the network acts as an approximating function for the relationship between the input and the output vectors. This technique has been used successfully in many studies and showed that facilitates greatly the reliability calculations and the optimisation during performance-based design (Shahin *et al.*, 2001; Goh & Kulhawy, 2003; Zhang & Foshi, 2004; Neto *et al.*, 2006; Chau, 2007; Tinoco, 2012).

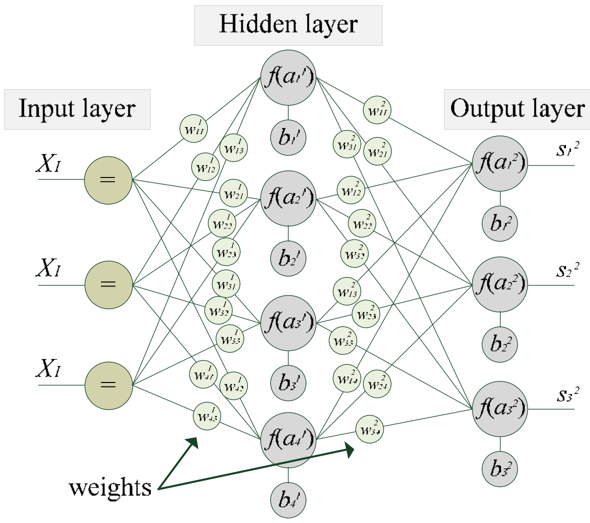

Figure 2.5 – Neural network (NN) representation

In Papadrakakis *et al.* (1996) study, the application of NN with MCS seemed very promising, because the application of RA level III in problems where FEM analysis is needed implies a great computational effort. Lee & Lee (1996), Nawari *et al.* (1999), Deng *et al.* (2003), Goh *et al.* (2005), Ardalan *et al.* (2008), Nejad *et al.* (2009) and Chan & Low (2012) are just some examples of helpful applications of NN together with reliability-based design of piles.

2.5.4 Code drafting and calibration

The development of a design code includes not only the determination of the SF values but also the verification of the nominal, also called characteristic, values of the parameters to be used and the calculation procedure. According to Frank *et al.* (2004), a code needs to contain:

- the scope and objectives (application range and levels of safety);
- demand function (the frequency of occurrence);
- target safety levels (based on recommendations or existing structures);
- format for presenting the code requirements;
- and design-checking formulas.

In calibration of the code one can use: (1) judgment and experience, fit with traditional design codes (WSD/ASD), (2) reliability analysis based on rational probability theory or (3) a combination of these two approaches. Reliability concepts and their application in recommended SF (such as LRFD and LSD) are well known in structural engineering, and their adoption in geotechnical engineering design is recommended (Bathurst *et al.*, 2008). This would allow a more consistent and rational framework of risk assessment in geotechnical engineering.

A reasonable number or code calibration works have been carried out in structural engineering since Ellingwood *et al.* (1982). However, geotechnical code calibration started only in the past decades, with Barker *et al.* (1991), Phoon *et al.* (1995), Honjo *et al.* (2000a), Paikowsky (2004), among others.

Moreover, the characteristic values have the same importance as the SF in design analyses. A bad choice of the characteristic values could lead to a behaviour of the structure far from the reality. Lacasse & Nadim (1996) defend that the SF sometimes is not the adequate parameter to quantify safety. How the characteristic value should be determined still brings discussion. For materials like steel and concrete, with quality control in production and therefore a low dispersion in their properties, the usual values adopted for the characteristic values of those properties are the 1 or 5% fractiles.

But in the case of a soil property that value would be very far from the mean value due to its high variability. Furthermore, a value so far from the mean value could lead, as referred already, to a behaviour of the element far from the most likely one. The selection of characteristic values is fundamental to all calculations, and their definition has been the most controversial topic in the whole process of drafting Eurocode 7 (Simpson & Driscoll, 1998; Frank *et al.*, 2004).

Some design engineers use the mean value, others use a more conservative value, or even use statistical methods, or for example half of standard deviation below mean. Also, there is the Bayesian approach based on comparable experience. Bayesian approaches are especially powerful because they provide probabilities on the state of nature rather than on the observations (Orr, 2000). In a Bayesian approach, the data analysis process starts with a given probability distribution (estimated based on previous experimental results, experience and professional judgement) that is called prior distribution. When additional data becomes available, we use it to update the prior distribution into a posterior distribution. The basic tool for this updating is the Bayes theorem which weights the prior information with the evidence provided by the new data (Ditlevsen *et al.*, 2000; Garbulewski *et al.*, 2009; Park *et al.*, 2012).

It is conventional that the R and E characteristic values are chosen as the lower and higher fractile values, respectively, to define resistances and load SF. The effectiveness of this method was discussed in Kieu Le (2008). For each fractile value of R (1 to 50%) and E (50 to 99%), load and resistance factors were calculated while changing the coefficient of variation (from 0.05 to 0.30). It was concluded that, for the case of two i.i.d. variables (R,E), Normal and Lognormal-distributed, and a linear performance function $M = R - E$, it is recommended to take the higher fractile value for E, however, using the low fractile value for R may not be as effective, instead, the mean value should be chosen.

Other concern in determination of SF is that there are an infinite number of them (different categories, combinations, conditions, structures, etc.). Therefore, as stated in Phoon *et al.* (2003), if the goal of the SF is to maintain uniform reliability, a single resistance factor is not adequate. The proposal originating from structural reliability-based design is to partition the parameter space (spanning typical ranges of deterministic and statistical parameters) into smaller domains and calibrate a single resistance factor for each domain. Phoon *et al.* (2003b) also emphasises that deviations from the target reliability index can be controlled to an acceptable level by adjusting these sizes of the domains.

The design value method (DVM) and FORM are normally used to assess partial SF for a specific case (Honjo *et al.*, 2002a,b; Phoon *et al.*, 2000). In DVM, the partial factor results from the relationship between the design point values and the characteristic values of the basic variables. The design point is the maximum likelihood point located on the limit state line. This method has some shortcomings, namely with the high non-linear nature of performance function (Honjo & Amatya, 2005). Thus, Kieu Le (2008) proposed a procedure to determine load and resistance factors by keeping the philosophy of DVM but using MCS instead of FORM (further information in Chapter 3, section 3.3).

In the geotechnical field, the design resistance of piles is very uncertain and the Eurocode 7 reckoned that the major uncertainty is not the strength of the *in situ* ground but the way the construction would interact with it. Therefore, the SF is essentially a factor of the resistance model, rather than on the strength of material. In such cases, it is appropriate and recommended to use resistance factor approach – RFA, such as LRFD or LSD, rather than material factor approach – MFA. This approach is adopted by Eurocodes (CEN, 2002a,b, 2007).

The Annex F presents some recommendations for SF and characteristic values ideas, by different design codes and the Figure 2.6 shows suggested minimum SF for compressed piles.

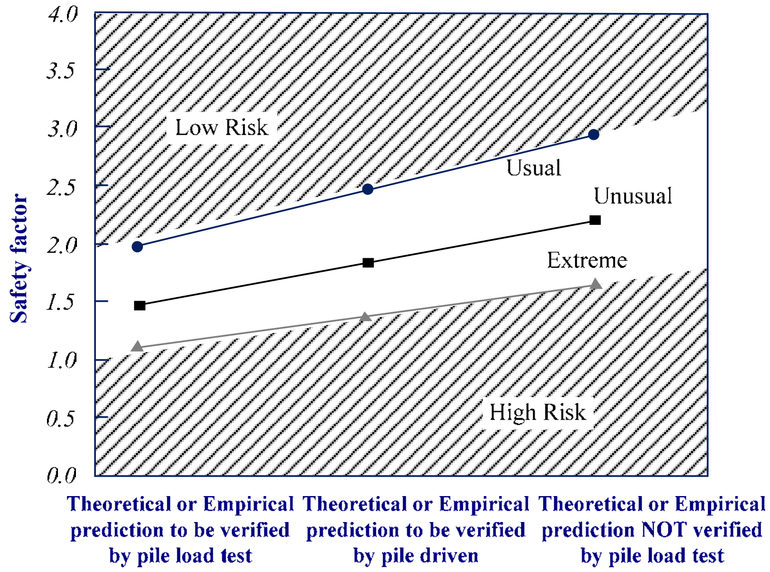

Figure 2.6 – Suggested minimum safety factor for compressed piles (data from US Army Corps of Engineers, 1999)

According to this figure, the usual ranges are between 2.0 and 3.0 (corresponding to multiplying SF[10] between 0.50 and 0.33). When comparing these values with the recommendation in European and North American codes, it is possible to realise that most of the cases in Eurocode's SF recommendation fall below this interval (SF<2.0). Only when considering piles in tension this SF rises to 1.6, but still below the denominated usual values. On the other hand, Canadian recommendations (NBC, CFEM and CHBDC) fall within this interval (between 1.67 and 3.33), while the American (USA) have lower recommendation (between 1.25 and 2.22). These values depend greatly on the type of design adopted (empirical methods, tests, etc.). Also, they depend on the type of construction (category) and type of pile selected. Nevertheless it is possible to say that, in daily practice and not so important construction works, these factors are just a mere indication, as for the real design engineers tend to use personal experience and engineering judgment.

When using the SF recommended by the design codes the engineer obtains a level of safety that is supposed to meet the recommended by those design codes. Remember that the partial SF can be assessed by RA level II or III and stating, for example, a reliability index of 3.8 (recommended in CEN, 2002a). The methodology to determine the level of safety of a structure that was designed by RA level I, taking into account the partial SF would be: (1) assume the shape of the PDF of R and

[10] It is important to notice the conventions/assumptions concerning SF. A safety factor should reduce resistances and enlarge actions, therefore if it is a multiplying SF: γ_R<1 and γ_E >1. However, in Eurocodes the SF are defined differently. The characteristic value of the resistance are normally divided by the resistance SF, as such, in this particular case $\gamma_{R'}$ >1. With the exception of the SF discussion in this page, this dissertation adopts multiplying SF convention γ_R<1 and γ_E >1.

E (usually Normal or Lognormal distributions are admitted) and (2) calculate the mean and standard deviation of R and E.

If one assumes a Normal distribution, the problem in eq.(2.17) where the unknown variables are the mean and standard deviation (μ_R, σ_R), sensitivity factor (α_R) and reliability index (β). These last two are parameters defined by the codes (e.g.: annex C of Eurocode 0).

$$\begin{cases} R_d = \gamma_R \cdot R_k \\ R_d = \mu_R - \alpha_R \cdot \beta \cdot \sigma_R \\ R_k = \mu_R - 1.645 \cdot \sigma_R \end{cases} \Leftrightarrow \begin{cases} \mu_R - \alpha_R \cdot \beta \cdot \sigma_R = \gamma_R \cdot (\mu_R - 1.645 \cdot \sigma_R) \\ \alpha_R = 0.8 \quad and \quad \beta = 3.8\ (defined\ in\ Eurocode\ 0) \\ R_k = \mu_R - 1.645 \cdot \sigma_R \end{cases} \tag{2.17}$$

2.6 Risk analyses (RA level IV)

The risk involved in each project or design can be studied by risk assessment and risk management. These types of analyses are the highest level of reliability one can admit. It allows the determination of what are the actions, strategies and investments necessary, worthwhile and cost-effective to minimise troubles or interruptions to the initial plans.

Risk and reliability analyses are multidisciplinary engineering fields requiring a solid foundation in one or several classical civil engineering disciplines, in addition to a thorough understanding of probability, risk and decision analyses. The general practice is to identify the threats (potential hazards), next to assess the likelihood of those threats (probability) and finally to evaluate its impact(s), check Figure 2.7.

Considering an activity with only one event with potential consequences, risk is the probability of occurrence of that event multiplied by the costs or consequences given the event occurs. In these studies, decision trees are drawn to understand the problem, and the risk will be assessed and decisions made (Figure 2.8). The task of risk-based analysis is to combine the variability of the inputs, based on knowledge of how the system operates, to obtain estimates of the variability of outputs. For complex systems with many sources of variability, this is clearly not a simple task.

As well, it is necessary to understand how to manage the risk. It is important to choose a cost-effective approach (the elimination of the risk should not cost more than the consequences of that risk, sometimes one should accept the risk instead of eliminate it). Moreover, management of the risk can be made in many ways, using old or new resources or contingency planning.

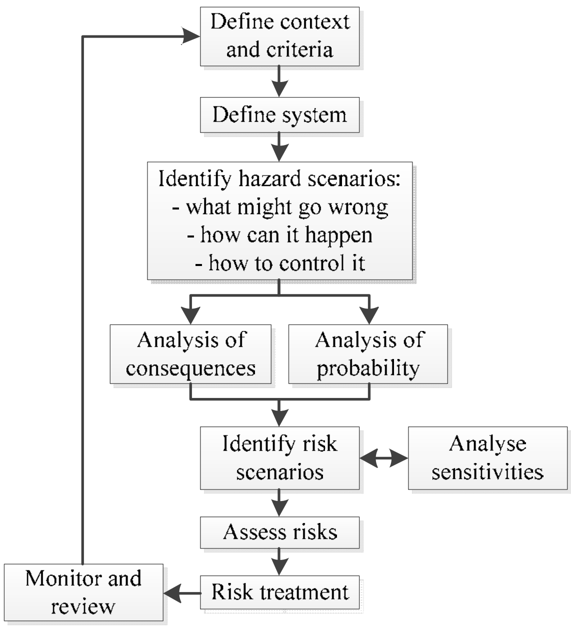

Figure 2.7 – Generic representation of the flow of risk-based decision analyses (Faber & Stewart, 2003)

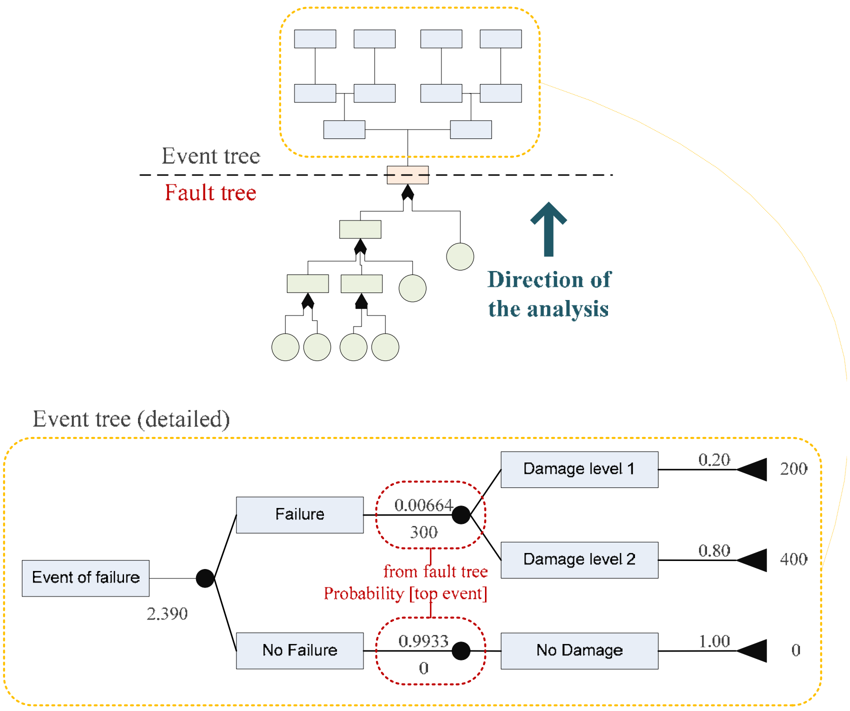

Figure 2.8 – Event tree and Fault tree relationship (adapted from Sousa, 2010)

In logic tree analysis (fault trees, event trees and cause/consequence charts), fault trees cannot directly accommodate dependent basic events, but for civil engineering applications this limitation is a serious one as dependency is a common feature rather than the exception for the events contributing to the risks. In principle, event trees can deal with such dependencies; though in practice, this requires great care during event-tree construction. This limitation is, however, not present for Bayesian probabilistic networks, which seem to be a very promising tool for risk analyses in general, for more detail see Jensen (1997), Faber (2007) and Sousa (2010).

Bayesian networks (BN) were developed as a decision support tool for AI engineering. Until then AI systems were mostly based on "rule based" systems, which besides many merits also have some problems in dealing with uncertainties, especially in the context of introducing new knowledge. BN models the domain of uncertainty, they are based on classical probability calculus and decision theory and instead of replacing the expert they support her/him. BN can be used at any stage of a risk analysis, and may readily substitute both fault trees and event trees in logical tree analysis.

2.7 USUAL AND RECOMMENDED RELIABILITY LEVELS

The reliability of a structure is assessed by the reliability index (β) or the probability of failure (pf). These two parameters have the relationships shown by eq.(2.12) and Figure 2.9 (see Annex G). Notice that in any area it is impossible to quantify all the uncertainties involved, therefore, the concept of probability of failure is simply a measure for comparison and not a real measure of the probability of the collapse.

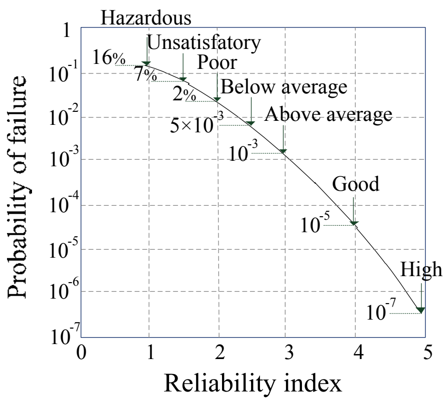

Figure 2.9 – Probability of failure vs. reliability index, with classifications proposed by US Army Corps of Engineers, adapted from Phoon (2008a)

The acceptable probability for non-compliance of a requirement can only be analysed as a function of costs associated with its non-compliance. This probability depends on many factors, such as the type of structure (its function, occupancy and lifetime/design working life), the social tolerance to a non-compliance (failure, rupture, among others) and the average number of victims in case of failure.

For the verification of ULS of structures should be considered the probabilities of failure given in Table 2.5, while for verification of ULS of geotechnical structures should be considered the values in Table 2.6. The Eurocode 0 (in annex B) and International Standard Organization (ISO) (CEN, 2002a; ISO, 1998) also provide recommendations for the reliability index of structures, see Table 2.7 and Table 2.8.

Table 2.5 – Values to be used for the probability of failure (and reliability index) in ULS – Henriques (1998)

Level of safety	Economic consequences		
	Not serious	Serious	Very serious
Low	pf=10^{-3} / β=3.1	pf=10^{-4} / β=3.7	pf=10^{-5} / β=4.3
Normal	pf=10^{-4} / β=3.7	pf=10^{-5} / β=4.3	pf=10^{-6} / β=4.8
Hight	pf=10^{-5} / β=4.3	pf=10^{-6} / β=4.8	pf=10^{-7} / β=5.2

Table 2.6 – Values to be used for reliability index in geotechnical structures, and correspondent probability of failure – Kamien (1995)

Expected performance level	β	pf	
Hazardous	1.0	0.16	(1.6×10^{-1})
Unsatisfatory	1.5	0.07	(7×10^{-2})
Poor	2.0	0.023	(2.3×10^{-2})
Below average	2.5	0.006	(6×10^{-3})
Above average	3.0	0.001	(10^{-3})
Good	4.0	0.00003	(3×10^{-5})
Hight	5.0	0.0000003	(1.6×10^{-7})

Table 2.7 – Recommended values for the reliability index, with a design of working life of 50 years – CEN (2002a)

Reliability class	Limit state	Minimum β	pf
RC3	ULS	4.3	8.5×10^{-6}
RC2	ULS	3.8	7.2×10^{-5}
	Fatigue	1.5 - 3.8	7.2×10^{-5} - 6.7×10^{-2}
	SLS	1.5 (irreversible)	6.7×10^{-2}
RC1	ULS	3.3	4.8×10^{-4}

Table 2.8 – Recommended values for reliability index by ISO and correspondent probability of failure – ISO (1998)

Relative cost of safety measures	Consequences of failure			
	little	some	moderate	great
High	β=0.0 / pf > 0.5	β=1.5 / pf=6.7×10^{-2}	β=2.3 / pf=10^{-2}	β=3.1 / pf=9.7×10^{-4}
Moderate	β=1.3 / pf=9.7×10^{-2}	β=2.3 / pf=10^{-2}	β=3.1 / pf=9.7×10^{-4}	β=3.8 / pf=7.2×10^{-5}
Low	β=2.3 / pf=10^{-2}	β=3.1 / pf=9.7×10^{-4}	β=3.8 / pf=7.2×10^{-5}	β=4.3 / pf=8.5×10^{-6}

Therefore, based on the recommendation shown, the ULS should have reliability index between 2.5 (for structures with low risks if failure occurs) and 4.0 (for a more important structure with high risks if failure happens). As for the SLS the probabilities admitted are always smaller (Phoon, 2006b), being between 10^{-2} and 10^{-1} (Henriques, 1998), equivalent to a reliability index of 2.3 and 1.3.

Next figures demonstrate some relationships between costs, probability of failure, risks and consequences for geotechnical structures. The Figure 2.10 represents what we live nowadays. Some public policy organisations have developed and adopted this type of plots for decision making. Adapted from Christian (2004), in Figure 2.11 is the example of Hong Kong Planning Department, Figure 2.12 was developed in The Netherlands and is proposed by the Australia New Zealand Committee on Large Dams (originally referred from Hong Kong Planning Department (in 1994), Versteeg (in 1987) and Australian New Zealand Committee on Large Dams - ANCOLD (in 1994)). The “Tolerable” label is sometimes referred as “ALARP” meaning “as low as reasonably practicable”.

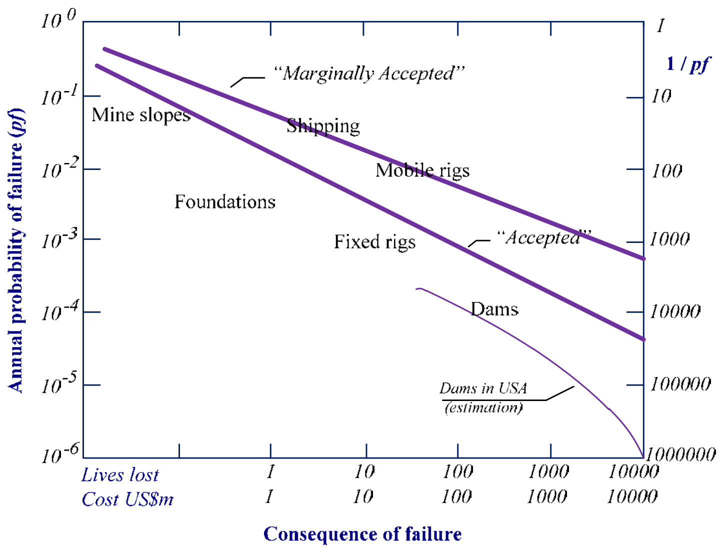

Figure 2.10 – Empirical rates of failure for civil engineering facilities, adapted from Phoon *et al.* (2000) and Beacher & Christian (2003), original from Beacher in 1982

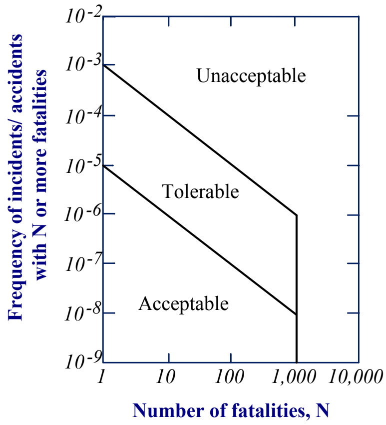

Figure 2.11 – Acceptable social risk for sliding, proposed by Hong Kong's Planning Department in 1994 (adapted from Christian, 2004)

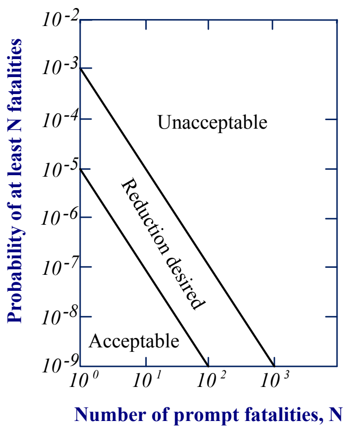

Figure 2.12 – Acceptable social risk for planning and design in Netherlands (adapted from Christian, 2004)

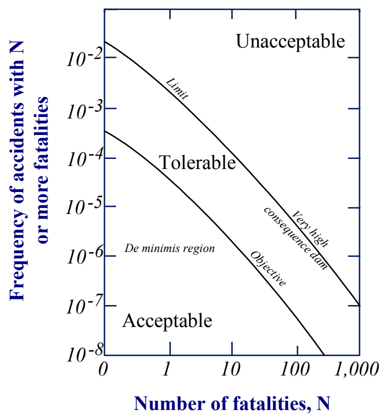

Figure 2.13 – Acceptable social risk ANCOLD (adapted from Christian, 2004)

As said earlier, the probability and acceptable risk that society is willing to take it is very difficult to know and depend on many factors. This is the reason why these proposals are different in each country, reflecting the different negotiations among the designers of the figures. Christian (2004) emphasises that current practice is not to be bound by bright lines separating the regions but to use them as guidelines. Also, regardless of how the figures were developed, they are convenient tools for comparing the results of reliability analyses with acceptable levels of risk.

2.8 APPLICATIONS OF RELIABILITY IN PILE DESIGN

In pile foundations design, the reliability tools have been firstly applied to offshore projects due to its importance and severe consequences in case of failure. Around the 90's, researches have started in uncertainty in design of offshore pile foundations (Tang, 1989; Wu *et al.*, 1989a,b; Tang *et al.*, 1993) and gradually in the XXI century more and more researchers and investments were put into the reliability-based tools. Gilbert & Gambino (1999) presented a study for offshore pile foundations concerning the reliability achieved without site specific boring. This study provides design charts for LRFD and WSD frameworks in order to provide equivalent levels of reliability independently of the use or not of site specific boring.

In spite of all efforts some geotechnical engineers and design engineers still have many reservations and issues concerning the application of reliability tools. "Why Consider Reliability Analysis for Geotechnical Limit State?" (Phoon *et al.*, 2003), intends to draw a clear distinction between accepting reliability analysis as a necessary theoretical basis for geotechnical design and downstream calibration of simplified multiple factor design formats (e.g.: LRFD), with emphasis on the former. Several studies demonstrate reliability analysis as a tool that provides a consistent method for propagation of uncertainties and a unifying framework for risk assessment across disciplines (structural and geotechnical design) and national boundaries.

However, only more recently, multiple researches and case studies have been published. The following tables present a review of the studies that have applied reliability theory in design or in calibration of safety factors of vertically loaded piles (Table 2.9) or laterally loaded piles (Table 2.10). Then, a discussion of the reviews is presented.

Table 2.9 – Resume of literature review in applications of reliability theory in pile foundations (vertical load)

Authors	Key words	Resume and main conclusions
Most cited investigation groups		
Becker (1996b, 2003)	Safety factors, LSD, code calibration	These works present the transition from the traditional WSD to design based on limit state concepts. In spite of some of the resistance SF not being established through rigorous reliability-based concepts, they are considered in the analyses and throughout the design recommendations.
Paikowsky *et al.* (1994); Paikowsky (2003, 2004)	FORM, FOSM, safety factors, LRFD, code calibration	Paikowsky's works provide recommendations and revisions to the driven piles and drilled shafts portions of section 10 (piles) of AASHTO specifications and a detailed procedure for calibrating deep foundation resistance SF.
(...)		

Table 2.9 – Resume of literature review in applications of reliability theory in pile foundations (vertical load)

Authors	Key words	Resume and main conclusions
Most cited investigation groups (...)		
(...)		
Foye *et al.* (2004, 2006a, 2011)	FOSM, safety factors, LRFD, code calibration	These works review the codes of USA, Canada and Europe based on the new trends of reliability-based designs. It provides guidance on the choice of values for load SF, develops recommendations on how to determine characteristic soil resistance under various design settings and develops resistance SF compatible with load SF and the method of determining characteristic resistance. The SF appear to be consistent for all codes (with just little differences between codes related to bridges and to buildings).
Allen (2005a,b)	FORM, FOSM, safety factors, LRFD, code calibration	This report summarises the historical development of the resistance SF developed for the geotechnical foundation design sections of the AASHTO LRFD Bridge Design Specifications, and recommends how to specifically implement recent developments in resistance SF for geotechnical foundation design. In addition, recommendations regarding the load SF for down-drag loads, based on statistical analysis of available load test data and reliability theory, are provided-
Phoon *et al.* (1990)	FOSM, FEM, pile settlement, LRFD, reliability	This early paper presents the evaluation of the reliability and probability of a pile foundation against an allowable settlement limit using FOSM and FEM analyses. The results are presented in concise design charts, easily understood and used by engineers.
Phoon *et al.* (1995, 2000, 2003b)	Reliability-based design, LRFD, safety factors	These works are one of the geotechnical initiatives in reliability-based code development / calibration or pile design. RFA and MFA (LRFD and PFM) are calibrated using FORM and a reliability index of 3.2 in order to produce designs that achieve a known level of reliability consistently.
Phoon *et al.* (2003a, 2011)	Reliability in geotechnical designs, safety factors	These two works discuss the consideration of reliability analyses in geotechnical design and the reliability of reliability-based SF. It is emphasised that "Reliability analysis provides a consistent method for propagation of uncertainties and unifying framework for risk assessment across disciplines (structural and geotechnical design) and national boundaries". However, when investigating the degree of deviation when using the LRFD formulas and SF (calibrated using FORM) developed by AASHTO to more realistic ground conditions (multiple layered soils) is possible to detect that the target reliability prescribed is not so easily achieved.
Phoon (2008a)	Reliability in geotechnical designs	This book presents different reliability theory and tools in different areas of geotechnical engineering. It presents practical computational methods that can be easily followed, and also geotechnical examples illustrating reliability analysis and design. This book aims to encourage geotechnical engineers to apply reliability-based design in a realistic context that recognises the complex variabilities in geomaterials and model uncertainties arising from a profession steeped in empiricism. This book serves as a valuable reference for engineers and a resource for students and it is especially relevant as geotechnical design becomes subject to increasing codification and to code harmonisation across national boundaries and material types.
Honjo *et al.* (1999)	Reliability-based design	This is one of the first works, of this research group, about combining the performance design concept and the reliability design methodology in pile foundation design for ULS and SLS.
Honjo *et al.* (2002a,b)	FORM, safety factors calibration	A procedure is developed to calculate partial SF based on FORM and DVM. The SF depend on length-diameter ratio and the authors conclude that the SPT N-value dependent system of design has serious limitations for improving the pile design
Honjo *et al.* (2008); Honjo (2009)	Random field, Soil spatial variability, reliability-based design	This work presents the influence of the degree of *in situ* investigation in characterisation of the soil variability (random field) and posterior use in pile reliability analyses. Also, it emphasises the importance of distinguishing between general and local problems and its considerations in reliability based designs.
(...)		

Table 2.9 – Resume of literature review in applications of reliability theory in pile foundations (vertical load)

Authors	Key words	Resume and main conclusions
Most cited investigation groups (...)		
(...)		
Honjo *et al.* (2010a)	Reliability-based design, MCS, uncertainty types, response surface	The examples set by the ETC10 are presented and designed using reliability tools and MCS (RA level III). This paper explains step by step how to perform these analyses and designs, based on different input data and models, and concluding that for these examples it is not the soil uncertainty that controls the major part of the uncertainty.
Honjo & Kusakabe (2000, 2002); Honjo (2003); Honjo *et al.* (2000b, 2003b, 2005, 2009, 2010d); Okahara *et al.* (2003); Honjo & Nagao (2007); Watabe *et al.* (2009); Nagao *et al.* (2009)	Reliability-based design, safety factors, code calibration	As seen previously, not only north American codes are being revised to incorporate reliability tools and reliability-based SF. In Japan considerable amount of work is also being done to revise the major Japanese structural and geotechnical design codes from the traditional descriptive specifications to performance-based specifications, and from working (or allowable) stress design codes to the limit state design codes.
Zhang *et al.* (2001)	Reliability comparison, LRFD, FORM	This work presents a reliability analysis (FORM) that compares and discusses the reliability achieved by a pile group and a single pile. It was concluded that when considering the pile system effect the reliability achieved is higher than when not considered or when compared to single piles.
Zhang & Tang (2002); Zhang (2004)	Bayesian tools, LRFD, resistance safety factors	This study shows how results from static pile load tests could be incorporated into pile design using Bayesian theory by updating the resistance SF in LRFD. A criterion of minimum acceptable test outcome is proposed for assisting decision-making in load tests.
Zhang *et al.* (2005)	Reliability, LRFD, resistance safety factors	Aiming international harmonisation of the failure criterion considerations, this paper studies the reliability levels associated with various failure criteria and actors for loads and resistances. The results presented that the bias of the failure criteria has a significant influence on the reliability of piles, and also the use of different SF (different codes recommendations) can cause considerable differences in the calculated reliability.
Zhang *et al.* (2009b)	Uncertainty types, reliability-based design, safety factors	This paper presents the characterisation of the uncertainties associated with large diameter bored pile design (parameters and model). It is shown that the parameter uncertainty alone cannot explain the disparity between predicted and measured pile capacities. Therefore it is presented a method to characterise with more reliability the model uncertainty, including Bayesian update and then how to proceed and evaluate resistance SF for pile design.
Zhang & Dasaka (2010)	Reliability, soil spatial variability	This work presents the study about the influence of the soil variability, spatial variability, soil investigation and their way of consideration in reliability-based design of pile foundations (founding depth). As expected, improvements are achieved when considering more information and kriging model (Zhang *et al.*, 2011a). However, design model errors, human judgement errors and construction effect also have influence.
Wang (2009a); Wang & Kulhawy (2008b)	Reliability-based design, SLS, FORM	This paper makes use of a relationship between β_{SLS} and β_{ULS} to infer the β_{SLS}, since ULS reliability is specified in some design codes. Three different design methods are considered: semi-empirical analysis using *in situ* and laboratory test data, analysis using static loading test results, and analysis using dynamic monitoring results. The results for the cases studied indicate that β_{SLS} varies slightly and it is generally larger than 3, corresponding to an expected performance level of “above average”.
(...)		

Table 2.9 – Resume of literature review in applications of reliability theory in pile foundations (vertical load)

Authors	Key words	Resume and main conclusions
(...)		
Wang *et al.* (2011b,c)	Reliability-based design, MCS	An expanded reliability-based design is presented. The procedure simple, based on MCS, and is equivalent to a sensitivity study on probability of failure versus the design parameters (length and diameter of the pile).
Most recent investigations		
Aoki *et al.* (2002a,b)	FORM, safety factors, reliability	The reliabilities achieved with the global SF of the Brazilian norm (1.6 to 3.0) were not sufficient to reach the reliability required, the main conclusion was that these SF need to be revised and adapted to new reliability trends.
Yamamoto & Karkee (2004)	Reliability	End bearing load transfer characteristics of bored precast piles equipped with expanded fabric bulb in tip region is investigated in this paper. The reliability investigations include the vertical load resistance aspects as well as the confidence limits for vertical movement.
Arrúa *et al.* (2005)	FORM, safety factors, reliability	Reliability-based analysis was applied to pile in loess (collapsible soil). SF were analysed and the respective probability of failure evaluated using FORM.
Yang (2006)	FORM, LRFD, safety factors	A new reliability-based quality control criterion on driven piles is developed (optimised LRFD). Plus, this paper provides a study on the number of pile load based on an acceptance criterion for the quality control of driven piles using various load test methods and target reliability indexes.
Yang & Liang (2006)	FORM, LRFD; reliability-based design	This study presents a statistical database to describe the increase in pile capacity with time. The research shows that normal distribution can be used to properly describe the set-up effect. The aim of this paper is to insert this type of information in reliability-based LRFD. The incorporation of this effect would reduce designed pile lengths.
Cherubini & Vessia (2007)	Reliability, FORM, safety factors	A reliability analysis is performed to take into consideration the variations in formulations/methods and values of the side resistance of bored piles. Normal and Lognormal distributions were studied and some differences in results were detected. The comparison between reliability and the partial SF approaches suggests that one be careful when poor statistical details are given on design variables.
Fenton & Griffiths (2007)	LRFD, reliability-based design	This work presents the results of a preliminary study into the effect of a soil's spatial variability on the settlement and ultimate load statistics of a pile. The results are used to provide recommendations on approaches to reliability-based deep foundation design at the SLS and ULS.
Misra *et al.* (2007); Roberts & Misra (2009)	Reliability, MCS, LRFD, safety factors	These works paper state that a probabilistic load-displacement analysis is desired over a traditional SF approach, due to all the uncertainties in pile foundations. The use of such probabilistic techniques can provide powerful methods for design of pile foundations. Based on the tests variability, MCS are performed and SF histograms developed for SLS.
Kunitaki *et al.* (2008)	MCS	Probabilistic and "possibilistic" approaches are considered, involving, respectively, the MCS and concepts of fuzzy arithmetic. The results are compared. Fuzzy approach showed efficiency in dealing with the uncertainties of the model, a very good computational efficiency.
Ching *et al.* (2009)	LRFD, code calibration	This research intends to solve the issue of not having complete information in load tests results (load test not conducted to failure) in calibrating LRFD. Using a probabilistic framework and a pile database, the results show that, although incomplete, this information helps in calibrating the resistance SF, and that they are in agreement with the SF in the current Taiwan design code.
Najjar & Gilbert (2009)	Reliability, model uncertainty, LRFD, code calibration	This paper presents a reliability-based methodology that inserts the information about a lower-bound limit on the resistance of the pile. The effect of this consideration is studied and it is concluded that it can cause significant increase in the calculated reliability even if it is an uncertain estimate Also, a proposal for LRFD evaluation is done, expecting to provide a more realistic quantification.
(...)		

Table 2.9 – Resume of literature review in applications of reliability theory in pile foundations (vertical load)

Authors	Key words	Resume and main conclusions
Most recent investigations (...)		
(...)		
Kohno *et al.* (2009)	Reliability-based design, uncertainty	The reliability index achieve for a grouped-pile foundation is assessed. The result of 3.1 was the same for the grouped-pile and shallow foundations studied, that were designed following current design specifications.
Park *et al.* (2009) Kwak *et al.* (2010)	FORM, MCS, safety factors, LRFD, code calibration	Different types of FORM and MCS were used to calibrate the resistance SF for static bearing capacity of driven steel pipe piles. The target reliability selected was 2.0 to 2.33 for pile group and 2.5 for single pile. Resistance SF are recommended for pile foundations, construction practice and soil conditions in South Korea.
Klammler *et al.* (2010)	Soil spatial variability, safety factors, LRFD	Site and shaft specific LRFD resistance SF are given based on the assumption of lognormal load and resistance distributions and existing formulas recommended by the FHWA. Results are efficiently represented in dimensionless charts for a wide range of target reliabilities, shaft dimensions, and geostatistical parameters including nested variograms of different types with geometric and/or zonal anisotropies.
Dithinde *et al.* (2011)	Model uncertainty, reliability, safety factors	The uncertainty of the model of the pile is studied for ULS and SLS design, using classic static formula and a pile database from South Africa. The uncertainty in the load-settlement prediction is characterised by fitting it with hyperbolic equation. The statistics reported are required for further reliability analyses and for calibration of the resistance SF
Elachachi *et al.* (2012)	Soil spatial variability, reliability	This research presents a model developed to include a description of the soil spatial variability within the framework of geostatistics, where the correlation length of soil properties is the main parameter.
Kim & Lee (2012)	Safety factors, LRFD, code calibration	This paper presents a framework for calculating LRFD for axially loaded pile foundations using ICP design method. Different levels of reliability are studied (2.33, 3.0, 3.5) and it was concluded that the effect of base-to-shaft capacity ratios on resistance SF are noticeable while the effect of dead-to-live load ratios is not.
Cai *et al.* (2012)	Model uncertainty, reliability	This work presents a proposal for evaluating the capacity of pile foundations in soft clay deposits in China based on CPTu results. Then, the capacity prediction's reliability is assessed by comparing it to load test results. A higher reliability was presented in the proposed methodology, when compared to other methods.
Stuedlein *et al.* (2012)	Reliability-based design, safety factors, LRFD	This paper proposed new recommendations for design of augered cast-in-place piles in granular soils. This proposal is assessed using pile load tests and resistance SF, for compression and uplift are calibrated for LRFD.
Park *et al.* (2012)	FORM, safety factors, LRFD, code calibration	This work presents an update on resistance SF of axially loaded pile foundations using Bayesian theory, load tests and FORM. It can be concluded that the update of the resistance SF with this information can provide a more economic design.

Table 2.10 – Resume of literature review in applications of reliability theory in pile foundations (lateral load)

Authors	Key words	Resume and main conclusions
Barakat *et al.* (1999)	Reliability-based design	Proposal of an approach that can be used to calibrate design SF to reflect the uncertainties in design of laterally loaded piles under both time-independent and time-dependent effects.
Tandjiria *et al.* (2000)	Response surface, FORM, MCS	The results of response surface methods (linear, reciprocal and quadratic approximate performance functions) in combination with Hasofer-Lind reliability index (FORM) are compared with the ones obtained by MCS, showing an agreement of the results
Phoon & Kulhawy (2005)	Model uncertainty, reliability	The evaluation of the lateral or moment limit and hyperbolic capacity are considered to define model error. The COV vary from 30-40 % while mean bias vary from 0.67 to 2.28. Lognormal probability model appear adequate.
Haldar & Babu (2008a)	Soil spatial variability, reliability, MCS	This paper shows the relevance of spatial variability of soil's undrained shear strength in laterally loaded pile design. Also, MCS technique combined with numerical analysis (DFM – difference finite method) was concluded to be a very useful approach in this regard.
Chan & Low (2009)	Response surface, SORM, MCS	This work proposed a two-step hybrid approach for reliability analysis, first using the response surface method and then using a neural network for modelling the performance function. Comparisons are made using MCS. Different parameter's influence on reliability were studied.

By analysing these tables that summarise the studies that use reliability tools in pile foundations problems, it is possible to emphasise the following:

- The most referred methods are FORM and MCS. These methods refer to RA level II and RA level III respectively. FORM is an approximate method, while ordinary MCS is a pure probabilistic method with higher accuracy. The MCS is a very straightforward method, while FORM has some limitations when complex performance functions are necessary and it is not possible to approximate to Normal distributions. Therefore, the usual procedure is to perform FORM analysis for simple performance functions, and comparing it with MCS results as a reference method.
- More than half of these studies use *in situ* characterisation to evaluate the pile ultimate bearing capacity, namely SPT and CPT. Many warn about the model error bias and how it affects the reliability of the pile. For this reason, most of the studies validate their results or calibrate their model errors with site-specific pile foundations databases and include information of pile load tests.
- It is possible to understand that almost equal amount of work was published concerning bored and/or driven pile foundations.
- From this review it is possible to understand that efforts are being made towards the calibration of safety factors to use with LRFD method, RA level I methodology. However,

it is clear that some downsides are still remaining for this methodology, such as the site-specific and pile-specific conditions.

2.9 CONCLUDING REMARKS

The engineers can deal with uncertainty by ignoring it, by being conservative, by using the observational method, or by quantifying it. In recent years, reliability analyses and probabilistic methods have found wide application in geotechnical engineering and related fields. Christian's paper (Christian, 2004) has concentrated on the imperfections in our knowledge and how they affect our ability to make decisions.

The reliability of an engineering system can be defined as its ability to fulfil its design purpose for some time period. The theory of probability provides the fundamental basis to measure this ability. In estimating this probability, system uncertainties are modelled using random variables with mean values, variances, and probability distribution functions.

This chapter introduces the fundamentals and the most important tools of reliability theory for geotechnical engineering. The probabilistic methods are used in reliability analyses, risk analyses and other similar terms. Starting with Freudenthal in the 50's, and increasingly in the last decades, a significant amount of literature has been published, proposing and detailing various methodologies and applications, firstly for structural engineer and then proceeding to geotechnical that involves certain complexities not found in structural problems.

As examples, the works of Fenton (1997), Baecher & Christian (2003), Christian (2004), Phoon & Honjo (2005) and Phoon (2008a) can be mentioned, as the ones most times referred in literature. Furthermore, in reliability of slopes and embankments we can mention the works of Matsuo & Kuroda (1974), D'Andrea & Sangrey (1982), Christian *et al.* (1994), Matsuo *et al.* (1995), Liang *et al.* (1999), Kanning (2005) and Honjo *et al.* (2010b), in pad foundations the works Honjo *et al.* (2000b), Honjo & Amatya (2005), Griffiths & Fenton (2005), Foye *et al*. (2006b), Fenton *et al.* (2008a,b), Honjo *et al.* (2010c), Wang (2009a,b, 2011b,c) and finally about the topic under study, the most discussed works of pile reliability-based pile designs were presented in Table 2.9. It can be concluded that a great effort is being made by the geotechnical community, to introduce this reliability tools into design of geotechnical structures. However, this subject still brings several discussions in geotechnical community, especially in geotechnical practitioners.

This chapter also showed that the reliability calculations can classified as RA level zero, I, II, III or IV. When deterministic analyses are considered and the uncertainties are not directly introduced in calculations, but through the use of safety factors, we have RA level zero that uses global safety factors and RA level I that uses partial safety factors. When using safety factors methodology, one must keep in mind that the uncertainties in combination with the safety factors are the ones that determine the reliability, and not the safety factors alone. See for example Figure 2.14, a structure with a low safety factor but with small uncertainties associated might have a larger reliability than a structure with a high safety factor and large uncertainties (Lacasse & Nadim, 1996; Christian, 2004; Kanning, 2005).

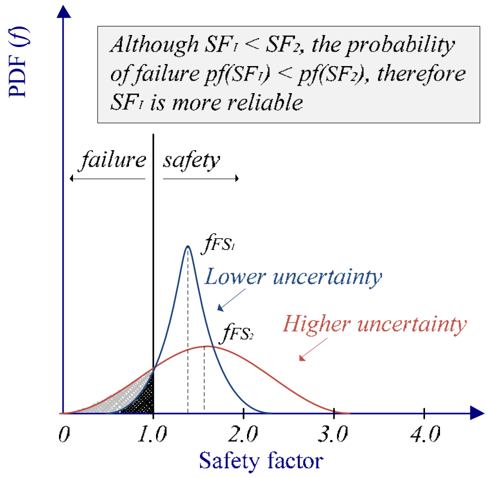

Figure 2.14 – Probability of failure for different safety factors, adapted from Lacasse & Nadim (1996)

The design methods implemented in codes/standards or regulations, use factored design equations (RA level I) that introduce safety factors for nominal (characteristic) actions and resistances that achieve approximated target reliability. These safety factors are optimally obtained from a minimisation of the differences between the target levels and the achieved reliabilities over a sufficiently large, representative number of "calibration design points" (Foschi *et al.*, 2002). The safety level that is inherent in any code or standard is supposed to represent a value judgment of the society; but the actual reliabilities achieved may vary from situation to situation and may differ substantially from the targets for cases other than the calibration points.

Lately, the design codes of Europe, North America and Japan were reviewed and adapted, to incorporate the reliability theory in its design methodologies (Kulhawy & Phoon, 2002; Honjo *et al.* 2010d). But still in use are the safety factors empirically determined.

Also an important thing in safety factor approach is the method to determine the characteristic value to use in RA level zero or RA level I. Some methodologies are reviewed concluding that the engineer should take the value that is expected in principle and not the mere average. The expected value (characteristic value to consider) should take into account the statistical errors in association with the testing method, the inhomogeneity of the soil and the limited number of the test data.

In RA level II and RA level III, the objective is to define the reliability index and the probability of failure respectively, while in risk analyses (RA level IV) a more complete and complex study is done. Risk analyses involve all the possibilities of failure, its probability, costs and/or consequences and decision analysis, hence much more difficult to implement. However, in all three there is a problem with engineering's lack of knowledge and insufficient information available.

As described in this chapter many methodologies are available for reliability analyses, especially when considering more simple problems as in structural area. For geotechnical environment special adaptations should be made, due to his particular aspects such as non-linearity, high coefficients of variation or complexity in determining the behaviour.

From the review done of reliability analyses in pile foundations the most used methods are first-order reliability method and Monte Carlo simulations. The main problems in these applications advent from the complex performance functions and/or the difficulty of getting necessary information for definition of the random variables. Concerning the latter issue, the related geotechnical literature can be consulted. But the soils have a high variability, therefore, the recommended values are not so easily applicable.

Furthermore, some authors argue that the uncertainty that mostly influence the probability of failure of a geotechnical structure is the modelling uncertainty, and not the soil variability. This idea is also defended in geotechnical Eurocode (CEN, 2007).

The models for geotechnical behaviour are numerous, but the geotechnical behaviour is many times difficult to understand and model. These models can be well founded and with little model error, but others have large, and largely unknown errors.

The first step for a reliability analysis should be the selection of the target reliability. This depends on many factors, such as the value given to human life by society, the material costs, loss of services, among others. It varies from location to location, conditions, culture, mentality, economy, and so on. The target reliability should be selected based on experience and recommended values, but also it can be determined by assessing existing structures that have shown a good performance.

In spite of these difficulties, it is believed that the hazard and risk associated with engineered constructions needs to be quantified, using the concepts of uncertainties, reliability, safety and risk. This should be true for any area, including geotechnical engineering. The probabilistic analyses complement the conventional deterministic safety factors and other analyses, and contribute to achieving safe and optimum design (Lacasse & Nadim, 2007). The probabilistic approach adds value to the results with a modest additional effort. The Lacasse & Nadim (2007) study emphasises the usefulness of risk assessment, the importance of engineering judgment in the assessment and the need for involving multidisciplinary competences to achieve reliable estimate of hazard and risk.

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Chapter 3

Reliability-based Methods for Design and Safety of Axial Piles

3.1 Introduction

The reliability of a system is a concern in many engineering areas. A reliability analysis evaluates the probability of a particular behaviour in a time period, with the knowledge of the input parameters randomness (uncertainties). In the case of civil engineering works, these uncertainties mostly concern geometry, constitutive properties and actions.

Unlike in structural engineering, the traditional procedure used in the geotechnical design still introduces the uncertainties through high global or partial safety factors (SF) based on past experience (reliability analysis (RA) level zero or RA level I). But this way of treating the uncertainties does not give a rational basis to understand their influence on the design (recall Chapter 2).

The structural engineering field has seen his design methodologies updated based on reliability analyses and theory of probability. On the other hand, in geotechnical engineering its application has been delayed because of all the inherent variability of the soils and uncertain models. Nevertheless, because of regulation codes and social concerns, geotechnical and foundation engineers need to increase their ability to deal and incorporate these methodologies in design.

This chapter demonstrates and explains a simple way and method to insert the variability and uncertainties on foundations design. Also, discusses how to deal and take into account the uncertainties that come from the soil intrinsic and spatial variability, errors in calculations models (errors in modelling, theoretical approaches and predictions) and human errors.

There are many ways to carry out a reliability-based design. Depending on the level of reliability required or the complexity of the problem, one can carry out RA level II or RA level III analysis, as previously explained in section 2.3 (Chapter 2). Here, reliability-based methodologies will be addressed to give guidelines for geotechnical engineers that aim to carry out reliability-based design in axial pile foundations design process. Also this chapter explains how to get valuable information and how to determine SF based on this type of analyses, taking into account the variability of a geotechnical problem.

All methodologies explained in the following sections are illustrated with a simple application example in the last section of the chapter. Application of these methodologies to real case studies of axial pile foundations will be presented in the next two chapters, Chapter 4 and Chapter 5.

As referred, the key of this methodology is the incorporation of the uncertainties in calculations, they are addressed next.

3.2 General uncertainties to consider

The design of pile foundations still has many limitations and uncertainties, mainly when there is not enough investment in soil characterisation and/or in pile load tests. In addition to pile design based on insufficient data and with theoretical approaches which do not have the model error well characterised, the engineers should consider the physical, statistical and spatial uncertainties and also human errors. The following sections try to guide engineers through the data assemblage and processing for uncertainties characterisation for further reliability analysis.

3.2.1 Deterministic variables

Normally, and also in this study, pile dimensions, such as length and diameter, are considered as deterministic, because they are believed to be manageable and to have low uncertainty when compared with other factors.

3.2.2 Human errors

Human errors can be considered as one of the major causes of structural failure (Nowak & Collins, 2000). They can come from within the construction process and/or from outside the construction process. The knowledge of this type of uncertainties is limited, however, is clear that it causes an increased uncertainty. Furthermore, as referred by Simpson (2011), human errors are not considered in reliability-based designs because it usually lacks a way of taking into account these uncertainties. In this dissertation the influence of human errors was not considered for the following reasons:

- its knowledge is incomplete, disorganised, largely inaccessible, many times intuitive;
- and it is believed that the control of these errors is possible and should be done. It is an important part of the strategy to improve the reliability, because it involves the reduction of causes (frequency of occurrence) and consequences. It is not like other uncertainties that one cannot minimise.

Nevertheless, its influence needs to be reduced by an adequate quality control; calculations should be checked and jobs inspected to control the quantity of errors on construction work.

3.2.3 Soil uncertainties

Soil variability includes the physical uncertainties of the material (soil), its inherent uncertain nature, and statistical uncertainties associated with the finite size and fluctuations in the soil samples used in estimation of relevant statistical parameters (Honjo & Setiawan, 2007b). Furthermore, geotechnical engineers need to take into account the soil spatial variability.

Also, measurement errors fall in this category. It comprises the accuracy of the precision of the measurement. The manufacturer of the test equipment can provide this type of uncertainties. However, the guidelines and testing equipment vary considerably; therefore, due to difficulties in its quantification and due to its small magnitude, many times it is not considered. If one wishes to add the measurement error in the final value of variability, measurement errors are normally around 15-45% for SPT and 5-15% for CPT (Phoon & Kulhawy, 1999a).

When data from the specific site in study is not available or is not sufficient to estimate variability, uncertainty can be characterised by the coefficients of variation (COV) observed at other sites assumed to be similar. As already mentioned in Chapter 2 (section 2.2), many literatures present and discuss geotechnical and soil uncertainties (Kulhawy & Mayne, 1990; Kamien, 1995; Phoon & Kulhawy, 1999a,b; Baecher & Christian, 2003; Uzielli *et al.*, 2005, 2007). Typical values of COV for soil properties and *in situ* test results have been compiled and reported by Phoon *et al.* (1995), Jones *et al.* (2002), and more recently by Phoon (2008a). Recall that some reference values can be consulted in Annex B.

All these types of soil uncertainties are normally considered together. For example, based on the data available from the site, one can evaluate its variability. Then, statistical estimation error, spatial variability and measurement error are included in the same value of variability.

The uncertainties and reliability analyses in this dissertation consider the *in situ* soil tests:

- SPT (Standard Penetration Test);
- CPT (Cone Penetration Test);
- PMT (Menard Pressuremeter Test).

These tests were selected mainly because of the available data of the two case studies of Chapter 4 and Chapter 5. Nonetheless, the SPT is one of the most used *in situ* tests worldwide for soil characterisation and design of pile foundations (Kikuchi *et al.*, 2005; Shariatmadari *et al.*, 2008; Lutenegger, 2009; Dung. *et al.*, 2011), but also CPT and PMT are very common tests, depending on the country and type of soil (Abu-Farsakh & Titi, 2004; Lehane *et al.*, 2007; Viana da Fonseca & Santos, 2008; Long, 2008; Xu *et al.*, 2008; Cai *et al.*, 2009, 2012; Baziar *et al.*, 2012). CPT has a higher range of application and better quality of the results for soil characterisation than SPT (Lunne *et al.*, 1997), while PMT is more indicated for certain types of soils. Besides, PMT is mainly used for deformability evaluation, and specially selected for settlements analysis (Menard, 1975). Taking this into account, the tests' wide range of application, and especially SPT's simplicity and common knowledge in numerous countries, these were the tests selected for the analyses. However, more emphasis is put into SPT-based results.

Table 3.1 presents some values recommended for uncertainties in this type of soil tests (including already the different components of the uncertainties).

Table 3.1 – Values recommended for coefficients of variation of SPT, CPT and PMT

Parameter	COV (%)	Reference(s)
N of SPT	15 to 45	Harr (1987); Kulhawy (1992)
N of SPT	10 to 70	Phoon & Kulhawy (1999a)
N of SPT [10 - 70] blows/ft	25 to 50	Phoon (2008a)
q_c of electric CPT	5 to 15	Kulhawy (1992)
q_c of mechanic CPT	15 to 37	Harr (1984); Kulhawy (1992)
q_t of CPT in clay [0.5 – 2.5] MN/m^2	< 20	Phoon (2008a)
q_c of CPT in clay [0.5 – 2] MN/m^2	20 to 40	Phoon (2008a)
q_c of CPT in sand [0.5 – 30] MN/m^2	20 to 60	Phoon (2008a)
pl of PMT in sand [1600 – 3500] kN/m^2	20 to 50	Phoon (2008a)

When a fair number of SPT, CPT or PMT tests were performed and are available, one can calculate the variability of the site under study. Firstly one determines the trend of the tests and then analyses the residual errors (comparing the trend to tests results), also an estimation of the autocorrelation distance is done (see detailed procedure in Annex C and Annex K) for spatial variability study. If information to carry this methodology is not provided, one can use the values for vertical correlation length recommended (Table 3.2). For horizontal autocorrelation length the values vary between 30 to 40 meters (Hong, 2008), this large value implies a smoothly varying field, while a small value (vertical direction) implies an irregular field. The way of introducing statistical estimation error and spatial variability was previously explained (Chapter 2, section 2.2).

Table 3.2 – Values recommended for autocorrelation for vertical direction of SPT and CPT values

Parameter	θ (m)	Reference(s)
N of SPT in sand	2.4	Vanmarcke (1977)
q_c of CPT in sand	2.2	Alonso (1976)
Friction ratio of CPT in sand	1.3	Alonso (1976)
q_c of CPT in clay	1.1	Alonso (1976)
q_c of CPT in clay	1.2	Vanmarcke (1977)

3.2.4 Calculation model errors

Calculation models (predictive models) are always used in design, for predictions or pre-assessment of proprieties. These models are more or less complex, based on theories and physical models. They can be numerical, such as finite elements method (FEM) or empirical (based on experience and database), with more or less accuracy. Basically, there are countless ways to determine geotechnical parameters or resistance values (Kulhawy & Mayne, 1990; Viana da Fonseca & Santos, 2008).

Bearing capacity of piles can be determined by (1) interpretation of data from full-scale pile loading tests, (2) dynamic analysis methods based on wave equation analysis, (3) dynamic testing by means of the Pile Driving Analyser (PDA), (5) static analysis by applying soil parameters in effective stress or total stress approaches or (6) methods using the results of *in situ* investigation tests, directly or indirectly (Kikuchi, 2007; Shariatmadari *et al.*, 2008).

As stated in Shariatmadari *et al.* (2008), among these different methods, pile load tests and dynamic tests (with PDA or signal matching) are the ones that represent the most reasonable results, but such tests are expensive, time-consuming, and the costs are often difficult to justify for ordinary or small constructions. Direct bearing capacity predicting methods for piles (empirically based) are developed based on *in situ* tests data, especially SPT and CPT, having applications that have shown an increase in recent years.

In fact, from early ages of foundation engineering, the *in situ* tests results have been extensively used in design, and specially for predicting the bearing capacity (Terzaghi & Peck, 1948; Meyerhof, 1956; Menard, 1975; Meyerhof, 1976; Bustamante & Gianeselli, 1982; Shioi & Fukui, 1982; Robertson, 1990; Armar *et al.*, 1991; Lunne *et al.*, 1997; Robert, 1997; Eslami & Fellenius, 1997; Yu & Mitchell, 1998; Lehane, 2003; Aoki *et al.*, 2003; Abu-Farsakh & Titi, 2004; Monzon, 2006; Fellenius *et al.*, 2007; Bustamante *et al.*, 2009, Teixeira *et al.*, 2010; Martins *et al.*, 2012). Thus, many of the major specifications (AASHTO, 2007; CGS, 2006; CEN, 2007; JRA, 2001; among others) have adopted pile bearing capacity estimation formulas based on *in situ* tests, especially SPT, but also CPT and PMT.

For this reason, this study concerning pile foundations design, takes the different *in situ* soil tests and predicts the resistance based on empirical formulas (direct way).

The model error component (bias) has a considerable weight in geotechnical engineering uncertainties and reliability (Wu, 2009). Aggravating this situation is the fact that these errors (bias) are most of the times unknown, not defined, which may be a barrier for performing reliability analyses (Phoon, 2005; Zhang *et al.*, 2004, 2012).

The errors in the predicted values (from formulas or program/software) are taken into account by a factor δ, called bias and given by eq.(3.1), where X corresponds to a parameter of the soil or the resistance of the system under study. This factor is characterised by its mean, variance and a convenient probability distribution, especially to model the tails.

$$\delta = \frac{X_{measured}}{X_{predicted}} \Leftrightarrow X_{measured} = \delta \times X_{predicted} \tag{3.1}$$

When considering the distribution of a resistance parameter or resistance value itself, the left side of the tail (the lowest values) is the part that governs the probability of failure (Figure 2.2, Chapter 2). In most reliability analyses, the foundation bearing capacity (resistance) is modelled using a Lognormal distribution (since it is always positive) and adopting a COV between 0.3 and 1.0 (Najjar & Gilbert, 2009).

The models that will be applied in reliability calculations of the case studies (next Chapters 4 and Chapter 5) are presented in Table 3.3. They are from the Japanese and French design recommendations. But also, other very known and used SPT-based models were studied, like Aoki &

Velloso and Shioi & Fukui (Aoki & Velloso, 1975; Shioi & Fukui, 1982). Table 3.3 summarises the information about the models chosen, including the model errors and reference(s). The formulation of each model and methodology can be consulted in Annex H.

For these last two models and fo the Japanese SHB model, the model error (bias and standard deviation) was evaluated with a Japanese database (Annex I). The S&F model is Japanese, and it was chosen because it was thought to be an adequate model for the evaluation of δ with the Japanese database. On the other hand, the A&V model is a very known model, selected for being referred many times in the literature, therefore assumed to be one of the most used, especially in Portuguese and Brazilian literature.

All choices taken ensure that all models have their formulas based on the well-known tests SPT, CPT and PMT, and have information about the error associated with the prediction. All are modelled with a Lognormal distributions.

Table 3.3 – Values recommended for the model error of the empirical models (pile bearing capacity prediction) based on of SPT, CPT and PMT

Resistance model	Input test	ID		Bias (δ)		Reference(s)
				mean	COV (%)	
Japanese code SHB[1], JGS[2]	SPT	SHB	Tip	1.12	63	Okahara *et al.*(1991); JRA (2001)
			Side	1.07	46	
Japanese code, LSD[3] guidelines, AIJ[4]	SPT	AIJ	Tip	1.14	28	AIJ (2000)
			Side (clay)	4.26	100	
			Side (sand)	2.14	76	
French recommendations	CPT	FRc	Total	1.36	43	Burlon (2011); Burlon *et al.* (2012); Banguelin *et al.* (2012)
	PMT	FRp	Total	1.10	22	
SHB	SPT	-	Total	1.16	39	Annex J
Shioi & Fukui	SPT	S&F	Total	2.86	36	Annex J
Aoki & Velloso	SPT	A&V	Total	2.47	60	Annex J

When comparing the uncertainty (bias) of the Japanese method SHB determined based on predictions using the Japanese database, with the ones published in Okahara *et al.* (1991), one can conclude that these values are very close to each other (check results in Chapter 4).

[1] Specifications for Highway and bridges
[2] Japanese Geotecnical Society
[3] Limit State Design
[4] Architectural Institute of Japan

3.2.5 Actions uncertainties

The uncertainties in actions, both permanent and variable components (G and Q), were adopted from structural engineering codes, namely the Japanese and American (Table 3.4).

In structural engineering the loads and its uncertainties are the ones that mostly influence the behaviour of the structure. But for geotechnical engineering the resistance models or other approximations done to achieve the geotechnical parameters, and even the soil itself, are the most important variable in a geotechnical reliability-based analysis (Najjar & Gilbert, 2009; Teixeira *et al.*, 2011a,b, 2012d). For this reason and since the main goal of this dissertation is the consideration of geotechnical and pile resistance uncertainties, these are the ones that receive more focus during the study.

Table 3.4 – Values recommended for actions uncertainties

Parameter	Bias (δ)		Distribution type	Reference(s)
	mean	COV (%)		
Permanent actions	1.00	10	Normal	JCSS (2001); Holicky *et al.* (2007)
Variable actions	0.60	35	Gumbel (Type I)	JCSS (2001); Holicky *et al.* (2007)
Dead (permanent actions)	1.05	10	Normal	Ellingwood (1996)
Live (50 years) (variable actions)	1.00	25	Gumbel (Type I)	Ellingwood (1996)
Live (25 years) (variable actions)	0.85	35	Gumbel (Type I)	Ellingwood (1996)

3.3 Procedure for reliability-based analysis with *in situ* tests

3.3.1 Methodology

The application of RA level II (FORM or other) or RA level III (MCS or other) is accomplished based on the same steps, for any kind of geotechnical design, as following:

Step 1. Select a target reliability index (β_T) or acceptable probability of failure (pf), eventually from codes (see Chapter 2, section 2.7). As the available codes are not directly concerned with geotechnical engineering it is recommended to obtain β from an existing structure with an acceptable performance (Ellingwood, 1996).

Step 2. Identify the significant failure modes and formulate their functions, named performance functions or limit state functions, $g(X)$; it is roughly defined as the resistances less actions. More specifically for pile foundations, it is possible to define it as a limit for displacement or a limit for bearing capacity. If the performance function is complex and/or requires quite amount of calculation efforts (e.g.: FEM), the surface response method or neural networks can be used to find a linear combination of the basic RV that give the same response as the complex performance functions (see Chapter 2, section 2.5.3).

Step 3. Define the calculation models for the necessary parameters in performance function:

a. the transformation uncertainty (error when transforming the test parameters into other parameters of the soil needed to calculate resistance),
b. and/or the model error, also called modelling uncertainty (error when transforming the test parameters or soil parameters into the resistance);

These models should have their error well-known, and its values are normally obtained in literature.

Step 4. Describe uncertainties, identifying the deterministic values and random variables (RV); along, also define:

a. its distribution types (PDF),
b. statistical parameters (mean, standard deviation or COV),
c. and dependencies between variables (correlation coefficients and covariance matrix – Annex A).

Statistical proprieties may be estimated based on the data obtained from laboratory and *in situ* testing, but if the quantity of information is not sufficient, one should refer to the statistical parameters obtained from similar sites and/or literature recommendations.

Step 5. Finally estimate the reliability, using RA level II or RA level III, of each failure mode defined in step 2 and compare it to the target β_T defined in step 1.

For example, for a pile foundation and soil investigation using SPT, the process of general consideration of the uncertainties for a reliability analysis would be like shown in Figure 3.1.

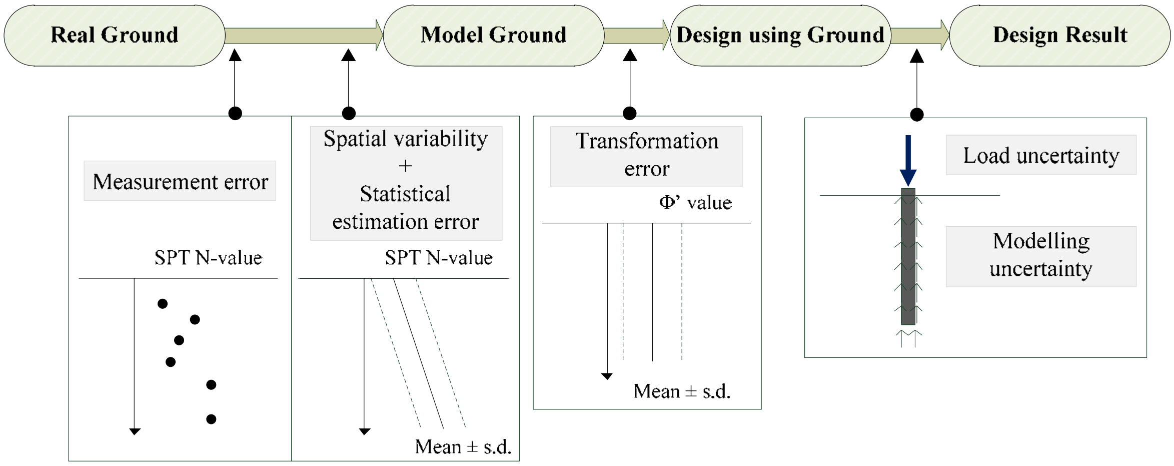


Figure 3.1 – Diagram of an example of a pad foundation and soil investigation by SPT, adapted from Honjo *et al.* (2010a,b,c)

For discussion and comparison, the two most used methods in RA from levels II and III were selected and here presented in more detail. The MCS are widely used because of its higher level and for being the most straightforward method in reliability calculations. The FORM method is very traditional and used since the first studies of structural reliability analysis, but it is an approximate method.

3.3.2 Target reliability

As previously referred in Chapter 2, the target reliability depends on many factors, such as the type of structure, the social tolerance to nonconformity, among many others. Also, it can be based on previous construction works or design codes.

Meyerhof (1970) suggests a probability of failure for foundations between 10^{-3} and 10^{-4} (corresponding to reliability index of 3.1 and 3.7 respectively). Furthermore, some probabilistic studies suggest that the general range of reliability index for pile foundations is 1.7 to 3.1 (Tang, 1989; Barker *et al.*, 1991b; Eslami & Fellenius, 1997; Whitman, 2000). They calculated that the reliability index for pile systems is somewhat higher and is approximately 4.0, corresponding to a lifetime probability of failure of 0.0005 (Paikowsky, 2004). Even though values between 2.5 and 3.0 may be appropriate (Barker *et al.*, 1991b; Paikowsky, 2004), when piles are used in groups these reliability index values can be reduced, since failure of one pile does not necessarily imply that the pile group will fail. This dissertation, and for the cases studied, the reliability index (β) and probability of failure (pf) selected as preferential was based on the recommended values for ULS (see Table 3.5). Thus, the target interval selected for β was [2.5; 4.0].

Table 3.5 – Compilation of the values recommended for target β and respective pf

Reliability index (β)	Probability of failure (pf)	Level	Reference(s)
3.0	1×10^{-3}	Above average	US Army Corps of Engineers (Kamien, 1995; Phoon, 2008a)
2.5	6×10^{-3}	Below average	US Army Corps of Engineers (Kamien, 1995; Phoon, 2008a)
4.3	1×10^{-5}	Normal with serious economic consequences	Henriques (1998)
3.7	1×10^{-4}	Normal with NO serious economic consequences	Henriques (1998)
3.8	7×10^{-5}	RC2	Eurocode 0 (CEN, 2002a)
3.1	1×10^{-3}	Moderate with moderate consequences	ISO 2494 (ISO, 1998)
2.3	1×10^{-2}	Moderate with some consequences	ISO 2494 (ISO, 1998)

3.3.3 Performance function

Concerning the performance function it has a very simple formulation, and is based on the principles: performance function equals the resistances less actions. Resistances (bearing capacity of the pile – ULS) are calculated based on empirical methods. Therefore, performance function is written as shown in eq.(3.2), where each component has their uncertainties taken into account by the factor δ. The failure zone is therefore defined by the condition M < 0 (same as $g(X)$ < 0).

$$M = \left(R_{tip} + R_{side}\right) - (G + Q) = \left(\delta_t \times Q_{tip} + \delta_f \times F_{side}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right)$$

or (3.2)

$$M = \left(R_{tip} + R_{side}\right) - (G + Q) = \delta_m\left(Q_{tip} + F_{side}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right)$$

Where M is the safety margin, R_{tip} the tip resistance of the pile, R_{side} the side resistance of the pile, G is the permanent action, Q is the variable action, δ are the factors to take into account the uncertainties (δ_t for model error uncertainty on tip resistance, δ_f for model error uncertainty in side resistance, δ_G for permanent actions uncertainties, δ_Q for variable actions uncertainties and δ_m for model error uncertainty on total resistance), Q_{tip} is the predicted tip resistance, F_{side} is the predicted side resistance, G_k the characteristic value of permanent actions and Q_k the characteristic value of variable actions.

3.3.4 Random variables and uncertainties

For a reliability analysis of a pile foundations, with performance function like shown in eq.(3.2), a total of three groups of uncertainty sources are identified:

- the modelling uncertainty (model error) in the evaluation of resistance by an empirical method or other method. Both side and tip components when considered in separate;
- the inherent soil variability considered through the *in situ* tests parameters, such as number of blows (N) of SPT or other used to calculate the resistance. Both side and tip components are considered (along the pile side and around the tip of the pile);
- and the physical uncertainties of actions. Both permanent and variable components.

There are cases where correlation(s) between parameters exists, even if they were independently obtained from different soil tests. These statistical characteristics of the correlation constants should be obtained based on sufficiently large databases. In some cases the correlation between parameters was found to be insignificant (Lumb, 1970), the assumption of independence in strength parameters simplifies the interpretation of the results and many cases leads to conservative actions (Cherubini, 1998). For these reasons, all uncertainties were considered as independent, including tip and side resistances, because no dependency or relationship is associated with the empirical method used. The consideration of any correlation between variables is taken into account in the RV simulation/generation step, being the performance function exactly the same.

As referred earlier in section 3.2, the model and actions uncertainties were mainly collected from the literature (Table 3.3). Meanwhile, the soil variability was considered in the variance of each test parameter (SPT, CPT and PMT). All calculations for soil variability are based in a routine written in R language (R Development Core Team, 2009) and following the next steps.

- first the trend is defined obtaining mean and standard deviation;
- then calculate the residuals (difference between the trend and actual values);
- analyse the residuals by plotting the histogram and Q-Q plot (graphical method to compare residual's distribution with Normal distribution);
- and plot the autocorrelation graph of the test to determine autocorrelation distance.

An example with each step is presented in Annex K. The autocorrelation distance for each test is determined. The soil variability/uncertainty can be reduced based on the autocorrelation as explained previously in section 2.2.2 (Chapter 2). The statistical estimation error was not considered. However, the values in Table 3.2 are also considered for the case study 1, in Chapter 4, because (1) it is thought that few data points were available for some of the *in situ* tests, and (2) the values in Table 3.2, as referred, have the statistical estimation error, spatial variability and measurement errors components all included in one value. This also allows an assessment of the influence of this parameter in the final results.

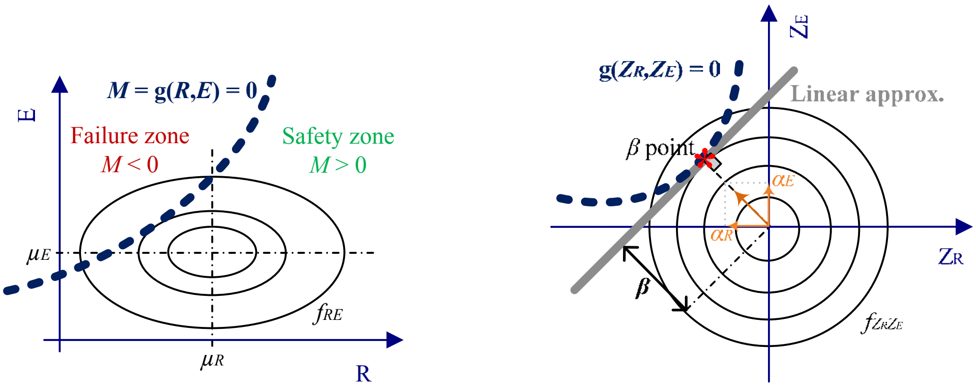
3.3.5 FORM

The FORM method is a traditional method based on successive linear approximations to a non-linear performance function. It can be used with statistically dependent or independent variables and non-normally or normally distributed. The procedure to determine the reliability index is described next, see also Figure 3.2.

i. transform all RV (named X) in standard normalised RV (named Z), this new variables will have mean equal to zero and standard deviation equal to one ($Z \sim N(0,1)$);
ii. rewrite the performance function with normalised RV ($g(Z)$);
iii. select the design point, named Z^*; the design point is the one closest to the origin in the normalised space; the distance between this point and the origin gives the reliability index β, therefore the design point is the one with the lowest β (Figure 3.2.);
iv. furthermore, sensitivity factors (α) can be calculated to know the influence of each variable in the reliability results

Sensitivity factors help to evaluate the influence of each RV; therefore, the necessity or importance of each of the basic RV of the problem is characterised by its sensitivity factor. Usually, and in all the following calculations and results of this dissertation, a positive α indicates that an increase in the corresponding RV means an increase in safety, while a negative α indicates the opposite. However, this sometimes depends on the methodology adopted to carry out FORM.

Evaluation of the sensitivity factors, as well as the sensitivity analyses, makes it possible to reduce the number of RV taken into account without compromising the accuracy of the reliability calculation. Because, even though there may be a great number of possible RV, only the variability of the most important and influential ones warrant consideration (Baecher & Christian, 2003). These calculations were performed using software that executes an iterative procedure based on the FORM process (Henriques *et al.*, 1999). This iterative procedure is necessary for problems with non-normal variables and/or non-linear functions, in order to make the approximations needed.



a) Graphical representation of the problem b) normalised space and linear approximation

Figure 3.2 – FORM method: transformation of the variables, definition of the design point (β point), reliability index and sensitivity factors (α)

3.3.6 MCS

The methodologies based on Monte Carlo technique are the most simple and easy ones, not requiring a lot of mathematical and statistical knowledge. This is important especially when the intention is to “sell” these methodologies to the practical engineers. The MCS are widely known and used as a reference method for approximation methods.

The methodology used in this study has been adapted from previously published work (Honjo *et al.,* 2010a,b,c). The goal is to remove the uncomfortable feelings that geotechnical engineers may have when using traditional reliability-based design tools, like confusion and loss of perception of the results (see Figure 3.3).

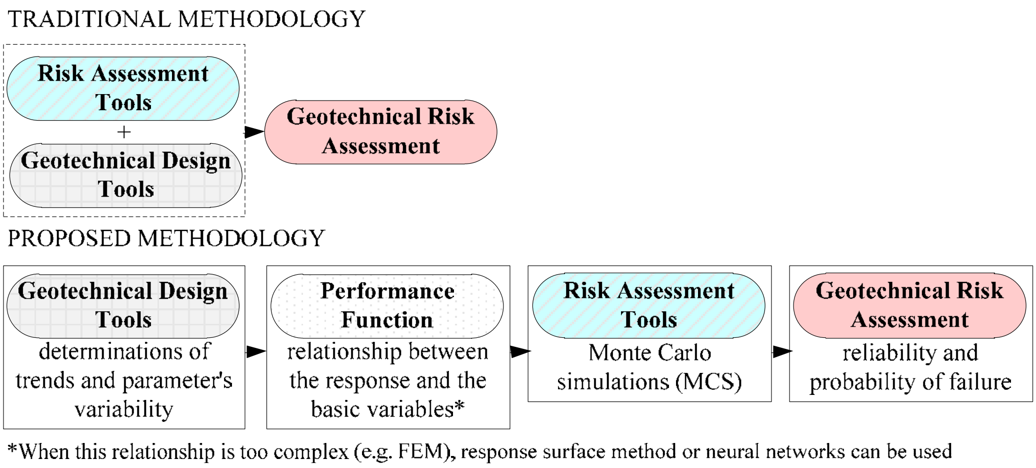


Figure 3.3 – Diagram of the traditional and proposed methodology used in the reliability analyses, adapted from Honjo *et al.* (2010a,b,c)

The “Geotechnical Design Tools” and the “Risk Assessment Tools” are separated as much as possible, allowing a better understanding of the different steps and responses obtained. Also the assessment of the reliability is done by the simplest method – MCS.

Simulation methods belong to the RA level III. They can be applied to RV with non-normal distribution and more complex performance functions. Its application uses all the statistical information of the RV, such as mean, standard deviation or COV and also PDF.

Even though many methodologies are proposed to decrease the volume of calculations of the MCS methodology (recall Annex E), its application here is not justified. So, following the generic steps described in the beginning of this section, are:

i. based on the desired reliability the number of simulations is selected – n;
ii. then generate n values for each RV considered, based on the variability information collected previously (mean, SD or COV and PDF);
iii. calculate, for each generation, the value of the performance function;
iv. and finally estimate the probability using eq.(3.3).

$$pf = \frac{1}{n} \cdot \sum_{1}^{n} I \qquad ; \qquad I = \begin{cases} 0 & if \;\; g(X) \leq 0 \; failure \\ 1 & if \;\; g(X) > 0 \;\; safety \end{cases} \tag{3.3}$$

Where pf is the probability of failure, n is the number of simulations, I the failure indicator and $g(X)$ is the performance function where X represents the RV.

The number of simulations must be chosen carefully, its stability should always be studied by repeating the set of n simulations and analyse the fluctuation of the final result. For the case studies presented in this dissertation, all MCS calculations were based in a routine written in R language. R is a free language and environment for statistical computing and graphics (R Development Core Team, 2009).

3.3.7 Minimum length (RBD) vs. allowable load (Safety evaluation)

Design methodology evolved from global SF (WSD/ASD), through partial SF (load and resistance factor design) and being nowadays replaced by reliability-based design methods in most important construction works. With the advances in science and increased knowledge it was possible to perfect these design methods and reach a point where more rational and comprehensible ways are employed in design.

The reliability-based methods can be used in different ways in a project/design of pile foundations. For example one way is the indirect use of reliability methodologies, for calibration of the SF for pile design. Other way is the use of reliability methods directly in the design of the pile, determining the minimum dimensions for a specific case, assuming the load and soil conditions. But also it is possible to evaluate, based on reliability analyses, the maximum bearing capacity for a defined case, knowing pile dimensions and soil conditions.

The first approach can be called RBD (reliability-based design) and it assumes a fixed load value and it allows the analysis of different lengths or diameters of the pile, and its influence in the probability of failure. Meanwhile, the latter approach can be called Safety evaluation, which assumes a fixed length and calculates the probabilities for different load values applied to the pile (Teixeira *et al.*, 2012b).

In both cases (RBD and Safety evaluation) the selection of the final value for design analysis (minimum dimension or maximum load) is based on a target reliability previously selected.

The reliability-based analyses quantify and give information about the parameters that mostly influence the behaviour under study, allowing the assessment of probability of failure and determination of the possible responsible causes for adverse effects on the structure (risk control). Therefore reliability-based methodologies in pile design are important and can lead to adequate efforts when gathering the information necessary to characterise the random variables that are important and influential in the evaluation of pile resistance and reliability.

3.3.8 Reliability-based safety factors

Geotechnical codes have been reviewed in the last decade, especially in Europe and North America, but also Japan (recall Chapter 2). The intention is the globalisation and harmonisation of design codes (Frank, 2002; Honjo & Kusakabe 2002a,b; Horikoshi & Honjo, 2006). But, as referred already, the geotechnical reality differs from structural reality. The new reliability-based methodologies are much more spread in structural engineering, which already apply partial SF based on RA for many designs. In the meantime, for geotechnical engineering the partial SF introduced in designs are largely determined by empirical ways and based on experience.

Therefore, while structural engineering field has seen the design methodologies updated based on reliability and probability theories, in geotechnical engineering its application has been delayed because of all the inherent variability of the soils and uncertain models. Nevertheless, because of regulation codes and social concerns, geotechnical and foundation engineers need to increase their ability to deal and incorporate these methodologies in design.

The work of Kieu Le (Kieu Le, 2008; Kieu Le & Honjo, 2007,2009) attempts to combine design value method (DVM) and MCS to assess load and resistance SF, which is believed to include the advantages of both methods, *i.e.* conceptual transparency, robustness, and flexibility of the calculation. DVM is based on FORM and is one of the powerful methods to assess the partial factors for RA level I. Here, β is defined as the shortest distance from the origin to the failure surface in the normalised coordinate system (see section 3.3.5).

In geotechnical design practice, the performance function can be very complex, containing several basic variables, some of which may appear in both components – resistance (R) and action/load (E) – of the performance function. Therefore, application of the DVM using FORM for estimation of load and resistance SF becomes very time-consuming or even impossible.

The need to use other techniques to calculate load and resistance SF, based on the idea of DVM has been taken into consideration and its combination with MCS was the solution (Kieu Le, 2008). Furthermore, the advantage of the DVM in determining partial SF is that it does not require redesign of the structure even if β at current design and the target (β_T) are different. This is because the method (implicitly) assumes that sensitivity factors calculated in the current design may not be terribly different from the sensitivity factors of the design that satisfies the target.

So, step-by-step, the way to calculate the multiplying SF (γ) [5] for each R and E is:

Step 1. Define PDF and probabilistic parameters of the RV needed for the evaluation of resistances (R) and actions (E). Statistical proprieties may be estimated based on the data obtained from laboratory and *in situ* testing, but if the quantity of information is not sufficient, one should refer to the statistical parameters obtained from similar sites and/or literature recommendations (the same as in the previous sections).

Step 2. Carry out MCS and calculate R, E and ratio R/E.

Step 3. Approximate Normal and Lognormal distributions to R and E results;

At this point it is considered the linear function $M = R - E$ to calculate the partial SF, and R and E are the two independent variables of the problem.

Step 4. Then, select the points close to the limit state line. Select the zone that satisfies the condition $R/E \in 1\pm0.02$, and evaluate the likelihood of each point within that zone – $f_R(R)$ and $f_E(E)$, where f is the PDF.

[5] Note that here the SF are both multiplying SF.

Step 5. The approximate design point is then calculated by one of two ways:

a. Maximum likelihood – $\max[f_R(R) \times f_E(E)]$ or $\max[\ln(f_R(R)) + \ln(f_E(E))]$,

b. or by normalising the space of the variables (Annex A - eq.(A.10)), then calculate their distance to the origin. The design point is the one with the shortest distance to the origin of the graph in normalised space (Figure 3.2).

Step 6. calculate the sensitivity factors, α_R and α_E:

a. using DVM formulas:

i. Normal fit – eq.(3.4),

ii. Lognormal fit – eq.(3.5).

b. or using Hasofer Lind normalised space:

i. Normal fit – eq.(3.6),

ii. Lognormal fit – eq.(3.7).

Step 7. And finally the multiplying SF, γ_R and γ_E:

i. Normal fit – eq.(3.8),

ii. Lognormal fit – eq.(3.9).

$$\begin{cases} \alpha_{DVM,N,R} = \dfrac{\sigma_R}{\sqrt{{\sigma_R}^2 + {\sigma_E}^2}} \\ \alpha_{DVM,N,E} = -\dfrac{\sigma_E}{\sqrt{{\sigma_R}^2 + {\sigma_E}^2}} \end{cases} \tag{3.4}$$

$$\begin{cases} \alpha_{DVM,Ln,R} = \dfrac{\mathrm{COV}_R}{\sqrt{{\mathrm{COV}_R}^2 + {\mathrm{COV}_E}^2}} \\ \alpha_{DVM,Ln,E} = -\dfrac{\mathrm{COV}_E}{\sqrt{{\mathrm{COV}_R}^2 + {\mathrm{COV}_E}^2}} \end{cases} \tag{3.5}$$

$$\begin{cases} \alpha_{HL,N,R} = \cos(angle) = \dfrac{Z_R}{\sqrt{{Z_R}^2 + {Z_E}^2}} \\ \alpha_{HL,N,E} = -\sin(angle) = -\dfrac{Z_E}{\sqrt{{Z_R}^2 + {Z_E}^2}} \end{cases} \tag{3.6}$$

$$\begin{cases} \alpha_{HL,Ln,R} = \dfrac{Z_{\ln(R)}}{\sqrt{{Z_{\ln(R)}}^2 + {Z_{\ln(E)}}^2}} \\ \alpha_{HL,Ln,E} = -\dfrac{Z_{\ln(E)}}{\sqrt{{Z_{\ln(R)}}^2 + {Z_{\ln(E)}}^2}} \end{cases} \tag{3.7}$$

$$\begin{cases} \gamma_{N,R} = \dfrac{\mu_R}{R_k} \times (1.0 - \beta_T \cdot \alpha_R \cdot \mathrm{COV}_R) \\ \gamma_{N,E} = \dfrac{\mu_E}{E_k} \times (1.0 - \beta_T \cdot \alpha_E \cdot \mathrm{COV}_E) \end{cases} \tag{3.8}$$

$$\begin{cases} \gamma_{Ln,R} = \dfrac{1}{\sqrt{1 + {\mathrm{COV}_R}^2}} \times \dfrac{\mu_R}{R_k} \times e^{(-\beta_T \cdot \alpha_R \cdot \mathrm{COV}_R)} \\ \gamma_{Ln,E} = \dfrac{1}{\sqrt{1 + {\mathrm{COV}_E}^2}} \times \dfrac{\mu_E}{E_k} \times e^{(-\beta_T \cdot \alpha_E \cdot \mathrm{COV}_E)} \end{cases} \tag{3.9}$$

Where μ_R, μ_E are the mean values of all R and E calculated after generation, R_k, E_k are the characteristic values, β_T is the target reliability index and COV_E, COV_S are the coefficients of variation of R and E (σ_R/μ_R and σ_E/μ_E respectively). The convention signals for sensitivity factors is: a positive α is for a RV on the safety side (resistances) and a positive α is for a RV against safety side (actions).

The definition of the characteristic values has been the most controversial topic in the whole process of drafting Eurocode 7. Despite all the proposals / studies / recommendations, the choice of the characteristic values is still an issue that leads to a lot of discussions. Nevertheless, the most conventional way is to choose the R and E characteristic values as the higher and lower fractile respectively (Annex F). The effectiveness of this method was discussed in Kieu Le (2008). For each fractile value of R (1 to 50%) and E (50 to 99%), load and resistance factors were calculated while changing the coefficient of variation (from 0.05 to 0.30). It was then concluded that, for the case of two statistical independent normal and lognormal-distributed RV (R and E) and a linear performance function ($M = R - E$), it is recommended to take higher fractile value for E. However, using the low fractile value for R may not be as effective, instead, the mean value should be chosen for R characteristic value. Based on this, the characteristic values (R_k and E_k) used in the following chapters for determination of the SF based on reliability analyses, were assumed as:

- the mean value for both R and $E \rightarrow R_k = \overline{\mathrm{R}}$ and $E_k = \overline{\mathrm{E}}$;
- the mean value for R and the high fractile of 95% for $E \rightarrow R_k = \overline{\mathrm{R}}$ and $E_k = E_{95\%}$.

3.3.9 Sensitivity analysis of uncertainty types

A sensitivity analysis can be performed in order to study the influence of the different uncertainties in the probability of failure. They can provide useful information for further calculations and improvement of the analyses. The results of sensitivity analyses also allow the engineers to better understand the behaviours, the results, and to have a more directional study of the variables to consider in a posterior full reliability analyses.

Sensitivity analyses of the different uncertainties are considered in this dissertation, especially the soil uncertainty and the modelling uncertainty (model error) components.

The calculation procedure is similar to a parametric study, where the calculation is repeated considering and not considering the different uncertainties. The impact on the performance of the structure and its reliability can be assessed by analysing different lengths and different combinations/considerations of the uncertainties (considering and not considering some). The Table 3.6 presents an example of the combinations of uncertainties that can be studied.

The sensitivity analysis will be carried out using MCS. However, FORM gives evaluations of the sensitivity factors, as a measure of the importance of the RV. Thus the uncertainties' influence can be studied one by one with each method and then both results can be compared.

Table 3.6 – Combinations of uncertainties studied for sensitivity analysis

	Model error		Soil variability		Actions' uncertainties	
Combination	Tip	Side	Tip	Side	Permanent	Variable
1	✓	✓	✓	✓	✓	✓
2	×	×	✓	✓	✓	✓
3	✓	✓	×	×	✓	✓
4	✓	✓	✓	✓	×	×

✓ means that uncertainty was considered
× means that uncertainty was NOT considered

3.4 APPLICATION EXAMPLE

An example that illustrates all the methodologies previously described is presented in this section. This example is a simple multivariable problem. A theoretical pile under vertical load, with 1 meter of diameter (B) and 10 meters length (D), considered as deterministic values.

Briefly recapping the section 3.3, the first step is to select the target reliability, then define the performance function, its RV, uncertainties and then calculate the reliability. For this application example lets consider the target reliability of β_T=3.0, equivalent to pf=0.001. The performance function is written in eq.(3.10), and the mean and standard deviation of the uncertainties shown in Table 3.7. The random variables (X) considered are the unit resistances q_{tip}, f_{side} and the action value E.

$$M = g(X) = R - E = \left(R_{tip} + R_{side}\right) - E = \left(A \times q_{tip} + U \times f_{side}\right) - E \tag{3.10}$$

Where M is the safety margin, R_{tip} the tip resistance of the pile, R_{side} the side resistance of the pile, E is the assumed vertical load applied to the pile, q_{tip} is the assumed unit tip resistance, f_{side} is the assumed unit side resistance, A the area of the tip of the pile ($\pi \cdot B^2$) and U the area of the pile in contact with the soil for side resistance ($\pi \cdot B \cdot D$).

The eq.(3.10) is equivalent to eq.(3.11), where X_1 corresponds to the q_{tip} variable, X_2 to the f_{side} variable and X_3 to the E variable.

$$M = g(X) = (3.14 \cdot X_1 + 31.4 \cdot X_2) - X_3 \tag{3.11}$$

Table 3.7 – Characterisation of the random variables of the application example

	Unit resistances (kN/m²)		Action (kN)
	tip (X_1)	side (X_2)	(X_3)
Mean value (kN)	92.3	19.4	550
Standard deviation (kN)	18.5	5.8	55
COV (%)	20	30	10
Distribution type	Normal	Normal	Normal

As one can see, the uncertainties are not considered through a factor δ, they are considered directly in the assumed values. Nevertheless, the procedures are the same as described before.

3.4.1 FORM

FORM methodology requires the determination of the new performance function with normalised RV (Z~N(0,1)). For the application example, this new normalised performance function is shown in eq.(3.12), for the length of 10 meters and load of 550 kN.

$$\begin{aligned} g(Z) &= \left(3.14 \cdot (Z_1 \cdot 18.5 + 92.3) + 31.4 \cdot (Z_2 \cdot 5.8 + 19.4)\right) - (Z_3 \cdot 55 + 550) = \\ &= 58.09 \cdot Z_1 + 182.12 \cdot Z_2 - 55 \cdot Z_3 + 348.98 \end{aligned} \tag{3.12}$$

Where $g(Z)$ is the normalised performance function and Z_1, Z_2, Z_3 are the normalised variables correspondent to X_1, X_2, X_3.

In order to understand how FORM determines the reliability, the limit functions $g(X) = 0$ and $g(Z) = 0$ are plotted in 3D in Figure 3.4 (3 dimensions are correspondent to the 3 variables of the problem). The value of β is calculated using eq.(3.13) that translates the distance to origin of the design point in normalised space. The limit of the performance function, the design points (X^* and Z^*) and mean values (the origin of the normalised space) are also depicted in Figure 3.4. All these calculations can be easily achieved with Excel, Matlab or R softwares.

$$\beta = \min\left(\sqrt{{Z_1}^2 + {Z_2}^2 + {Z_3}^2}\right) = \sqrt{Z_1^{*2} + Z_2^{*2} + Z_3^{*2}} \tag{3.13}$$

The results of the FORM comprehend the value of β, the value of the design point and the sensitivity factors (Figure 3.5). For the application example the results are:

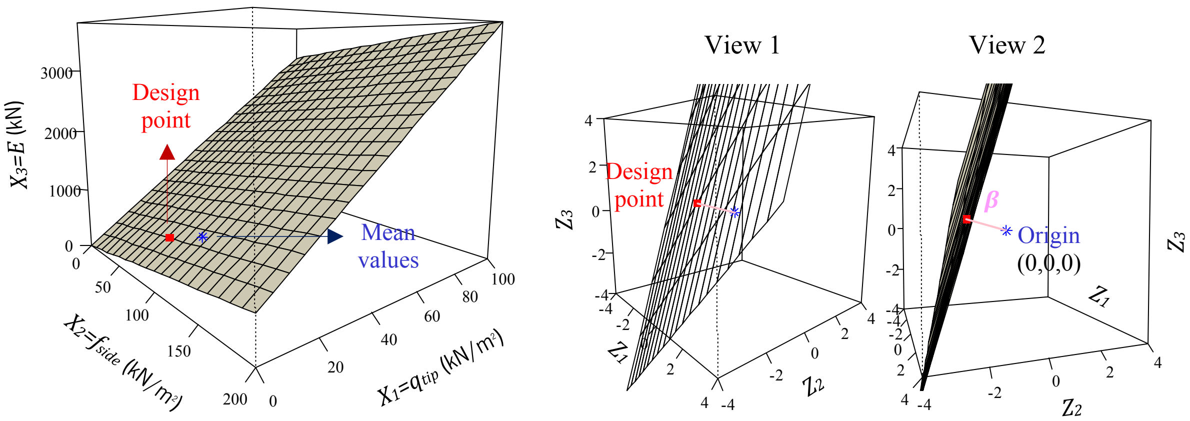
- reliability index of $\beta = 1.76$, correspondent to a probability of failure of $pf = 0.04$;
- design point values are $q_{tip}=X_1=82.8$ kN/m², $f_{side}=X_2=10.1$ kN/m² and $E=X_3=577$ kN, correspondent to the normalised space $Z_1= -0.51$, $Z_2=-1.61$ and $Z_3=0.48$;
- and sensitivity factors of $\alpha_1=-0.29$, $\alpha_2=-0.92$ and $\alpha_3=0.27$, indicating the influence of the RV.

For the case studies presented in next Chapters, a software with an iterative procedure based on the FORM methodology is used (Henriques *et al.*, 1999). The iterative procedure is necessary for problems with non-normal variables and/or non-linear functions, where approximations need to be done. But even though the iterative procedure is not needed for this simple case (application example), the software inputs and outputs are explained in more detail while applied to this case.

The input for the software is a matrix of the data containing: the number of variables, the coefficients of the performance function (independent, linear or quadratic), the characterisation of the RV' uncertainties and its correlations, and also the maximum number of interactions and admitted error between interactions. Consult the input file and output file with results in Annex L. The comparison between software and analytical calculation by FORM can be consulted in Table 3.8.

As one can compare the results are exactly the same, because the RV are normally distributed and the performance function is a linear combination of the RV. This allowed the iterative process (software) to converge immediately.

Finally, when comparing the application example result with the target it can be concluded that it does not meet the required value ($\beta = 1.76 < \beta_T = 3.0$).



a) Representation of the random variables, X

b) Representation of the normalised space, Z

Figure 3.4 – Graphical representation of FORM results for the application example

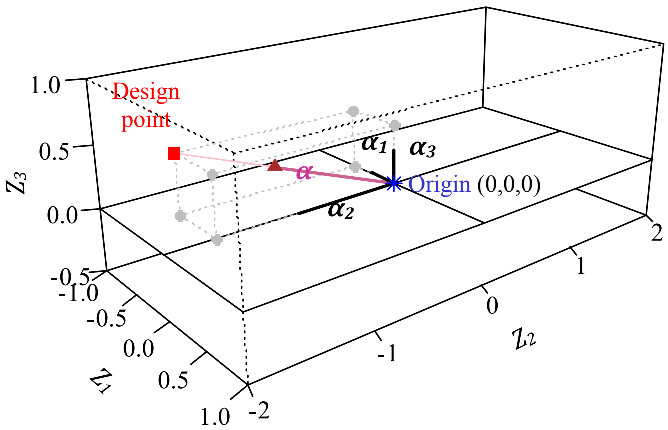


Figure 3.5 – Sensitivity factors graphical representation (FORM) for the applications example

Table 3.8 – Results of FORM analysis of the application example

	Graphical method			*Software*		
	Unit resistance (kN/m²)		Actions (kN) (X_3)	Unit resistance (kN/m²)		Actions (kN) (X_3)
	tip (X_1)	side (X_2)		tip (X_1)	side (X_2)	
Design point X^*	82.8	10.1	577	82.8	10.1	577
Normalised Z^*	-0.51	-1.61	0.48	-0.51	-1.61	0.48
Sensitivity α	0.29	0.92	-0.27	0.29	0.92	-0.27
Reliability	β = 1.76 → pf = 0.04			β = 1.76 → pf = 0.04		

3.4.2 MCS

For MCS methodology one needs only do select the number of simulations (n) and generate n values for each RV, based on its mean and standard deviation (Table 3.7). With those values it is possible to calculate the result of the performance function (eq.(3.11)), and then evaluate the probability of failure (eq.(3.3)).

Three sets of simulations, n = [50; 1,000; 100,000], were repeated to explain their influence in the results, see eq.(3.14a), (3.21b) and (3.21c) and Figure 3.6. Only simulation results of n=50 are presented in Table 3.9 as illustration. Figure 3.6 shows the difference in pf results, also indicating computational time of each number of simulations.

Computational time increases exponentially with the number of simulations (n), but since this method (MCS) is the reference, giving precise results, and since nowadays supercomputers are available, the use of variance reduction techniques is not required. Nevertheless, the number of simulations can be optimised studying the stability of results. As one can see in Figure 3.6, the results for n=50 simulations are far from stable.

Table 3.9 – Results of one generation of n=50 MCS for the application example

#	X_1	X_2	X_3	$g(X)$	I	#	X_1	X_2	X_3	$g(X)$	I
1	86.75	33.81	541.1	792.7	0	26	82.81	14.68	554.9	165.9	0
2	68.66	19.97	505.2	337.5	0	27	128.79	18.87	504.6	492.2	0
3	96.82	14.28	572.9	179.6	0	28	108.31	21.77	547.1	476.5	0
4	115.91	14.81	627.4	201.7	0	29	136.25	23.48	552.4	612.7	0
5	114.47	14.69	595.1	225.6	0	30	80.18	14.17	501.0	195.6	0
6	124.36	19.89	675.8	339.2	0	31	123.04	26.12	538.8	667.6	0
7	51.90	20.57	592.4	216.5	0	32	62.47	29.66	548.0	579.6	0
8	87.97	9.57	508.3	68.3	0	33	102.27	19.79	682.7	260.0	0
9	112.56	16.74	482.7	396.4	0	34	108.99	27.82	625.3	590.6	0
10	72.20	19.57	601.9	239.4	0	35	127.80	15.76	612.1	283.9	0
11	79.53	15.44	580.6	154.1	0	36	90.74	21.82	573.9	396.3	0
12	61.03	19.67	647.8	161.5	0	37	82.61	21.92	611.7	336.1	0
13	58.13	21.25	507.3	342.4	0	38	104.79	15.74	566.7	256.5	0
14	74.21	32.60	551.0	705.8	0	39	85.25	19.60	556.2	327.1	0
15	77.99	8.30	568.4	-63.0	1	40	106.32	19.56	460.8	487.1	0
16	53.12	19.22	446.6	323.6	0	41	65.58	14.56	631.2	32.0	0
17	81.79	30.47	596.7	617.0	0	42	76.63	24.57	497.6	514.3	0
18	84.83	21.52	563.3	378.9	0	43	64.15	15.16	559.3	118.3	0
19	94.79	21.07	497.5	461.7	0	44	85.59	24.01	592.2	430.3	0
20	85.54	14.11	624.3	87.4	0	45	91.70	19.13	583.4	305.2	0
21	86.25	15.55	571.7	187.5	0	46	92.82	18.74	543.5	336.3	0
22	85.45	20.45	559.2	351.3	0	47	86.33	19.05	648.0	221.2	0
23	117.14	27.59	486.2	748.0	0	48	132.90	23.26	572.2	575.4	0
24	90.72	22.89	527.7	475.7	0	49	60.06	29.52	500.7	614.8	0
25	88.86	29.42	610.3	592.5	0	50	78.68	21.57	517.5	406.9	0

$$pf = \frac{1}{50} \cdot \sum_{1}^{50} 1 = 0.02 \qquad ; \qquad n = 50 \tag{3.14a}$$

$$pf = \frac{1}{1\,000} \cdot \sum_{1}^{1\,000} 41 = 0.041 \qquad ; \qquad n = 1\,000 \tag{3.15b}$$

$$pf = \frac{1}{100\,000} \cdot \sum_{1}^{100\,000} 4247 = 0.0424 \qquad ; \qquad n = 100\,000 \tag{3.15c}$$

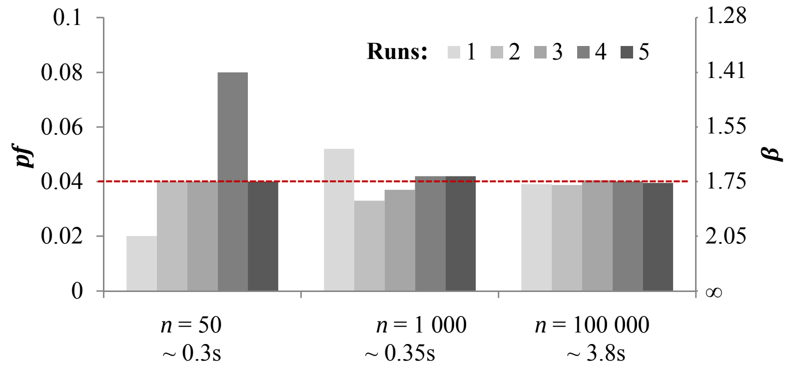


Figure 3.6 – Results of the different number of MCS for the application example

The final results are achieved with n=100,000. The generated RV and the graphical representation of each step, until reaching the probability of failure (pf) and then the reliability index (β) is depicted in Figure 3.7.

For the application example, the results using MCS were:

- reliability index of β = 1.76, correspondent to a probability of failure of pf = 0.04.

These results are completely coherent with the FORM results.

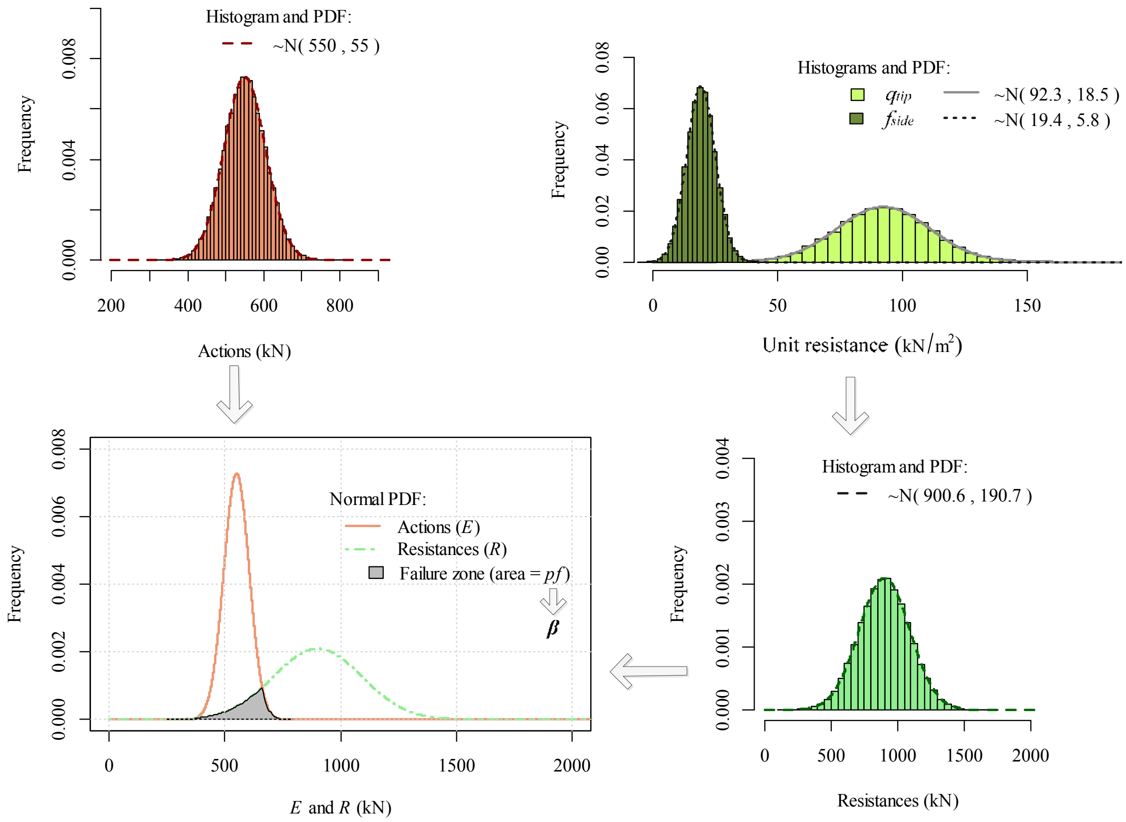


Figure 3.7 – MCS methodology for reliability analysis of the application example (B=1m, D=10m)

3.4.3 Minimum length (RBD) vs. allowable load (Safety evaluation)

The RBD (considering different lengths with fixed load value) and Safety evaluation (considering different load values and a fixed length) approaches were performed using both FORM and MCS methodologies. Therefore the calculations were repeated for each method and each following case:

- RBD: design with lengths 7, 9, 10, 15, 20 and 25 meters;
- Safety evaluation: safety evaluation of loads 200, 300, 400, 550, 600 and 700 kN.

All the results are depicted in Table 3.10 and Figure 3.8.

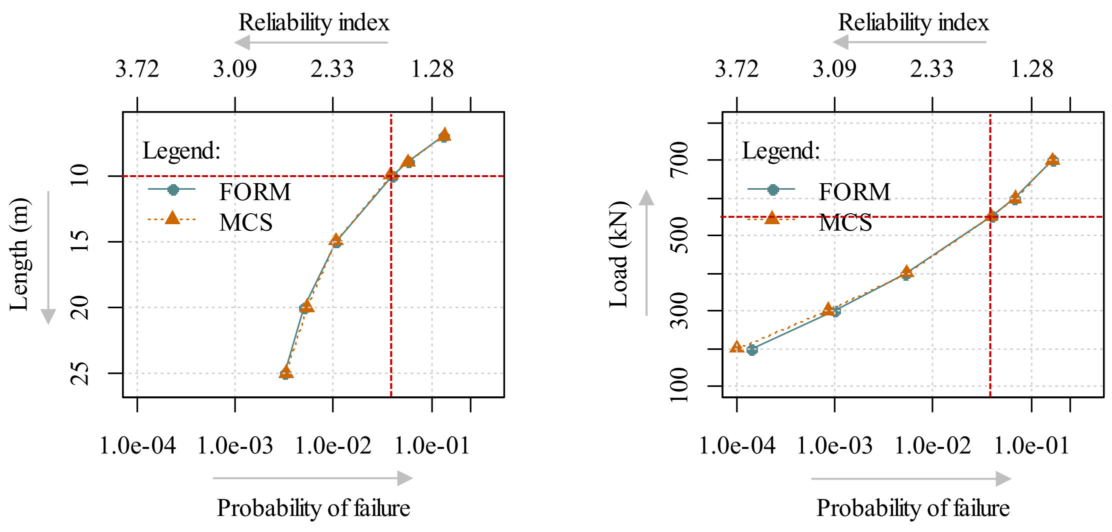
Table 3.10 – Results of reliability-based approaches using FORM and MCS for the application example

a)

Lengths (m)	RBD FORM	RBD MCS	Δ (%)
7	1.106439	1.102602	0.35
9	1.581017	1.577850	0.20
10	*1.755898*	*1.767766*	*0.68*
15	2.297027	2.300498	0.15
20	2.570064	2.552347	0.69
25	2.732165	2.712390	0.72

b)

Loads (kN)	Safety evaluation FORM	Safety evaluation MCS	Δ (%)
200	3.637226	3.808168	4.70
300	3.096342	3.059104	1.20
400	2.556042	2.537022	0.74
550	*1.755898*	*1.758750*	*0.16*
600	1.493847	1.494443	0.04
700	0.979248	0.985293	0.62



a) RBD (different lengths) b) Safety evaluation (different loads)

Figure 3.8 – Results of reliability-based approaches using FORM and MCS for the application example

Considering the RBD results and the target $\beta_T = 3.0$, it is seen that even if the length of the pile was duplicated the required reliability could not be reached. Since the soil was considered as an homogeneous stratum with same unit resistance at any depth, the tip resistance does not change with the length of the pile, therefore only side resistance increases. Side resistance of the pile (mean of 19.4 kN/m^2) is a low value, therefore β does not vary considerably with length of the pile. On the other hand, Safety evaluation presents a higher variability with load changes. It is concluded that for this pile one should redesign the pile (increase diameter) or design a second pile, since for the required target reliability is only achieved with load value around 300 kN (each pile, B=1m and D=10m). A similar calculation is done with different diameter values is presented in Figure 3.9, concluding that a diameter higher than 1.3 m would contribute for $\beta > 3.0$. This RBD analyses, using different lengths or different diameters of the pile, allows a cost-effective analyses.

Concerning the comparison between FORM and MCS results, it can be seen that they are the same, considering an error lower than 1%. Excepts when the probabilities of failure are very low ($pf < 10^{-3}$), but still errors are lower than 5%. Since the RV are normally distributed and the performance function is a linear combination of them, the FORM method gives exact result for this problem. Therefore it can be said that for this theoretical example, the n=100,000 simulations provide results with enough accuracy.

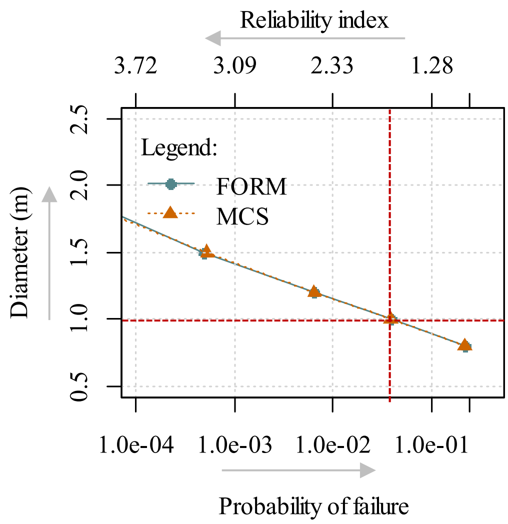


Figure 3.9 – Results of reliability-based approach (different diameters) using FORM and MCS for the application example

3.4.4 Reliability-based safety factors

Concerning the determination of reliability-base SF, the following steps briefly resume the methodology previously described:

- carry out MCS to calculate R, E and R/E;
- approximate Normal and Lognormal distributions to R and E results;
- select the points close to the limit state and evaluate its likelihood;
- determine design point;
- calculate sensitivity factors;
- and finally the load and resistance SF (formulas in section 3.3.8).

All the results are exhibited next. Figure 3.10 shows the PDF approximation to MCS results, Figure 3.11 presents the MCS in 2D graph, being each dimension the two RV considered (R and E), and Figure 3.12 represents the normalised space with the limit state point zone.

Finally Figure 3.13 and Table 3.11 present the range of SF obtained for different reliability indexes, while Table 3.12 presents the preliminary results.

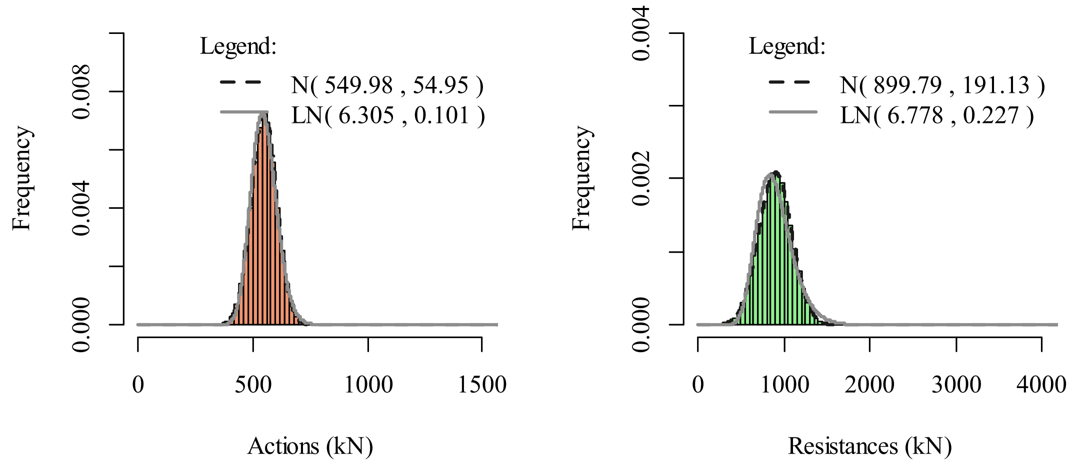


Figure 3.10 – Histograms and PDF approximations to E and R, achieved with n=100,000 MCS for the application example (B=1m, D=10m)

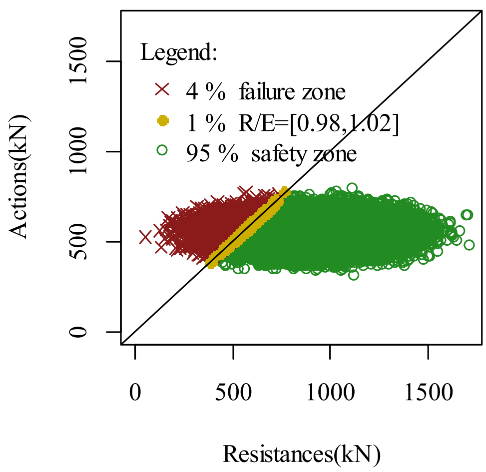


Figure 3.11 – Graphical representation of the MCS points near limit state line, failure zone and safety zone, achieved with n=100,000 for the application example

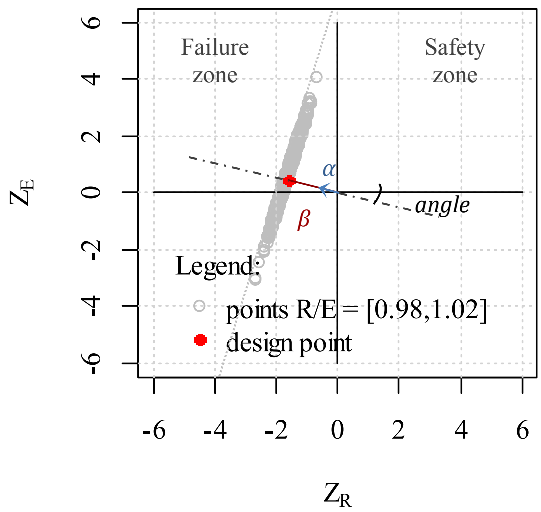


Figure 3.12 – Graphical representation of the MCS points near limit state line in normalised space, design point and reliability index for the application example

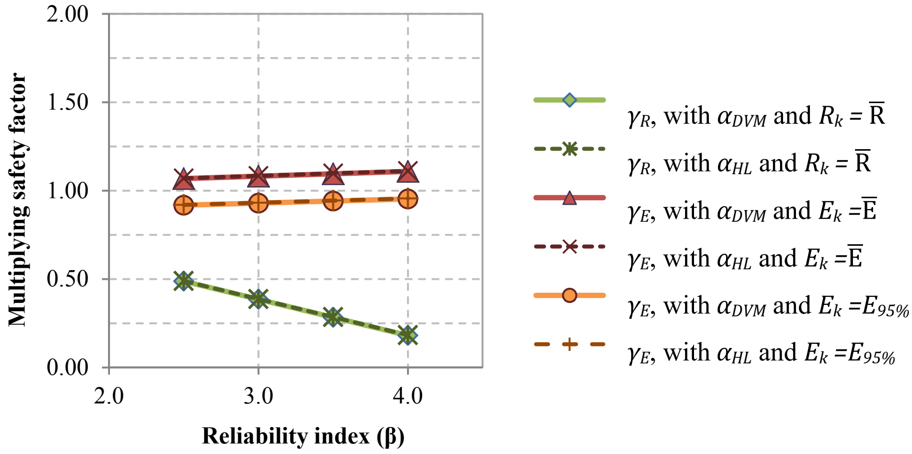


Figure 3.13 – SF based on reliability analyses for the application example, considering a Normal distribution for R and E

Table 3.11 – Sets of SF based on reliability for the application example, considering different reliability targets

	DVM				HL			
	Using $R_k = \overline{R}$, $E_k = \overline{E}$		*Using* $R_k = \overline{R}$, $E_k = E_{95\%}$		*Using* $R_k = \overline{R}$, $E_k = \overline{E}$		*Using* $R_k = \overline{R}$, $E_k = E_{95\%}$	
β_T	γ_R	γ_E	γ_R	γ_E	γ_R	γ_E	γ_R	γ_E
2.5	0.49	1.07	0.49	0.92	0.49	1.07	0.49	0.92
3.0	0.39	1.08	0.39	0.93	0.39	1.08	0.39	0.93
3.5	0.28	1.10	0.28	0.94	0.29	1.10	0.29	0.94
4.0	0.18	1.11	0.18	0.95	0.18	1.11	0.18	0.96

Table 3.12 – Safety factors' preliminary results based on reliability analyses for the application example, considering a Normal distribution for R and E

	Approximation to Normal	
	Resistances	Actions
Approx. Mean values (kN)	899.636	550.283
Real Mean values (kN)	899.438	550.000
Approx. Standard deviation (kN)	240.3318	54.6572
Real Standard deviation (kN)	191.4934	55.0000
Design point (kN):		
- using max likelihood	587.86	576.42
- using min(β)	587.86	576.42
	Z_R = -1.628	Z_E = 0.478
	β=1.70 →	pf=0.045
Sensitivity factors (α)		
- DVM	0.962	-0.274
- HL	0.959	-0.282

The values obtained for SF, based on Normal PDF fit, and with DVM or HL formulas to assess the sensitivity factors (α), are the same. The SF values are calculated for mean characteristic values (R_k = $\overline{\text{R}}$, E_k = $\overline{\text{E}}$), but also considering the common fractile of 95% for actions (R_k = $\overline{\text{R}}$, E_k = $E_{95\%}$). These values should be lower than one (γ_R < 1) for resistances, and higher than one (γ_E > 1) for actions. The resistance multiplying SF resulted between 0.18 and 0.49, while for actions the 1.07 and 1.11. They always depend on the target reliability, on the variability of the RV and, of course, on the characteristic values assumed.

Furthermore, it is possible to understand that the variability of the SF varies directly with the importance of the RV in study. Resistance has a higher variability therefore its SF will also have a higher variability. The results allow the assessment of SF, but in order to use it in design codes/standards, they have to be calibrated with adequate number of cases, contemplating different types of soils and different types of piles.

3.4.5 Sensitivity analysis of uncertainty types

The sensitivity analysis methodology explained in the previous section 3.3.7, is applied to the theoretical example using MCS, in order to study the influence of each component uncertainty (R_{tip}, R_{side} and E). The calculations are performed for the combinations shown in Table 3.13 and also considering different lengths of the pile.

Table 3.13 – Combinations of uncertainties studied for sensitivity analysis

Combination		Resistances		Actions
		tip (X_1)	side (X_2)	(X_3)
1	All uncertainties considered	✓	✓	✓
2	Actions' uncertainties removed	✓	✓	×
3	Resistance's uncertainties removed	×	×	✓
4	R_{tip} uncertainties removed	×	✓	✓
5	R_{side} uncertainties removed	✓	×	✓

✓ means that uncertainty was considered
× means that uncertainty was NOT considered

The results of the sensitivity analysis are depicted in Table 3.14 and Figure 3.14. From this type of sensitivity analyses is possible to understand and assess the influence of each uncertainty type taken into account in the reliability of the pile. For this case, it is possible to understand that there is a very low variability in the results when comparing combinations 1, 2 and 4. On the other hand it is noticeable that when removing the variability of resistance (combination 3) the reliability of the pile changes completely. Furthermore, it is possible to recognise that the uncertainty that mostly influence the results is the side resistance uncertainty, which for this case is precisely the uncertainty with higher COV (Table 3.7). The same results, or consistent results, are observed for both sensitivity analyses with RBD (Figure 3.14 a) and Safety evaluation (Figure 3.14 b) approaches.

Recalling the results of the sensitivity factors of FORM (Table 3.8), it is safe to say that FORM and sensitivity analyses based on MCS present consistent and coherent results of the influence of the RV considered.

Table 3.14 – Comparison of the relative influence of the uncertainties (sensitivity analysis results) using MCS method for the application example

Combination		RBD (different lengths)	Safety evaluation (different loads)
2	Actions uncertainties removed	2%	2%
3	Resistance uncertainties removed	98%	98%
4	R_{tip} uncertainties removed	3%	6%
5	R_{side} uncertainties removed	97%	94%

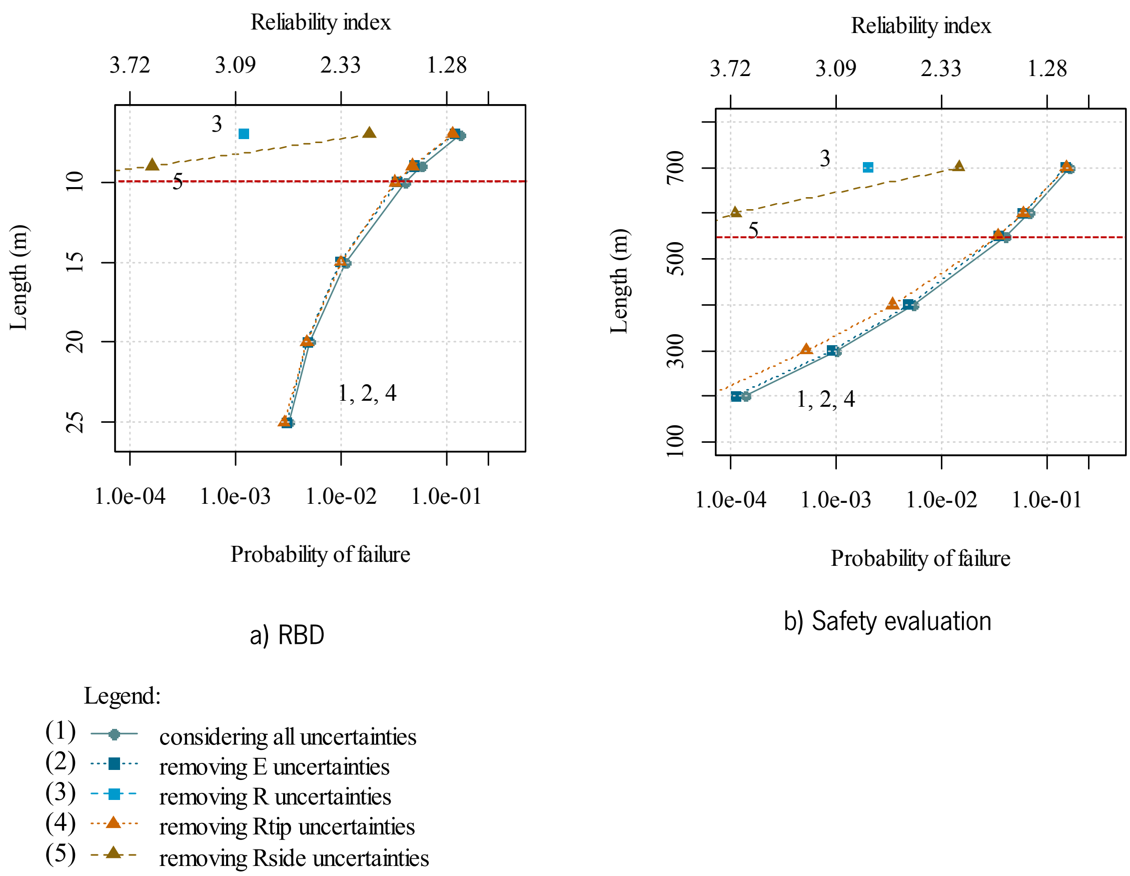


Figure 3.14 – Reliability-based sensitivity analyses results using MCS for the application example

3.5 CONCLUDING REMARKS

This chapter present reliability concepts for pile foundations design and safety aspects. These methodologies in geotechnical practice are not as current as they are in structural engineering, needing some adaptions to be applied and considered in a geotechnical environment.

Therefore, based in the reliability theories, some proposals are presented, describing the main steps for different reliability analyses, including uncertainties classification, consideration and assessment, and different approaches referred as RBD (reliability-based design), Safety evaluation, reliability-based safety factors and reliability-based sensitivity analysis. These proposals are mainly based on the works developed by professor Honjo's research group (Kieu Le, 2008; Honjo *et al.*, 2010a,b,c).

The methodologies and approaches presented are believed to be very easy and can help support the design of pile foundations; also, they try to eliminate the possible confusions and difficulties that traditional reliability methodologies used in structures can cause to geotechnical designers in practice.

For better understanding of these methodologies, a simple theoretical application example of a pile is presented; on which techniques like FORM (first-order reliability method) and MCS (Monte Carlo simulations) are used to assess the probabilistic response.

RA level II and RA level III are the ones commonly used for evaluation of the probability of failure, and within literature the most referred methods are FORM and MCS (RA level II and RA level III respectively). The MCS are widely used because of its higher level of accuracy and for being the most straight forward method for reliability analyses while FORM method is very traditional and it is used since the first studies of structural reliability. Nevertheless FORM is an approximated method and sometimes does not have the capacity to incorporate every detail, namely a specific probability density function or a complex limit conditions.

The uncertainties considered are (1) modelling uncertainty (or model error), (2) the inherent and spatial soil variability and (3) the physical uncertainties of actions. The spatial variability of the soil allows the reduction of soil parameter's standard deviation based on the autocorrelation of the *in situ* tests parameters. It is obvious that this will lead to a more reliable result; therefore, not considering this reduction results in more conservative values although technically incorrect.

In particular for the application example of a theoretical pile foundation problem, it shows how to apply, in a simple way, the methodologies proposed. This application example has three random normally distributed variables, two on the safe side (resistance) and one against the safe side (actions). The calculations led to results of reliability index of 1.8 for both FORM and MCS methods. Also, RBD and Safety evaluation approaches performed using FORM and MCS, presented the same results, having just little differences when probabilities of failure are very low (due to number of MCS chosen). It is also shown a alternative of the RBD method. If instead of analysing different pile lengths (usual procedure), if one also analyses different diameters of the pile, it is possible to better understand the influences and proceed with a more cost-effective design.

Concerning the sensitivity analysis the random variable with higher COV (coefficient of variation) present itself as being the most influence in the results of probability of failure. The MCS sensitivity analyses results agree with the FORM sensitivity factors, being the side resistance the most influent RV, and being the tip resistance uncertainty and actions uncertainty of the application example very similar in its influence in the reliability result.

Finally, it is consensual that these reliability-based analyses allow a more rational way to deal with uncertainties of a problem, instead of just introducing a "blind" factor of safety. Also, it is in agreement with new regulation codes. Moreover, sensitivity analysis of uncertainties would help save time and optimise resources on investigations of variables in pile reliability, since uncertainties characterisation is not an easy task in geotechnical engineering.

Chapter 4

APPLICATION OF RELIABILITY TO AN EXPERIMENTAL SITE

4.1 INTRODUCTION

In any area of civil engineering the calibration of design methods and design codes plays an important role in practice. But differently from the structural or mechanical areas that deal with materials of their own making, geotechnical engineering has to deal with materials given by nature, and sometimes mixtures of it. Therefore, for the geotechnical practice, complete databases covering different types of soil, structures and different construction processes are a very important part for knowledge and code calibration. Databases are important not only for calibrating a design method or a set of safety factor (SF) but they are also important for validation of methodologies and for the definition of appropriate/suitable uncertainties for reliability analyses.

Concerning pile foundation engineering, a complete database for analysis should include all the information of each pile foundation site, such as (1) soil characterisation, *in situ* and laboratory tests and geotechnical reports, (2) pile information, such as geometry, dimensions (length, diameter, etc.), materials used, their quantities, construction method (driven, bored, with or without cast, cast lost or recovered), and also including information about the driving and integrity tests, and lastly and very important, (3) the results of load testing, such as dynamic, static, or others.

Only with access to rich databases one can make reliable assessments, study variability, evaluate uncertainties and calibrate methods to improve reliability, safety and economy in designs. The following Table 4.1 shows references of some pile databases that can be used for those purposes.

Table 4.1 – Databases of pile foundations

Name	Development	Reference	Access
DFLTD[1]	TMA[2] Webgeotech	http://www.webgeotech.com/	FREE
NGES[3] Database	NSF[4] and FHWA[5]	http://www.unh.edu/nges/	FREE
Japanese Database	PWRI[6]	Okahara *et al.* (1991)	RESTRICT (accessed)
Experimental site of FEUP[7]	FEUP[7]	Viana da Fonseca & Santos (2008)	RESTRICT (accessed)
Caltrans[8] Database	Caltrans[8]	Monzon (2006)	RESTRICT (partial access)
Portuguese databases	-	Viana da Fonseca (2007), Santos (2007)	RESTRICT (partial access)
Brazilian databases	-	Cavalcante*et al.* (2007)	RESTRICT (no access)
Geotech Database	UOF[9] and FDOT[10]	http://fdot.ce.ufl.edu/	RESTRICT (no access)
PD/LT Database	UML[11]	Paikowsky *et al.* (1994)	RESTRICT (no access)
PILEACT	RTAPR[12] and UPR[13]	Granell (2005)	RESTRICT (no access)
GRLWEAP[14] Database	GRL Engineers, inc.	Thendean *et al.* (1996)	RESTRICT (no access)

The case study presented in this chapter, denoted “Case study 1”, was selected from an extensive research concerning pile databases in 2009/2010 (Table 4.1). The databases found were developed for various reasons, such as code calibration, LRFD calibration, definition of resistance

[1] Deep Foundations Load Test Database
[2] Technology and Management Applications
[3] National Geotechnical Experimentation Sites
[4] National Science Foundation
[5] Federal Highway Administration
[6] Public Works Research Institute
[7] Faculty of Engineering of University of Porto
[8] California Department of Transportation
[9] University of Florida
[10] Florida Department of Transportation
[11] University of Massachusetts Lowell
[12] Roads and Transport Authority of Puerto Rico
[13] University of Puerto Rico
[14] GRL Wave Equations Analysis Program

models, determination of model errors or simply from surveys. For example, DFLTDatabase has information from all over the world, including thousands of piles and types of soils (consult Annex M). In the case of the Japanese database, it contains many data from different parts of Japan and different types of piles (more than 4 hundred), while Caltrans database have information about 2 hundred deep foundations from California, containing static or dynamic load tests.

Case study 1, from the experimental site of FEUP, is pertained to a bored pile. This case was selected as the more suitable to carry out the reliability studies using different *in situ* tests and different design methods. Bored piles are the most common practice in Portuguese construction projects, and this experimental site has a very extensive and diversified soil characterisation.

The previously described reliability-based methodologies (Chapter 3) are applied to this case study and presented next. A comparison between different methods is done and discussed using mainly MCS-based and SPT-based results. Reliability analyses using FORM and using CPT-based and PMT-based resistance models are also included. Finally, a sensitivity analysis to the different uncertainties considered is performed.

4.2 CASE STUDY 1 – FEUP

The case study 1 is referred to the experimental site in Porto, north of Portugal (Figure 4.1). FEUP performed this experimental site with 14 piles for a prediction event in 2004 (International Site Characterization – ISC'2 conference[15]). Further information can be found in Esteves (2005), Fellenius *et al.* (2007) and Viana da Fonseca & Santos (2008). The layout of this site is depicted in Figure 4.2.

4.2.1 Soil characterisation

Residual soil from granite, a very common type of soil in the northwestern part of Portugal, is found at this site. The site is geologically formed by an upper layer of heterogeneous residual (saprolitic[16]) granite soil of varying thickness, overlaying a relatively weathered granite in contact with high-grade metamorphic rocks (Viana da Fonseca & Santos, 2008). Bedrock is found at a depth of approximately 20 m, and the Ground Water Line (GWL) is found at a depth of approximately 10 m - Figure 4.3.

[15] http://paginas.fe.up.pt/isc-2/main.html

[16] Chemically weathered rock

Figure 4.1 – Geographic location of experimental site of case study 1 (FEUP), adapted from Viana da Fonseca & Santos (2008)

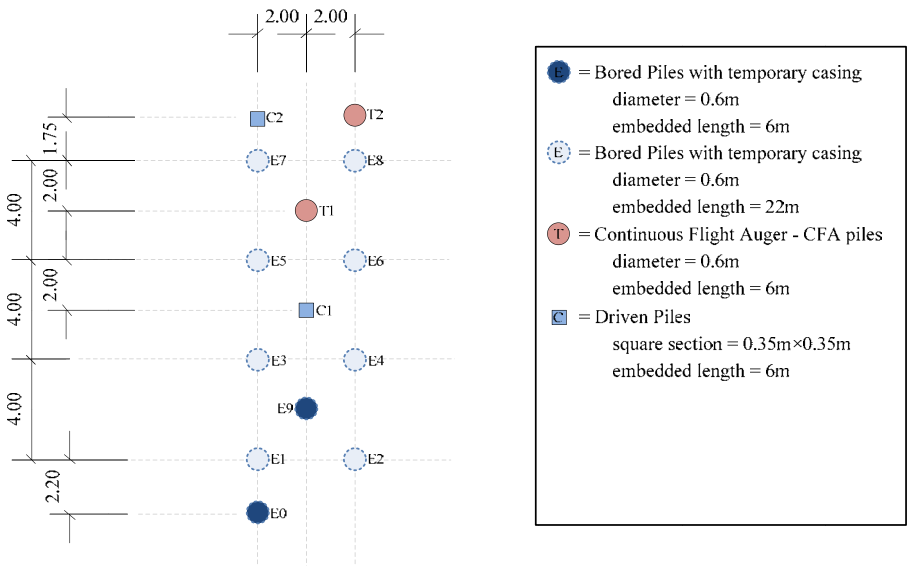


Figure 4.2 – Plan view of experimental site of case study 1 (FEUP), adapted from Viana da Fonseca & Santos (2008)

An extensive *in situ* and laboratorial investigation and characterisation of the experimental site was conducted, see Figure 4.3. Tests like SPT (Standard Penetration Test), CPT (Cone Penetration Test), DMT (flat plate Dilatometer Test), PMT (Menard Pressuremeter Test), surface and borehole seismic, and laboratory tests (triaxial extension, triaxial compression, oedometer and resonant column tests), were performed. As referred in Chapter 3, the SPT, CPT and PMT are the tests used for the following analyses (mainly SPT) – refer to section 3.2. Thus, the following figures present the detailed results of SPT (Figure 4.4), CPT (Figure 4.5) and PMT (Figure 4.6).

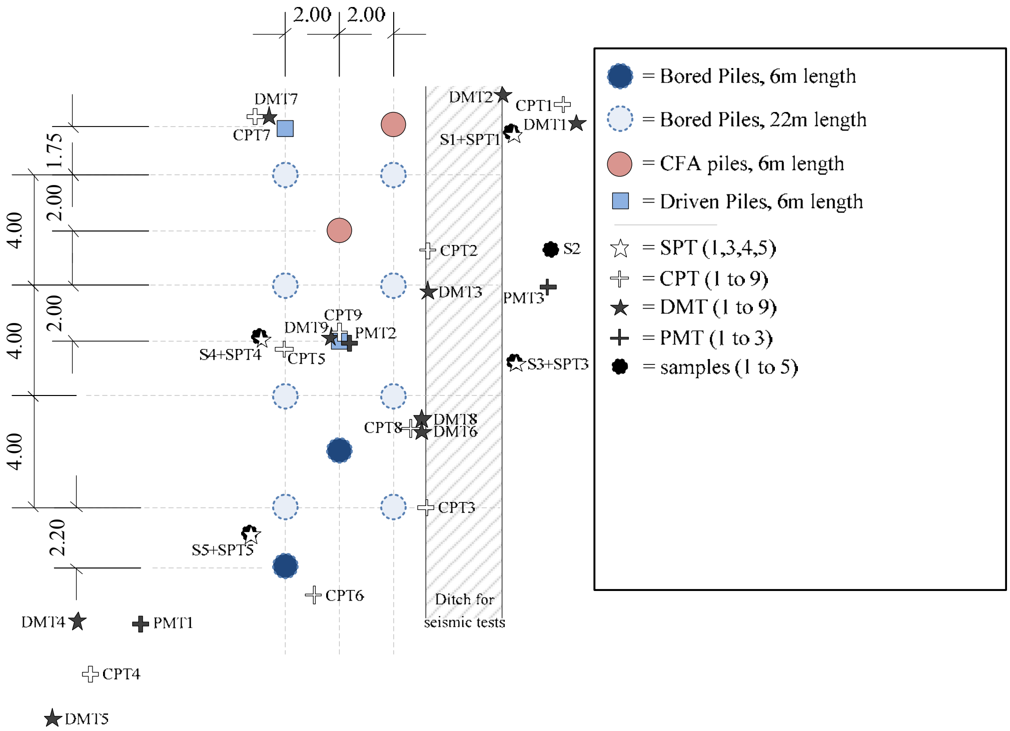


Figure 4.3 – Layout of the experimental site's investigation tests of case study 1 (FEUP), adapted from Viana da Fonseca & Santos (2008)

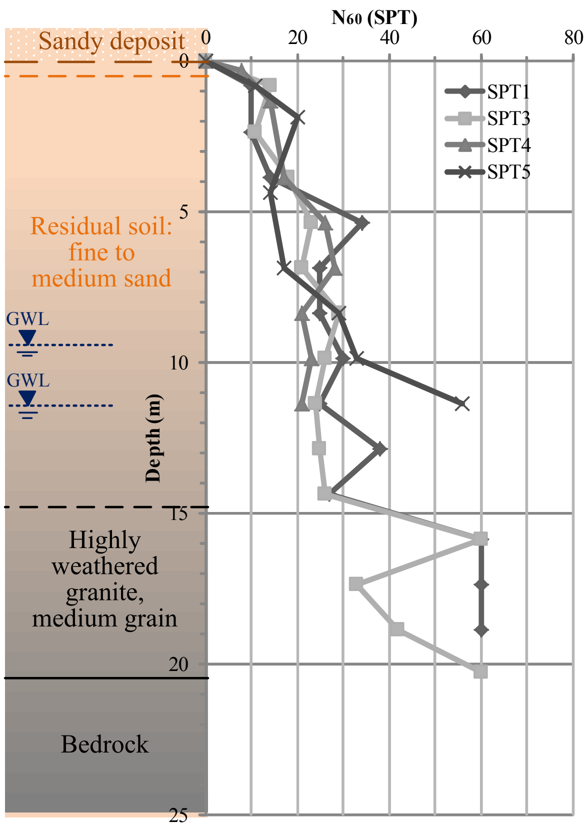
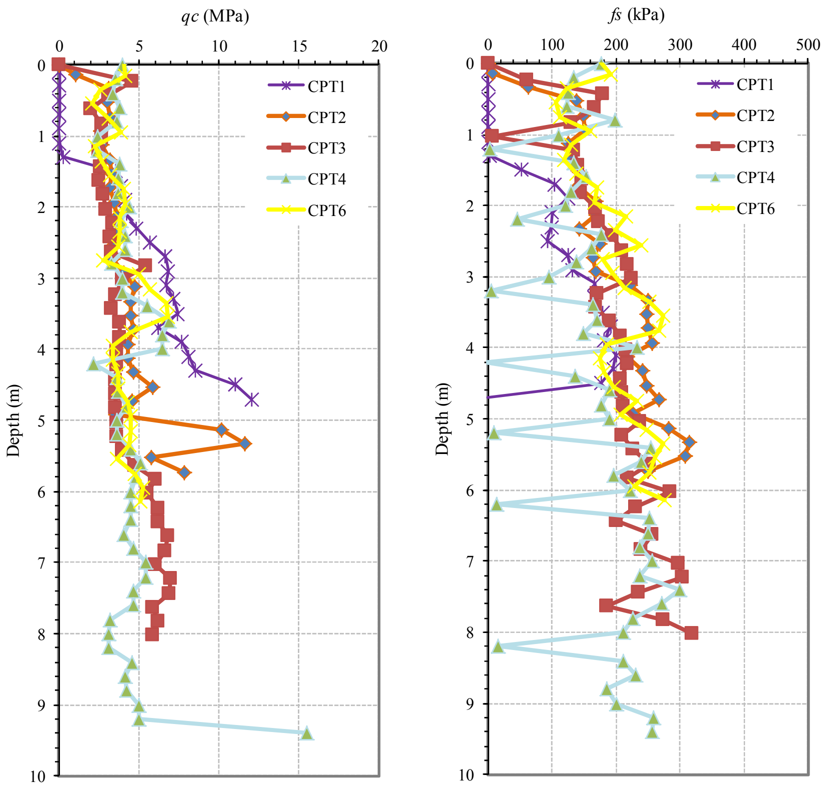


Figure 4.4 – Geological profile and SPT results from experimental site of case study 1 (FEUP), adapted from Esteves (2005)

CPT tests performed before the installation of the piles



CPT tests performed after the installation of the piles

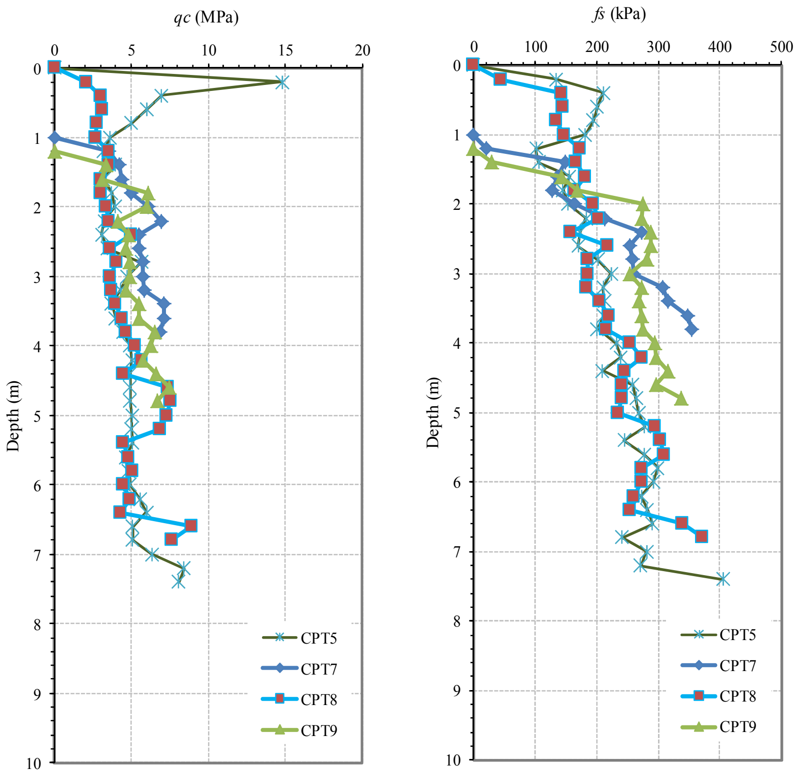


Figure 4.5 – CPT results from experimental site of case study 1 (FEUP), adapted from Esteves (2005)

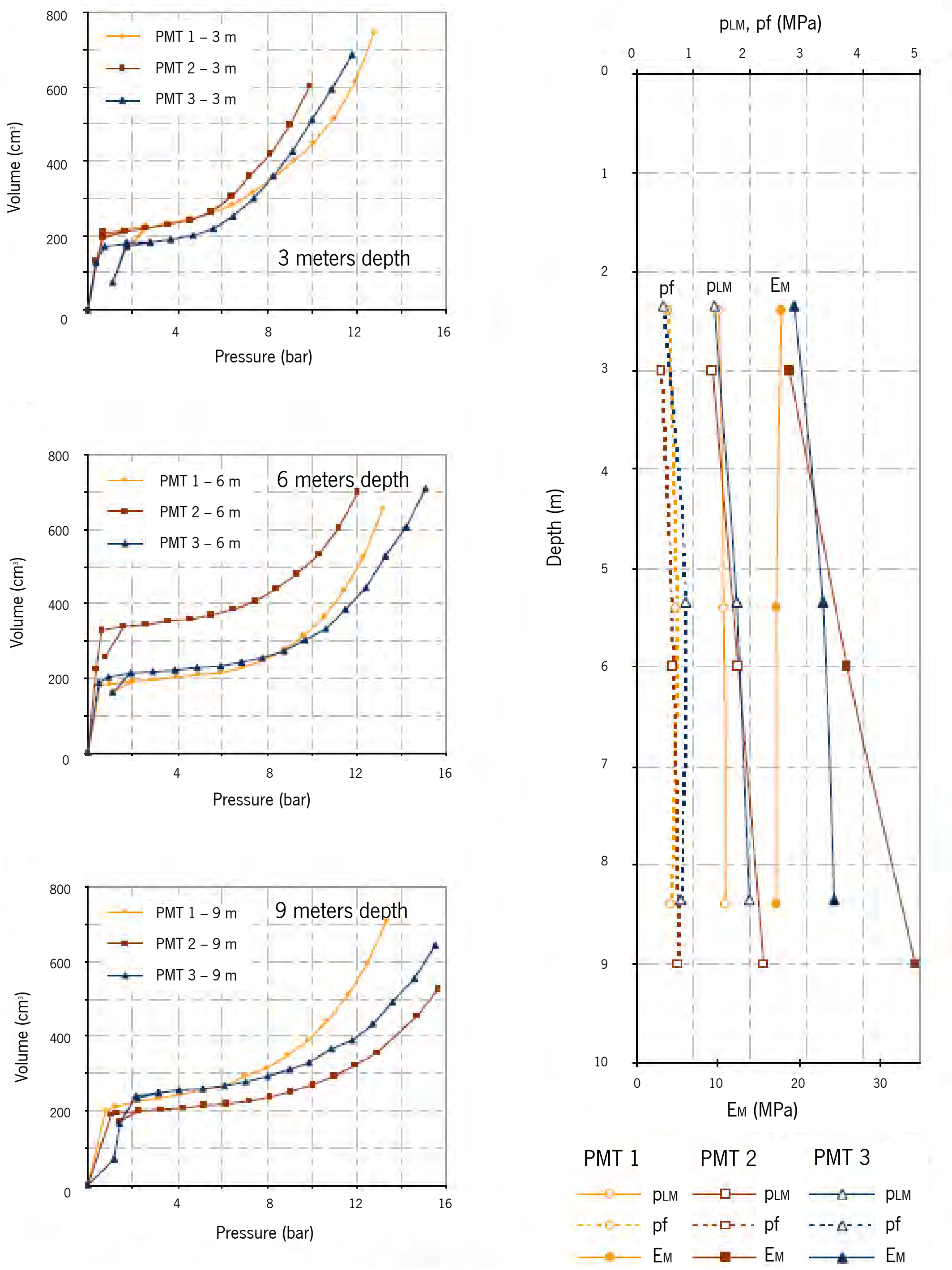


Figure 4.6 – PMT results from experimental site of case study 1 (FEUP), adapted from Esteves (2005)

According to Robertson's classification (Robertson, 1990; Esteves, 2005; Fellenius *et al.*, 2007) the charts of each CPT performed identify parts of the soil as overconsolidated and cemented (Robertson's classification areas: "Very stiff fine grained*" and "Sand to clayey sand*", * Overconsolidated or cemented). In Fellenius *et al.* (2007), they add: "However, the mechanical response of the residual soils to loading is often quite different from sedimentary soils with similar densities and grain-size distributions. Nontextbook geomaterials always require experimental verification"

4.2.2 Pile information

The pile considered is the axial bored pile, denoted E9 (Figure 4.2) with the characteristics shown in Table 4.2. This pile was tested to failure with static load (Figure 4.7). The load program is presented in Figure 4.8 and the ultimate bearing capacity obtained was 1350 kN.

Table 4.2 – Information of the bored pile E9 (reinforced concrete), case study 1 (FEUP)

Type	Section (m)	Length (m)	Embedded length (m)	Reinforcement		f_{ck} (MPa)	f_{cm} (MPa)
				Longitudinal	shear		
Compression Bearing pile	Circular 0.6	6.0	6.0	A500 12Ø25	Ø12//10cm	27.7	30.9

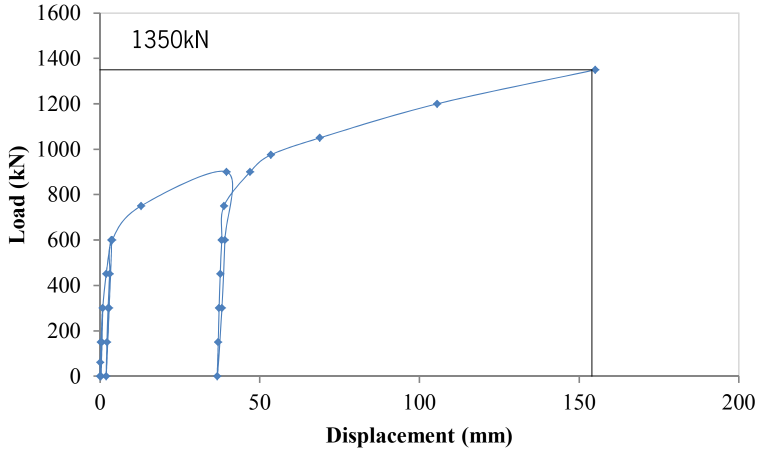


Figure 4.7 – Static load test result for bored pile E9, case study 1 (FEUP) adapted from Esteves (2005)

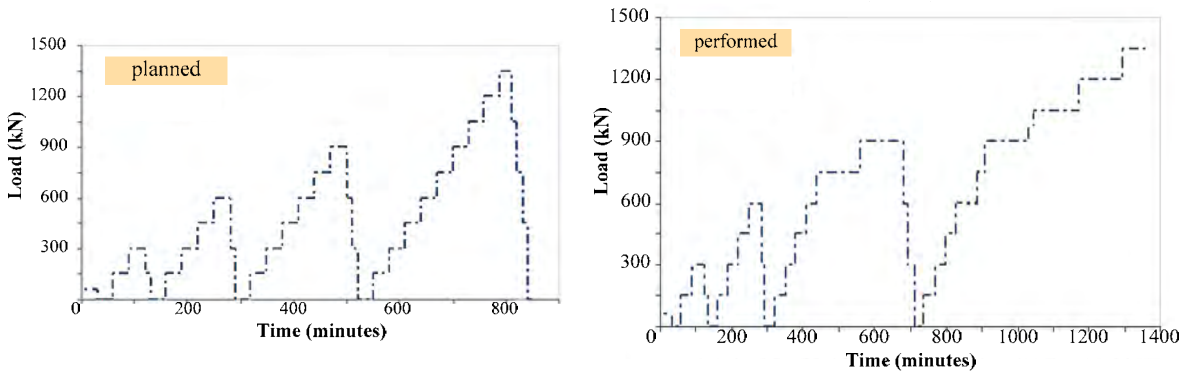


Figure 4.8 – Load steps planned and performed for static load test of pile E9, case study 1 (FEUP) adapted from Esteves (2005)

4.2.3 Actions

The evaluation of the actions requires knowledge of the project and design documentation. When no information is provided, the permanent and variable loads can be estimated based for example on the prediction of the vertical bearing capacity.

For this case study the design loads (actions) value was not available. The piles were installed for experimental purposes; therefore, they were not designed based on specific actions. Piles from this experimental site were designed based on costs limitations, sponsorships and testing necessities (*i.e.,* E1 to E8 are reaction piles for load test performed to E9 - Figure 4.2).

Consequently, the value of the actions adopted for further calculations was assumed and determined using the load test result (1350 kN - Figure 4.7). Also, an estimate of the actions value is done based on the resistance prediction models SHB, AIJ, FRc, FRp, S&F and A&V (Annex H) and applying partial SF proposed by design codes – eq.(4.1). The permanent and variable loads were considered equal in magnitude and the partial SF (γ, φ) from Eurocodes (CEN, 2002a,b; CEN, 2007) were applied for the estimation of the actions.

$$\begin{gathered}\frac{R_{load\ test}}{\gamma} > \varphi_G \times G_k + \varphi_Q \times Q_k \Leftrightarrow (assuming\ G_k = Q_k = Load) \\ \Leftrightarrow \frac{1350}{1.15} > 1.35 \times Load + 1.50 \times Load\end{gathered} \tag{4.1}$$

This also allowed the assessment and comparison of the prediction results with the load test value. These results are presented in Table 4.3. The values of the bearing capacity presented are a mean value from the prediction using all testing types. Accordingly, the final total value of 800 kN (Q_k=G_k=400 kN) was chosen as the design load value to carry out the following reliability analyses. This decision was based on the results depicted in Table 4.3, but mainly taking into account the value of bearing capacity that resulted from the load test.

Table 4.3 – Prediction of the vertical bearing capacity and estimation of the design load based on different methods considering G_k=Q_k, case study 1 (FEUP)

Method	Based on:	Bearing capacity (kN)	$Load$ (kN)
Static load test	-	1350	412
SHB	SPT	1639	500
AIJ	SPT	1328	405
French recom.	CPT	718	219
French recom.	PMT	1497	457
Shioi & Fukui	SPT	1073	328
Aoki & Velloso	SPT	2109	644

4.2.4 Uncertainties to consider

For the following reliability analyses of the case study 1, a performance function like shown in eq.(3.2) (Chapter 3) is used. Therefore, the following uncertainties are identified, as previously stated:

- the model error;
- the soil variability;
- the actions' uncertainties.

The model error (characterised by its bias and standard deviation) is summarised in Tables 4.4, 4.5 and 4.6 (refer to Table 3.3). It represents the error in the prediction of each model used.

Table 4.4 – Model uncertainties based on *in situ* test SPT (Honjo *et al.*, 2002; Okahara *et al.*, 1991)

Model error from:				
	SPT empirical method (SHB)		SPT empirical method (AIJ)	
	δ_t - tip	δ_f - side	δ_t - tip	δ_f - side
Mean	1.12	1.07	1.14	2.14
COV (%)	63	46	28	76
Standard deviation	0.706	0.492	0.319	1.626
PDF	Lognormal	Lognormal	Lognormal	Lognormal

Table 4.5 – Model uncertainties based on *in situ* test SPT and determined based on Japanese database (Annex J)

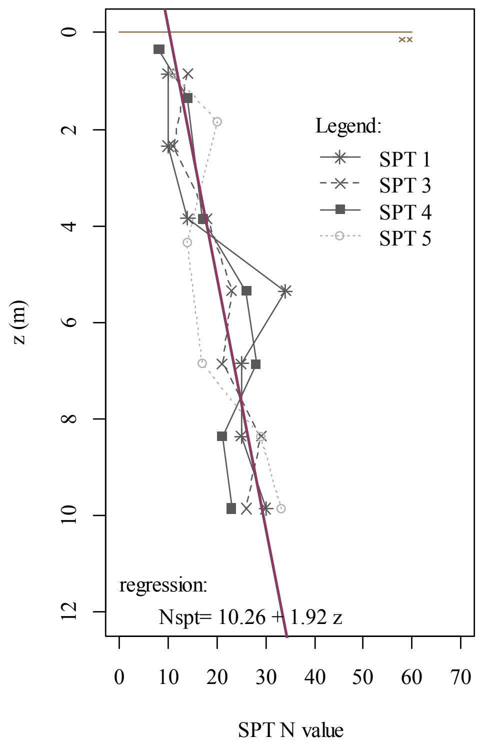
Model error from:			
	SPT empirical method (S&F)	SPT empirical method (A&V)	SPT empirical method (SHB)
	δ_m - total	δ_m - total	δ_m - total
Mean	2.86	2.47	1.16
COV (%)	36	60	39
Standard deviation	1.038	1.482	0.452
PDF	Lognormal	Lognormal	Lognormal

Table 4.6 – Model uncertainties based on *in situ* test CPT and PMT (Burlon, 2011; Banguelin *et al.*, 2012; AFNOR, 2012)

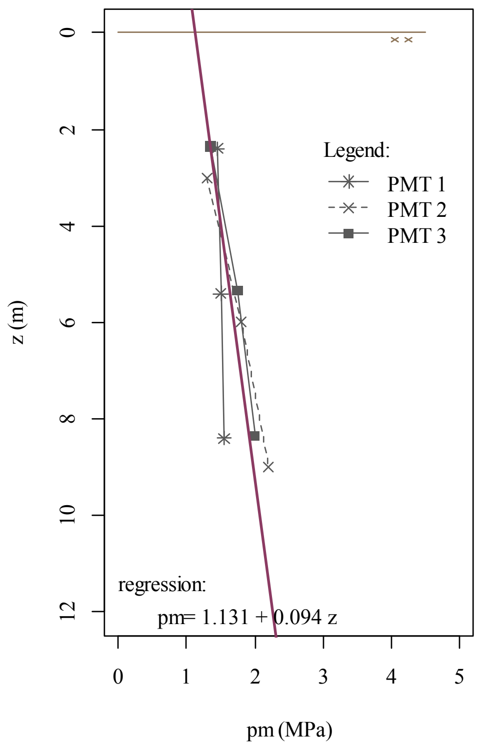
Model error from:		
	CPT empirical method	PMT empirical method
	δ_m - total	δ_m - total
Mean	1.36	1.10
COV (%)	43	22
Standard deviation	0.578	0.244
PDF	Lognormal	Lognormal

Concerning the soil, its variability is considered in the variance of the test parameter for each *in situ* test. For SPT in the N value, for CPT in qc value and for PMT in pl value. The results are presented next, including test trends (Figure 4.9 and Figure 4.10), Q-Q plots of the residuals (Figure 4.11) and autocorrelation graph (Figure 4.12) determined by the standard moment estimation method and considering the mean of all tests like explained in Annex C.

From the CPT results it is possible to detect some outliers, reporting perhaps a technical problem while performing the test. This is especially noticeable in CPT 4. If one needed to use these results (e.g. fs from CPT 4) it would probably be a good judgement to ignore these tests. Nevertheless, for the following performed computations all CPT in Figure 4.10 were used, since only the qc parameter is placed as input of the CPT-based resistance model (FRc model). Note that only CPT performed before pile installation (Figure 4.5) are presented in Figure 4.10 and then used for the uncertainties quantification. See more detailed procedure in Annex K, concerning CPT results.

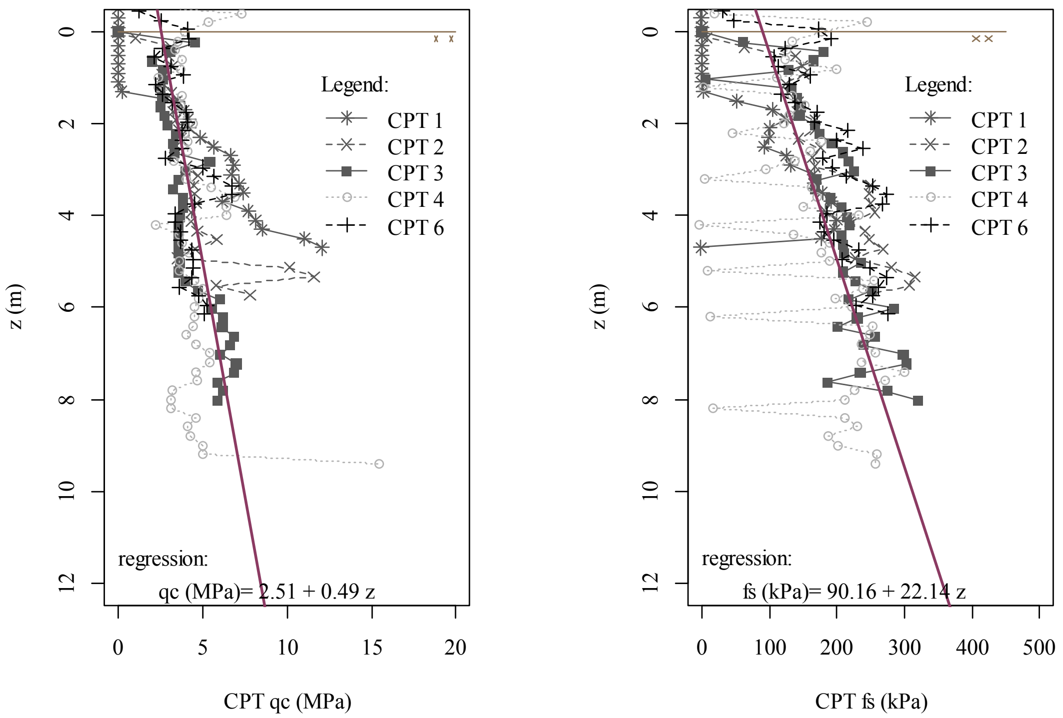


a) SPT's



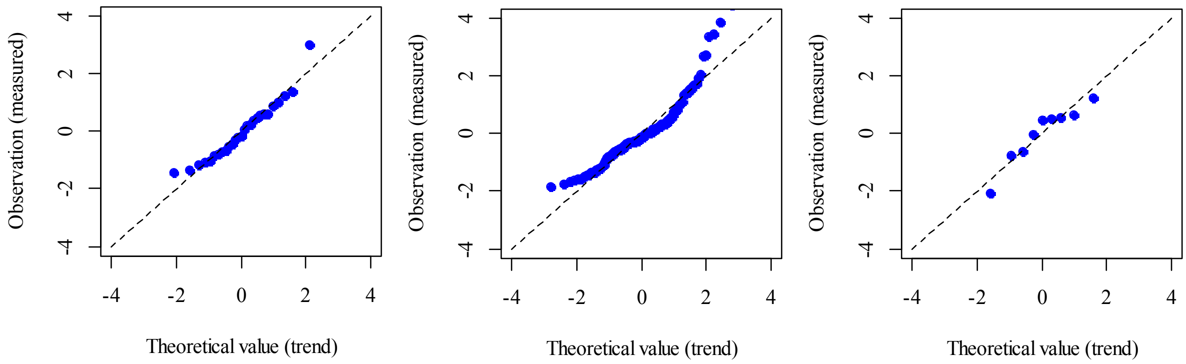
b) PMT's

Figure 4.9 – Case study 1 *in situ* tests trends



a) *qc* b) *fs*

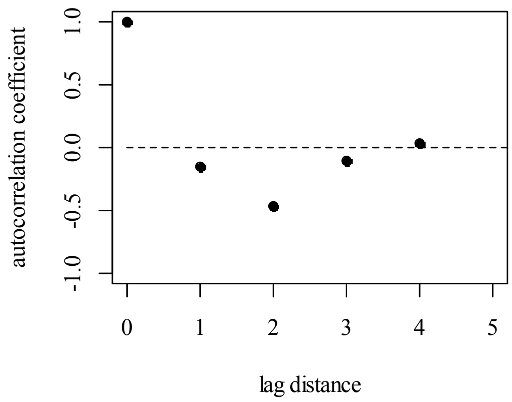
Figure 4.10 – Case study 1 *in situ* CPT trends



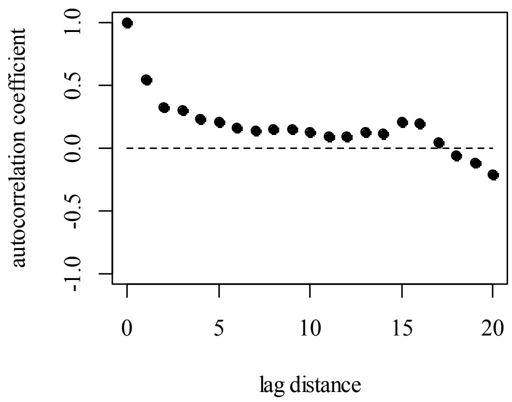
a) SPT N value b) CPT *qc* value c) PMT *pl* value

Figure 4.11 – Standard normal Q-Q plots for *in situ* tests of case study 1 (FEUP)

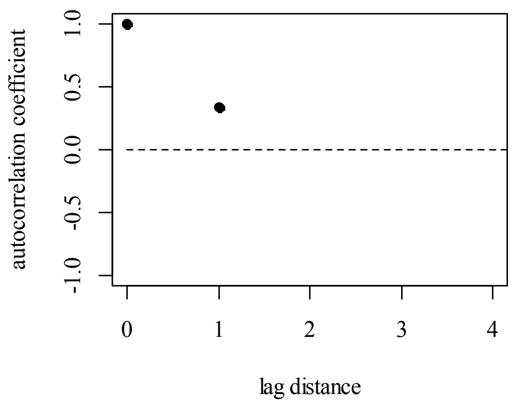
The SPT and CPT Q-Q plots display a good approximation to Normal of the residuals. PMT provided just few points for analysis; therefore, the approximation is not so easy to understand. Still, the Normal PDF was adopted for this *in situ* test.



a) SPT N value



b) CPT *qc* value



c) PMT *pl* value

Figure 4.12 – Autocorrelation plot for *in situ* tests of case study 1 (FEUP)

Regarding to autocorrelation of the *in situ* tests parameters (Figures 4.12), the graphs show the relationship between the different points in depth (vertical direction). The lag distance refers to the intervals of the test values. For SPT, lag distance corresponds to 1.5 m and for CPT corresponds to 0.2 m. The PMT would have a lag distance of 3 meters, a very high value, and also it has only 3 points (corresponding to two correlations) for each test performed to make this analysis (Figure 4.12c); therefore, conclusions about PMT autocorrelation results cannot the strained.

For SPT, the autocorrelation graph shows that this data has no correlation, because the value drops to zero right in the second lag point. This is to be expected in such type of tests (low number of points and high lag distance). Concerning the CPT (qc value), the autocorrelation value falls around 0.5 - 0.7 m (exponential fit). This is an important value for the control of the error in estimation of prediction of intermediate parameters and is also used for reducing the variance of the parameter.

Based on these analyses, the final uncertainties values for case study 1's soil variability are presented in Table 4.7. Additionally, the actions uncertainties are reviewed in Table 4.8 (refer to Table 3.4).

Table 4.7 – Characterisation of the soil uncertainties of case study 1 (FEUP)

Soil variability from:						
	SPT		CPT (MPa)		PMT (MPa)	
	N tip	N side	qc tip	qc side	pl tip	pl side
Mean	10.26+1.92z		2.51+0.49z		1.131+0.094z	
SD	4.6*	4.6**	1.87*	1.87**	0.1925*	0.1925**
PDF	Normal	Normal	Normal	Normal	Normal	Normal

* to be reduced by taking into account the influence zone on the pile tip (3×B) by averaging over the thickness

** to be reduced by taking into account the length of the pile by averaging over the thickness

Table 4.8 – Actions uncertainties of case study 1 (FEUP)

	Actions' uncertainties	
	δ_G - permanent	δ_G – variable
Mean	1.0	0.6
COV (%)	10	35
SD	0.10	0.21
PDF	Normal	Gumbel

4.3 RELIABILITY-BASED METHODOLOGIES

Although this case study has a generous amount of soil data and tests, the analysis presented in this section will be based on SPT only. Nevertheless, the follow up section presents the comparison between RBD using different *in situ* tests, namely SPT, CPT and PMT.

In order to make the comparative analysis, this section presents different reliability methodologies using SHB empirical model (SPT-based resistance model for pile bearing capacity prediction). This is one of the methods presented in section 3.2.3 with bias defined separately for tip and side resistance, closer to 1.0 and also low COV.

4.3.1 Reliability analyses FORM vs. MCS

The steps for a reliability analyses were listed in section 3.3, is summary they are:

- select target reliability index $\rightarrow \beta_T$;
- select performance function(s) $\rightarrow g(X)$;
- define calculation model(s);
- define random variables (RV);
- and finally estimate reliability based on FORM (section 3.3.3) or MCS (section 3.3.4).

Since there is still no agreement about which should be the target reliability index for such structure, the interval 2.5 to 4.0 was assumed based on the recommendations (section 3.3.1). As for the performance function, considering the ULS, the bearing capacity is compared with the actions applied to the pile.

Based on this, and on basic eq.(3.2), the performance function to take into account for the analyses in this section is eq.(4.2)[17]. The uncertainties of the RV (test parameter, model error and actions uncertainties) were previously defined.

$$M = \left(\delta_t \times A \times 100N_{tip} + \delta_f \times U \times 5\overline{N_{side}}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right) \tag{4.2}$$

FORM calculations

The eq.(4.2) is not a linear combination of the RV, and some RV are not all normally distributed. Therefore, FORM method will provide an approximation of the results to compare with MCS. For FORM calculations the following equation was used:

$$\begin{gathered} M = g\left(\delta_t, N_{tip}, \delta_f, \overline{N_{side}}, \delta_G, \delta_Q\right) = g(X_1, X_2, X_3, X_4, X_5, X_6) \Leftrightarrow \\ \Leftrightarrow M = (X_1 \times A \times 100X_2 + X_3 \times U \times 5X_4) - (X_5 \times G_k + X_6 \times Q_k) \Leftrightarrow \\ \Leftrightarrow M = (X_1 \times 28.27 \times X_2 + X_3 \times 56.55 \times X_4) - (X_5 \times 400 + X_6 \times 400) \end{gathered} \tag{4.3}$$

This information will be inputted in the same software referred in example of Chapter 3 (Henriques *et al.*, 1999). The independent term is null, and the linear and quadratic coefficients (Bi and Cij) are presented in Table 4.9. It was assumed a β-error between interactions of 10^{-4}. After 7 interactions the results obtained are the ones presented in Table 4.10.

[17] Unit tip resistance and unit side resistance have a limit value of 3,000 kPa and 200 kPa respectively.

Table 4.9 – Performance function coefficients for FORM iterative calculations of case study 1 (FEUP)

	X_1	X_2	X_3	X_4	X_5	X_6
Bi	0	0	0	0	-400	-400
Cij	*1*	*2*	*3*	*4*	*5*	*6*
1	0	28.27	0	0	0	0
2	0	0	0	0	0	-
3	0	0	0	56.55	-	-
4	0	0	0	-	-	-
5	0	0	-	-	-	-
6	0		-	-	-	-

Table 4.10 – Results of FORM iterative calculations of case study 1 (FEUP)

	Safety side – Resistance				Against safety side – Actions	
	X_1	X_2	X_3	X_4	X_5	X_5
Design point X^*	0.48	19.8	0.53	14.4	1.03	0.73
Sensitivity α	0.58	0.20	0.66	0.27	-0.13	-0.33
Reliability achieved	$\beta = 2.31 \rightarrow pf = 0.0105 = 10^{-2}$					

MCS calculations

Monte Carlo simulations (MCS) are used as reference method, and will give accurate results for comparison with FORM approximations. For better accuracy, and since only one run is needed for this calculation, n = 1,000,000 was selected. As will be shown and explained later, n around 200,000 would suffice for this case. The MCS results for case study 1 are depicted in Figure 4.13, as one can see the generated RV lead to a distribution of resistance and actions to finally achieve the probability of failure of 0.0125 and the correspondent reliability index of 2.24. The time consumed for the number of simulations n = 1,000, 000 was 2.5 minutes.

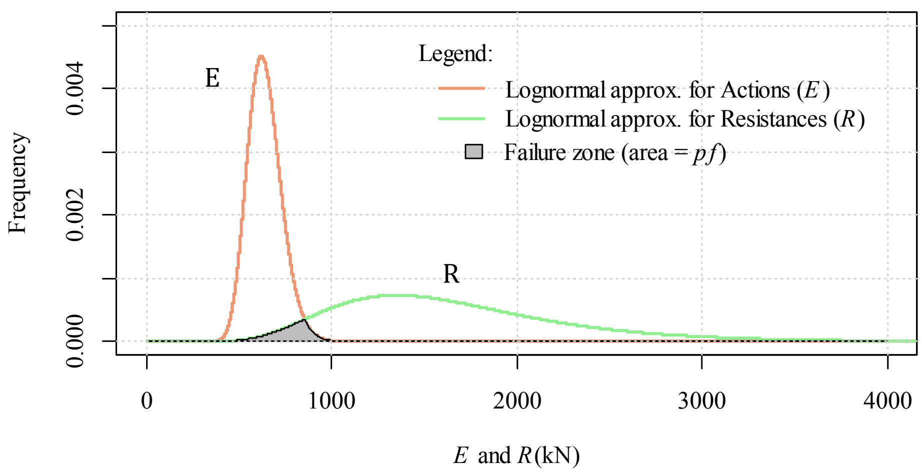


Figure 4.13 – Lognormal PDF approximations of E and R and failure zone (shaded area), achieved with n=1,000,000 MCS for case study 1 (FEUP)

Summarising, for the case study 1, the results using these two methodologies were:

- FORM: reliability index β = 2.31 and a probability of failure pf = 0.0105.
- MCS: reliability index β = 2.24 and a probability of failure pf = 0.0125.

FORM result is consistent with MCS result. For this reason, the values for α, called sensitivity factors (Table 4.10.) can be considered as acceptable for further analyses and conclusions.

4.3.2 Minimum length (RBD) vs. allowable load (Safety evaluation)

This section presents the results of the RBD and Safety evaluation, using both FORM and MCS. For MCS, a stability study was performed (Annex N). The stability for case study 1 was achieved for n=200,000 considering the interval [2.5; 4.0] for reliability index values (discussion in section 3.3.2). Nonetheless, the results for n=1,000,000 are also presented in the following figures for comparison.

For these reliability analyses the following lengths and loads were deliberated:

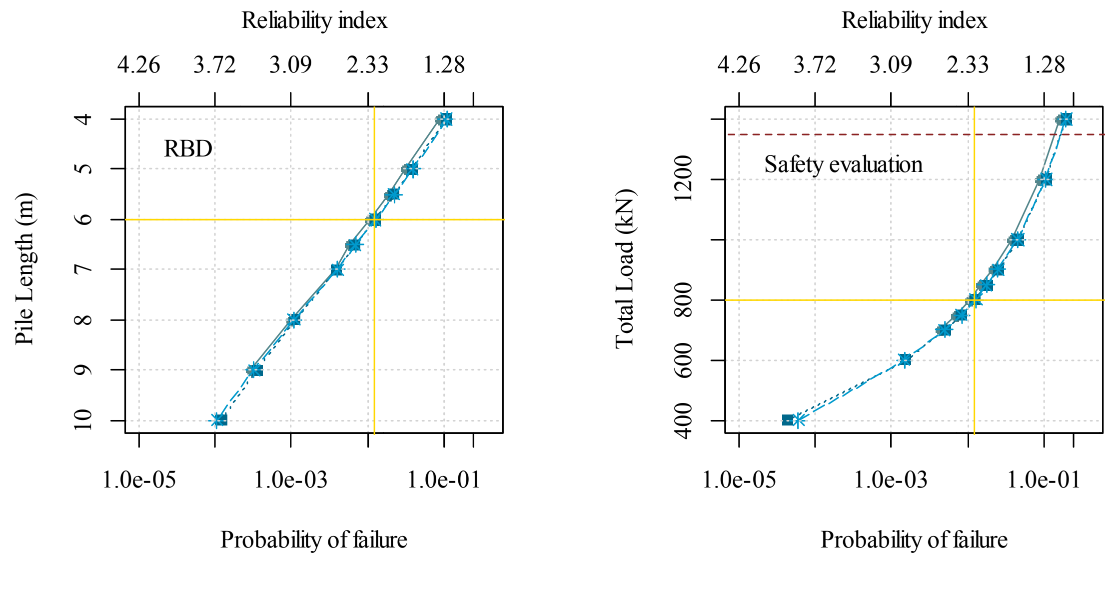
- RBD approach: different lengths of the pile D=[4; 5; 5.5; 6; 6.5; 7; 8; 9; 10] m, being the actual length of the pile installed 6 m (results in Figure 4.14.a);
- Safety evaluation approach: different load values E=[400; 600; 700; 750; 800; 850; 900; 1000; 1200; 1400] kN (results in Figure 4.14.b).

The time consumed for each approach was 25 to 35 min total when n=1,000,000 and 4 to 6 min total when n=200,000.

The following figures (Figure 4.14.a and 4.16.b) also depict a light line marking the 6 m (length of the pile), the value of the design load considered (total of 800 kN) and the load test result (1350 kN). It is possible to conclude that the actual pile installed in the experimental site does not achieve the reliability index normally recommended.

Also, when comparing the simulations n=1,000,000 and n=200,000 the results are consistent for both RBD and Safety evaluation approaches.

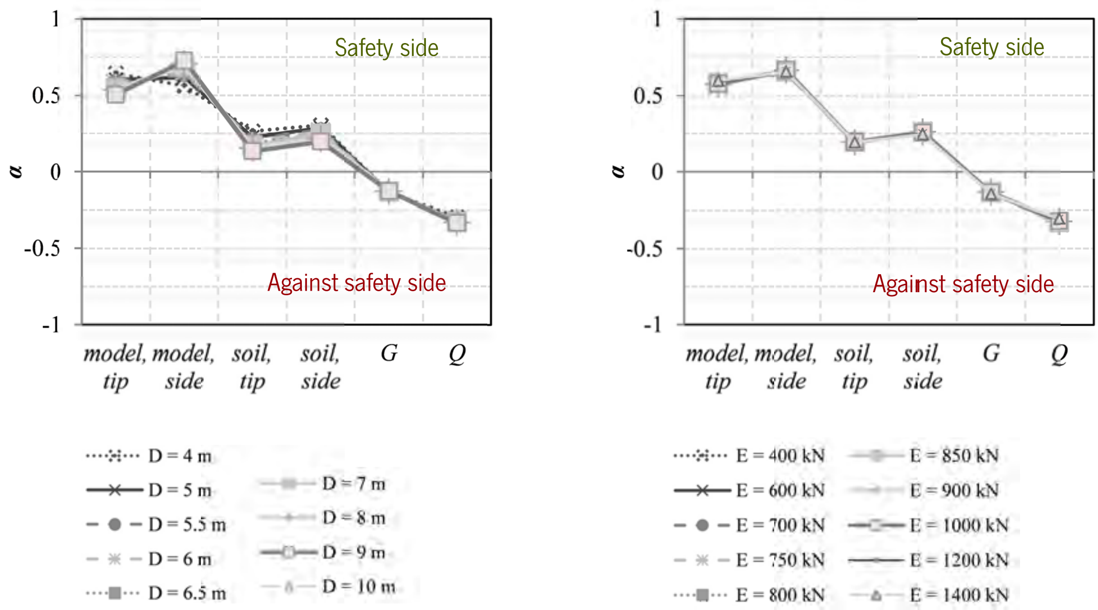
Concerning the comparison between FORM and MCS reliability results, the slight deviations are due to the type of RV (normally and non-normally distributed) and also the performance function (not linear). Nevertheless, the FORM results give very acceptable approximations. Accordingly, the following Figure 4.15 presents the obtained values for α factors for each approach.



a) RBD

b) Safety evaluation

Figure 4.14 – Results of reliability-based approaches using FORM and MCS for case study 1 (FEUP)



a) RBD

b) Safety evaluation

Figure 4.15 – Sensitivity factors from FORM analyses for case study 1 (FEUP)

It is possible to conclude that these reliability-based approaches (RBD and Safety evaluation) present the same results for α values. Also, as expected, the actions variables (G and Q) have negative α and the resistances variables have a positive α. For this case study and based on the FORM results, the uncertainty that has more influence in pile reliability is the model error (α around 0.60) being the other uncertainties considerably less important (α around 0.25).

4.3.3 Reliability-based safety factors

The procedure to determine reliability-based SF used in this section was proposed by Kieu Le (2008) and is described in detail along the section 3.3.6. This method is based on the combination of DVM formulas and MCS. DVM is based on FORM, a powerful method to determine the SF for RA level I. This combination includes the advantages of both methods, *i.e.,* conceptual transparency, robustness, and flexibility of the calculations.

According to the procedure it is necessary to define the statistical parameters of the variables R and E (mean, SD, COV and PDF type), the characteristic values and also the target reliability index (β_T). The characteristic values (R_k and E_k) used for determination of the SF based on reliability analyses, were assumed as:

- the mean value for both R and $E \rightarrow R_k=\overline{\mathrm{R}}$ and $E_k=\overline{\mathrm{E}}$;
- the mean value for R and the high fractile of 95% for $E \rightarrow R_k=\overline{\mathrm{R}}$ and $E_k=E_{95\%}$.

After definition of the target reliability indexes, performance function and characterisation of the RV, the resistance and load SF (multiplying factors) can be determined.

The results of the n=1,000,000 MCS were used (section 4.3.1) to determine the load and resistance SF for the single pile foundation (case study 1, FEUP, 6 m length and 0.6 m of diameter). The histograms were obtained for the resistances and loads, and Normal and Lognormal distributions (PDF) were fitted to data – see Figure 4.16. It is possible to conclude the following:

- the Lognormal PDF has a better fitting than the Normal;
- and, as expected, the resistances have a much higher dispersion (larger uncertainty) than the loads/actions.

After fitting the PDF type, all points simulated (R,E) are divided in three groups as presented in Figure 4.17. The first group comprehends the points near the limit state line named limit state zone ($R/E \in$ 1±0.02) → 3,178 limit points (~0.3%); then the points in failure zone ($R/E <$ 0.98) → 11,009 fail points (~1%); and then the points in safety zone ($R/E >$ 1.02) → 985,813 safe points (~98.6%).

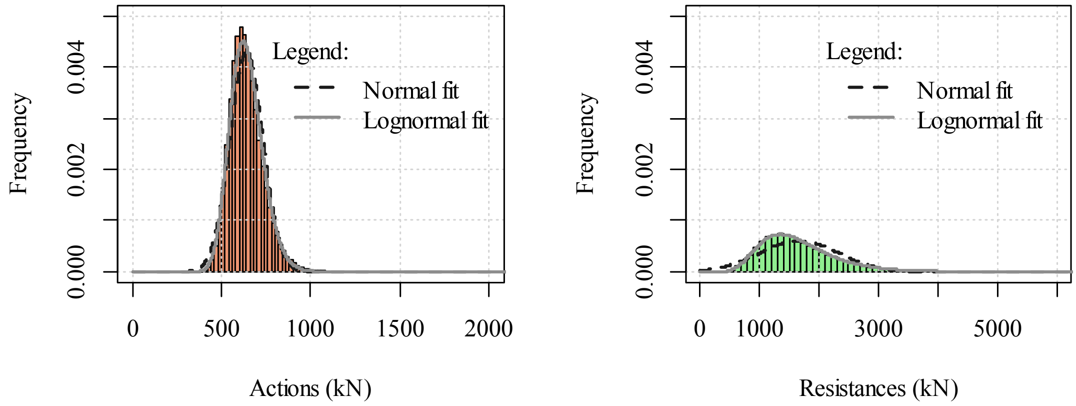


Figure 4.16 – Histograms and PDF approximations to E and R, achieved with n=1,000,000 MCS for case study 1 (FEUP, B=0.6m, D=6m and total load=800kN)

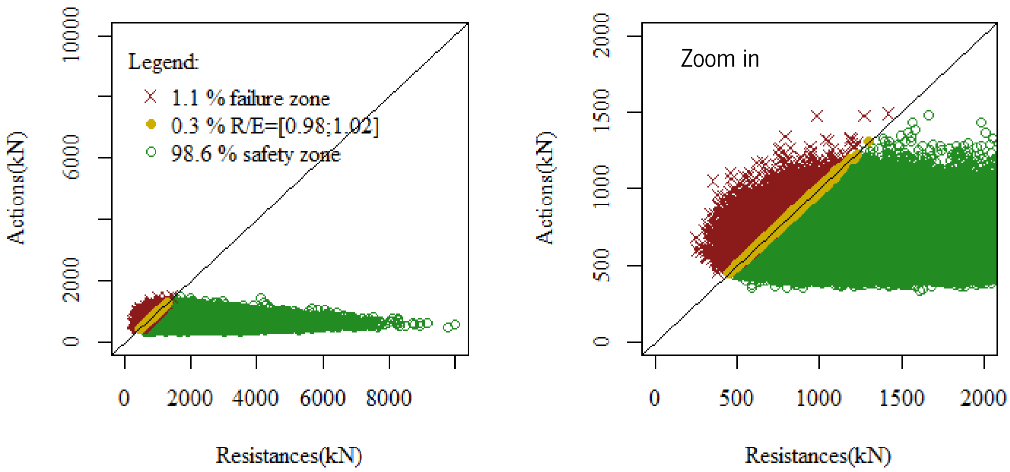


Figure 4.17 – Graphical representation of the MCS points near limit state line, failure zone and safety zone, achieved with n=1,000,000 for case study 1 (FEUP)

The points of the limit state zone are then represented in a normalised space, see Figure 4.18, and the design point is determined. Finally Figure 4.19 presents the range of SF obtained for different reliability indexes assumed within the recommendations, β=[2.5,4.0]. While Table 4.11 presents the preliminary results of this reliability-based SF methodology, considering a lognormal distribution for R and E.

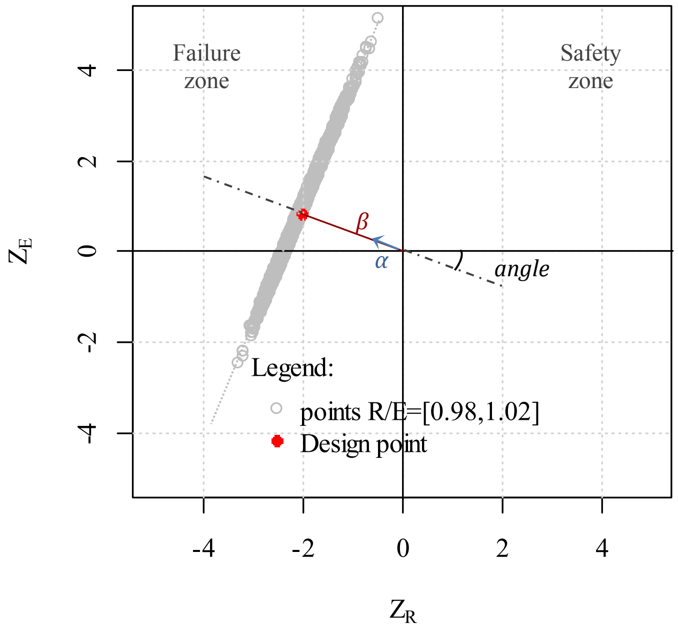


Figure 4.18 – Graphical representation of the MCS points near limit state line in normalised space, design point (Z^*) and reliability index (β) representation for case study 1 (FEUP)

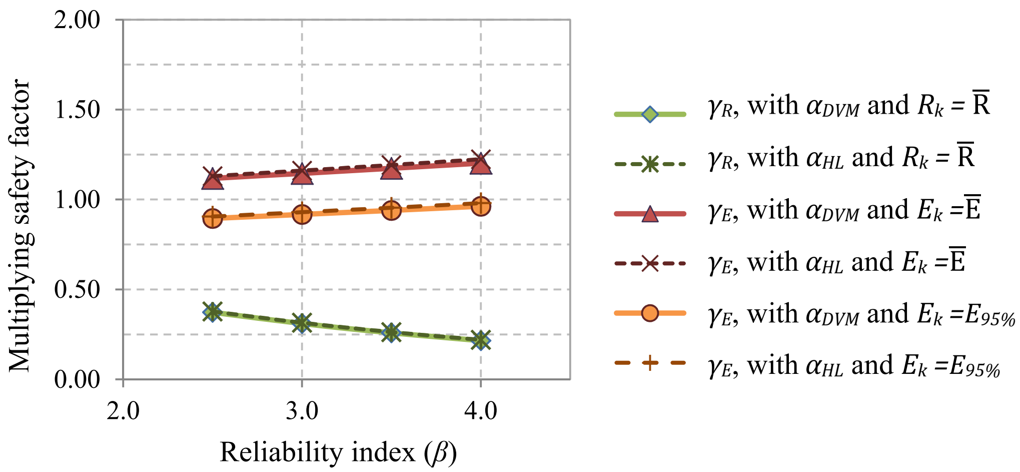


Figure 4.19 – Multiplying SF based on reliability analyses for case study 1 (FEUP), considering a Lognormal distribution for R and E

The SF values were calculated for mean characteristic values ($R_k = \overline{R}$, $E_k = \overline{E}$), but also considering the commonly used fractile of 95% for actions ($R_k = \overline{R}$, $E_k = E_{95\%}$). From the results, using Lognormal approximation, it is possible to highlight that:

- the values obtained for the SF, based on Lognormal PDF fit, with DVM or HL formulas are the same;

- the resistance multiplying SF resulted between 0.2 and 0.4. These low values may result from the high number of points with a very high resistance, a thick tail – high uncertainty (as one can see in Figure 4.17);
- the values for actions SF should be higher than one, but when using $E_k = E_{95\%}$ the values obtained are slightly lower than one (0.9 to 1.0), due to its low variability and necessity to reduce, since its sensitivity factor is relatively low;
- when using the mean value for the characteristic value of the loads, the SF falls between 1.1 and 1.2 that, although still low, are higher than one;
- finally it is noticeable that the resistance SF has a noticeable higher variability with the reliability index than the actions SF.

The values recommended by the Eurocode 7 (Annex A of CEN, 2007 – multiplying resistance SF between 0.7 and 1.0 and load SF between 1.0 and 1.5) are higher than the ones calculated here, the reason could be the fact that the reliability index for this case study 1, β=2.2, is far from the target one.

Table 4.11 – Safety factors' preliminary results based on reliability analyses for case study 1 (FEUP)

	Approximation to Lognormal	
	Resistances	Actions
Approx. Mean values (kN)	1670	640
Approx. Standard deviation (kN)	651	91
Design point (kN):		
- using max likelihood	695	681
- using min(β)	725	711
	Z_R = -2.03	Z_E = 0.82
	β=2.2 →	pf=0.014
Sensitivity factors (α)		
- DVM	0.94	-0.34
- HL	0.93	-0.38

4.4 RBD USING DIFFERENT *IN SITU* TESTS

This section presents the RBD approach applied to the case study 1, but using different *in situ* tests for prediction of the vertical bearing capacity of the pile (resistance).

4.4.1 Resistance based on SPT

With the SPT results (Figure 4.9a) it is possible to assess the vertical bearing capacity of the pile (resistance). Four different SPT-based models are presented next, namely SHB, AIJ, S&F and A&V (recall Annex H), and then the reliability is evaluated with RBD approach using MCS. Based on this, and on eq.(3.2), the performance functions to take into account for the analyses are presented in eq.(4.4), (4.5), (4.6) and (4.7).

SHB[18]: $M = \left(\delta_t \times A \times 100N_{tip} + \delta_f \times U \times 5\overline{N_{side}}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right)$ (4.4)

AIJ[19]: $M = \left(\delta_t \times A \times 100N_{tip} + \delta_f \times U \times 3.3\overline{N_{side}}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right)$ (4.5)

S&F: $M = \delta_m\left(A \times 100N_{tip} + U \times \overline{N_{1,side}}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right)$ (4.6)

A&V: $M = \delta_m\left(A \times K/3.5 \times N_{tip} + U \times \alpha' \times K/7 \times \overline{N_{1,side}}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right)$ (4.7)

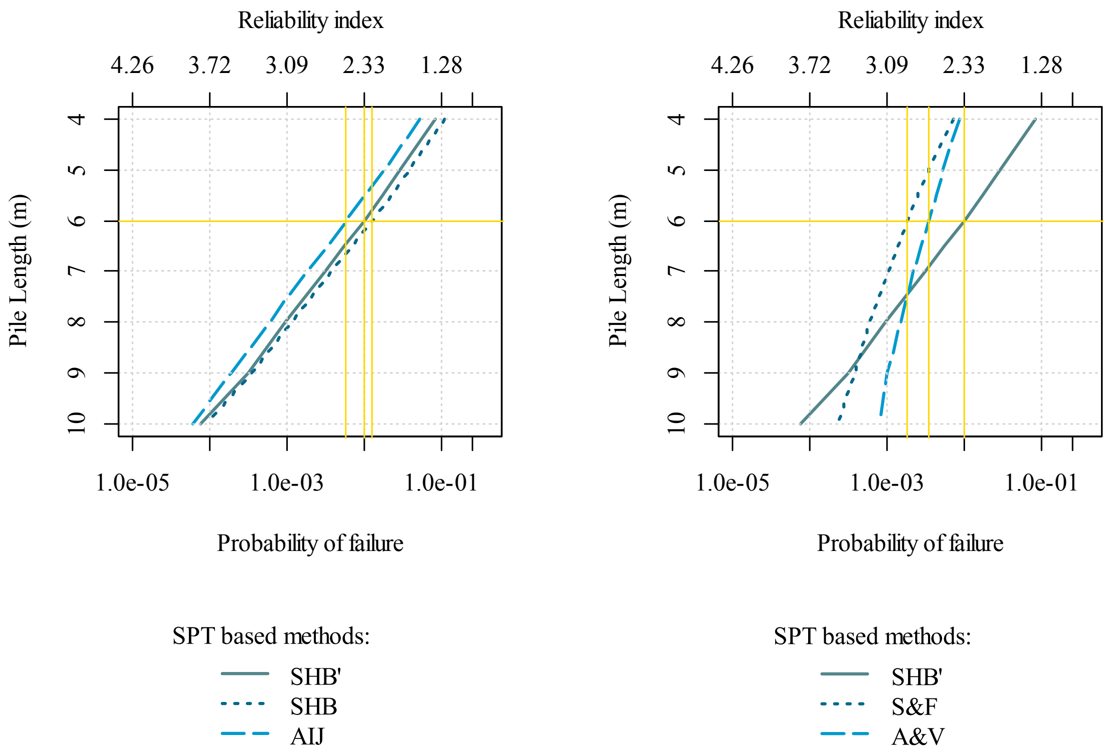
The δ value for the model errors are summarised in Tables 4.4 and 4.5. The uncertainties of SPT N value and δ value for actions are defined in Tables 4.7 and 4.8 correspondingly. The parameters K e α' are related to the type of soil. A e U are the areas in contact with soil, respectively the area at the tip and the area along the pile. These two variables are considered deterministic for the reasons previously mentioned in Chapter 3 (section 3.2).

The following Figure 4.20 presents the results of RBD approach allowing comparison of the influence of each pile resistance model adopted.

In both images of the Figure 4.20 the results using SHB model error calculated based on Japanese database (denoted SHB') are presented for easiness of comparison. Note that SHB and AIJ methods have the model error characterised for each tip and side resistance (separately) and that SHB', S&F and A&V have the model error characterised for the total resistance (tip + side).

The SHB and AIJ methods present very similar results as expected (these methods have a similar basis of formulation), while for S&F and A&V results, some deviations are detected due to its high model error COV. It is also noticed that the SHB and SHB' results are the same, as expected.

[18] Unit tip resistance and unit side resistance have a limit value of 3,000 kPa and 200 kPa respectively.
[19] Unit tip resistance has a limit value of 10,000 kPa.



a) SHB', SHB and AIJ pile resistance models

b) SHB', S&F and A&V pile resistance models

Figure 4.20 – Results of the RBD approach using MCS and different SPT-based models for predicting bearing capacity for case study 1 (FEUP)

Additionally, the same calculations are repeated using different uncertainties for SPT N value (COV of 10%, 20% and 50%)[20] recommended in literature (Table 3.3). The following figures present the result of the probability of failure's variation with the alternative SPT N value COV (Figure 4.21).

The results presented are consistent regarding the changes in probability of failure. The *pf* variation with SPT N value COV increases as the model error magnitude decreases, meaning that if a method has a high model error (e.g. A&V model) the variance of a soil property uncertainty will not affect as much as when the model error are lower (e.g. SHB or AIJ models). The results obtained for the SHB and AIJ present a high variation when COV is 50%, but this is a very high COV value and not commonly considered. Furthermore, it is possible to confirm that the uncertainty calculated from the *in situ* SPT performed is within the interval of COV 10-20% (recommendations interval).

[20] Correspondent to a low, medium and high COV.

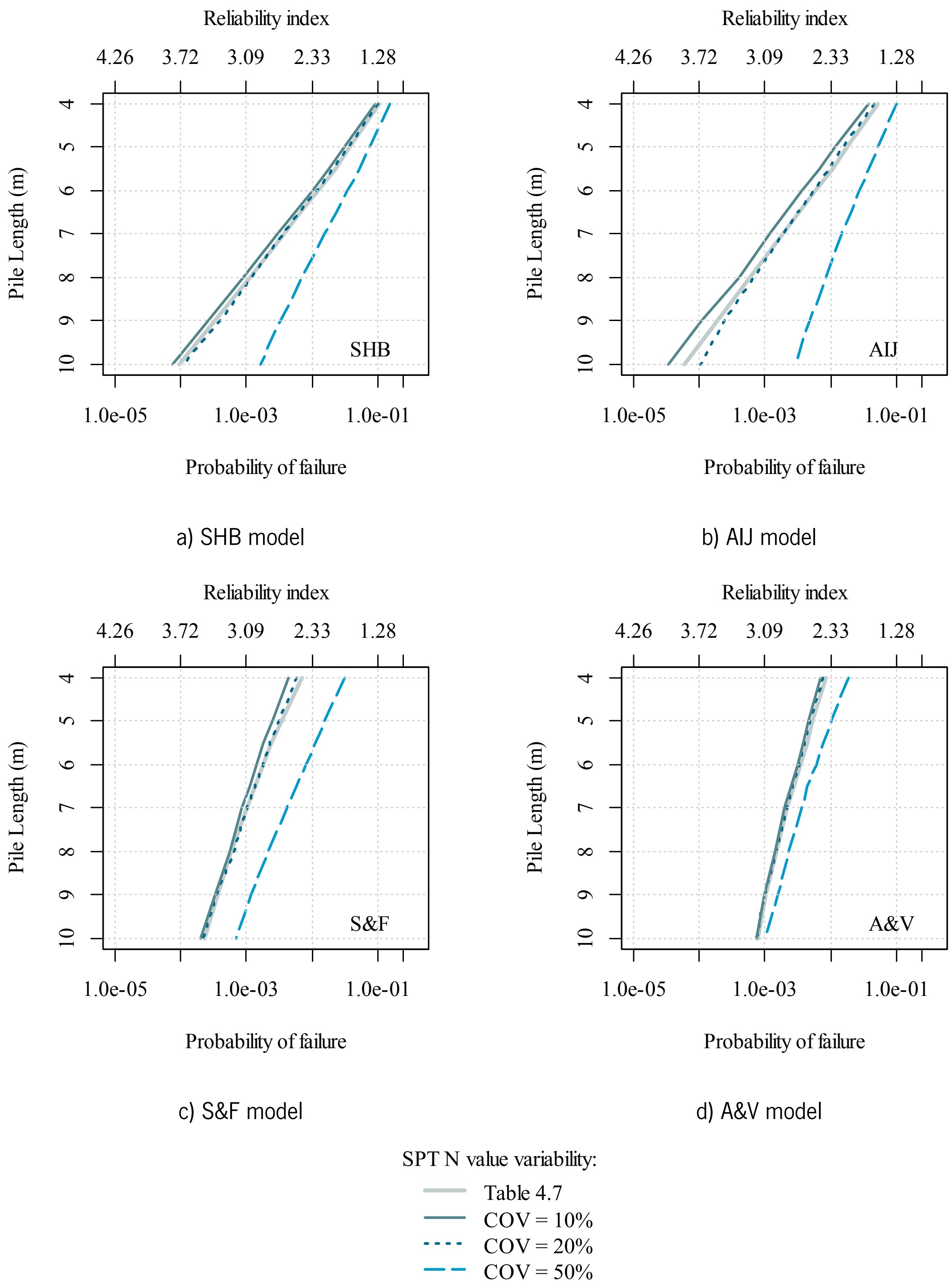


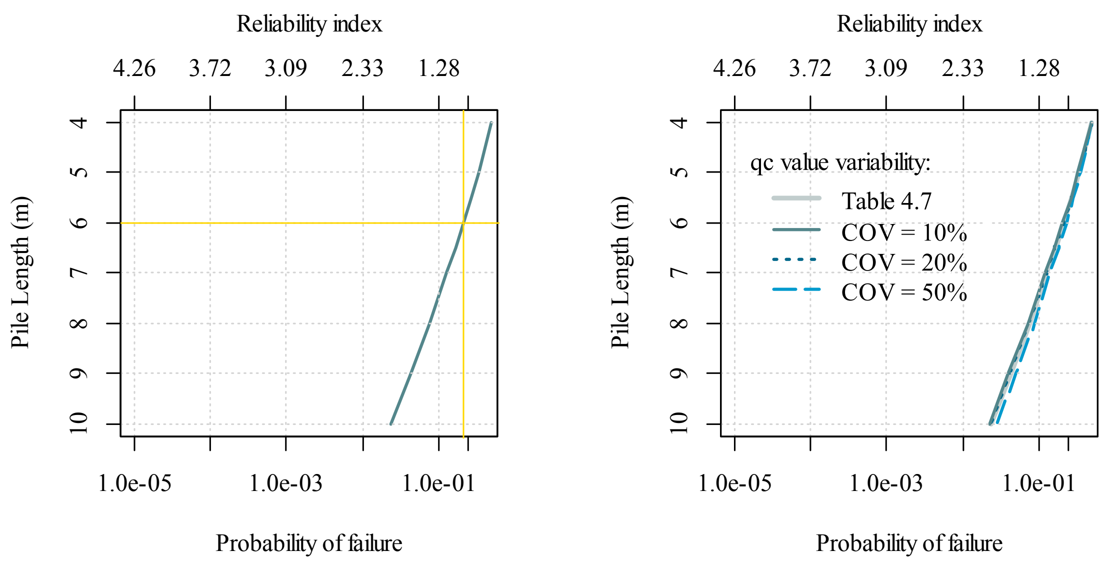
Figure 4.21 – Results of the RBD approach using MCS and different SPT-based models for predicting bearing capacity for case study 1 (FEUP)

4.4.2 Resistance based on CPT

The CPT is also a very widely known method in geotechnical practice. It has a higher reliability than SPT but it is more expensive, time consuming and requires special machines and trained technicians to perform the tests. The method from French recommendations (AFNOR, 2012; Banguelin *et al.*, 2012) is presented as a possible method to assess the vertical bearing capacity of the pile (resistance) and to perform RBD approach using MCS for case study 1. As such, the performance function is presented in eq.(4.8).

$$M = \delta_m\left(A \times q_{tip} + U \times f_{side}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right) \tag{4.8}$$

The uncertainties were presented in Tables 4.6, 4.7 and 4.8, correspondingly for CPT-based model error (δ_m), CPT qc value variability (inside the formulas of q_{tip} and f_{side}) and actions uncertainties ($\delta_G,\ \delta_Q$). As implemented in the previous section, the same calculations are performed assuming different uncertainties for CPT value of qc (COV of 10%, 20% and 50%). The Figure 4.22 presents all the results of these CPT-based analyses.



a) Using calculated soil variability (Table 4.7)

b) Using recommended values for soil variability (Table 3.3)

Figure 4.22 – Results of the RBD approach using MCS and CPT-based model (FRc) for predicting bearing capacity for case study 1 (FEUP)

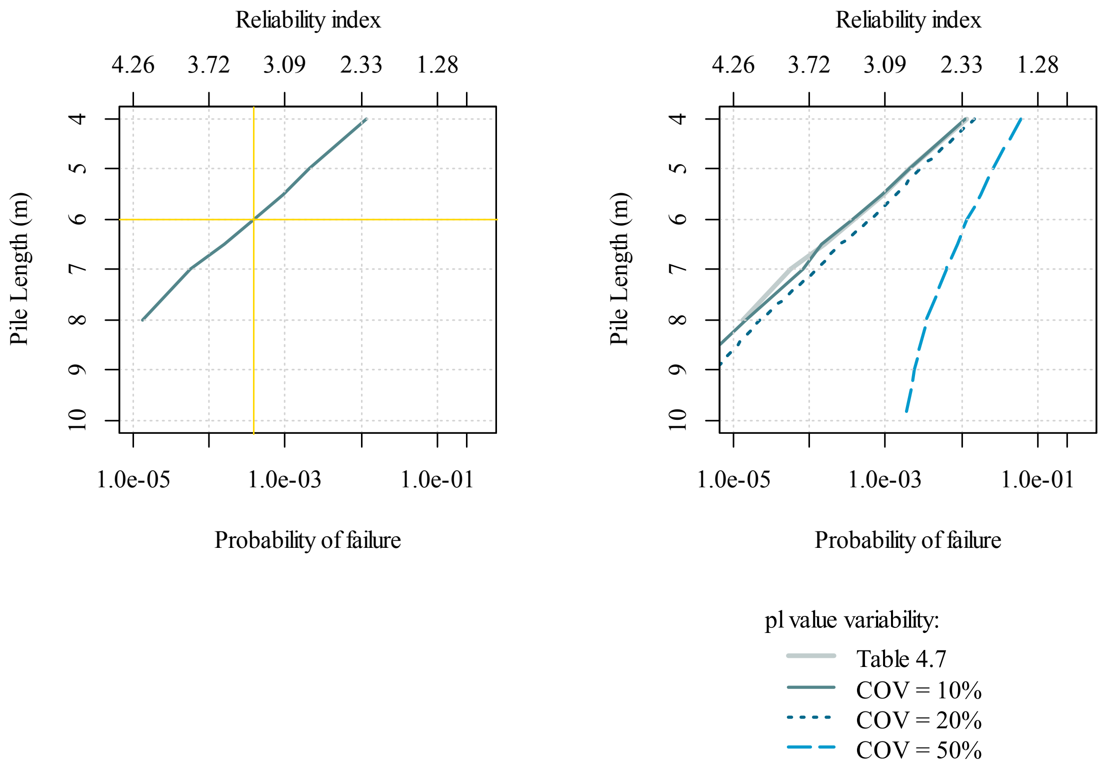
The results using CPT-based model present slight variations when qc COV value changes. This shows that, not only the uncertainties have influence in the sensitivity of the results but also the level of reliability in which the problem is, influences this sensitivity.

4.4.3 Resistance based on PMT

The PMT is also included in this study because Burlon (2011), Bengalin *et al.* (2012) and Burlon *et al.* (2012) studies have its model error characterised and because the case study 1 has an extensive *in situ* and laboratory soil characterisation to allow the analysis. The method from French recommendations is presented as a possible method to assess the vertical bearing capacity of the pile (resistance) and to perform RBD approach using MCS for case study 1. As such, the performance function is presented next in eq.(4.9).

$$M = \delta_m\left(A \times q_{tip} + U \times f_{side}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right) \tag{4.9}$$

The uncertainties were presented in Tables 4.6, 4.7 and 4.8 respectively for PMT-based model error (δ_m), PMT pl value variability (inside the formulas of q_{tip} and f_{side}) and actions uncertainties ($\delta_{G,}$ δ_Q). Once again, the same calculations were performed assuming different uncertainties for PMT value of pl (COV of 10%, 20% and 50). The results using PMT-based method present also variations supporting the results obtained previously (Figure 4.25).



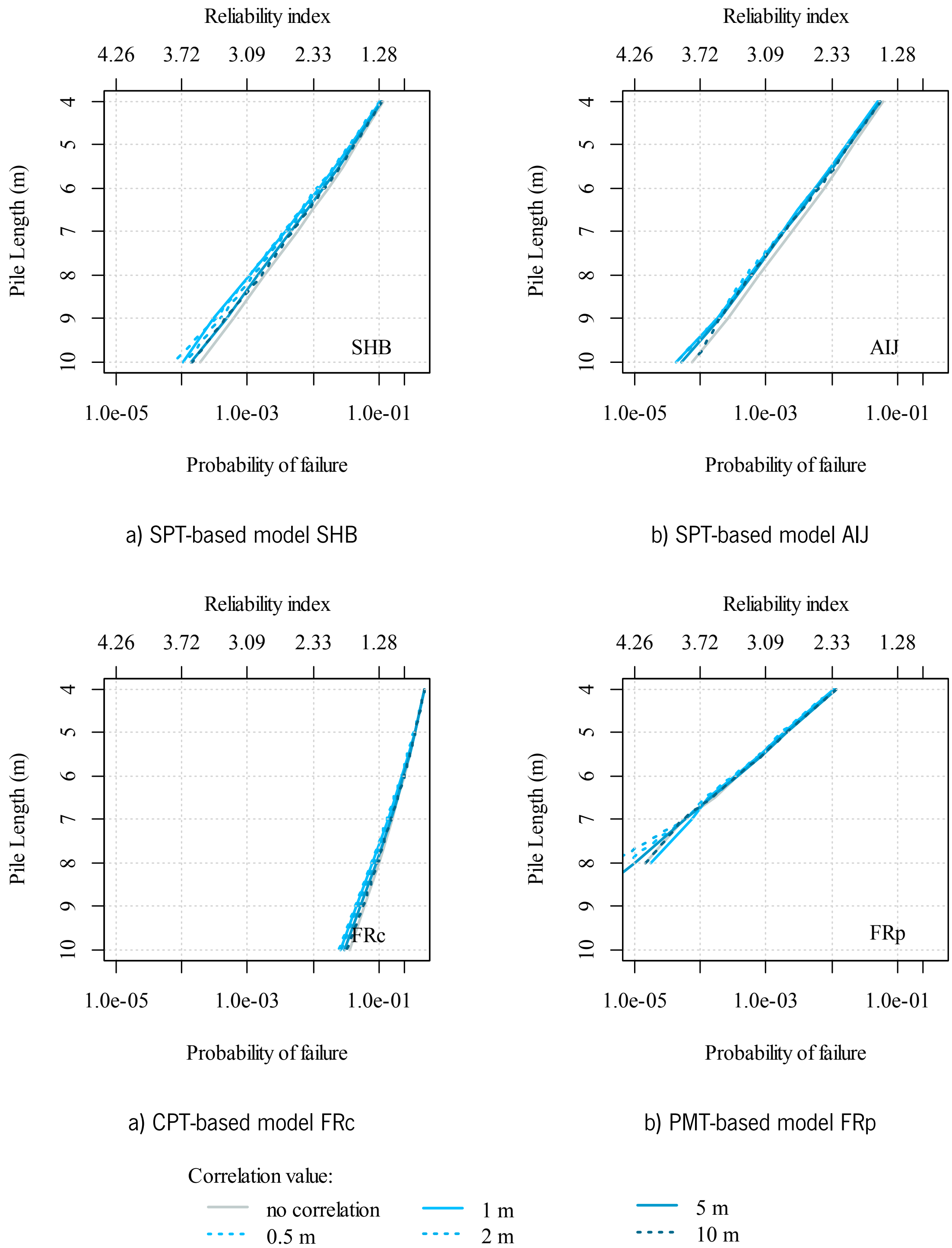
a) Using calculated soil variability (Table 4.7)

b) Using recommended values for soil variability (Table 3.3)

Figure 4.23 – Results of reliability-based approach RBD using MCS and PMT-based model (FRp) for predicting bearing capacity for case study 1 (FEUP)

4.4.4 Soil spatial correlations

To understand the influence of the consideration and not consideration of the spatial variability, RBD approach was performed for different soil correlation lengths and for for each *in situ* test and models. The interval of autocorrelation values considered was θ=[0; 0.5; 1; 2; 5; 10] m - Figure 4.26.



a) SPT-based model SHB

b) SPT-based model AIJ

a) CPT-based model FRc

b) PMT-based model FRp

Figure 4.24 – Reliability-based sensitivity analysis to soil autocorrelations, using MCS and different models for predicting bearing capacity for case study 1 (FEUP)

A large value of autocorrelation will imply a smoothly varying field, while a small value will imply a ragged field. This is a very important parameter for random field modelling, especially when considering a great volume of soil, like for example a slope stability reliability analysis (Kuo, 2008; Shen, 2012). As one can see, the variations are very smooth, independently of the autocorrelation. Also, all of them, when considered turn the analyses less conservative.

The next section presents a more detailed sensitivity analysis, and a full discussion of the pf's sensitivity to all the variations studied is done in the last section of this chapter (section 4.6).

4.5 SENSITIVITY ANALYSIS OF UNCERTAINTY TYPES

The main goal of the sensitivity analyses is to evaluate the relative influence of the uncertainty associated with each RV on the final result (*i.e.,* the probability of failure). These analyses are similar to those of a parametric study where the impact on both the performance of the pile and its reliability is assessed by analysing different lengths (RBD) and different combinations of the uncertainties (considering and not considering a specific uncertainty).

This section presents a study of the pf's sensitivity to consideration and not consideration of the different uncertainties regarding model error, actions and soil variability. All analyses use RBD approach and MCS. Concerning the resistance model for pile bearing capacity, the SHB, AIJ (both SPT-based), FRc (CPT-based) and FRp (PMT-based) models were chosen to perform the analyses.

The Table 4.12 presents the combinations of uncertainties studied for SPT-based models (with δ_t and δ_f) and the Table 4.13 presents the combinations of uncertainties studied for CPT-based and PMT-based models (with only δ_m).

Table 4.12 – Combinations of uncertainties studied for the sensitivity analysis using SHB and AIJ models for case study 1 (FEUP)

Combination	Model error		Soil variability		Actions' uncertainties	
	Tip	Side	Tip	Side	Permanent	Variable
1	✓	✓	✓	✓	✓	✓
2	×	×	✓	✓	✓	✓
2*	×	✓	✓	✓	✓	✓
2**	✓	×	✓	✓	✓	✓
3	✓	✓	×	×	✓	✓
3*	✓	✓	×	✓	✓	✓
3**	✓	✓	✓	×	✓	✓
4	×	✓	×	✓	✓	✓
5	✓	×	✓	×	✓	✓
6	✓	✓	✓	✓	×	×

✓ means that uncertainty was considered

× means that uncertainty was NOT considered

Table 4.13 – Combinations of uncertainties studied for the sensitivity analysis using FRc and FRp models for case study 1 (FEUP)

Combination	Model error	Soil variability		Actions' uncertainties	
	Total	Tip	Side	Permanent	Variable
1	✓	✓	✓	✓	✓
2	×	✓	✓	✓	✓
3	✓	×	×	✓	✓
3*	✓	×	✓	✓	✓
3**	✓	✓	×	✓	✓
6	✓	✓	✓	×	×

✓ means that uncertainty was considered

× means that uncertainty was NOT considered

In review, the combinations studied using RBD approach and MCS are:

(1) calculation considering all uncertainties;

(2) calculations considering all uncertainties except model error;

(2*) all uncertainties except tip component of model error;

(2**) all uncertainties except side component of model error;

(3) calculations considering all uncertainties except soil variability;

(3*) all uncertainties except tip component of soil variability;

(3**) all uncertainties except side component of soil variability;

(4) calculations considering all uncertainties except tip component uncertainties;

– all uncertainties except tip component of model error (2*);

– all uncertainties except tip component of soil variability (3*);

(5) calculations considering all uncertainties except side component uncertainties;

– all uncertainties except side component of model error (2**);

– all uncertainties except side component of soil variability (3**);

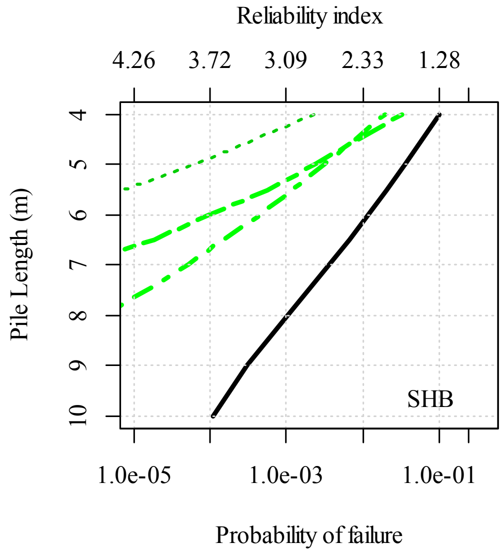
(6) calculations considering all uncertainties except actions uncertainties.

The results are presented in the following figures for each analysis. Also, because of the logarithmic scale adopted for the probability of failure representation, the same results are presented in a real/linear scale of the reliability index value. This is to avoid distorted evaluations in logarithmic scale. The following results are depicted:

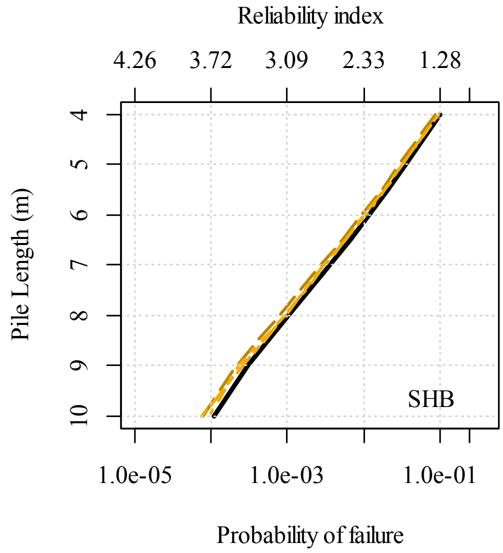
- Figure 4.25 (log scale) and Figure 4.26 (linear scale), present the results using SPT-based SHB model for the RBD sensitivity analysis;
- Figure 4.28 (log scale) and Figure 4.29 (linear scale) depict the results using SPT-based AIJ model for the RBD sensitivity analysis;
- Figure 4.31 (CPT-based model FRc) and Figure 4.32 (PMT-based model FRp) show the results for both scales of the RBD sensitivity analysis;
- Figure 4.27 (SHB model), Figure 4.30 (AIJ model), Figure 4.33 (FRc model) and Figure 4.34 (FRp model) present the relative influence of each uncertainty studied;
- finally, Figure 4.35 depicts an overall graph of the sensitivity results.

The main outcomes of these RBD sensitivity analyses show that soil uncertainties do not exhibit an importance as great as was expected. However, model uncertainties contributed greatly to the probability of failure for all models studied. Meanwhile, the contribution of tip and side uncertainties depends on the type of pile and the ratio between these two resistances.

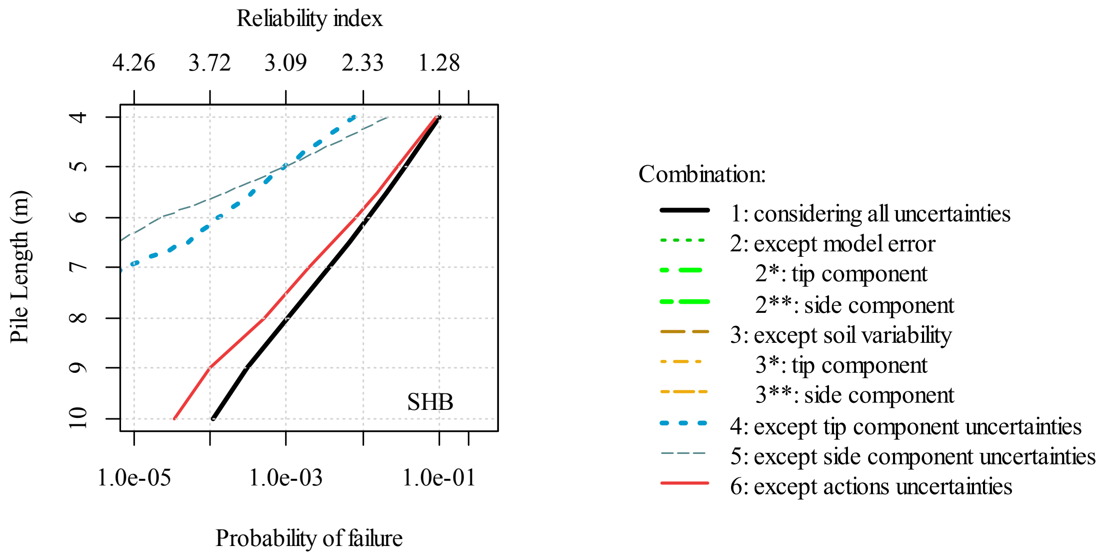
Next section will present a more detailed discussion about these results.



a) Combinations 1, 2, 2* and 2**

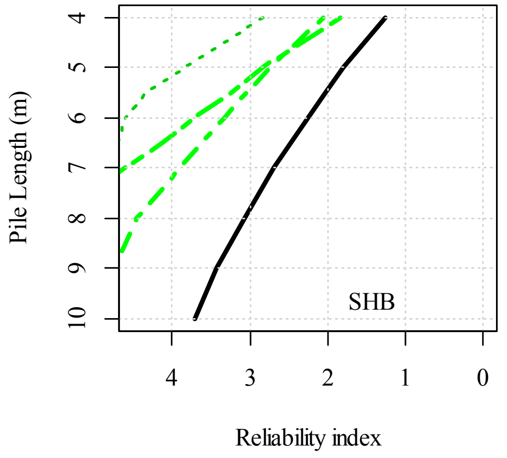


b) Combinations 1, 3, 3* and 3**

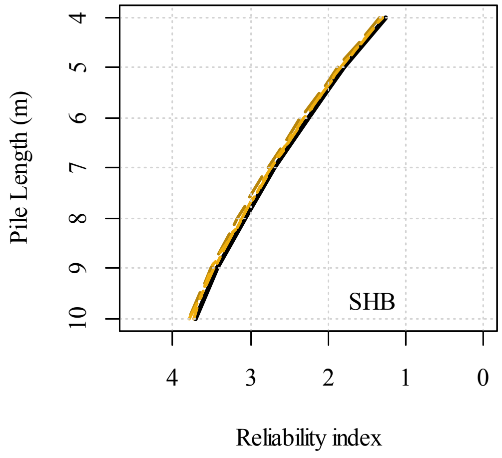


c) Combinations 1, 4, 5 and 6

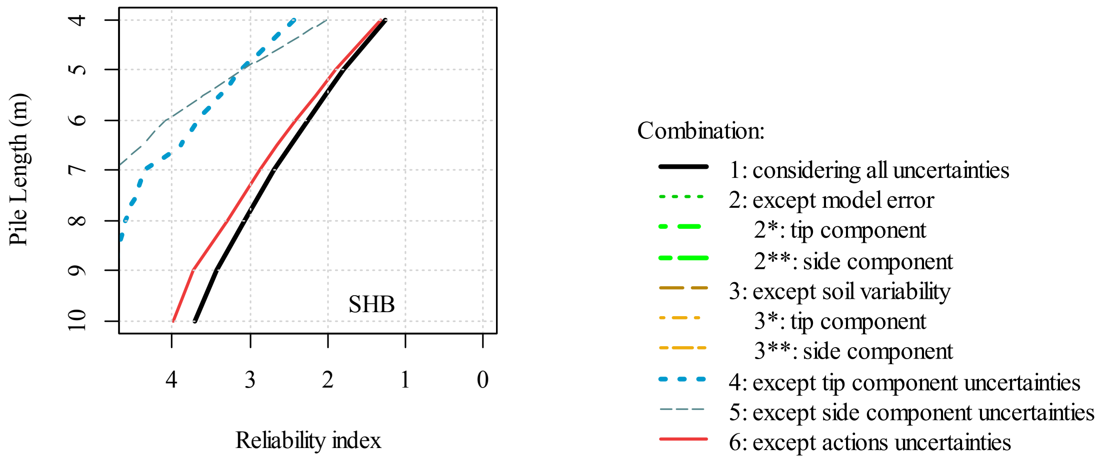
Figure 4.25 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS and SPT-based SHB model for predicting bearing capacity for case study 1 (FEUP)



a) Combinations 1, 2, 2* and 2**

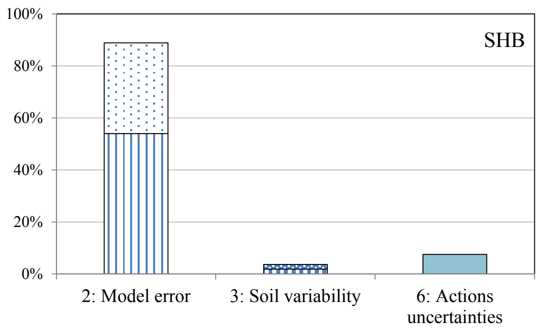


b) Combinations 1, 3, 3* and 3**

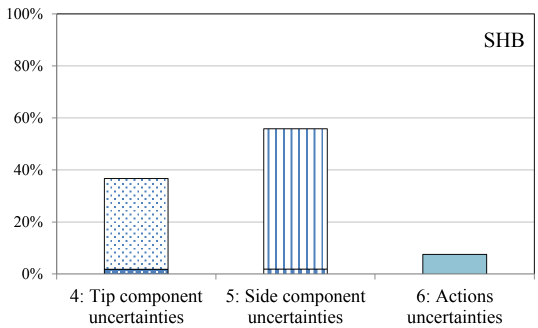


c) Combinations 1, 4, 5 and 6

Figure 4.26 – Reliability-based sensitivity analysis results using (linear scale) MCS and SPT-based SHB model for predicting bearing capacity for case study 1 (FEUP)

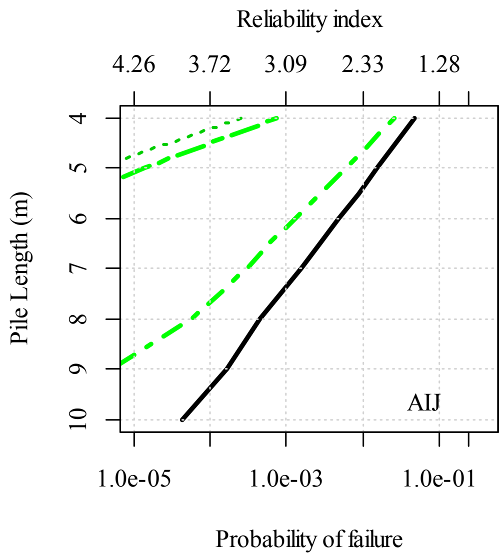


a) Combinations 2,3,6

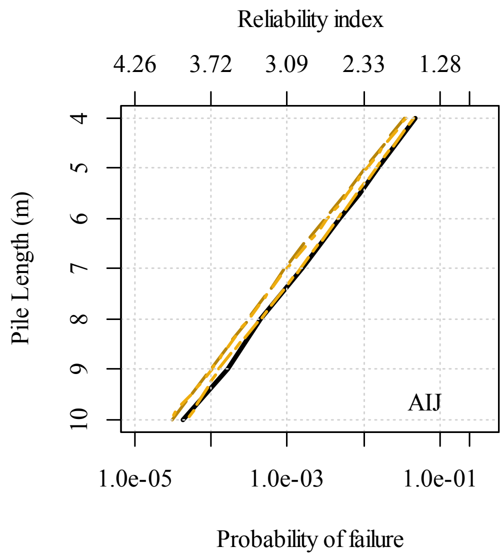


a) Combinations 4,5,6

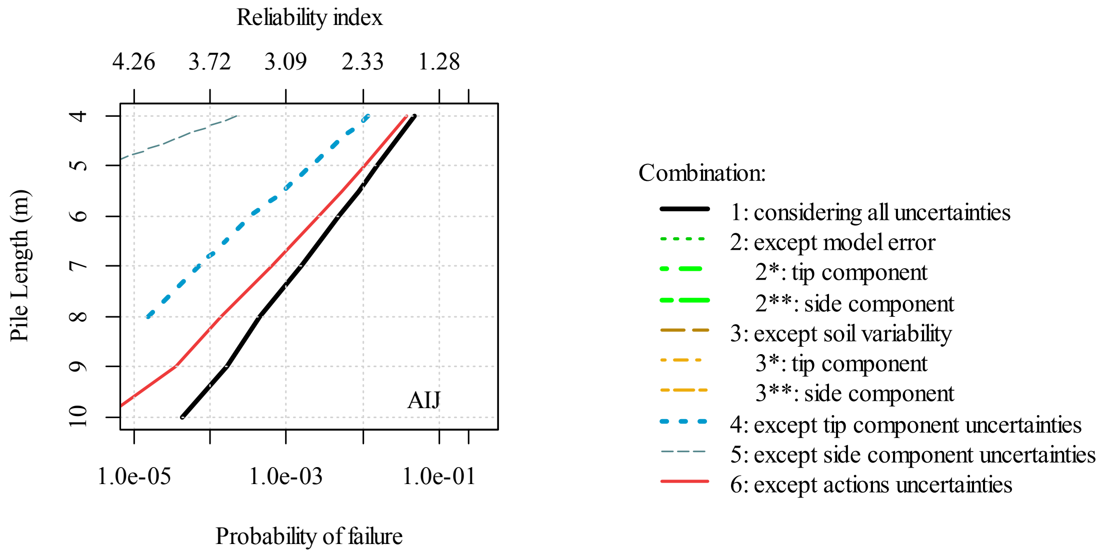
Figure 4.27 – Relative influence results from reliability-based sensitivity analyses, using MCS and SPT-based SHB model for predicting bearing capacity for case study 1 (FEUP)



a) Combinations 1, 2, 2* and 2**

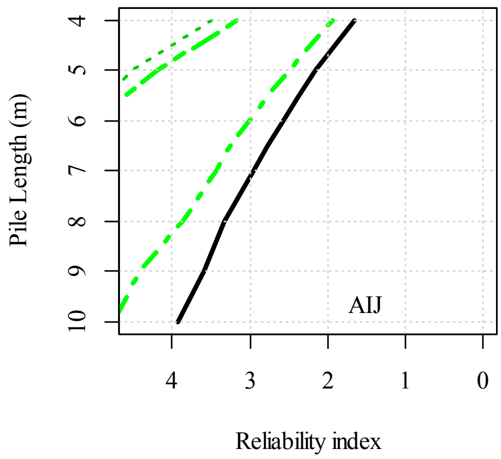


b) Combinations 1, 3, 3* and 3**

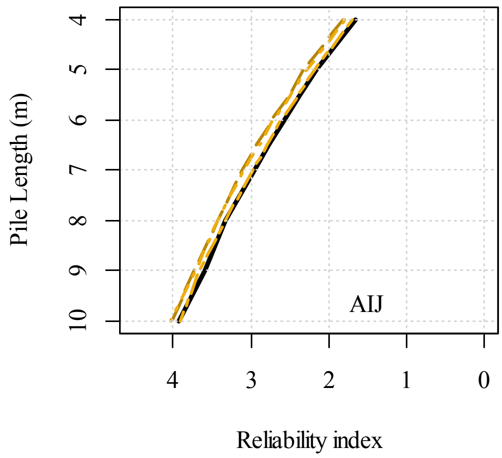


c) Combinations 1, 4, 5 and 6

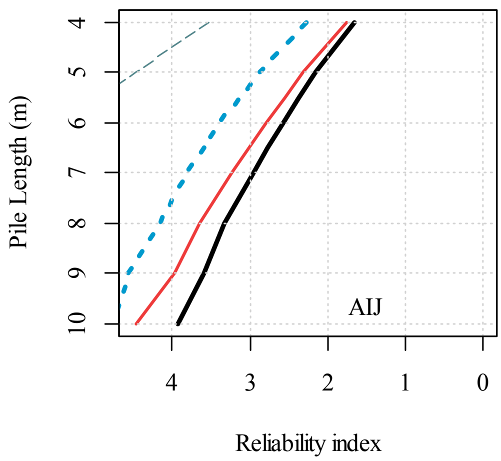
Figure 4.28 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS and SPT-based AIJ model for predicting bearing capacity for case study 1 (FEUP)



a) Combinations 1, 2, 2* and 2**



b) Combinations 1, 3, 3* and 3**



c) Combinations 1, 4, 5 and 6

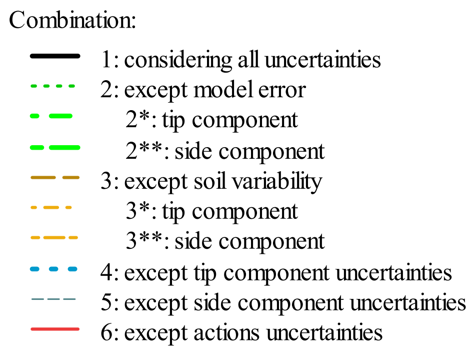
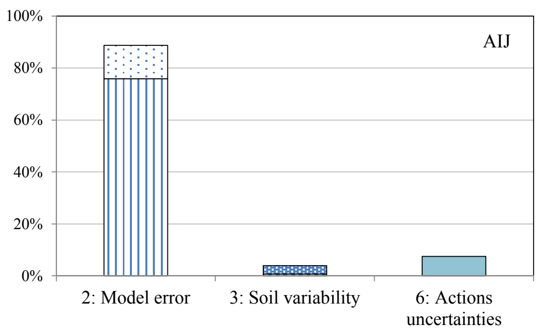
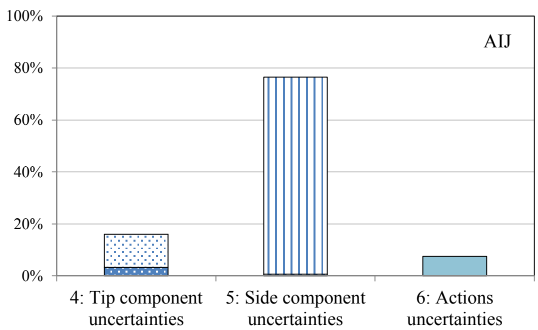


Figure 4.29 – Reliability-based sensitivity analysis results (linear scale) using MCS and SPT-based AIJ model for predicting bearing capacity for case study 1 (FEUP)



a) Combinations 2,3,6



a) Combinations 4,5,6

Figure 4.30 – Relative influence results from reliability-based sensitivity analyses, using MCS and SPT-based AIJ model for predicting bearing capacity for case study 1 (FEUP)

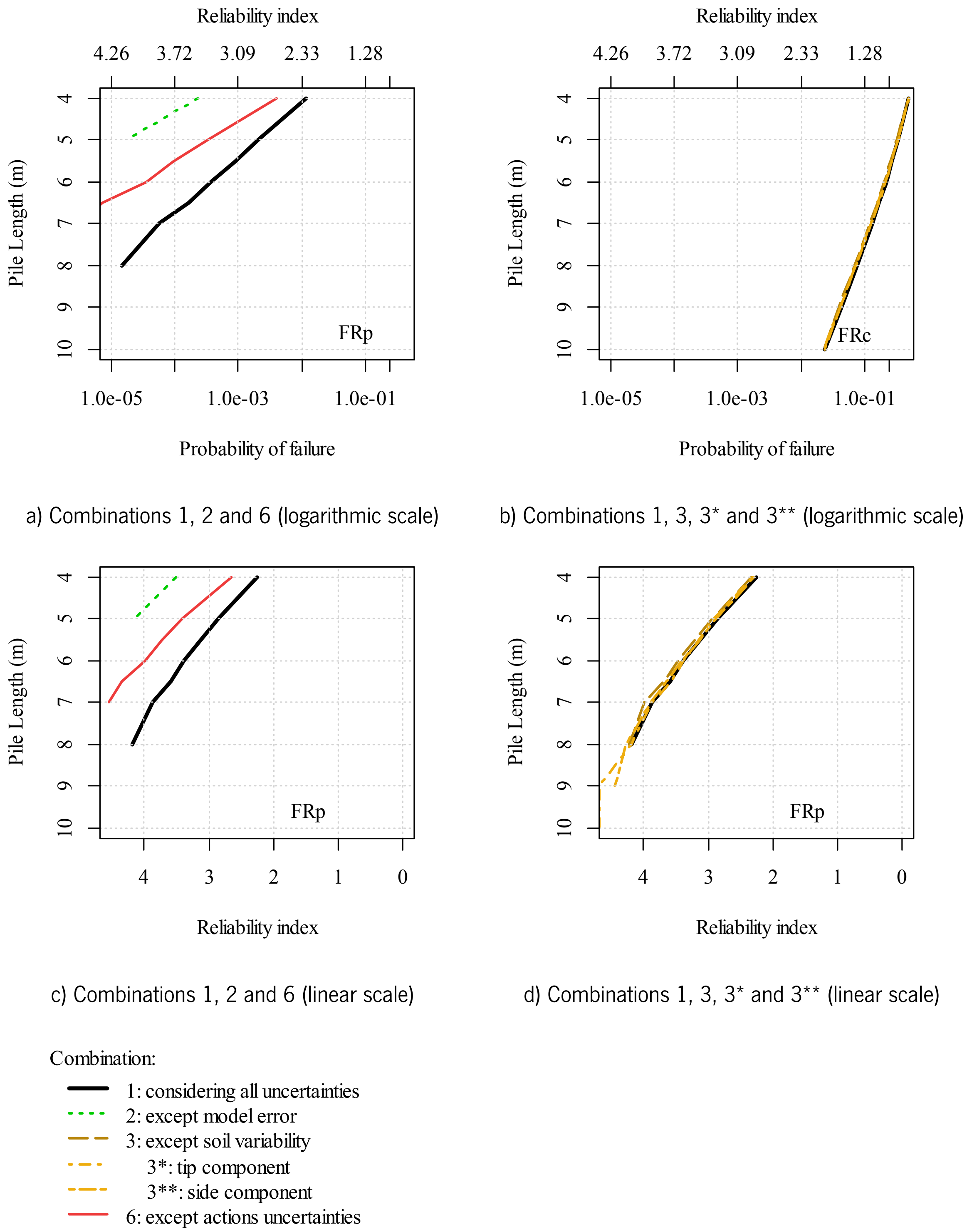
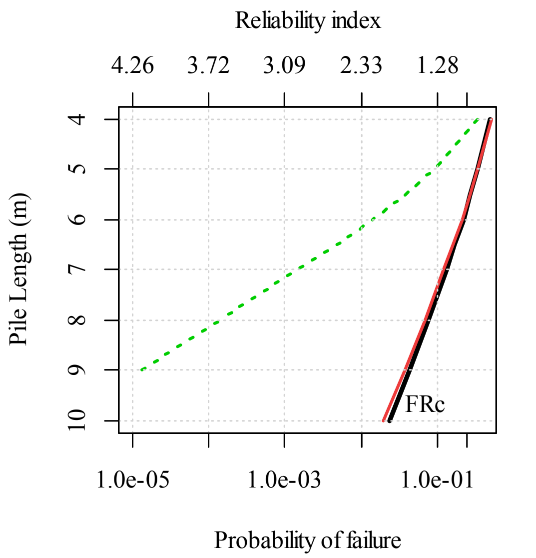
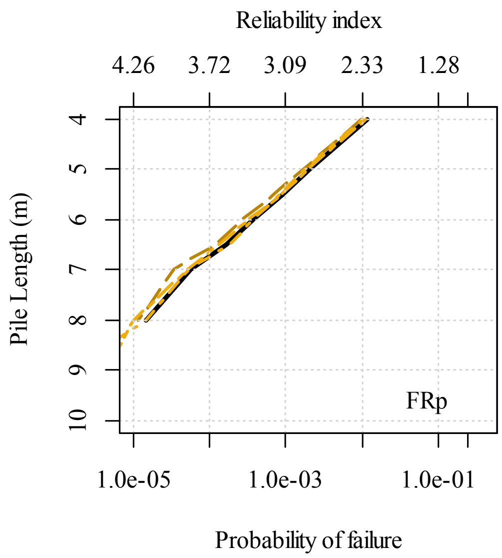


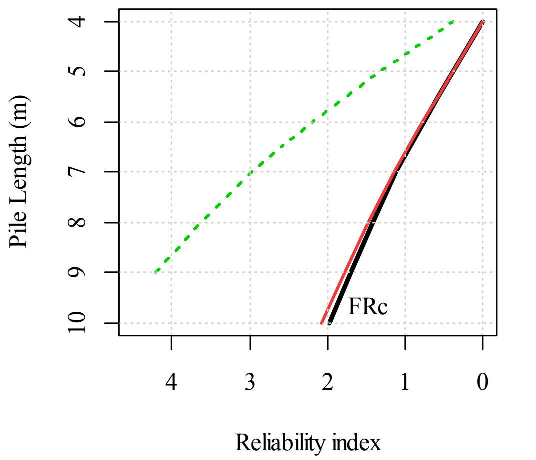
Figure 4.31 – Reliability-based sensitivity analysis results using MCS and CPT-based FRc model for predicting bearing capacity for case study 1 (FEUP)



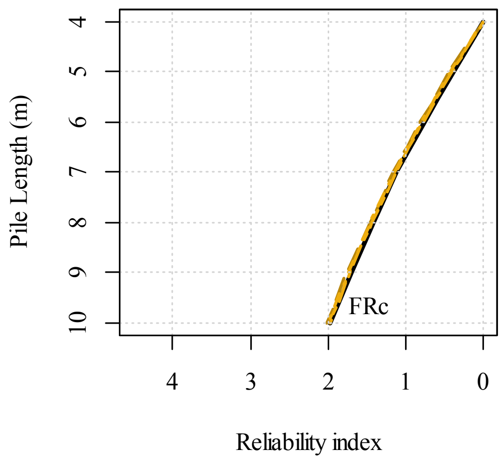
a) Combinations 1, 2 and 6 (logarithmic scale)



b) Combinations 1, 3, 3* and 3** (logarithmic scale)



c) Combinations 1, 2 and 6 (linear scale)



d) Combinations 1, 3, 3* and 3** (linear scale)

Combination:
1: considering all uncertainties
2: except model error
3: except soil variability
3*: tip component
3**: side component
6: except actions uncertainties

Figure 4.32 – Reliability-based sensitivity analysis results using MCS and PMT-based FRp model for predicting bearing capacity for case study 1 (FEUP)

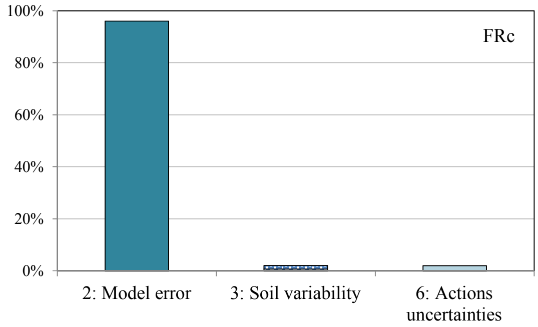


Figure 4.33 – Relative influence results from reliability-based sensitivity analyses, using MCS and CPT-based FRc model for predicting bearing capacity for case study 1 (FEUP)

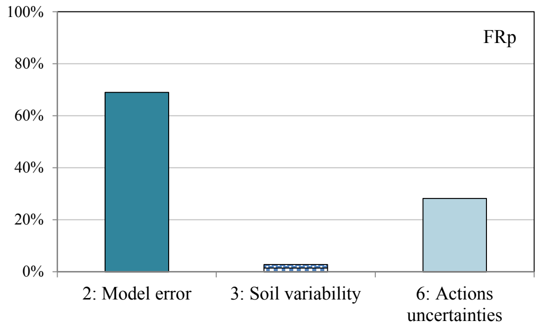


Figure 4.34 – Relative influence results from reliability-based sensitivity analyses, using MCS and PMT-based FRp model for predicting bearing capacity for case study 1 (FEUP)

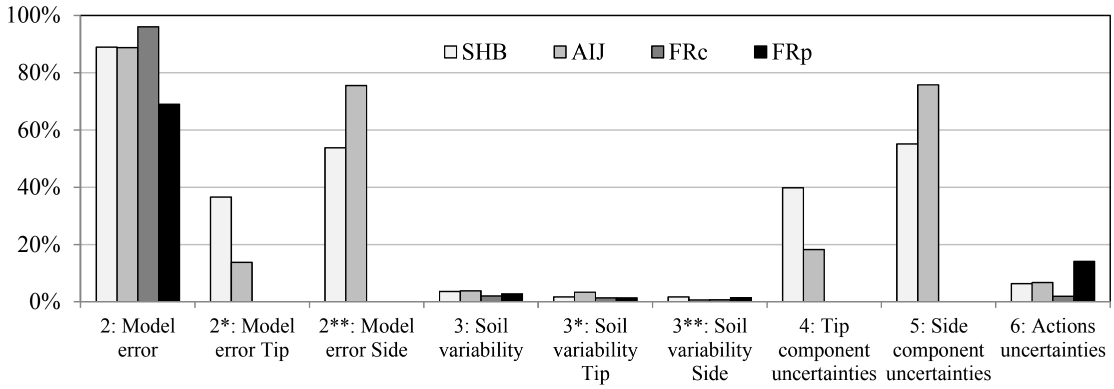


Figure 4.35 – Relative influence results from reliability-based sensitivity analyses, using MCS and different models for predicting bearing capacity for case study 1 (FEUP)

4.6 CONCLUDING REMARKS

This chapter presented different applications of the previously described reliability-based methodologies to a case study. This case study pertains to an experimental site in the north of Portugal. Here, data is available from different pile foundations types, 2 driven piles, 10 bored piles and 2 CFA (Continuous Flight Auger) piles, and a significant number of tests for soil characterisation and also pile load tests. Residual soil from granite is found on at this site. The bedrock is found at approximately 20 m and GWL at approximately 10 m. The pile considered for this study is a bored pile, with 0.6 m of diameter and 6 m of length, with ultimate bearing capacity of 1350 kN by static load test to failure.

The first section of this chapter presents the characterisation of the case study entitled "Case study 1 (FEUP)". Then, the most used reliability methods, FORM and MCS, are compared. The results of FORM and MCS are very close to each other (β_{FORM} = 2.31, β_{MCS} = 2.24). For a more extensive comparison, the application of these using RBD and Safety evaluation approaches was done (study of different pile lengths and different values of the total load, respectively).

MCS was performed in all calculations without any variance reduction technique, since the computational effort is not extensive. A MCS for one case (assumed length, diameter and total load) takes few minutes if n=1,000,000, and only few seconds if n=200,000 (value for which the case study 1 achieved stability of the results). Therefore, a full RBD or Safety evaluation analysis would take around 5 min (considering 10 cases each and stability at n=200,000).

Concerning the results of these two approaches, they verified a good agreement between the MCS and FORM results. A summary of the results obtained for the two different approaches (RBD and Safety evaluation) is presented in Table 4.14, where SPT and an empirical method were used to perform the analyses.

Table 4.14 – Results of the RBD and Safety evaluation for case study 1 (FEUP)

Case study 1	Obtained reliability	Fall inside recommended interval?	RBD, value of the length of the pile (m)		Safety evaluation, allowable total load (kN)	
			β =2.5	β =4.0	β =2.5	β =4.0
Pile installed with 6 m, total load of 800 kN	2.2	No	6.5	10.4	750	400

In order to have a more realistic value of the probability of failure for comparison of the following analyses, MCS were performed with the values of actions (800 kN) and the value of the load test value (1350 kN). Uncertainties for actions are the same and a COV of 5, 10 and 20% was assumed for the load test. Usually, load tests have low COV (Baecher & Christian, 2003; Phoon, 2008), unless one is evaluating a capacity of a similar pile installed some meters apart from the one that was actually tested. Results are presented in Table 4.15 and the COV=10% should be the most realistic. The achieved result is within the recommendations of the Eurocode, which agrees with the calculations, since the load was determined based on Eurocodes' SF.

Table 4.15 – Results of MCS using load test result, pile installed with 6 m, total load of 800 kN, case study 1 (FEUP)

	COV of load test (%)		
	5	10	20
pf	$2.4\text{x}10^{-5}$	$7.3\text{x}10^{-5}$	$1.7\text{x}10^{-3}$
β	4.1	3.8	2.9

Also, all results (RBD approach and Safety evaluation approach) exhibited a visible relationship between the probability of failure (on a log scale) and the length of the pile (on a linear scale), and also between the probability of failure (on a log scale) and the total load on the pile (on a linear scale).

Concerning the reliability-based safety factors (resistance and loads multiplying safety factors) it was found, as expected, that they reflect directly the importance of the uncertainties. Furthermore, the characteristic values have the same importance as the partial safety factors in reliability analyses, and a bad choice of the characteristic values could lead to a behaviour of the structure far from the reality. The results of this case study allow the assessment of the safety factors for a particular case, so, in order to use such values in design codes/standards or recommendations, they have to be calibrated with adequate number of representative cases, namely different types of soils and different types of piles.

This chapter also presented different analyses concerning the sensitivity of the probability of failure (or reliability) to the variance in some parameters (section 4.4 and 4.5). Based on those analyses results using MCS, when comparing the various combinations the following was observed for all resistance models considered (SHB, AIJ, FRc and FRp):

- Combination 1 vs. combination 2 and combination 3 (model error and soil variability), it was concluded that with no doubt, for this case, the model error is the most important uncertainty. Even though soil variability is always described as being very important, that was not true in this case, as evidenced by the combination 3 results being very similar to the combination 1 results.

- Combination 1 vs. combination 4 and combination 5 (tip and side component uncertainties), the results of combination 4 are similar to combination 5 and they are very similar to the combination 1 results. Yet, This comparison yields information on the influence of the tip and side components. Even if side resistance has a high magnitude, tip component has a higher uncertainty, yielding similar influences of the final results.
- Combination 1 and combination 6 results are very similar, accommodating the low influence of the actions uncertainties in this case study.
- The influence of each uncertainty was the same by each method (FORM and MCS). The results of the RBD sensitivity analyses using MCS agree and support the results obtained for the sensitivity factors of the FORM analyses. For both it is noticeable that model error is the most important uncertainty considered.

When analysing the results of the soil variability's sensitivity analysis (section 4.4), the following points can be highlighted:

- The soil COV of 10% or 20% presented a low influence in the probability of failure. It was proven that the considered soil variability assessed based on the *in situ* soil tests (SPT, CPT and PMT) falls within this range. However, COV of 50%, a very high and not common COV, present a higher variance in the reliability of the pile.
- The soil autocorrelation presented a very low influence in the probability of failure. The spatial autocorrelation allows for the reduction of the standard deviation of the soil variability. It is obvious that this will lead to a more reliable result and when this reduction is not considered, the result is more conservative, but certainly not correct, especially in terms of economy.
- Furthermore, it is also observed that, not only the uncertainties have influence in the sensitivity of the results but also the level of reliability in which the problem is, influences this sensitivity.

Finally, this case study, and all the results obtained using different reliability-based methodologies, return and emphasise the need for well defined and characterised resistance models, the need for accurate knowledge of the model error and also how important the load tests can be to achieve more reliable results and economic designs

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Chapter 5

APPLICATION OF RELIABILITY TO A BRIDGE PILE FOUNDATION

5.1 INTRODUCTION

This chapter presents the application of reliability to case study 2, a steel pipe pile foundation that belongs to a bridge project. Different reliability-based methodologies are applied to this second case study, identical to the previous chapter. This case study is distinct from the case study 1, since it is not from an experimental site, it is a different category of pile (bored vs. driven) and it has a different type of load test associated (static vs. dynamic).

A comparison between methodologies and methods is presented after an extensive reliability-based sensitivity analysis of the different uncertainties considered. Different load combinations values and uncertainties sources are studied, however with slight higher emphasis to the influence of the actions uncertainties. In this second case study only SPT (Standard Penetration Tests) were performed. Therefore, all the analyses presented are based on it and use SHB empirical model (SPT-based resistance model for pile bearing capacity prediction – Annex H).

5.2 CASE STUDY 2 – BRIDGE

The case study 2 is referred to a railway bridge in southern Portugal. This new segment connects the railway to the Pinheiro station (Estação do Pinheiro) over the Sado River (Figure 5.1). The pile under study is part of the set of individual piles designed to withstand the loads transmitted to the temporary towers during the movement and elevation process of the bridge deck/girder and the bridge arches (construction phase).

5.2.1 Soil characterisation

The soil for each pile foundation of this bridge is different, particularity along the riverbed and on the riverside. The pile under study is installed in riverbed; therefore, the water line (WL considered z=0m) is above the soil surface (Figure 5.2).

The soil around it consists of an upper layer of mud (soft and dark grey) over a layer of dense to medium-dense slightly clayey sand, a layer of medium to coarse sand and, finally, at a depth of approximately 35 to 40 m, a layer of very dense carbonate sand and marl. The results from the soil investigation by SPT and the geological profile are depicted in Figure 5.2 (GeoDrive Technology, 2008). Notice that the profile on the right side is the one that corresponds to the specific place where the pile under study was installed.

5.2.2 Pile information

The pile considered is an axial open-ended steel pipe pile, denoted PPR1-B, with the characteristics shown in Table 5.1. A dynamic load test was carried out to determine the axial compression bearing capacity. Re-drives were done to verify soil setup with time. The pile was tested and driven to refusal with dynamic load testing. The magnitudes of static and dynamic components were determined by signal matching. A stress wave model of the pile was made and the soil is modelled per layer. These results are presented in Annex O. The dynamic analysis indicates a total bearing capacity of approximately 4000 kN (GeoDrive Technology, 2008).

Table 5.1 – Information of the open-ended pipe pile PPR1-B (steel), case study 2 (bridge)

Type	Section (m)	Length (m)	Embedded length (m)
Compression Friction pile	Circular, pipe 1.12m, 12.4mm*	43.5	33.5

* Thickness of the pipe

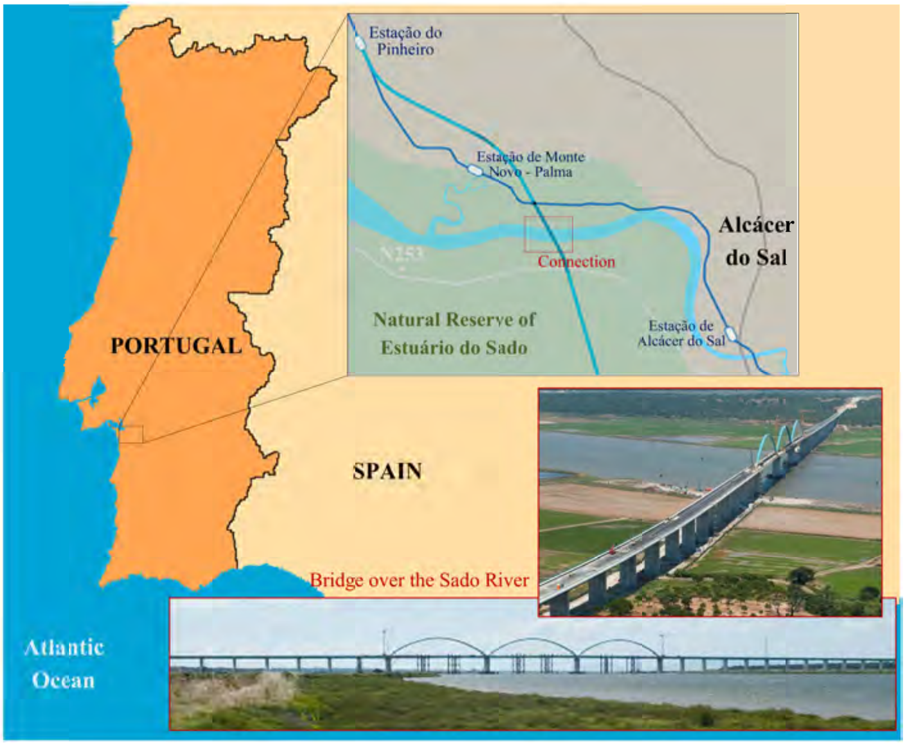


Figure 5.1 – Location of case study 2 (bridge)

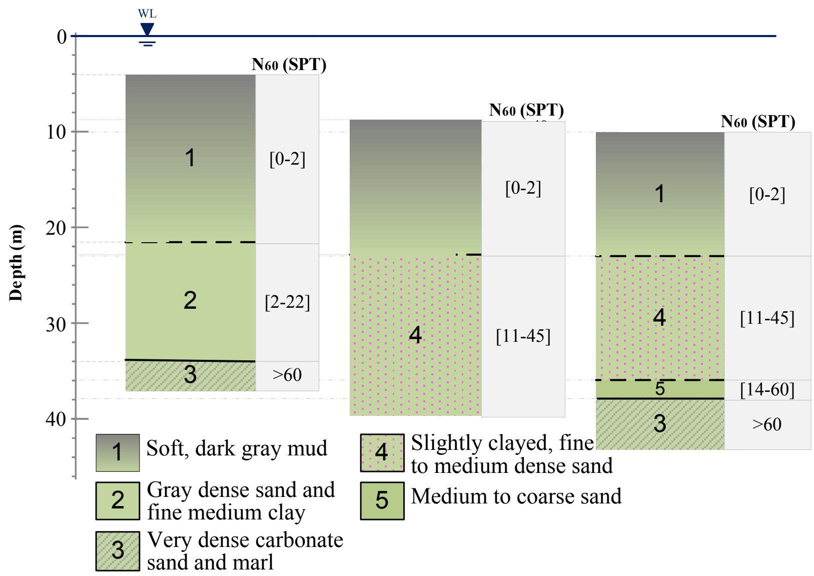


Figure 5.2 – Geological profile and SPT results from case study 2 (bridge), adapted from GeoDrive Technology Report (2008)

5.2.3 Actions

For this case study the values of actions (load combinations) used for design are numerous, which is expected for a project such as this. Therefore, only the most important combinations were adopted and used for the case study 2 computations. These combinations are presented in Table 5.2.

Table 5.2 – Actions and load combinations values for case study 2 (bridge)

Combinations	Actions (kN)		TOTAL (kN)
	Permanent *Load* (G_k)	Variable *Load* (Q_k)	($G_k + Q_k$)
LC-1*	6100 / 1.35▲ = 4520	400 / 1.50▲= 267	4787 (approx. 95%+5%)
LC-2**	2400 / 1.35▲ = 1778	4100 / 1.50▲ = 2733	4511 (approx. 40%+60%)

* Launching of the bridge deck/girder
** Wind combination
▲ Actions' safety factors (CEN, 2002b)

For load combinations of actions LC-1 and LC-2 the design values were reduced dividing them by the respective SF. To evaluate the SHB model prediction, the result was compared with the load test (Table 5.3 and Figure 5.3). For this, the permanent and variable loads were considered equal in magnitude and the partial SF from Eurocodes (CEN, 2002a,b; CEN, 2007) were applied for the estimation of the actions using eq.(4.1), alike for case study 1.

Table 5.3 – Prediction of the vertical bearing capacity and estimation of the design load based on different methods considering G_k=Q_k, case study 2 (bridge)

Method	Based on:	Bearing capacity (kN)	*Load** (kN)
Static load test	-	4000	1220
SHB	SPT	10076	3074

* eq.(4.1)

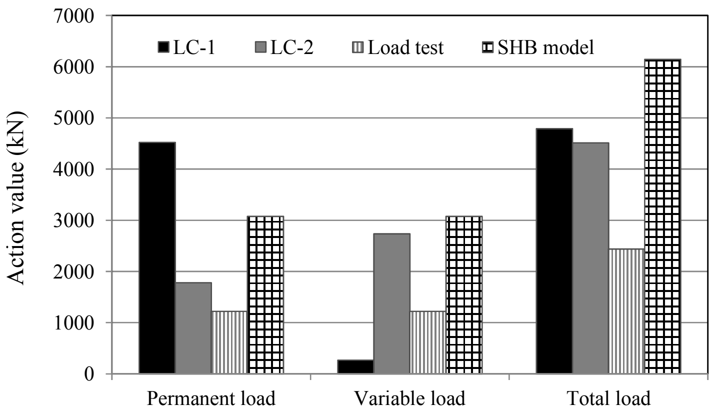


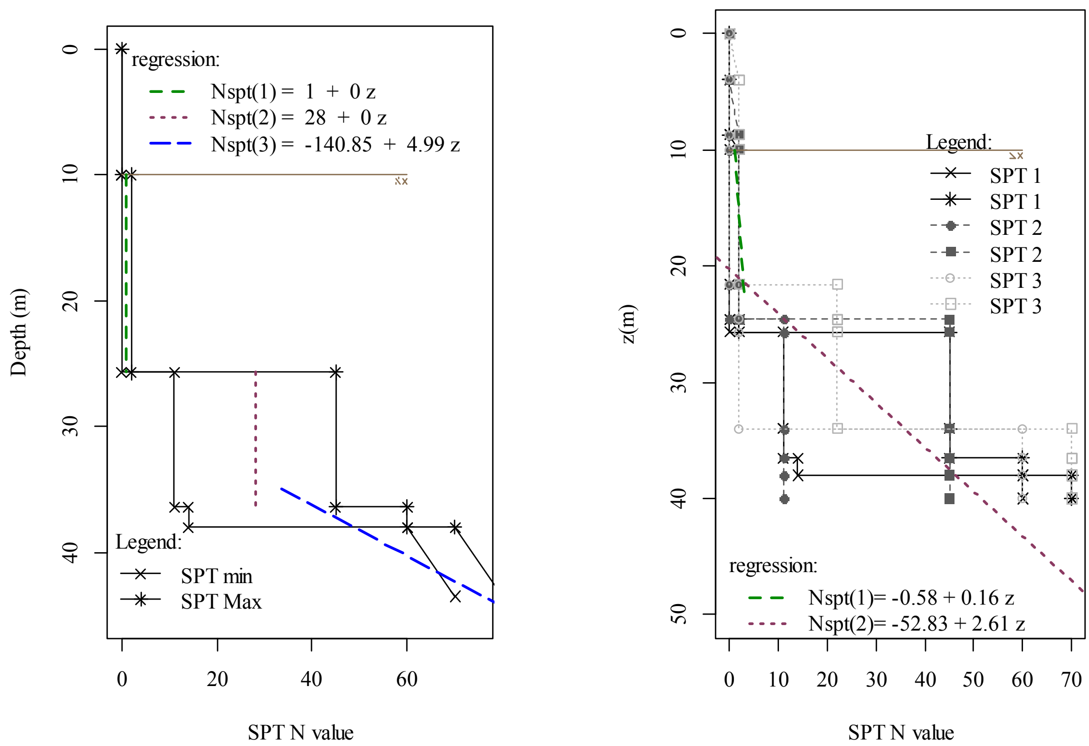
Figure 5.3 – Comparison between loads of case study 2 (bridge)

The values adopted for further calculations are the ones presented in Table 5.2 and denoted load combinations LC-1 and LC-2.

5.2.4 Uncertainties to consider

As previously stated, in pile foundation's reliability analyses, with performance function like shown in eq.(3.2) (Chapter 3), a total of six uncertainty sources, from three distinct types are considered: (1) the model error, (2) the soil variability and (3) the actions' uncertainties. Note that once again the pile dimensions, such as length and diameter, were considered as deterministic values.

The model error (characterised by its bias and standard deviation) is summarised in Table 5.4 (refer to Table 3.3). The soil variability is considered in the variance of the test parameter (SPT N value). However, for case study 2 the SPT profile shown in Figure 5.2 (the one on the right side, starting at 10 m depth) corresponds to the specific place where the pile under study was installed (Figure 5.4.a). Therefore this profile should be the one adopted. If one was going to use the three profiles to determine soil variability (Figure 5.4.b), the results should present a higher variability when compared with the variability to be assumed using only the local SPT profile.



a) Variability from 1 SPT in place

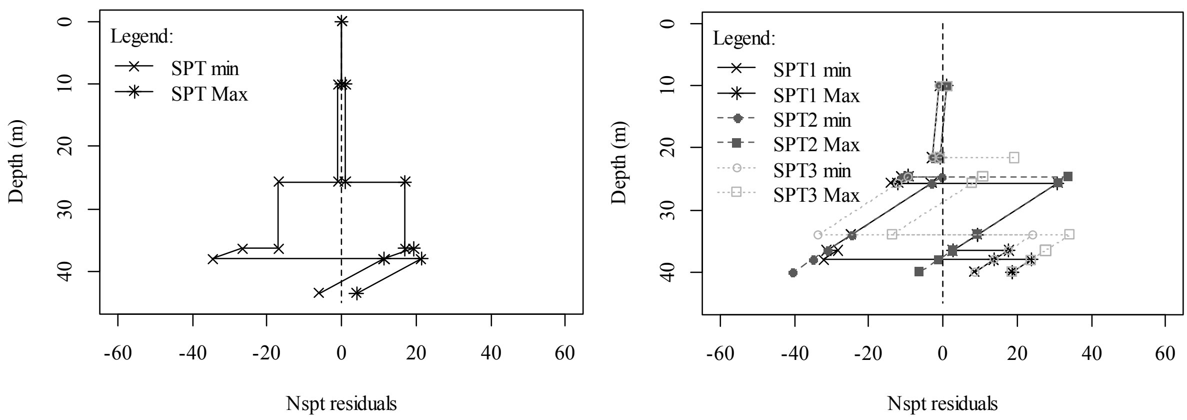
b) Variability from all SPT

Figure 5.4 – Case study 2 *in situ* tests trends

So, this case study can be presented as an example of the local estimation consideration for determining local average, where one wants to design a foundation and makes a detailed soil investigation at the specific place; in this case the uncertainties to be considered in ground conditions can be lower. Furthermore, for this reason, in case of 1 SPT, the autocorrelation does not need to be considered in the analyses, which simplifies calculations.

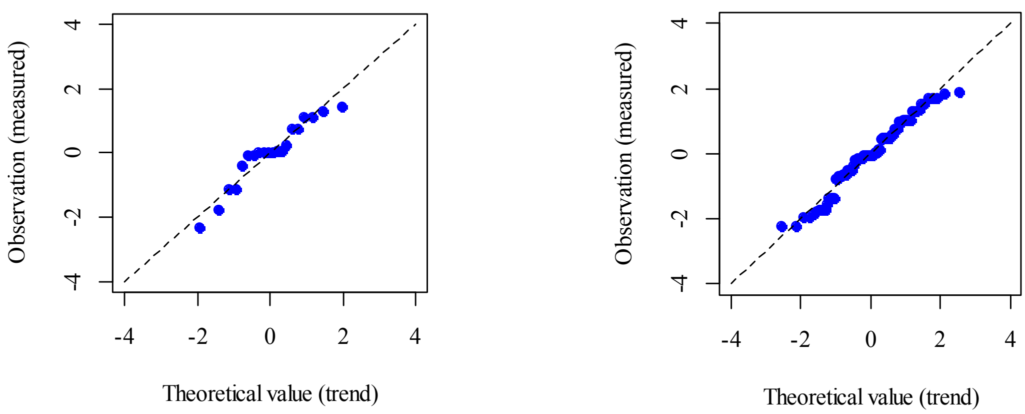
Nevertheless, these two cases will be considered in the following calculations (using only 1 SPT or all SPT). Trend residuals and its Q-Q plots are presented in Figure 5.5 and Figure 5.6. Note that the SPT N value was divided in braches for a better representation of the vertical variability of the soil (Figure 5.4):

- for 1 SPT, 3 branches are considered, [10; 26] m,]26; 36] m and > 36 m;
- for all SPT, 2 branches are considered, one above and one below the 22 m.
-



a) Variability from 1 SPT in place b) Variability from all SPT

Figure 5.5 – Case study 2 *in situ* tests residuals (measured subtracted by trend)



a) Variability from 1 SPT in place b) Variability from all SPT

Figure 5.6 – Case study 2 standard normal Q-Q plots of *in situ* tests residuals

Based on the previous considerations, the following tables exhibit the uncertainties values for case study 2's reliability computations:

- model error in Table 5.4;
- soil variability in Table 5.5;
- and actions' uncertainties in Table 5.6, as this study will highlight the actions uncertainties, two sets of uncertainties are considered for the actions uncertainties (refer to Table 3.4).

Table 5.4 – Model uncertainties based on *in situ* test SPT (Honjo *et al.*, 2002; Okahara *et al.*, 1991)

	SPT empirical method (SHB)	
	δ_t - tip	δf - side
Mean	1.12	1.07
COV (%)	63	46
Standard deviation	0.706	0.492
PDF	Lognormal	Lognormal

Table 5.5 – Characterisation of the soil uncertainties of case study 2 (bridge)

a) Variability from 1 SPT in place

	N tip	N side
Mean	(1) 1+0z ; (2) 28+0z (3) −140.9+4.99z	
Standard deviation	(1) 0.5; (2) 8.5 (3) 22.6	
PDF	Normal	

(1) branch from 10 to 26 meters
(2) branch from 26 to 36 meters
(3) branch from 36 to 60 meters

b) Variability from all SPT

	N tip	N side
Mean	(1) −0.58+0.16z (2) −52.8+2.6z	
Standard deviation	(1) 5.11 (2) 20.3	
PDF	Normal	

(1) branch from 10 to 22 meters
(2) branch from 20 to 60 meters

Table 5.6 – Actions uncertainties of case study 2 (bridge)

a) Set 1 (JCSS, 2001; Holicky *et al.*,2007)

	δ_G - permanent	δ_G – variable
Mean	1.0	0.6
COV (%)	10	35
Standard deviation	0.10	0.21
PDF	Normal	Gumbel

b) Set 2 (Ellingwood, 1996)

	δ_G - permanent	δ_G – variable
Mean	1.05	1.00
COV (%)	10	25
Standard deviation	0.105	0.25
PDF	Norma	Gumbel

5.3 SENSITIVITY ANALYSIS OF UNCERTAINTY TYPES

Since the uncertainties for this case study can have different considerations, for both actions (load combination value and uncertainty set) and soil variability (1 SPT or all SPT), the results of the different reliability-based methodologies and methods are presented and compared only after a comparison of the results of an extensive reliability-based sensitivity analysis.

These calculations include both considerations of the soil variability (using 1 SPT or all SPT, Table 5.5), both load combinations (LC-1 or LC-2, Table 5.2) and both sets of actions uncertainties (Set 1 or Set 2, Table 5.6), in order to make a comparative and critical analysis. The notation used for presenting the results of the different analyses and considerations is presented in Figure 5.7.

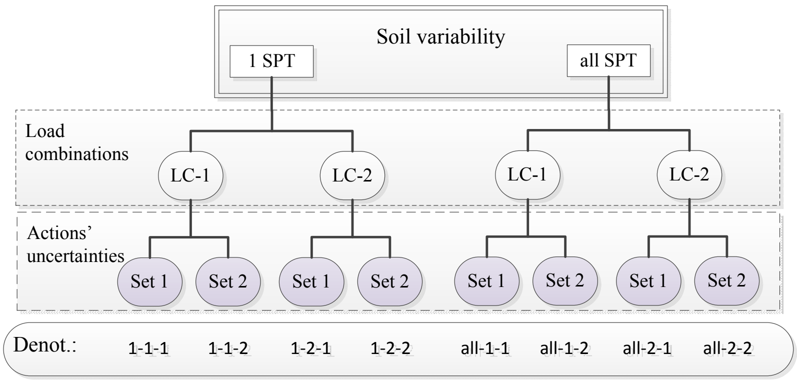


Figure 5.7 – Notation of the cases studied concerning case study 2 (bridge)

As such, this section presents the sensitivity analyses of the different uncertainties considered, one by one, for case study 2 using MCS and RBD approach (recall Chapter 3). As for the previous case study, a MCS stability study was performed (Annex P). The stability for case study 2 was achieved for n=500,000 considering the interval [2.5; 4.0] for reliability index values (discussion in section 3.3.2).

The calculation procedure is similar to a parametric study, where the calculation is repeated considering and not considering the different uncertainties and pf's sensitivity is analysed. The combinations studied are divided in groups to contemplate all sets and uncertainty types. For each one, the following combinations are considered (see also Table 5.7):

(1) calculation considering <u>all uncertainties</u> ;

(2) calculations considering all uncertainties <u>except model error</u>;

(3) calculations considering all uncertainties <u>except soil variability</u>;

(4) calculations considering all uncertainties <u>except tip component uncertainties</u>;

(5) calculations considering all uncertainties <u>except side component uncertainties</u>;

(6) calculations considering all uncertainties <u>except actions uncertainties</u>;

(6*) all uncertainties except permanent component of actions;

(6**) all uncertainties except variable component of actions.

For the study using all SPT the combination (1†) was included, that comprises the calculation considering all uncertainties except the reduction of variance based on soil autocorrelation (Vanmarcke, 1977). The results are presented in the following subsections for each analysis. Notice that the results are depicted in a log scale for the probability of failure. With case study 1's results was concluded that the graphical assessment of the sensitivity analyses outcomes do not differ greatly when comparing the pf log scale with the β real/linear scale, therefore, for case study 2 the results are only presented in logarithmic scale.

Table 5.7 – Combinations of uncertainties studied for the sensitivity analysis of case study 2 (bridge)

	Model error		Soil variability		Actions' uncertainties	
Combination	Tip	Side	Tip	Side	Permanent	Variable
1	✓	✓	✓	✓	✓	✓
1†	✓	✓	✓†	✓†	✓	✓
2	×	×	✓	✓	✓	✓
3	✓	✓	×	×	✓	✓
4	×	✓	×	✓	✓	✓
5	✓	×	✓	×	✓	✓
6	✓	✓	✓	✓	×	×
6*	✓	✓	✓	✓	×	✓
6**	✓	✓	✓	✓	✓	×

✓ means that uncertainty was considered

× means that uncertainty was NOT considered

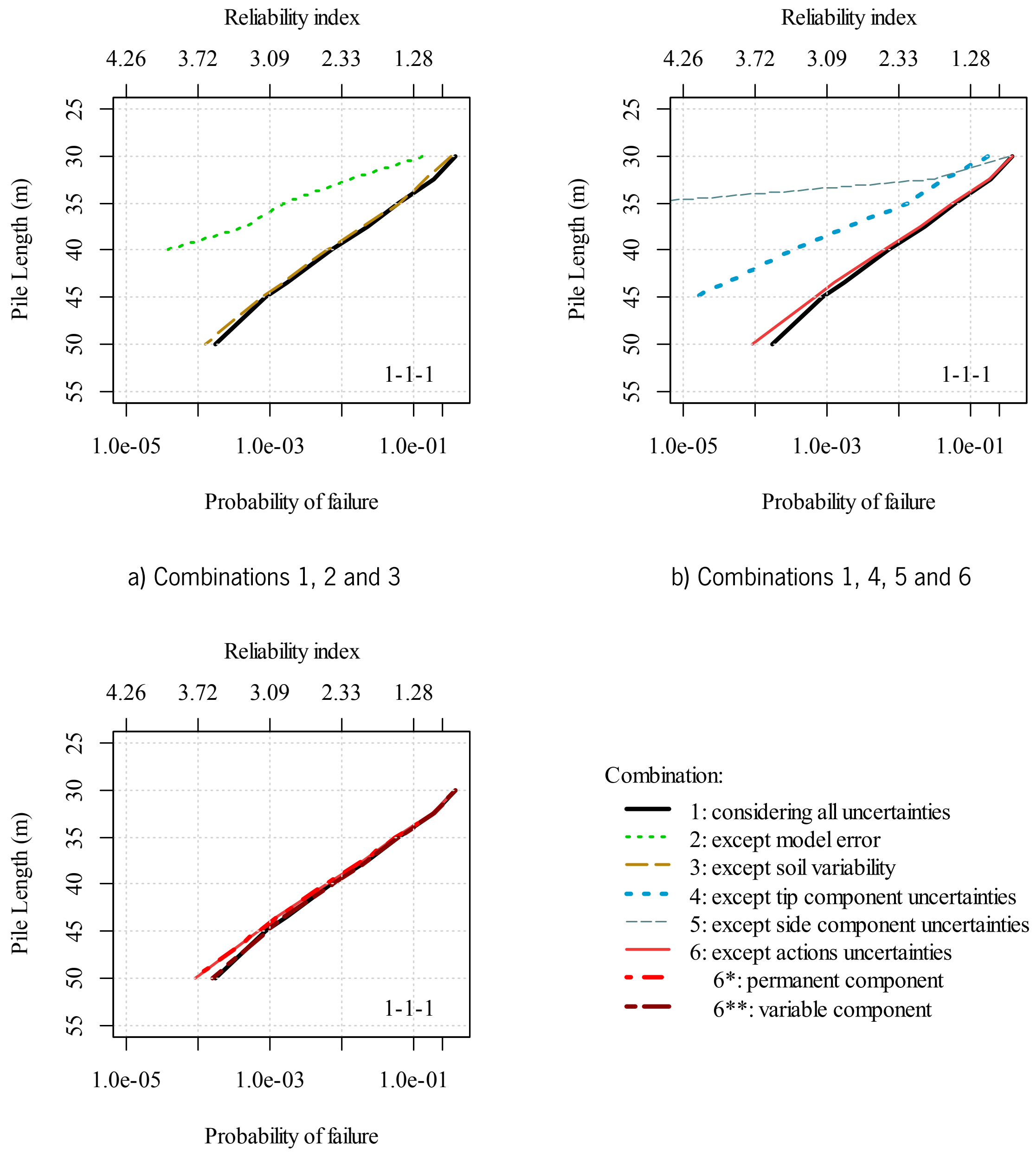
† means that reduction of the variance (spatial variability) was NOT considered (for "all SPT" cases)

5.3.1 Soil variability from 1 SPT

This subsection presents the results of the reliability-based sensitivity analyses using soil uncertainties of Table 5.5.a, obtained from the 1 SPT performed at the specific site where the pile was installed.

LC-1 with uncertainties Set 1 (1 SPT)

Figure 5.8 presents the results of load combination LC-1 using actions' uncertainties Set 1 (1-1-1).



a) Combinations 1, 2 and 3

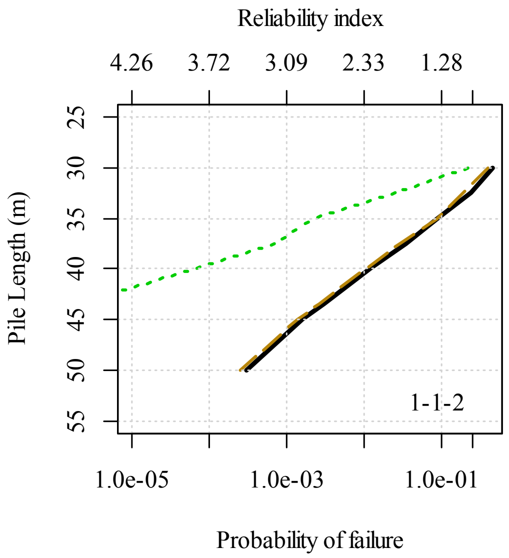
b) Combinations 1, 4, 5 and 6

c) Combinations 1, 6, 6* and 6**

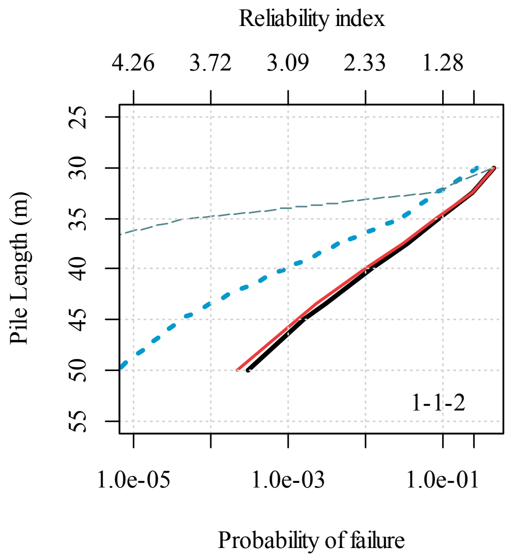
Figure 5.8 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from 1 SPT, LC-1 & Set 1 for case study 2 (bridge)

LC-1 with uncertainties Set 2 (1 SPT)

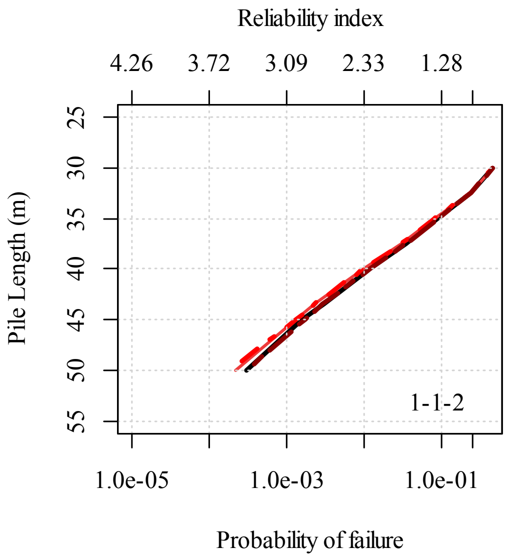
Figure 5.9 presents the results of load combination LC-1 using actions' uncertainties Set 2 (1-1-2).

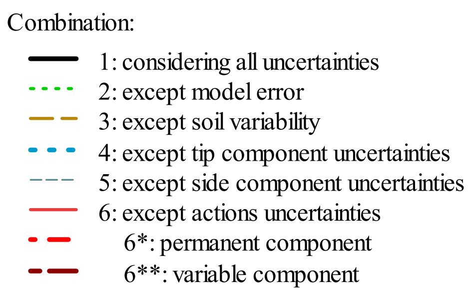


a) Combinations 1, 2 and 3



b) Combinations 1, 4, 5 and 6



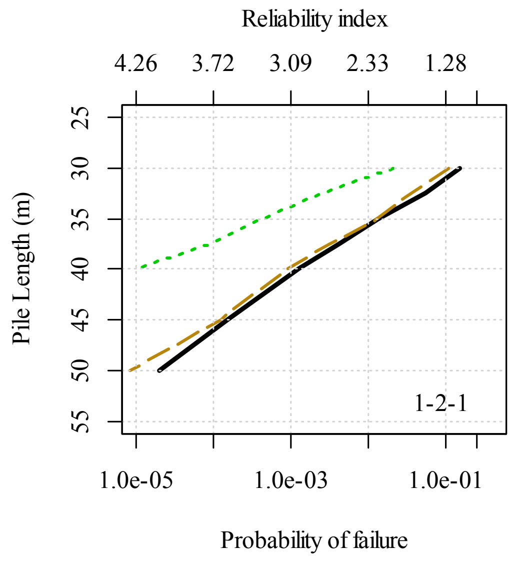


c) Combinations 1, 6, 6* and 6**

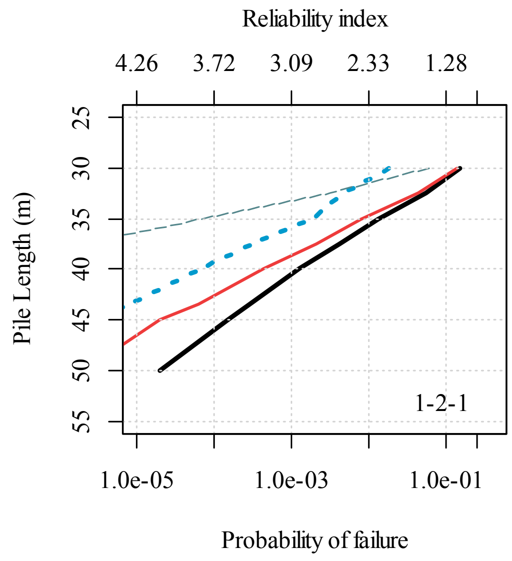
Figure 5.9 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from 1 SPT, LC-1 & Set 2 for case study 2 (bridge)

LC-2 with uncertainties Set 1 (1 SPT)

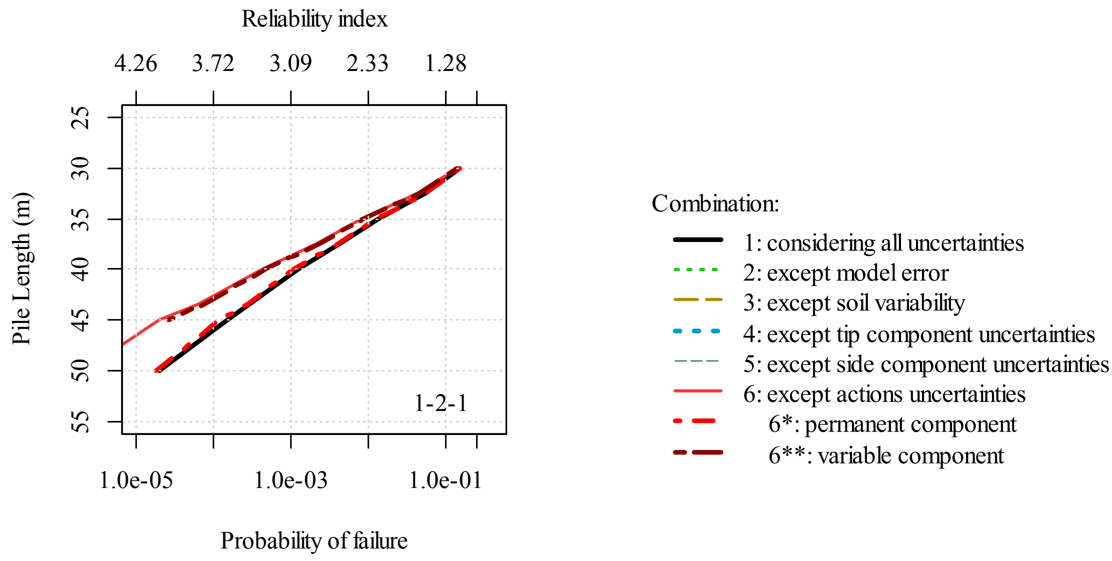
Figure 5.10 presents the results of load combination LC-2 using actions' uncertainties Set 1 (1-2-1).



a) Combinations 1, 2 and 3



b) Combinations 1, 4, 5 and 6

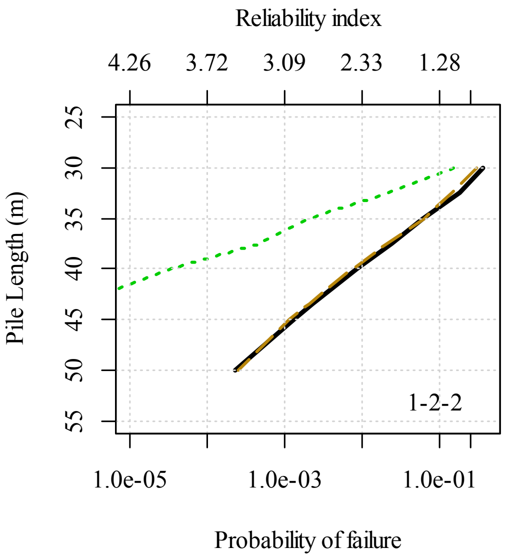


c) Combinations 1, 6, 6* and 6**

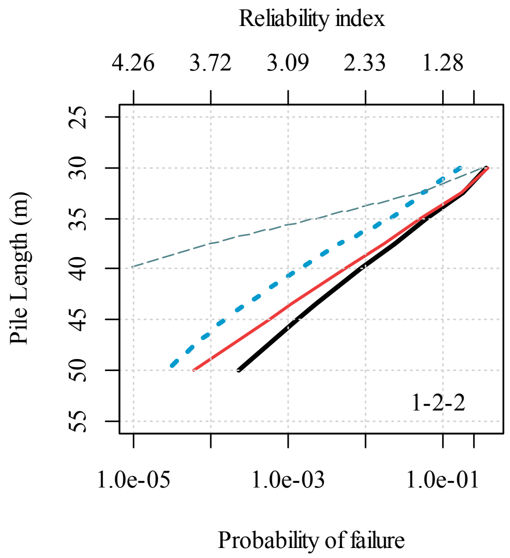
Figure 5.10 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from 1 SPT, LC-2 & Set 1 for case study 2 (bridge)

LC-2 with uncertainties Set 2 (1 SPT)

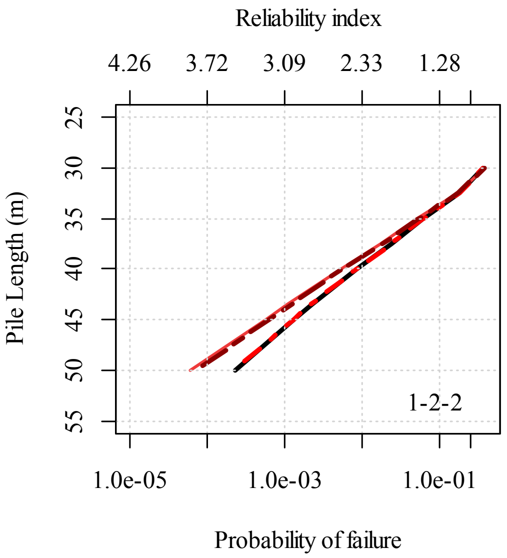
Figure 5.11 presents the results of load combination LC-2 using actions' uncertainties Set 2 (1-2-2).

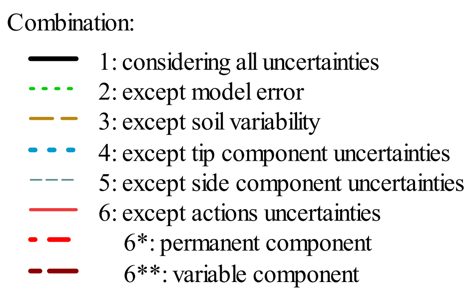


a) Combinations 1, 2 and 3



b) Combinations 1, 4, 5 and 6





c) Combinations 1, 6, 6* and 6**

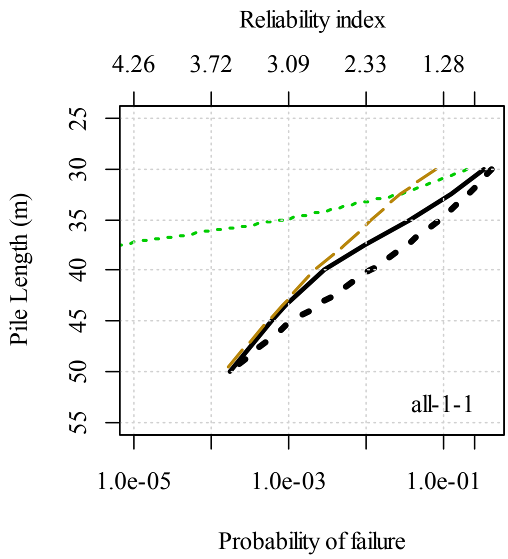
Figure 5.11 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from 1 SPT, LC-2 & Set 2 for case study 2 (bridge)

5.3.2 Soil variability from all SPT

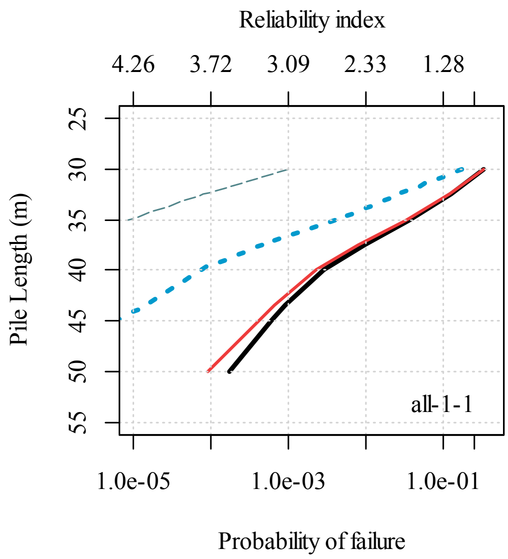
This subsection presents the results of the reliability-based sensitivity analyses using soil uncertainties in Table 5.5.b, achieved considering all SPT performed during construction phase.

LC-1 with uncertainties Set 1 (all SPT)

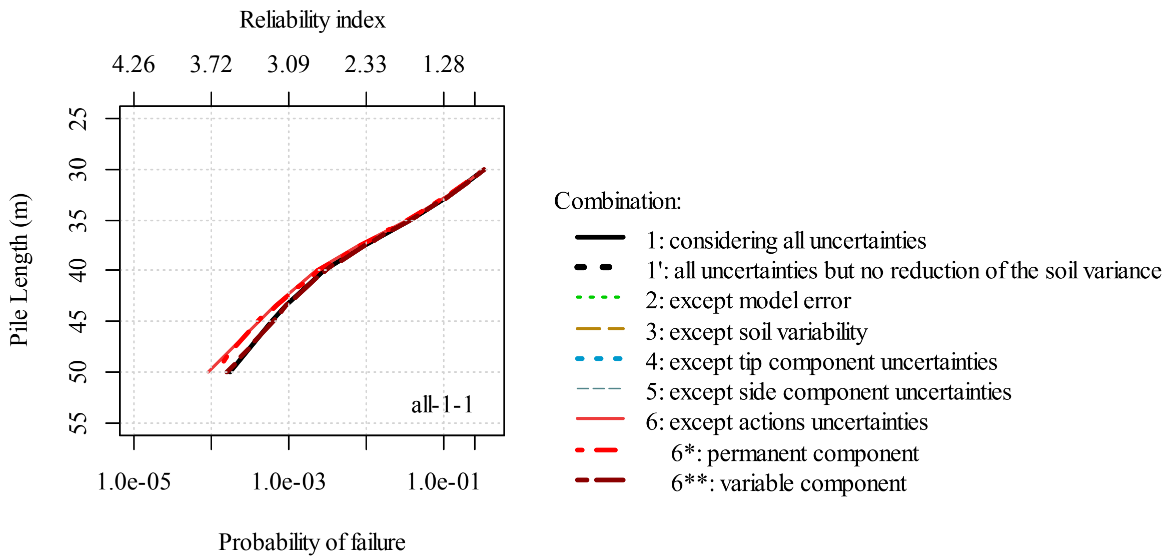
Figure 5.12 presents the results of load combination LC-1 using actions' uncertainties Set 1 (all-1-1).



a) Combinations 1, 1', 2 and 3



b) Combinations 1, 4, 5 and 6

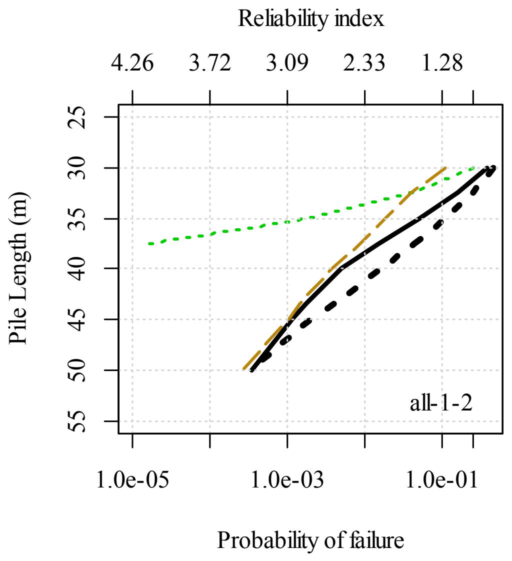


c) Combinations 1, 6, 6* and 6**

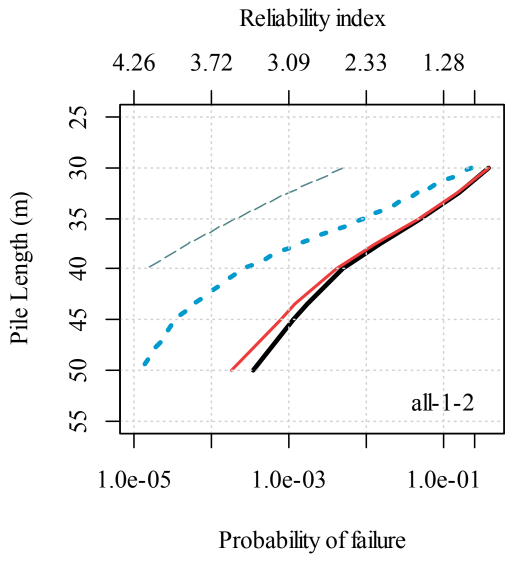
Figure 5.12 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from all SPT, LC-1 & Set 1 for case study 2 (bridge)

LC-1 with uncertainties Set 2 (all SPT)

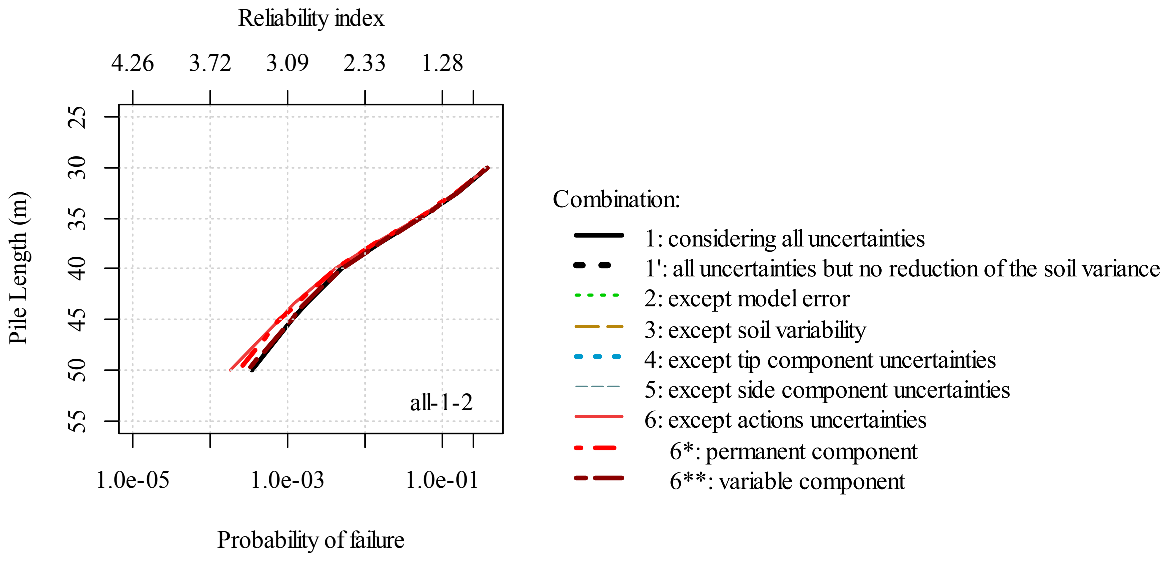
Figure 5.13 presents the results of load combination LC-1 using actions' uncertainties Set 2 (all-1-2).



a) Combinations 1, 1†, 2 and 3



b) Combinations 1, 4, 5 and 6

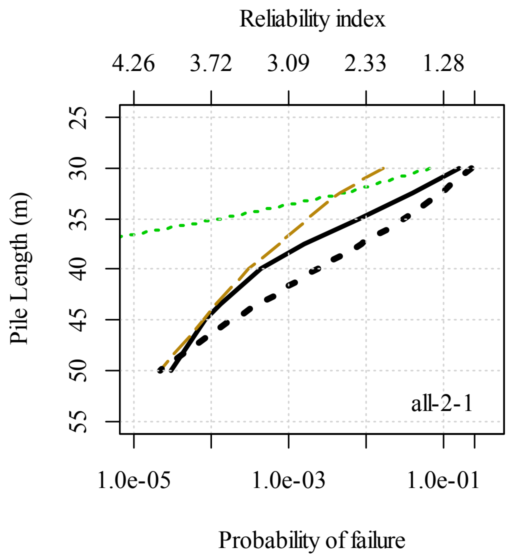


c) Combinations 1, 6, 6* and 6**

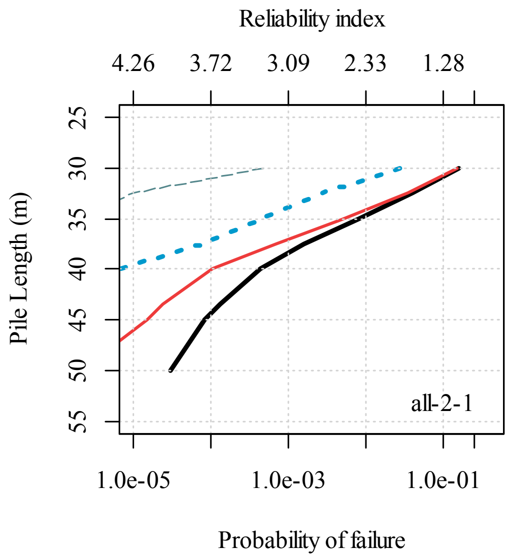
Figure 5.13 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from all SPT, LC-1 & Set 2 for case study 2 (bridge)

LC-2 with uncertainties Set 1 (all SPT)

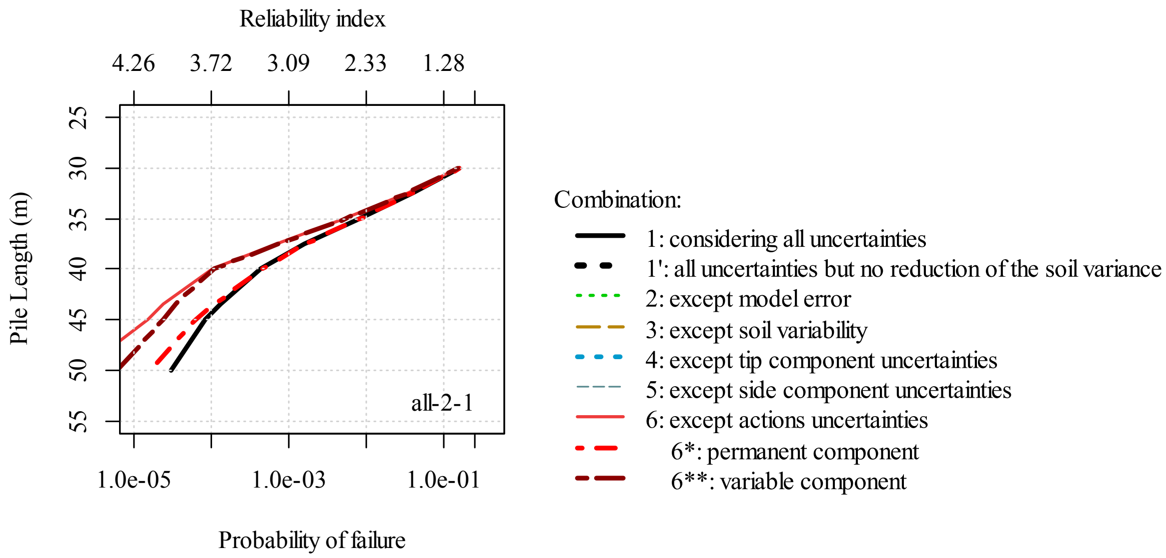
Figure 5.14 presents the results of load combination LC-2 using actions' uncertainties Set 1 (all-2-1).



a) Combinations 1, 1†, 2 and 3



b) Combinations 1, 4, 5 and 6

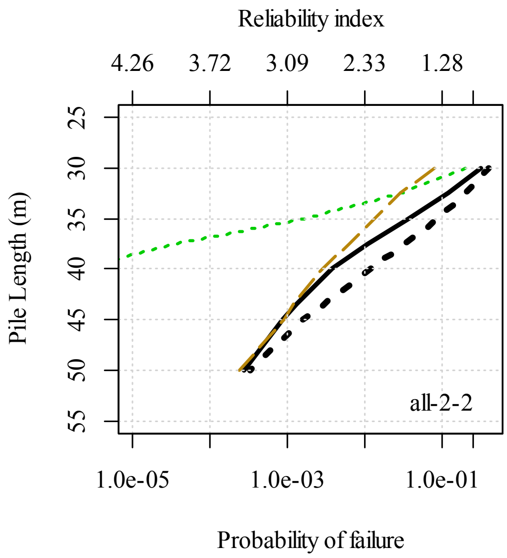


c) Combinations 1, 6, 6* and 6**

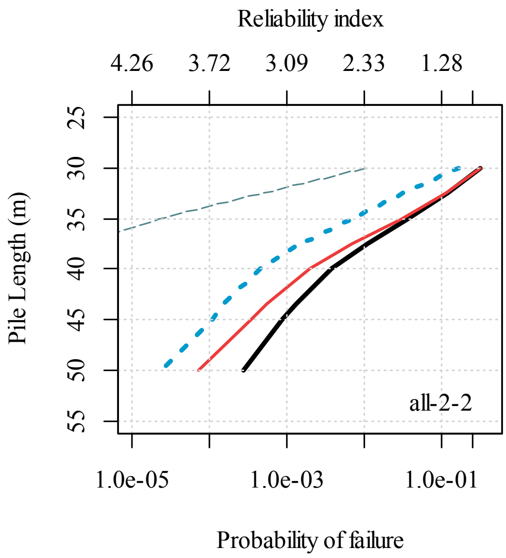
Figure 5.14 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from all SPT, LC-2 & Set 1 for case study 2 (bridge)

LC-2 with uncertainties Set 2 (all SPT)

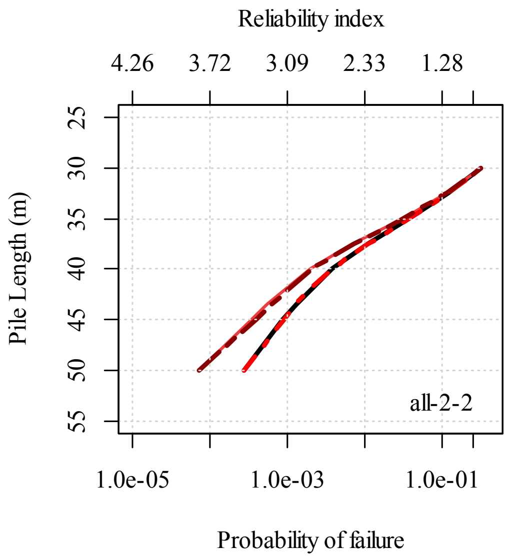
Figure 5.15 presents the results of load combination LC-2 using actions' uncertainties Set 2 (all-2-2).

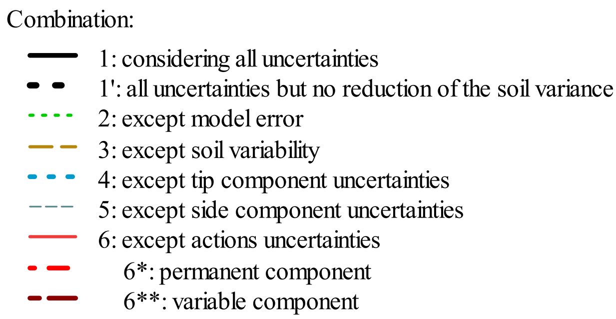


a) Combinations 1, 1†, 2 and 3



b) Combinations 1, 4, 5 and 6





c) Combinations 1, 6, 6* and 6**

Figure 5.15 – Reliability-based sensitivity analysis results (logarithmic scale) using MCS, soil variability from all SPT, LC-2 & Set 2 for case study 2 (bridge)

5.3.3 Results comparison and relative influence

The following figures present the comparison between all the results. Figure 5.16 and Figure 5.17 demonstrate the relative influence of each uncertainty studied for each case ("1 SPT" cases and "all SPT" cases, respectively). Individual results can be consulted in Annex Q.

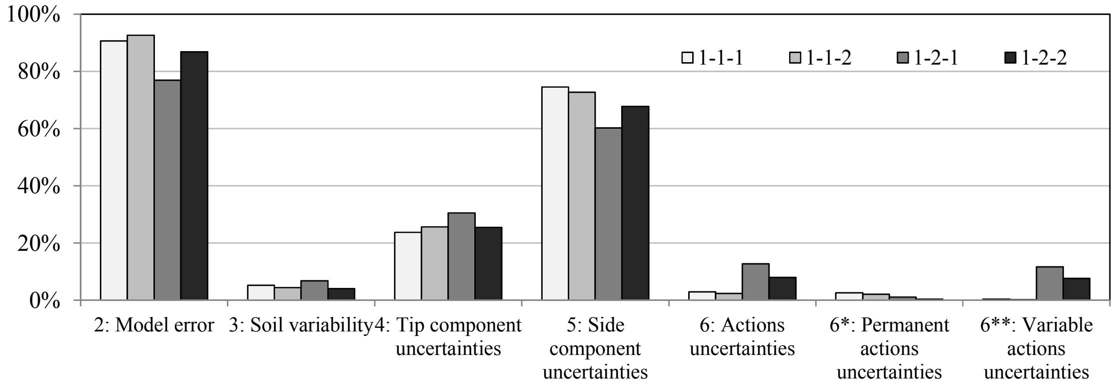


Figure 5.16 – Relative influence results from reliability-based sensitivity analyses, using MCS and soil variability from 1 SPT, for case study 2 (bridge)

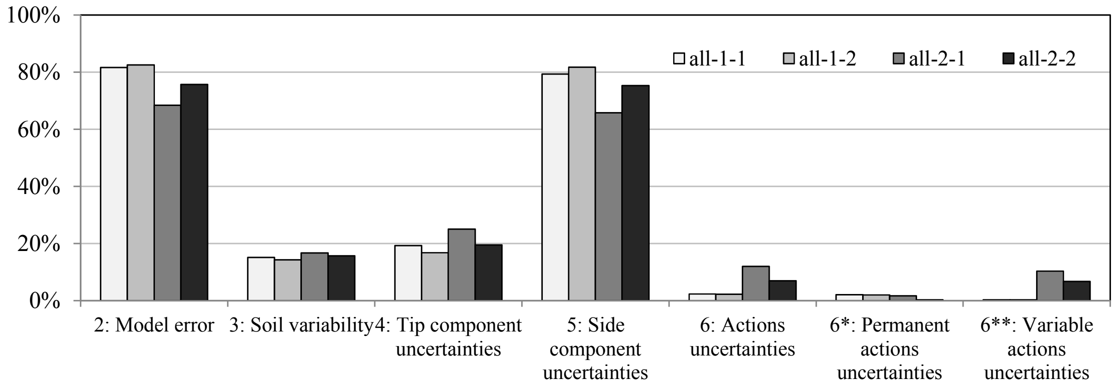


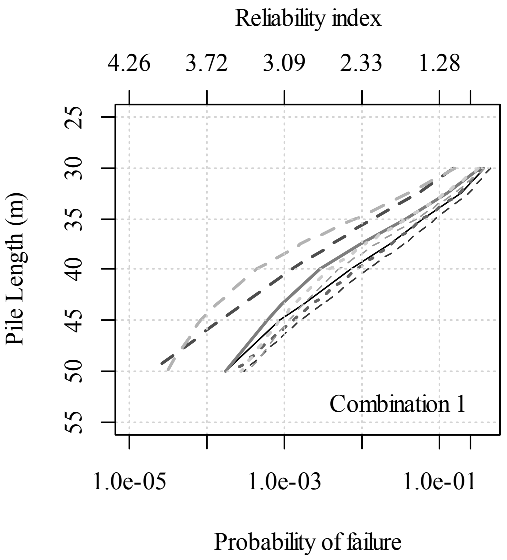
Figure 5.17 – Relative influence results from reliability-based sensitivity analyses, using MCS and soil variability from all SPT, for case study 2 (bridge)

The results presented in these graphs are undeniably consistent. For this case study 2, the model uncertainties (model error) contribute greatly to the probability of failure for all cases studied.

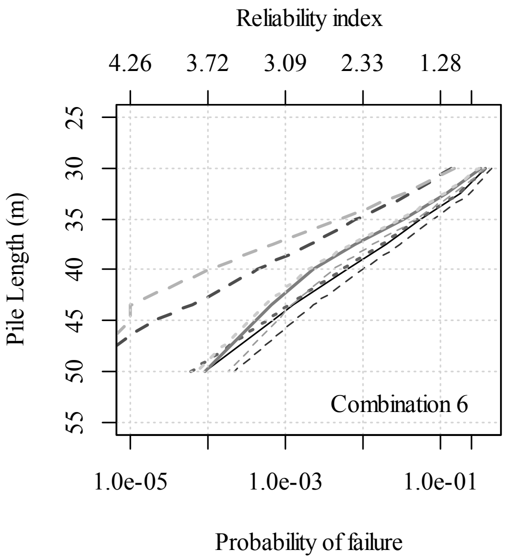
In these sensitivity analyses, when comparing combinations 1 with 2 and 3 (model error and soil variability) one can see that model error is easily highlighted, with a much higher influence than soil variability (combination 3). Also, when comparing combinations 1 with 4 and 5 (tip and side components uncertainties) one can see a high importance of the side component in this pile, which has a high embedded length. Combination 6 presents an almost insignificant influence, except when load combination LC-2 is used.

Concerning combinations 1 and 1†, when comparing the results, it can be seen that they are close to each other. Although, if one does not reduce the variance based on spatial autocorrelation it will lead to a conservative result.

In Figure 5.18 one can see the comparison between combinations 1 and 6 (that consider all uncertainties and except actions uncertainties) for each case (1-1-1; 1-1-2; 1-2-1; 1-2-2; all-1-1; all-1-2; all-2-1; all-2-2). This comparison allows detecting the most and least favourable of cases (higher and lower reliability), that is the case “all-2-1” and case “1-1-2” (see Table 5.8).



a) Combination 1 results



b) Combination 6 results

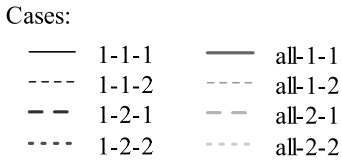


Figure 5.18 – Comparison of reliability-based sensitivity analysis, combinations 1 and 6, of all cases (logarithmic scale) using MCS, for case study 2 (bridge)

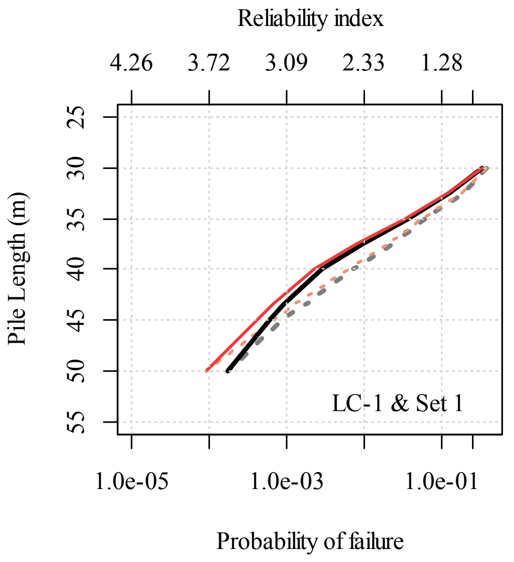
Table 5.8 – Reliability achieved using MCS for each case, with 43.5 m length, of case study 2 (bridge)

Cases	pf	β	
all-2-1	0.000127	3.66	Most favourable
1-2-1	0.000286	3.44	-
all-1-1	0.000910	3.12	-
all-2-2	0.001330	3.00	-
all-1-2	0.001732	2.92	-
1-1-1	0.001726	2.92	-
1-2-2	0.002318	2.83	-
1-1-2	0.003044	2.74	Least favourable

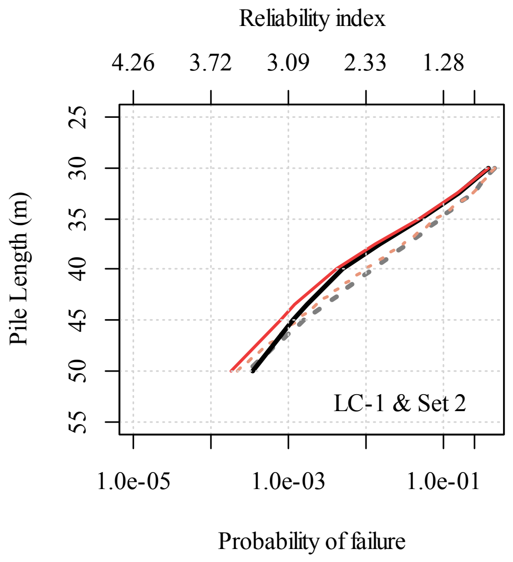
Also it is interesting to analyse case by case. In Figure 5.19 are presented the cases:

- “1-1-1” vs. “all-1-1” (Figure 5.19.a);
- “1-1-2” vs. “all-1-2” (Figure 5.19.b);
- “1-2-1” vs. “all-2-1” (Figure 5.19.c);
- “1-2-2” vs. “all-2-2” (Figure 5.19.d).

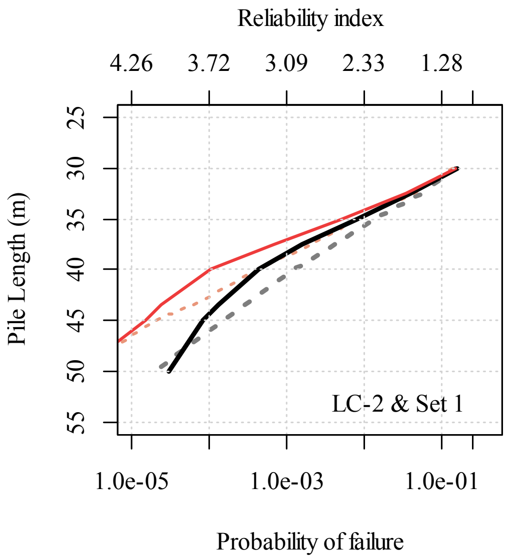
From these individual comparisons is possible to understand that the reliabilities are somehow similar for the cases when 1 SPT and/or all SPT are used. This closeness of the results may be caused by the use of the variance reduction theory in “all SPT” cases. Also, in spite of the cases “1 SPT” having slight lower soil variability, the results show similarities because the main influence comes from the soil model, that is the same in both cases (“1 SPT” and “all SPT”).



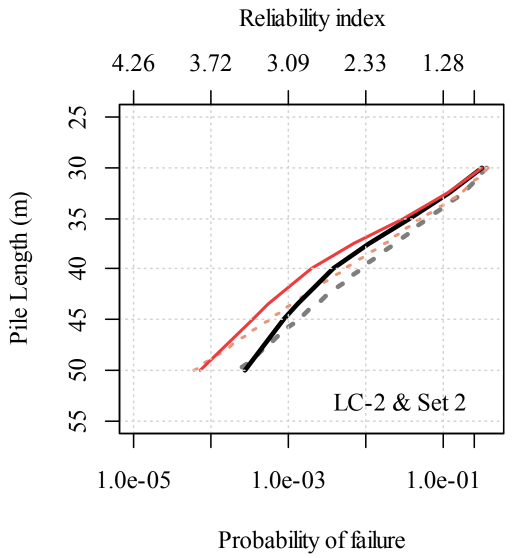
a) LC-1 and uncertainties Set 1 results



b) LC-1 and uncertainties Set 2 results



c) LC-2 and uncertainties Set 1 results



d) LC-2 and uncertainties Set 2 results

Combination:
case 1 SPT
1: considering all uncertainties
6: except actions uncertainties
case all SPT
1: considering all uncertainties
6: except actions uncertainties

Figure 5.19 – Individual comparison of reliability-based sensitivity analysis, combinations 1 and 6, of all cases (logarithmic scale) using MCS, for case study 2 (bridge)

5.4 RELIABILITY-BASED METHODOLOGIES

As seen in the previous sections, for case study 2, the action uncertainties (Set 1 or Set 2) and the load combination considered (LC-1 or LC-2, in this case with similar total magnitude) have a relatively small influence in the results of reliability. The most influent parameter is the model error (by empirical formulas).

Thus, the following calculations for reliability-based methodologies are only presented for the most and least favourable of cases, that is case “1-1-2” and case “all-2-1” respectively (recall Table 5.8). Nonetheless, for the “Reliability-based safety factor” section (5.4.3), the final result (SF) is presented for all cases, especially for assessment of the actions’ SF variations with the uncertainties set and load combination used.

5.4.1 Reliability analyses FORM vs. MCS

The reliability analyses steps followed are (more details in section 3.3):

- select target reliability index → β_T;
- select performance function(s) → $g(X)$;
- define calculation model(s);
- define random variables (RV);
- and finally estimate reliability based on FORM or MCS.

Case study 2 is referred to a bridge pile foundation. In this case it is easier to assume a failure consequence and cost, and adopt a target reliability index based on the recommendations. Nevertheless, and as for case study 1, it is assumed a suitable target reliability index between 2.5 and 4.0, but it is also assumed a desirable reliability index of 3.1 (considering a moderate level for relative cost of safety measures and consequences of failure – Table 2.8, page 34).

Concerning the ULS performance function, the bearing capacity is compared with the actions applied on the pile (LC-1 or LC-2). Based on eq.(3.2), the performance function taken into account is presented in eq.(5.1) for both case “1-1-2” and case “all-2-1”. Note that the unit tip resistance and the unit side resistance have a limit value of 3,000 kPa and 150 kPa (clay) or 200 kPa (sand) respectively (see Annex H). Also know that $\overline{N_{side}}$ has different values associated, depending on the depth.

$$M = \left(\delta_t \times A \times 100N_{tip} + \delta_f \times U \times (10\ or\ 5)\overline{N_{side}}\right) - \left(\delta_G \times G_k + \delta_Q \times Q_k\right) \tag{5.1}$$

FORM calculations

The performance function, as for case study 1, is not a linear combination of the RV. Therefore, FORM method will provide an approximation of the reliability of the pile and then compare it with MCS results. The following eq.(5.2) is the input for the FORM calculations:

$$M = g\left(\delta_t, N_{tip}, \delta_f, \overline{N_{side}}, \delta_G, \delta_Q\right) = g(X_1, X_2, X_3, [X_4, X_5, X_6], X_7, X_8) \Leftrightarrow$$
$$\Leftrightarrow M_{1-1-2} = \left(X_1 \times A \times 100X_2 + X_3 \times U \times (10X_4 + 10X_5 + 5X_6)\right) - (X_7 \times G_k + X_8 \times Q_k)$$
or
$$\Leftrightarrow M_{all-2-1} = \left(X_1 \times A \times 100X_2 + X_3 \times U \times (10X_4 + 5X_5)\right) - (X_7 \times G_k + X_8 \times Q_k) \tag{5.2}$$

As one can see the $\overline{N_{side}}$ variable has three RV (X_4 , X_5 , X_6) for case "1-1-2" and two RV (X_4 , X_5) for case "all-2-1". The RV X_4 , X_5 and X_6 in case "1-1-2" are referring to braches 1, 2 and 3 respectively ([10; 26] m,]26; 36] m and > 36 m) while RV X_4 and X_5 in case "all-2-1" are referring to braches 1 and 2 respectively ([10; 22] m and > 22 m). In case "all-2-1" formulation the RV X_6 is omitted so that actions uncertainties have the same numbering for both cases (X_7, X_8).

Considering this, the independent terms are null, and the linear and quadratic coefficients are presented in Table 5.9 for case "1-1-2" and in Table 5.10 for case "all-2-1".

Table 5.9 – Performance function coefficients for FORM iterative calculations of case study 2 (1-1-2)

	X_1	X_2	X_3	X_4	X_5	X_6	X_7	X_8
Bi	0	0	0	0	0	0	–4520	–267
Cij	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*
1	0	100 A	0	0	0	0	0	0
2	0	0	0	0	0	0	0	-
3	0	0	0	10 U	10 U	5 U	-	-
4	0	0	0	0	0	-	-	-
5	0	0	0	0	-	-	-	-
6	0	0	0	-	-	-	-	-
7	0	0	-	-	-	-	-	-
8	0	-	-	-	-	-		-

Table 5.10 – Performance function coefficients for FORM iterative calculations of case study 2 (all-2-1)

	X_1	X_2	X_3	X_4	X_5	X_6	X_7	X_8
Bi	0	0	0	0	0		–1778	–2733
Cij	*1*	*2*	*3*	*4*	*5*		*7*	*8*
1	0	100 A	0	0	0		0	0
2	0	0	0	0	0		0	-
3	0	0	0	10 U	5 U		-	-
4	0	0	0	0	0		-	-
5	0	0	0	0	-		-	-
6								
7	0	0	-	-	-		-	-
8	0	-	-	-	-			-

The FORM results are presented in the following tables: Table 5.11 for "1-1-2" and Table 5.12 for "all-2-1". Sensitivity factors are depicted in Figure 5.20.

Table 5.11 – Results of FORM iterative analysis for case "1-1-2" of the case study 2 (bridge)

	Resistance side						Actions side	
	X_1	X_2	X_3	X_4	X_5	X_6	X_7	X_8
Design point X^*	0.307	47.6	0.443	0.92	14.2	20.9	1.12	0.98
Sensitivity α	0.521	0.350	0.488	0.388	0.412	0.411	–0.167	–0.232
Reliability achieved	β = 3.94 → pf = 0.0000412							

Table 5.12 – Results of FORM iterative analysis for case "all-2-1" of the case study 2 (bridge)

	Resistance side					Actions side	
	X_1	X_2	X_3	X_4	X_5	X_7	X_8
Design point X^*							
Sensitivity α	0.345	0.197	0.213	0.290	0.824	–0.481	–0.174
Reliability achieved	β = 3.42 → pf = 0.000309						

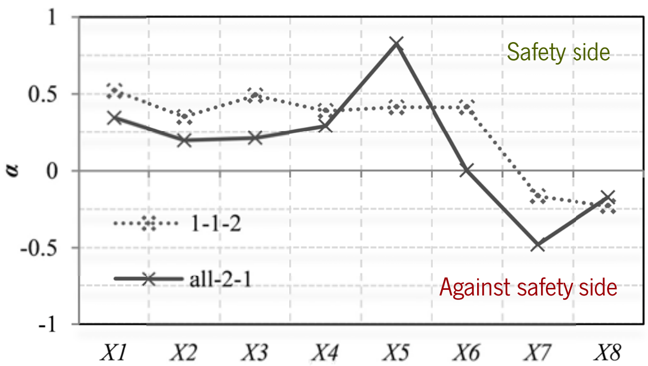


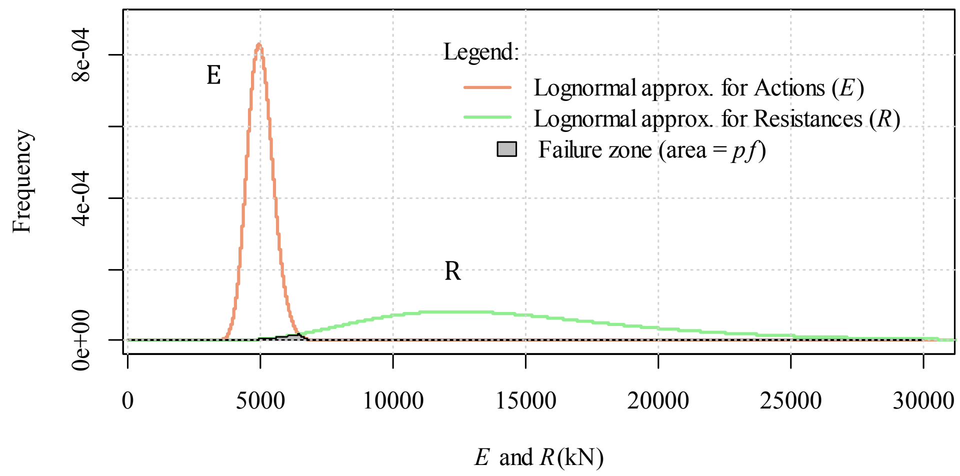
Figure 5.20 – Sensitivity factors from FORM analyses for case study 2 (bridge)

During FORM application to case study 2 some difficulties were observed in convergence of the reliability indexes. It was also noted that the reliability indexes, although not very far from MCS, presented inconsistent values during the analyses. Also, the sensitivity factor values also presented erratic results. The reason for this might be the type of empirical method used (SHB method's formulation, Annex H), that presents a limit condition. This characteristic (truncated prediction/limit condition), added to a non-normally distributed RV and non-linear function, causes this kind of difficulties when trying to perform normally distributed and linear approximations with FORM method.

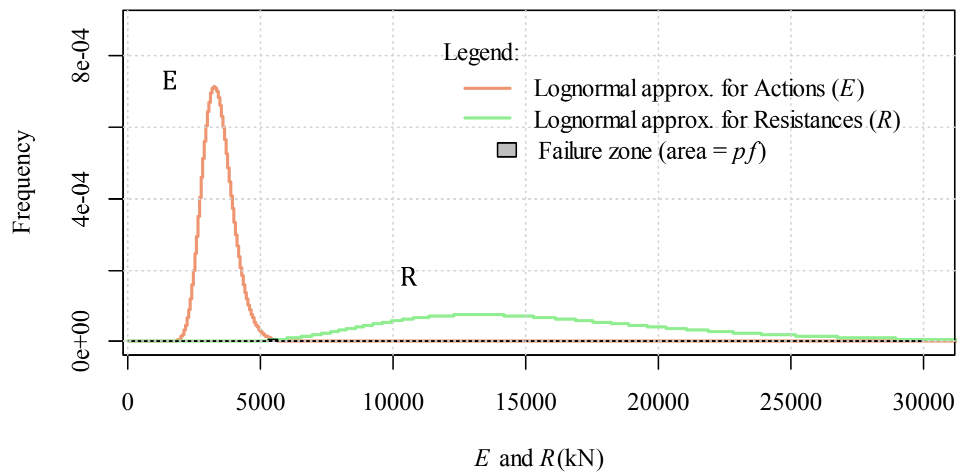
As conclusion, FORM displays here its limitations with particular conditions in performance functions. For this reason this method is considered not applicable to case study 2.

MCS calculations

The MCS are widely known and used as a reference method for approximation methods. For better accuracy, and since it will be the only run for this calculation the n = 1,000,000 was selected, however, as explained earlier, n around 500,000 would suffice for this case study 2. The MCS results for cases “1-1-2” and “all-2-1” are depicted in Figure 5.21. These cases achieved the probability of failure of $pf_{1\text{-}1\text{-}2}$=0.00304 ($\beta_{1\text{-}1\text{-}2}$=2.74) and $pf_{all\text{-}2\text{-}1}$=0.000127 ($\beta_{all\text{-}2\text{-}1}$=3.66) respectively. In Figure 5.22 are depicted the graphical representation of these probabilities ($pf_{1\text{-}1\text{-}2} > pf_{all\text{-}2\text{-}1}$).

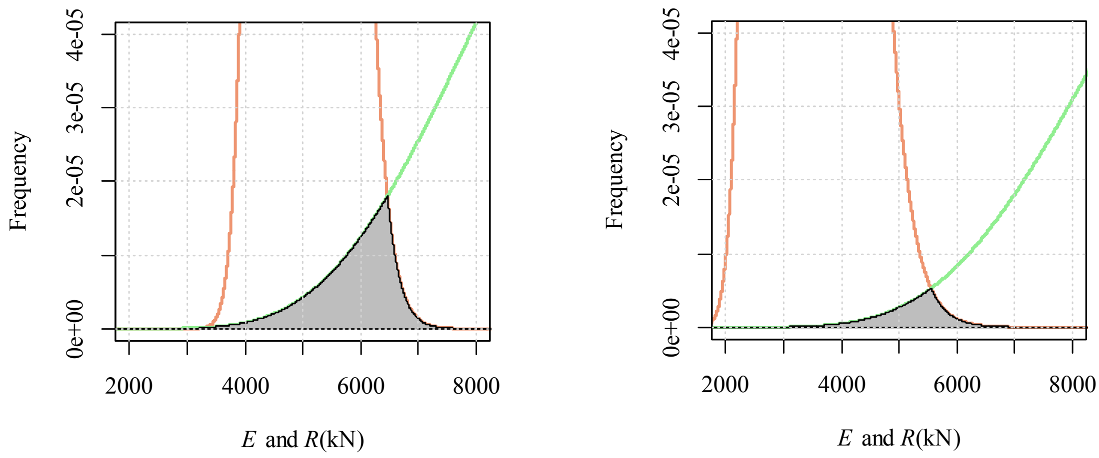


a) Case 1-1-2



b) Case all-2-1

Figure 5.21 – Histograms of E and R achieved with n=1,000,000 MCS for case study 2 (bridge)



a) Case 1-1-2

b) Case all-2-1

Figure 5.22 – Graphical representation of the probability of failure achieved with n=1,000,000 MCS, for case study 2 (bridge)

5.4.2 Minimum length (RBD) vs. allowable load (Safety evaluation)

RBD and Safety evaluation are methodologies based on Monte Carlo technique. The stability for case study 2 was achieved for n=500,000 (see Annex P) considering the interval [2.5; 4.0] for reliability index values, as discussed beforehand.

These reliability approaches are carried out for each case, "1-1-2" and "all-2-1", and the following lengths and loads were considered:

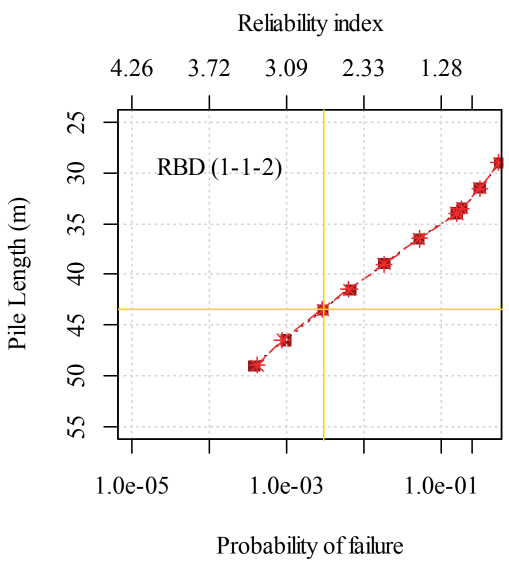
- RBD approach: different lengths of the pile D=[30, 32.5, 35, 37.5, 40, 43.5, 45, 50] m, being the actual length of the pile installed 43.5 m [1] (Figure 5.23.a and 5.23.b);
- Safety evaluation approach: different load values E=[2000, 3000, 4000, 4500, 6000, 7000, 8000, 9000, 10000, 12000] kN (Figure 5.23.c and 5.23.d).

The next figures depict the results and also a light line marking the length of the pile (43.5 m), the total value of the load combination (LC-1, LC-2), and the load test result (4000 kN).

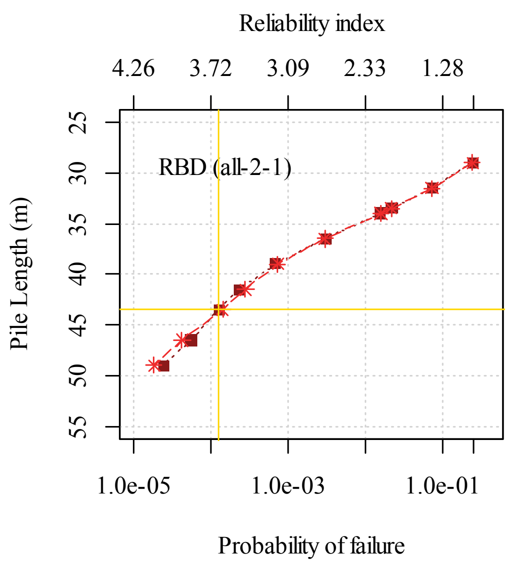
It is possible to conclude that, in cases "1-1-2" and "all-2-1", the reliability of the pile installed falls between the pre-established interval (β=[2.5; 4.0]). Nevertheless, if we consider the desirable reliability of 3.1, only case "all-2-1" achieves this value. Curiously, the reliability of the pile achieved is higher when considering general soil variability (all SPT). The reason for this is probably the fact that this parameter (soil variability) is not one of the most influential ones and also the fact

[1] Recall that 10 m of this length is between water table and soil level, therefore, the embedded length corresponds to D minus 10 m.

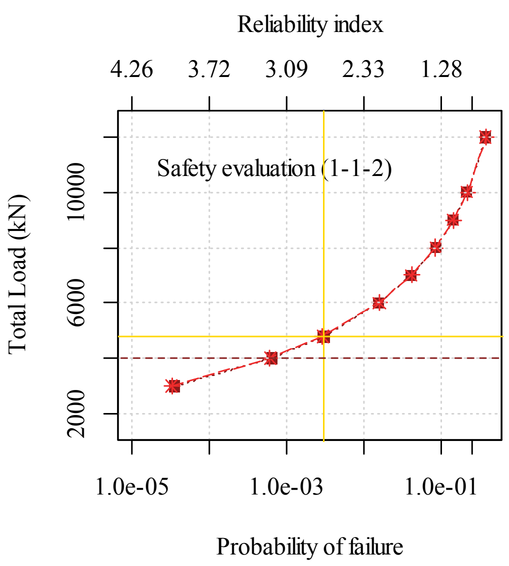
that when all SPT is considered, it is possible to reduce the variance based on autocorrelation of the soil (increased reliability).



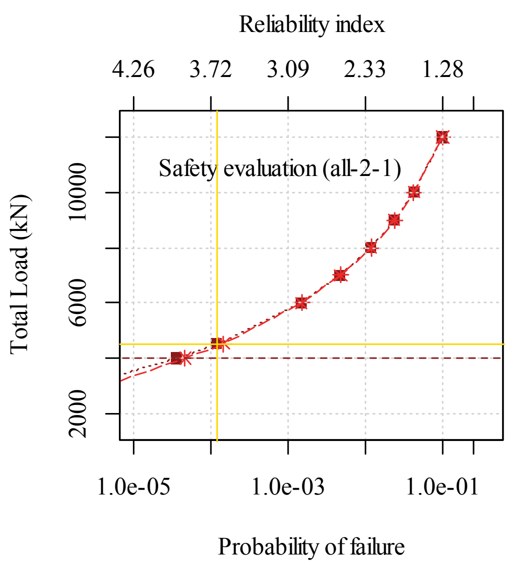
a) RBD of case 1-1-2



b) RBD of case all-1-1



c) Safety evaluation of case 1-1-2



d) Safety evaluation of case all-1-1

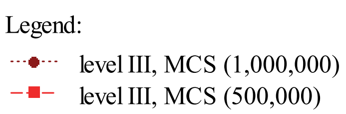


Figure 5.23 – Results of reliability-based approaches RBD and Safety evaluation, using MCS for case study 2 (bridge)

5.4.3 Reliability-based safety factors

The reliability theory methods can be used in a direct or indirect way. An indirect way is for example the use of safety factors (SF) calibrated based on reliability theory, differing from the previous section, where the reliability theory is directly used for the design of the pile (determining the minimum dimensions for a specific case (for specific load and soil conditions) or evaluating the maximum bearing capacity for a specific case (for specific dimensions and soil conditions).

This section presents the evaluation of the multiplying SF for the case study 2, considering all cases denoted in Figure 5.7.

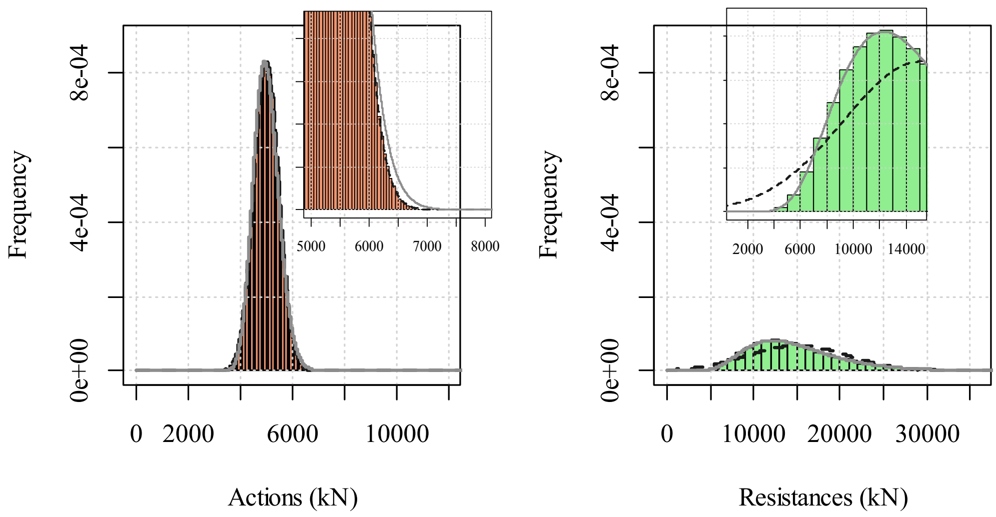
The results will be compared with the SF recommended by different design codes (Annex F). According to the procedure (Chapter 3 – 3.3.8), it is necessary to define the statistical parameters of the variables R and E (mean, SD, COV and PDF type), the characteristic values and also the target reliability index (β_T= [2.5, 4.0]).

The results of n=1,000,000 MCS were used to evaluate the statistical parameters of R and E. Only the intermediate results of the cases "1-1-2" and "all-2-1" are presented in detail, then final SF results are presented for all cases for comparison (Figure 5.27).

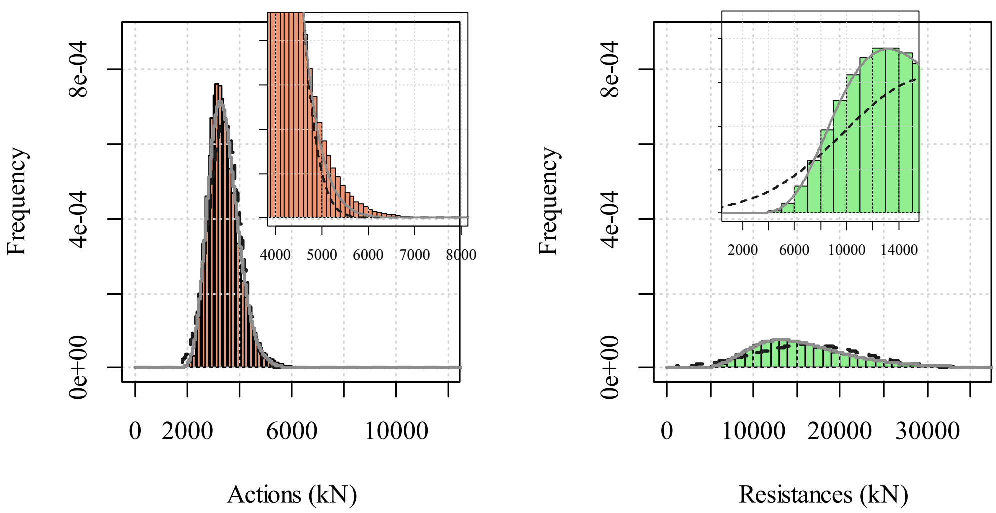
For each case Normal and Lognormal distributions (PDF) were fitted to the histograms of each variable (R,E) – for cases "1-1-2" and "all-2-1" this step is presented in Figure 5.24 a,b. After fitting the PDF type, all points simulated (R,E) are divided in three groups. The first group comprehends the points near the limit state line, then the points in failure zone and then the points in safety zone (Figure 5.25 presents these results for cases "1-1-2" and "all-2-1"). Finally, Figure 5.26 presents the range of SF obtained for cases "1-1-2" and "all-2-1", while Figure 5.27 presents the all the final SF results for all cases considered (different load combinations and sets of actions uncertainties) and different target reliability indexes.

Recall that the characteristic values (R_k and E_k) were assumed as:

- the mean value for both R and $E \rightarrow R_k=\overline{\mathrm{R}}$ and $E_k=\overline{\mathrm{E}}$;
- the mean value for R and the high fractile of 95% for $E \rightarrow R_k=\overline{\mathrm{R}}$ and $E_k=E_{95\%}$.



a) Case 1-1-2

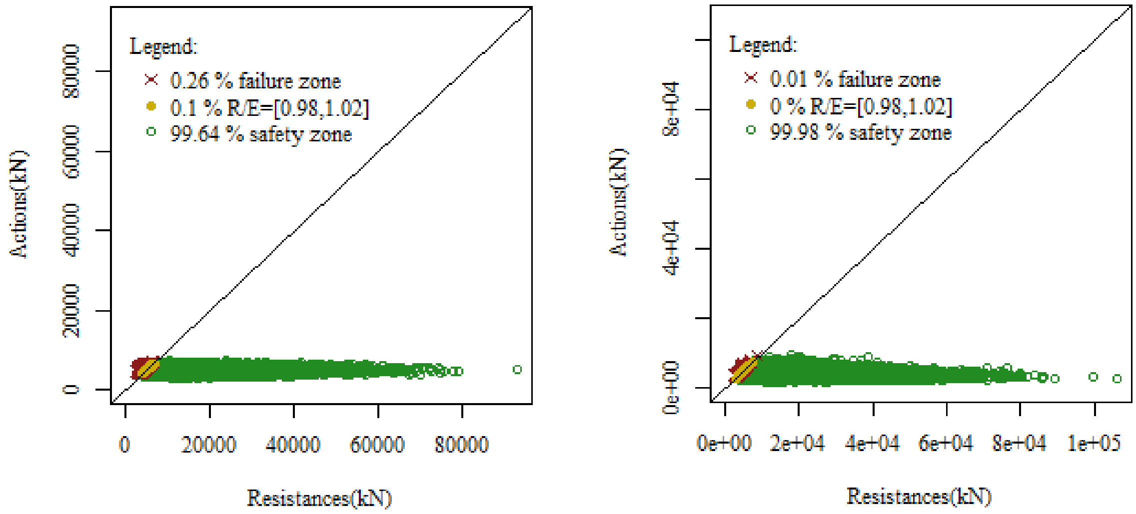


b) Case all-2-1

Legend:
Normal fit
Lognormal fit

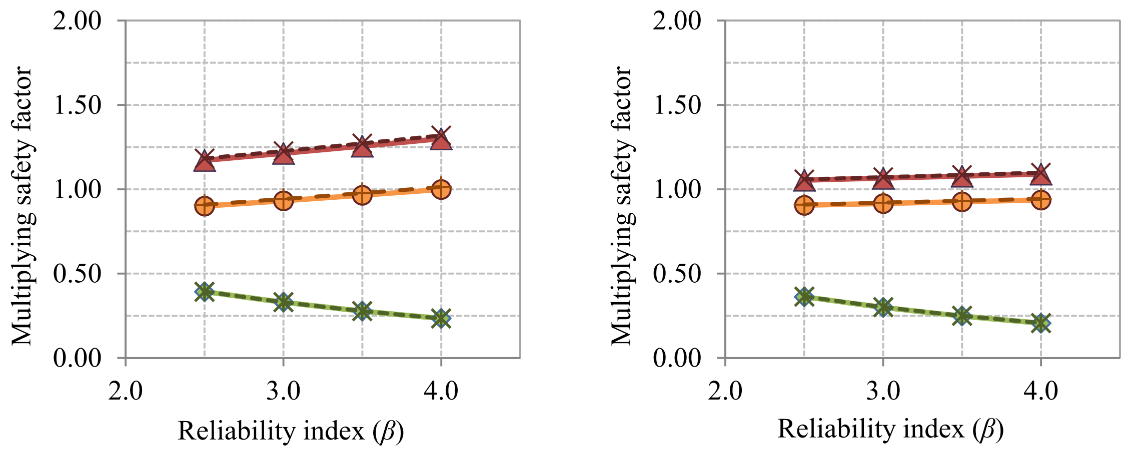
Figure 5.24 – Histograms and PDF approximations to E and R, achieved with n=1,000,000 MCS for case study 2 (bridge)

It is possible to conclude that the Lognormal PDF has a better fitting for most of the cases. However, in case "1-1-2", for actions histogram, the Normal PDF is the one that better fits when considering the right side of the tale. Also, and as anticipated, the resistances present a considerably higher dispersion than the loads/actions.



a) Case 1-1-2 b) Case all-2-1

Figure 5.25 – Graphical representation of the MCS points near limit state line, failure zone and safety zone, achieved with n=1,000,000, case study 2 (bridge)



a) Case 1-1-2 b) Case all-2-1

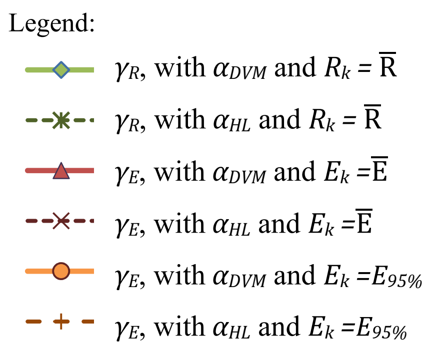


Figure 5.26 – Multiplying SF based on reliability analyses for case study 2 (bridge)

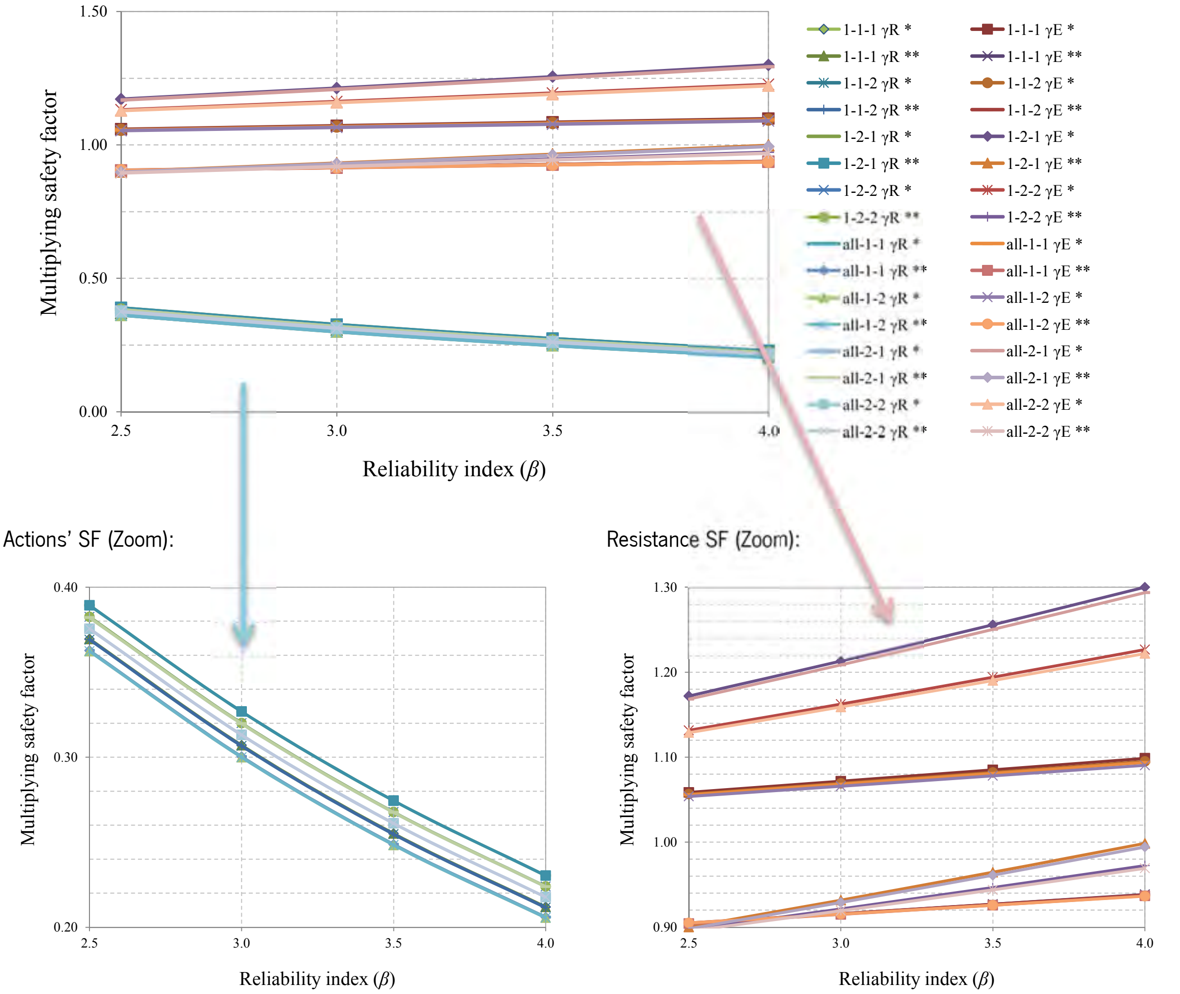


Figure 5.27 – Multiplying SF based on reliability analyses for case study 2 (bridge), * α_{DVM}, ** α_{HL}

The SF values were calculated for ($R_k = \overline{\mathrm{R}}$, $E_k = \overline{\mathrm{E}}$) and ($R_k = \overline{\mathrm{R}}$, $E_k = E_{95\%}$). From the results presented in the previous graphs, it is possible to highlight that:

- the values obtained for the SF, with DVM or HL formulas are the same;
- the resistance multiplying SF resulted between 0.21 and 0.36 as a lower limit by cases “all-1-1” and “all-1-2”; and 0.23 and 0.39 as a higher limit by case “1-2-1”. These results can be considered as equal (0.2-0.4). These low values result from the high variability of resistances in all cases;
- the actions' multiplying SF resulted between 0.91 and 0.94 as a lower limit by cases “1-1-1”, “1-1-2”,.“all-1-1” and “all-1-2”; and 1.17 and 1.30 as a higher limit by case “1-2-1” and “all-2-1”;

- once again, notice that the values for actions' SF should be higher than one, but when using $E_k = E_{95\%}$ the values obtained are slightly lower than one (0.9 to 1.0). This is due to its low variability and necessity to reduce, since its sensitivity factor is relatively low;
- The values of actions' SF are different for the different cases, but some groups can be distinguished. From top to bottom, and for $E_k = \overline{\mathrm{E}}$, it presents the LC-2 with Set 1 of uncertainties (will be denoted group 1), next, LC-2 with Set 2 (group 2) finally LC-1 with Set 1 and Set 2 (group 3). This tendency repeats for $E_k = E_{95\%}$;
- the group 1 and group 2 have a higher variability with the reliability index, yielding that the LC-2 has a higher uncertainty associated.

The values recommended by the Eurocode 7 (Annex A of CEN, 2007 – multiplying resistance SF between 0.7 and 1.0 and load SF between 1.0 and 1.5) are higher than the ones calculated here. However these values are close to the North American recommendations (Annex F).

5.5 Concluding remarks

This chapter presented a real life case study of a pile foundation from a bridge project. The reliability-based methodologies explained and described in Chapter 3 are applied to this case, denoted "Case study 2". Case study 2 pertains to a pile from a railway bridge located in the south of Portugal. The pile under study is installed in the riverbed and encounters a geological formation compound mainly by an upper layer of mud followed by layers containing mixtures of sand, clay and marl. The SPT results, from three distinct places around the pile installation site are available for analyses. Water line is 10 m above the soil level. The pile is an open-ended steel pipe pile, with a length of 43.5 m (33.5 m of which is embedded), a diameter of 1.12 m and a wall thickness of 12.4 mm, with an ultimate bearing capacity of 4000 kN by dynamic load testing (signal matching).

In spite of the fact that this dissertation is mostly focused on the geotechnical part of the reliability analyses (soil and modelling uncertainties), this case study presents also some emphasis to load combinations and actions uncertainties influence. Two load combinations are considered (LC-1 and LC-2) and two sets of uncertainties for actions studied (Set 1 and Set 2).

Moreover, as different considerations were made for actions, the soil variability was also considered in two different ways. In this case the uncertainties to be considered in ground conditions can be based on 1 SPT or on all SPT. As such, and considering all these cases, a total of eight combinations are possible, denoted cases 1-1-1; 1-1-2; 1-2-1; 1-2-2; all-1-1; all-1-2; all-2-1; all-2-2 (SPT - LC# - Set#).

A reliability-based sensitivity analysis was the first step of this chapter, in order to identify the most and least favourable of cases (higher and lower reliability respectively). The sensitivity analyses, for the various cases will help the engineer on decision-making concerning the selection and study of the uncertain variables and proper evaluation of the different cases. Based on those analyses the following points can be highlighted:

- All cases present:
  - consistent results in terms of relative influence of each uncertainty;
  - a noticeable relationship is exhibited between of probability of failure (on a log scale) and the length of the pile (on a linear scale), and also between the probability of failure (on a log scale) and the total load on the pile (on a linear scale).
- Combination 1 vs. combination 1† (considering and not considering reduction of variance for cases with all SPT); the spatial autocorrelation allows for the reduction of the standard deviation of the N value (SPT). It is obvious, and evidenced by the results, that this will lead to more reliable results. When this reduction is not considered, the result is more conservative, but not correct, especially in terms of economy.
- Combination 1 vs. combination 2 and combination 3 (modelling uncertainty and soil variability), it is possible to realise over and over that the model error is a very influent and against safety uncertainty type. Meanwhile, the soil variability presents a medium to low influence.
- Combination 1 vs. combination 4 and combination 5 (tip and side component uncertainties), the results of the combination 5 present much higher variability than the combination 4 results. As evidenced by combination 4 results being very similar to combination 1 results. The importance of the side is highlighted since this pile has a considered high embedded length, which results in a much bigger influence of the side component uncertainty, as expected.
- Combination 1 vs. combination 6, 6*,6** (actions uncertainties), the analyses present an almost insignificant influence of actions uncertainties, except when load combination LC-2 is used.
- The most favourable case (higher reliability) is case “all-2-1”.

– The least favourable case (lower reliability) is case “1-1-2”.

After identifying these cases, reliability computations using FORM (RA level II) and MCS (RA level III), are presented and compared. The results obtained for FORM are an approximation, not successfully achieved for this case study 2. The deviations in the results are due to FORM limitations to the complexity of the performance function. The formula used to predict the capacity of the pile is truncated and it was not possible to introduce this on FORM calculation, and it also difficulted the linear approximations. A more extensive comparison between FORM and MCS was done, considering different cases, proving once again that FORM is not applicable to this case because of its limitations (the reason why the results were not all presented).

The MCS results, with n=1,000,000, indicate $\beta_{1\text{-}1\text{-}2}$ = 2.7 and $\beta_{all\text{-}2\text{-}1}$ = 3.7. Concerning the results and comparison of the two different approaches, RBD (study of the probability of failure with the variations in the length of the pile) and Safety evaluation (study of the probability of failure with the variations in the value of the total load applied), a summary is presented in Table 5.13.

Table 5.13 – Results of the RBD and Safety evaluation for case study 2 (bridge)

Case study 2 Pile installed with 43.5 m	Obtained reliability	Fall inside recommended interval?	RBD, value of the length of the pile (m)		Safety evaluation, allowable total load (kN)	
			β =2.5	β =4.0	β =2.5	β =4.0
Total load:						
4787 kN (case 1-1-2)	2.7	Yes	42	55	5128	2989
4511 kN (case all-2-1)	3.7	Yes	36	48	7326	6881

It is possible to conclude that, in cases “1-1-2” and “all-2-1”, the reliability of the pile installed falls between the pre-established interval of β=[2.5; 4.0]. Furthermore and unexpectedly, the reliability of the pile achieved is higher when considering general soil variability (all SPT). The reason for this may be fact that this (soil variability) is not one of the most influential uncertainties and also the fact that when all SPT is considered, it is possible to reduce the variance based on autocorrelation of the soil.

The final section of this chapter presented the evaluation of resistance and load multiplying safety factors, and its comparison with the recommendations by different design codes. All cases’ SF were evaluated and the values of actions’ SF are different for each one. However, some groups can be distinguished. The LC-2 with Set 1 of uncertainties denoted group 1 (higher actions’ SF), next, LC-2 with Set 2 as group 2 (intermediate actions’ SF) and LC-1 with Set 1 and Set 2 as group 3 (lower actions’ SF). The group 1 and group 2 have a higher variability with the reliability index, yielding that the LC-2 has a higher uncertainty associated, as also stressed with the sensitivity analyses. Concerning the resistance SF results, they can be considered as the same for all cases (between 0.2-0.4).

When comparing these values/results with the recommended by the Eurocode, they present a smaller magnitude. However, they are closer to the North American recommendations. These differences are to be expected since many considerations contribute for the calculation of both values. Once again, it is necessary to mention that this assessment cannot be extrapolated, since it is related to one case only. In order to make some recommendations it is necessary to evaluate and calibrate many different cases.

Finally, this case study, and all the results and conclusions, also emphasise the need of well understanding of these reliability tools for comprehensive analyses and the need for better defined and more accurate resistance models.

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Chapter 6

COSTS AND RISKS ASSOCIATED TO AXIAL PILES

6.1 INTRODUCTION

In the present, there are major concerns with the preservation of the environment, but the global financial crisis (2007~2008) has brought a bigger concern – achieving economy. This trend and the new regulation design codes are imposing geotechnical engineers to adjust the traditional design methodologies. Thus, nowadays and more than ever, the designs must be economic, sustainable and reliable, all at the same time.

Concerning the reliability part of the design, different approaches can be adopted, as explained in the previous chapters of this dissertation (refer to Chapter 2 and Chapter 3). All reliability-based approaches provide very useful tools for modelling the uncertainties and for quantifying their influence on the behaviour under study, helping at the same time, achieve a more rational design, as showed by practical applications in Chapters 4 and 5.

However, and as stressed before, achieving economy is also a very important aspect of the designs and construction processes. It is important to invest in quality and cost optimisation; thus, studies on the behaviour of costs are of significant importance (Baecher, 1987; Banafa *et al.*, 1990; Velazco *et al.*, 2003).

In Flor's research (Flor, 2007) it is clearly stated that the final costs of a construction work exceeds hugely the contract value. These big deviations cannot be simply attributed to random or unpredictable causes. The uncertainties that lead to these deviations are sometimes not known (not identifiable and/or quantifiable); however, increasing the knowledge of what is identifiable and quantifiable can many times reduce these deviations. According to this study the information can be divided as following:

- known and quantifiable, 60%;
- known but not quantifiable, 25%;
- unknown, 15%.

These conclusions were based on 73 different construction woks (public and private), such as Expo'98 (Lisbon), metro of Lisbon and football stadiums for Euro 2004 competition. Flor's goal was to develop a tool to improve the quality of decision, allowing the prediction of the financial deviation that will be associated to it depending on the characteristics of the project (construction work) and on the type of management that is implemented.

In particular for geotechnical projects, the knowledge of the characteristics and behaviour of the site and soil is never complete, and investments on the geotechnical investigation are normally only around 2% of the total costs of the project. Therefore, the geotechnical engineers deal with a high component of uncertainty, which can result in increased costs to the final project/construction work (Einstein, 2001; Mrabet & Giles, 2002). It is important and necessary the assessment of the adverse consequences as a result of the underestimation of its geotechnical component (Chapman & Marcetteau, 2004).

Thus, when combining reliability analyses with a cost-effect study is possible to match the needs for a particular problem and adequate the efforts to gather the information necessary to achieve certain reliability level. At the same time it is possible to decrease the chances of additional costs and/or other implications due to unforeseen ground conditions or pile capacity (Figure 6.1).

This combination of reliability (probabilities) and costs during preliminary investigations, design, construction phase and study of possible consequences of failure, generates the concept of risk. Risk can be defined as the product of the probability of an event with an undesirable impact and the consequences of that event. Hence, for these purposes, this chapter intends to study the interaction/relationship between the reliability of a design procedure adopted and the corresponding costs. Such information would support cost-benefit decisions, helping and guiding the research of a pile foundation project, and also avoiding spending/investing where it is not appropriate/favourable for the reliability of the pile, as schematised in Figures 6.2 and 6.3.

Gilbert (2003) provides real-world examples where the benefit of reliability-based design as a decision-making tool has been achieved. Meanwhile Jaksa *et al.* (2003) defends that is urgently needed a series of guidelines that link the scope of a site investigation with the probability that the foundation will be under-designed (resulting in some form of failure), or be overdesigned (resulting in more funds being spent on the foundation than would have otherwise been necessary had a more appropriate site investigation been carried out).

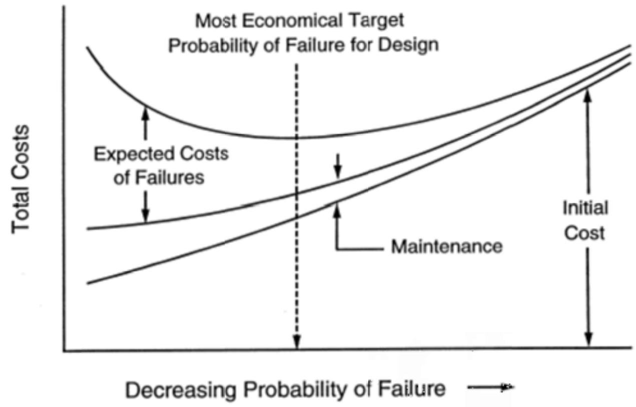


Figure 6.1 – Illustrative cost-benefit analysis (Phoon *et al.*,2000)

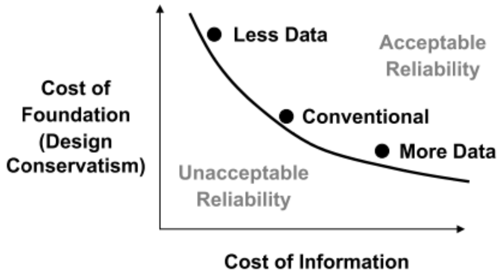
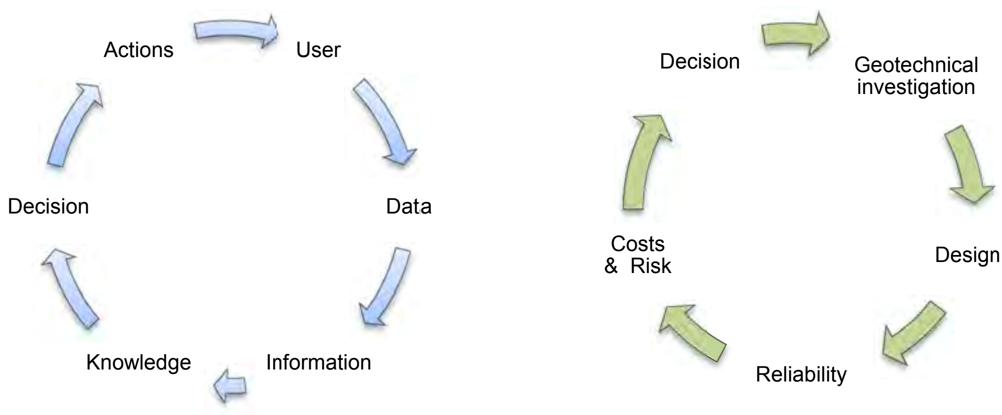


Figure 6.2 – Conceptual relationship between decision making and reliability-based design (Gilbert, 2003)



a) Generic case

b) Geotechnical engineering case

Figure 6.3 – Data, information and decision cycle

Based on this, the analyses presented, concerning the case study 1 (FEUP – experimental site, Chapter 4), take into account the costs, reliability and risk. It demonstrates the use of such approaches and considerations, both in the site investigation stage and the pile design stage.

6.2 Brief introduction on costs

Nowadays it is essential to control costs, which could result in a lower final price, making companies more competitive and profitable in relation to others. More and more companies invest in programs for quality and cost optimisation, in order to gain competitiveness (ARUP is an example, refer to Chapman & Marcetteau, 2004 and NHBC Foundation, 2010). Thus, studies on the behaviour of costs are taking relevant importance not only in academic circles, but also in areas that deal directly with business activities.

The use of parametric costs represents an excellent control tool for a company. When data from the past is documented they can be correlated to establish standard costs, using regressions and statistics, and to represent reality. With them, it is possible to build models that are much simpler than the reality and yet predict and explain the phenomena with a high degree of accuracy.

For the purposes of decision-making, it is essential to know the fundamental differences between the main costs along with the condition of their use in decision-making. Therefore, and while computing these costs, it is essential to have understanding of the different types of costs. Costs can be classified in many different ways (Truett & Truett, 2006):

- Total costs, Average costs and Marginal costs;
- Fixed costs and Variable costs;
- Direct costs and Indirect costs;
- Short-run costs and Long-run costs;
- Controllable costs and Non-controllable costs;
- Incremental costs and Marginal costs;
- Explicit costs and Implicit costs;
- Urgent costs and Postponable costs;
- Actual cost, Opportunity cost, Sunk cost, Book cost, Out of pocket costs, Accounting costs, Economic costs, Historical costs, Replacement costs, Shutdown costs, Abandonment costs, Business cost, Full cost, etc.

These classifications depend on the kind of industry, product, situation (output) analysed, among others. Commonly, the way to denominate costs is as fixed and varible. Economists often use this classification and the simplest function of total cost (equal to fixed costs plus variable costs). The fixed costs (or constant) do not change with different factors, such as quantity of the activity or duration, or even the production scale (the output). Meanwhile, the variable costs change in a direct proportion to the activity (the output volume). It is important to stress that fixed costs are "fixed" within a certain range of activity or over a certain period of time. If enough time passes, all costs become variable.

Other common way is to use the classification of direct costs and indirect costs. The direct costs are associated with a particular cost object/production/operation. It is assignable or traceable. On the other hand, the indirect costs are non-assignable or non-traceable. They are concerned with the costs that cannot be easily and definitely traced to a object/production/operation.

Finally, the total costs are normally expressed as the average cost per unit, and also sometimes a marginal cost is considered to express the change in total cost value that arises when the quantity produced changes by one unit.

6.3 BRIEF INTRODUCTION ON RISK

The dictionary defines risk as an "exposure to the chance of injury or loss; a hazard or dangerous chance; the degree of probability of such loss; the type of loss, as life, fire, marine disaster, or earthquake, against which an insurance policy is drawn".

The importance of recognising the uncertainties in a geotechnical engineering project was primarily addressed by Casagrande (1965), who defended that the risks could not literally be calculated or quantified. Risk and reliability studies are multidisciplinary engineering fields requiring a solid foundation in one or several classical civil engineering disciplines, in addition to a thorough understanding of probability, risk and decision analyses (Hui & Weiji, 2008).

The probability theories were explained and explored with the reliability analyses performed in the previous chapters, stressing that in any area it is impossible to quantify all the uncertainties involved, therefore, the concept of probability of failure is simply a measure for comparison and not a real measure of the probability of collapse. The risk analyses' concept is to identify potential problems (risk) ahead of time, before these were to pose negative impacts.

The task of risk-based analysis is to combine the variability of the inputs, based on knowledge of how the system operates, to obtain estimates of the variability of outputs. As well, it is necessary to understand how to manage the risk involved. It is important to choose a cost-effective approach (the elimination of the risk should not cost more than the consequences of that risk, sometimes one should accept the risk instead of eliminate it).

For these reasons, decision analyses (normally using different types of tree analyses) is also a very important part of the process. As presented in Chapter 2 review, these processes include three types of nodes: (1) decision node, which represents the option of exploring or not; (2) chance node, which represent the outcome of the exploration; and (3) utility node, which represents the cost associated with the exploration – see also the following example.

According to the literature, there are different definitions for risk (see Baecher, 1981 and Vanmarcke & Bohnenblust, 1982). The simplest form to express risk is:

$$Risk = \mathrm{P}[Ev] \times C \tag{6.1}$$

However, an undesirable event (Ev) can lead to different consequences (C) and in this case vulnerability levels are associated (Einstein, 1997), and risk is defined as:

$$Risk = \mathrm{P}[Ev] \times Vulnerability\ of\ Ev\ (= \mathrm{P}[C|Ev]) \times Utility\ of\ C \tag{6.2}$$

The utility of the consequences is very often expressed in monetary values, but they are also sometimes used to express effects associated with other types of consequences such as environmental or social. According to Sousa (2010), the typical utility function for monetary values will depend on the decision maker's risk preference (neutral, risk averse, or risk prone). It is stated in Sousa’s work that, “although a decision maker maybe risk prone for positive values (i.e. gain or profit), he/she is normally risk averse for negative values such as costs”.

Consider a "classic" risk analysis technique, through a simple example. An example, from Sousa (2010) dissertation, where an engineer is faced with the choice of two different construction strategies for a tunnel. Analysing one section of the tunnel (section 1 - full face excavation with nominal support) it is considered the following data/information:

- prior geological states (state variables) for tunnel section 1 are presented in Table 6.1;
- the construction strategies (decision variables) and associated costs for section 1 are shown in Table 6.2;
- probability of failure given the construction strategy and the geological state, *i.e.* the vulnerabilities, are presented in Table 6.3.

- and the consequences (utilities) associated with failure are presented in Table 6.4.

The cost of the construction of the tunnel's section 1 and the associated risk, was obtained by Sousa (2010), considering this section independently from others and using the probabilistic model and decision trees. Figure 6.4 shows the decision tree for section 1 of the tunnel.

Table 6.1 – Prior geotechnical state for tunnel section 1[1]

Geotechnical states	*Probability*
G1	0.40
G2	0.60

Table 6.2 – Construction strategy costs [1]

Construction strategy	$U = -cost$
CS1	–15
CS2	–10

Table 6.3 – Probability of failure given construction strategy and geological state (vulnerability)[1]

	CS1		CS2	
	G1	G2	G1	G2
Failure	0.01	0.001	0.10	0.005
No failure	0.99	0.999	0.90	0.995

Table 6.4 – Consequences of Failure (Utilities)[1]

	CS1		CS2	
	G1	G2	G1	G2
Failure	–35	–25	–90	–70
No failure	0	0	0	0

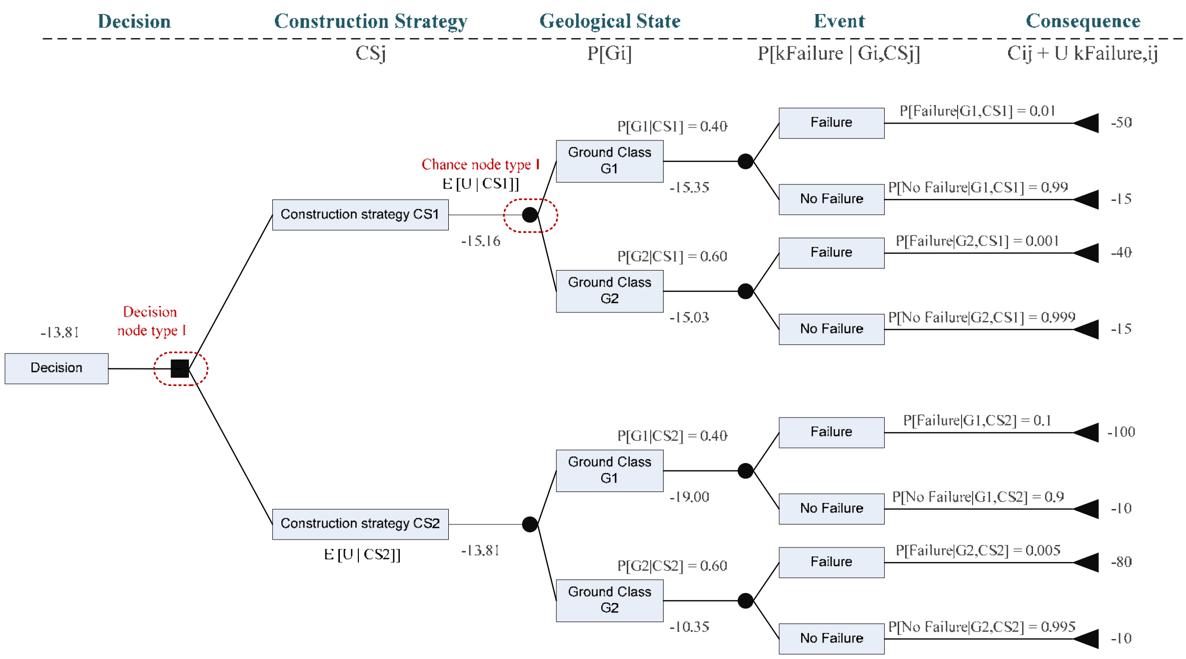


Figure 6.4 – Representation of the decision tree for a risk-based decision analysis, example of a tunnel - section 1 (adapted from Sousa, 2010)

[1] Data from Sousa (2010)

Note this:

- The construction strategies do not necessarily imply construction methods; strategies can refer to the same construction method, but in this case the construction strategies refer to the same construction method.
- In this example, the engineer is worried about failure of the face of the tunnel during construction. It is assumed that there is only one mode of failure.

At the decision node type I, the maximum expected utility over all construction strategies is computed from eq.(6.3). Chance nodes type I show the expected utilities for a given construction strategy, and are computed from eq. (6.4) and the expected utility of the tunnel section is computed from eq.(6.5).

$$max_{j=1}^{l} = \{E[U|CSj]\} \tag{6.3}$$

$$E[U|CSj] = \sum_{i=1}^{n}\left[\mathrm{P}[Gi] \times \sum_{k=1}^{m}[\mathrm{P}[kFailure|Gi,CSj] \times (Cij + U\ kF,ij)]\right] \tag{6.4}$$

$$E[U] = max_{j=1}^{l}\left\{\sum_{i=1}^{n}\left[\mathrm{P}[Gi] \times \sum_{k=1}^{m}[\mathrm{P}[kFailure|Gi,CSj] \times (Cij + U\ kF,ij)]\right]\right\} \tag{6.5}$$

Where, Cij represents the cost of CSj and Gi, CS is the construction strategy, G represents the geologic state, U is the utility, $U\,kF,ij$ is the utility associated with $Failure$ mode k, in Gi with CSj, $\mathrm{P}[kFailure|Gi,CSj]$ represents the probability of $Failure$ mode k, in Gi with CSj (k=1 means failure, k=2 means no failure).

Decisions are then made regarding the optimal construction strategy(s) based on expected values of utility given the uncertain geology and possible failure mode. In the example above, the maximum utility value is (–13.81).

These tools are being applied to validate claims for safety or demonstrate the need for further improvement, and used to support management decision-making, forming the new area of risk management (Whitman, 1984; Staveren & Knoeff, 2004).

6.4 COST-RELIABILITY-RISK ANALYSES

It is the main goal to perform costs, reliability and risk analysis of an axial pile, based on statistical data. It allows the estimation of the cost associated with a certain reliability level, reflecting different types of costs, the uncertainties of the variables in the calculation process and the risk of failure.

6.4.1 Description of the problem

In geotechnical engineering, before any design, it is necessary to conduct prospection/exploration works (geological and geotechnical characterisation) and to study the models that better reflect the geotechnical site and the type of construction work in hands (Ordem dos Engenheiros, 2004). In general, geotechnical engineers interact with a considerable volume of soil; therefore, the knowledge of the entire soil mass is obviously impossible and financially impracticable. The selection of the location of the boring holes, volume of soil tested and amount of research conducted, depend significantly on the level of knowledge and experience of the engineers involved, but also, they depend greatly on the time and financial constraints imposed by the project's owner.

For these reasons, the geotechnical projects are more vulnerable to risk, since the engineers only get to know a small, or very small, part of the soil mass. But, it can be said that, up to a certain level, more information about the soil would contribute to a higher degree of accuracy and reliability in the designs of engineering solutions (Goldsworthy *et al.*, 2004a,b; Ching & Phoon, 2012). The quantity and/or quality of the geotechnical characterisation should be proportional and compatible with the complexity and scale of the construction work, minimising the risk of undesirable events along the construction phase but also during its life cycle. However, and as concluded throughout this dissertation's examples (Chapters 4 and 5), the way an engineer carries the design also has a very important role in the final results of probability of failure, reliability and risk. This was also concluded in Zhang *et al.* (2009b), Honjo *et al.* (2010a) and Teixeira *et al.* (2011a, 2012d).

Hence, also the design method and/or its validation have to be considered and be proportional and compatible with the complexity and scale of the construction work.

Based on these statements, the follow up section will perform cost-reliability-risk analyses, using the data of Chapter 4 (experimental site of FEUP, ~120 m^2) and assuming that 4 bored piles need to be designed and installed. The lengths 4 m, 6 m and 8 m, and the diameters 0.4 m, 0.6 m and 0.8 m, will be studied. The piles should withstand a total load of 800 kN each (50% permanent load and 50% variable load).

It will be considered that the distance between piles is sufficient and the capacity of a pile will not be influenced by the nearby pile. Also, the worries about failure are concerned to pile failure alone (geotechnical / soil failure), and this is assumed to be the only mode of failure.

For each case studied, three components are analysed and computed in order to perform the cost-reliability-risk analyses:

- evaluation of the investigation costs (I$), including soil testing, samples and load tests;
- evaluation of the pile(s) construction cost (C$) and its reliability ($pf$ - refer to Chapter 3);
- and finally the evaluation of the consequence(s) of failure (F$), in order to evaluate risk.

According to Robertson (1998) the risk level associated to a geotechnical structure and subsequent site investigation plan correlates as presented in Figure 6.5. The application example presented is pertained to an experimental site; the risks of failure are low. However, in order to assess the influence of different consequence(s) of failure levels, different risk levels will be considered and studied.

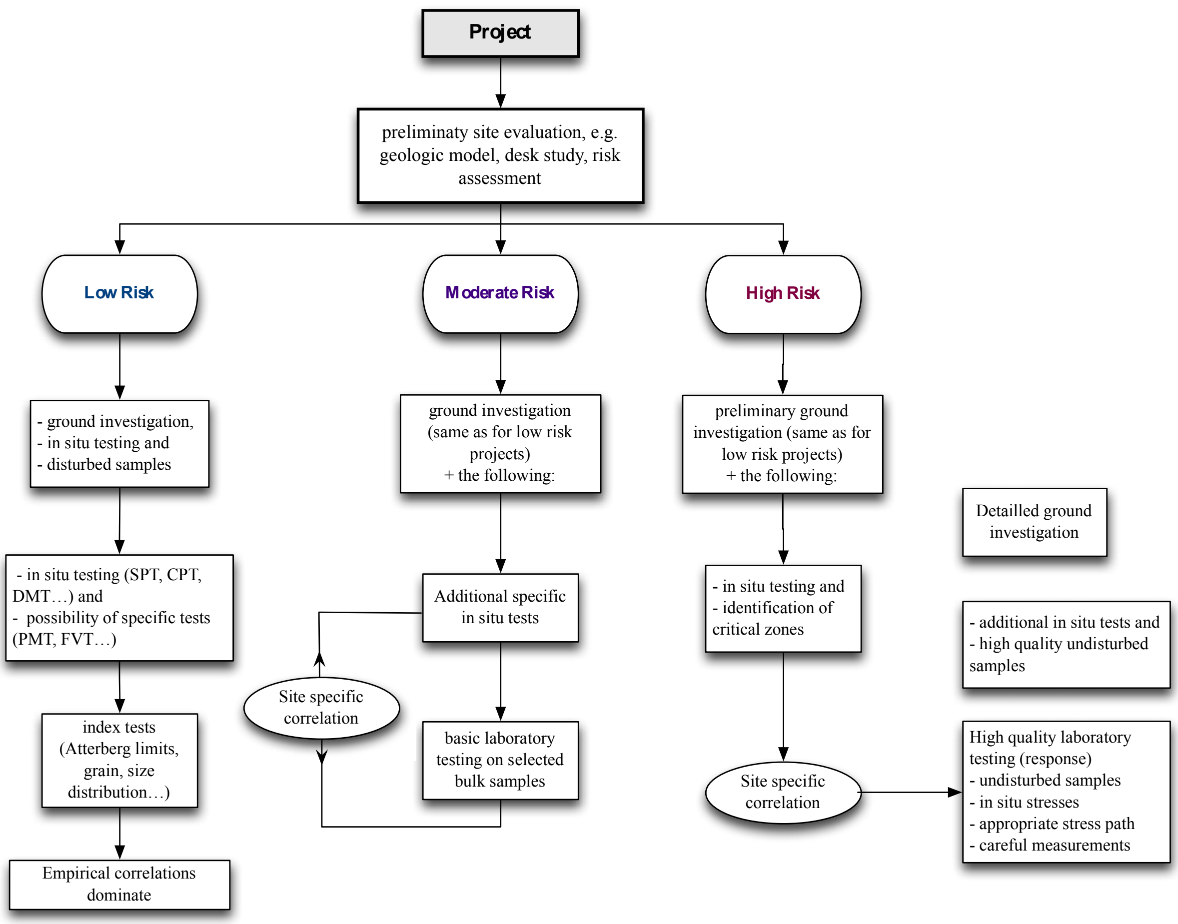


Figure 6.5 – Risk levels to consider in geotechnical engineering, adapted from Robertson (1998)

So, in order to carry out the cost-reliability-risk analyses with this application example, two main uncertainties are considered and studied. They are the amount and reliability of the soil investigation and reliability of the design method (resistance evaluation with empirical formulas or based on load tests). Both depend on the geological state and whether or not the engineer has the financial support to explore these choices; therefore, the first step is to define the strategy to approach the pile foundation design problem.

The following scenarios (S) are considered:

- S1 → perform just one SPT and get additional soil samples, in this case pile resistance will be based on empirical formulas;
- S2 → perform more than one SPT and possibly get some additional soil samples; as for S1, the pile resistance will be based on empirical formulas;
- S3 → perform one pile load test and maybe some SPT or soil samples for correlation of pile resistance results.

Thus, a tree is used for the analysis of these strategies/scenarios. The following Figure 6.6 presents the different case scenarios, according to the recommendations and risk levels previously discussed.

For each case scenario a graph will be drawn depicting the "Investment costs" versus "Expected probability of failure". To achieve that information it is necessary that the costs (I$, C$, F$) and reliability (*pf*) are evaluated. Finally, the risk (consequences of failure, F$) will be added to the plotted results and the final outcomes will be compared and discussed. The cases studied are depicted in Table 6.5.

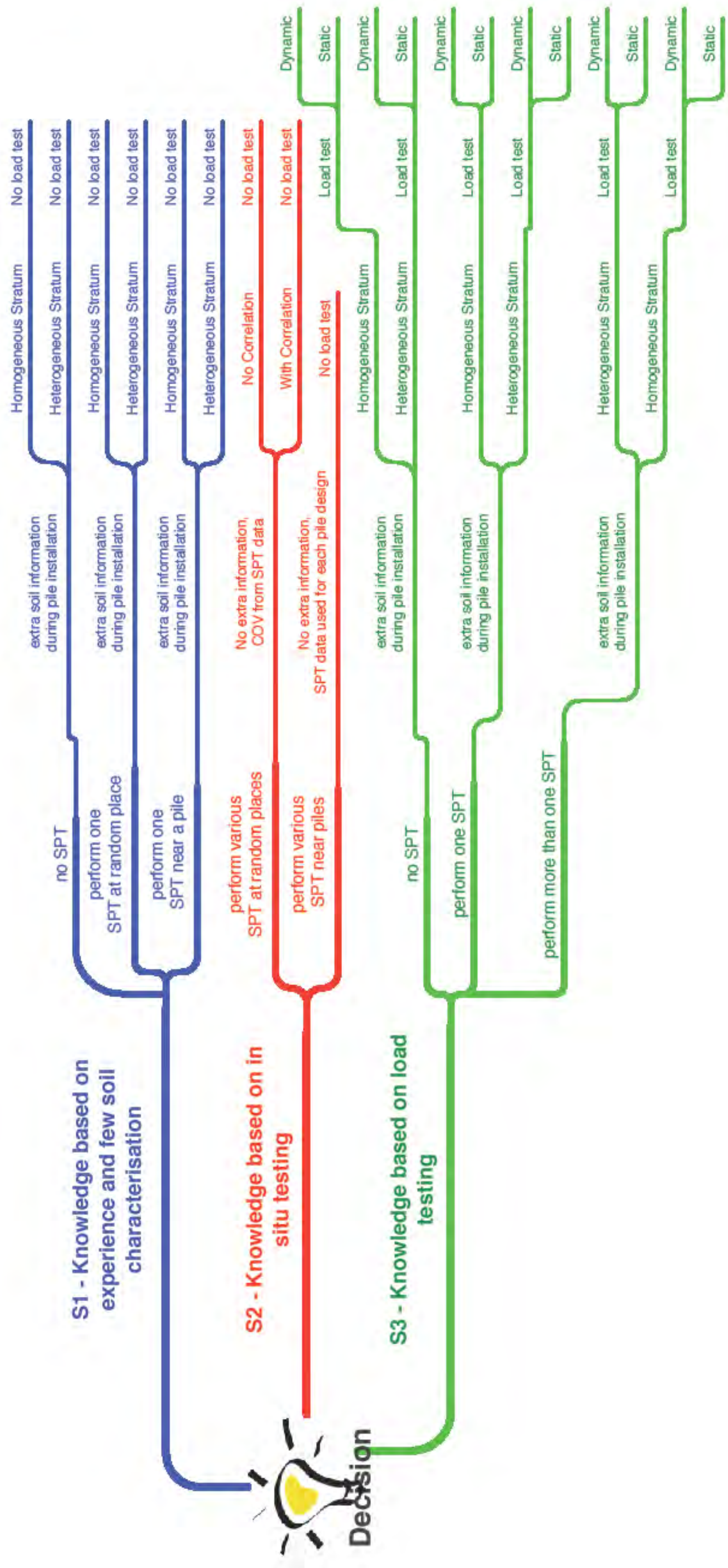


Figure 6.6 – Representation of the case scenarios S1, S2 and S3 for the example under study (Case study 1 – FEUP experimental site)

Table 6.5 – Case scenarios studied for cost-reliability-risk analysis (Case study 1 – FEUP experimental site)

Strategy	Case	Uncertainty level to consider to determine pile resistance
S1*	A#	A low COV will be assumed for the pile installed near the SPT profile, and a high COV for the other 3 piles.
	B#	A low COV will be assumed for the pile installed near the SPT profile, and a medium COV for the other 3 piles.
	C#	A low COV will be assumed for the pile installed near the SPT profile, and a very high COV for the other 3 piles.
	D#	Medium to high COV adopted to design all piles.
	E#	Low to medium COV adopted to design all piles.
	F#	High to very high COV adopted to design all piles.
S2*	A#	Very low COV for all piles.
	B##	COV calculated based on SPT results (with no spatial correlation, conservative choice)
	C##	COV calculated based on SPT results (with consideration of spatial correlation, less conservative choice)
S3**	A#	Low COV assumed for the pile tested and a high COV for the other 3 piles resistance.
	B#	Low COV assumed for the pile tested and a medium to high COV for the other 3 piles resistance.
	C#	Low COV assumed for the pile tested and a very high COV for the other 3 piles resistance.
	D#	Low COV assumed for the pile tested and a medium COV for the other 3 piles resistance.
	E#	Low COV assumed for the pile tested and a medium to low COV for the other 3 piles resistance.
	F#	Medium COV assumed for the pile tested and a high COV for the other 3 piles resistance.
	G#	Medium COV assumed for the pile tested and a medium to high COV for the other 3 piles resistance.
	H#	Medium COV assumed for the pile tested and a very high COV for the other 3 piles resistance.
	I#	Medium COV assumed for the pile tested and a medium COV for the other 3 piles resistance.
	J#	Medium COV assumed for the pile tested and a medium to low COV for the other 3 piles resistance.

* COV concerning the N value from SPT, to use in empirical predictions of pile resistance
** COV concerning the results of the load test, to determine pile resistance
# COV values from literature (Table 6.7)
COV values computed

6.4.2 Costs and risk evaluation

Concerning the costs for design and construction of the pile foundations, they will be divided into two categories, as previously referred, the investigation costs and construction costs. The investigation costs will comprise:

- the soil investigation costs, that depend on the type of soil, its variability and volume to analyse (*in situ* tests and samples);
- the costs of evaluating the resistance, such as with static load test or other.

There are different types of pile load tests, which provide different levels of information. These include preliminary static load tests (load-settlement behaviour evaluation of a test pile), static load test on working piles (working pile tested to the design verification load plus 50% of the safe working load) and dynamic load tests (working pile tested, does not provide direct measures, but it allows predictions – cheaper and quicker). When design is on the basis of calculation methods, especially empirical based, static load tests can reduce considerably the uncertainty and subsequent risk control and cost savings (Hajduk *et al.*, 2004). Also, load test can confirm the variability of the ground conditions across the site.

The use or performance of such load tests should be assessed not only on the basis of any regulatory requirements (codes and standards), but also in the level of risk or uncertainty posed by the site conditions, together with the potential benefits that may be derived in terms of more efficient desings (Hachich & Santos, 2006).

The execution/construction costs include:

- the materials supply (concrete and steel) and
- construction of the pile designed (length and diameter).

The costs are expressed in terms of monetary units (MU). Their values were collected from a few Portuguese companies that perform this type of works, and are relative to the fiscal years of 2009 and 2010 – Table 6.6

For each case, a mean values of the referred intervals in Table 6.6 are assumed. The fixed costs are equal in all case scenarios; therefore, they were not included in the final cost (Table 6.6b).

The risk or consequence(s) evaluation is often quantified in terms of costs and delays, however delays (time) can also be quantified as a monetary cost (although it depends on many factors). Numerous sources of risk can be considered, depending on the project and/or part of the project that is being analysed. Namely, they can be related to the geotechnical conditions of the specific site, to the construction technique(s), environmental issues, materials and the construction itself, among others (Rosa *et al.*, 2012). In other words, the risk is a measure of the uncertainties and how, or in what way, can they be accepted.

As such, their values are assumed as following:

- Risk level 1 = 10,000 MU (low);
- Risk level 2 = 50,000 MU (medium);
- Risk level 3 = 200,000 MU (high).

Table 6.6 – Costs evaluation, in terms of monetary units (MU)

a) Soil investigation costs

Costs	Soil investigation		
	Samples (ϕ101mm)	Undisturbed samples	SPT
One unit	40-60 MU/meter	70-90 MU/unit	22-25 MU/unit
Machinery	1000-1500 MU		
Geotechnical Report	500-1500 MU		

b) Construction costs

Construction Costs	Bored piles (length < 12m)		
	ϕ 400mm	ϕ 600mm	ϕ 800mm
Construction	~21 MU/meter	~32 MU/meter	~37 MU/meter
Concrete (C30/37) supply	~10 MU/meter	~20 MU/meter	~35 MU/meter
Steel (A500) supply and casting (~90kg/m^3)	~15 MU/meter	~25 MU/meter	~53 MU/meter
TOTAL	40-55 MU/meter	80-95 MU/meter	120-150 MU/meter
Fixed costs	Machinery, workmanship, transportations (...)		

c) Pile testing costs

Costs	Pile load tests (LT)			
	LT1	LT2	LT3	LT4
Performance of the test	1,000 MU	5,000 MU	10,000 MU	50,000 MU

6.4.3 Reliability evaluation

Regarding the probability of failure, different combinations of COV need to be computed for each case scenario (S1, S2 or S3), see Table 6.7. The “Very low”, “Low”, “Low-meidum” and “Medium” COV values mainly concern measurement and statistical errors.

Table 6.7 – Coefficients of variation (COV) categories (literature recommendations)

	Soil
Very low	2%
Low	5%
Low-medium	7%
Medium	10%
Medium-high	15%
High	20%
Very high	30%

Lengths 4 m, 6 m and 8 m, and diameters 0.4 m, 0.6 m and 0.8 m were considered for the pile foundations. For each dimension combination (3 x 3 = 9 combinations) and each soil COV (assumed from literature or calculated), the probabilities of failure (reliability) were computed based on the methodology preciously described in Chapter 3. In order to achieve the following results, a performance function like shown in eq.(3.2) (Chapter 3) is used, and the following uncertainties considered:

- the model error (Table 3.3 – SHB model);
- the soil variability (Table 4.7 – SPT-based, Chapter 4);
- the actions' uncertainties (Table 3.4 – JCSS recommendations).

The results of probability of failure for S1 and S2 computations are presented in Table 6.9 (assumed COV values) and Table 6.10 (SPT-based COV values). Their graphical representations are in Figure 6.7 and Figure 6.8, respectively. For S3 the probability of failure values are presented in Table 6.11 and depicted in Figure 6.9.

The Figures 6.7 and 6.8 present agreement of the results. The calculated COV and consequent probability of failure falls within the intervals of COV assumed based on literature recommendations. This behaviour was predictable and was also confirmed in Chapter 4 computations. Once again it is clearly detected a linear relationship between the length of the pile and the probability of failure (log-scale).

Bellow the Figures 6.7 and 6.8, the Table 6.8 depicts the costs of each pile dimension. Also, the reliability graphs depict the derided interval for probability of failure (10^{-4} and 10^{-2}). From these two sources of information, reliability graphs and costs, it is possible to understand that adequate reliability levels can be achieved with (D=8m;B=0.6m) or (D=6m;B=0.8m), with correspondent costs of 752 MU/pile and 924 MU/pile. For a set of 4 piles (total of 3008 MU or 3696 MU → +23%), if chosen the dimensions of (D=8m;B=0.6m), it would correspond to a significant saving on construction costs for the same level of reliability.

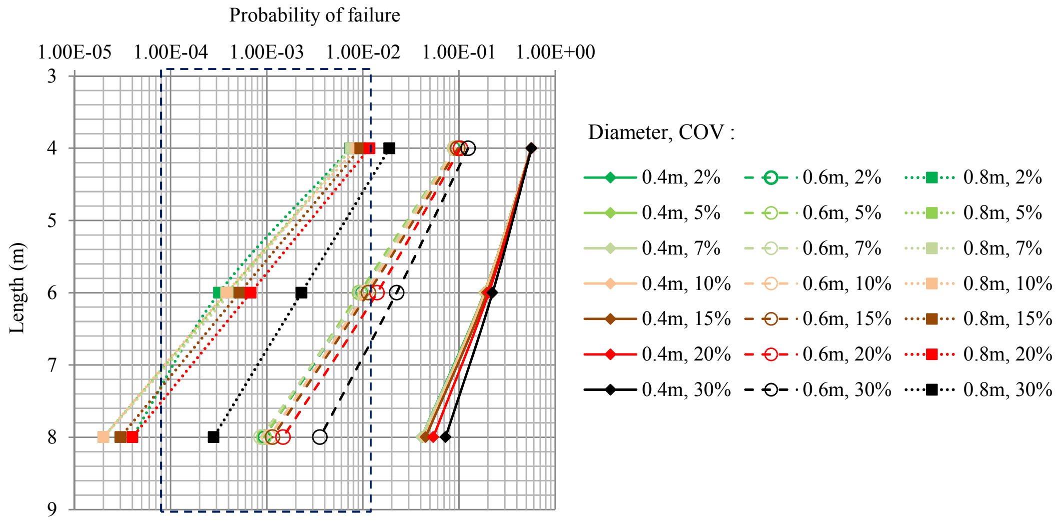


Figure 6.7 – Probabilities of failure (log-scale), using different COV, for each case and each pile (Case study 1 – FEUP experimental site)

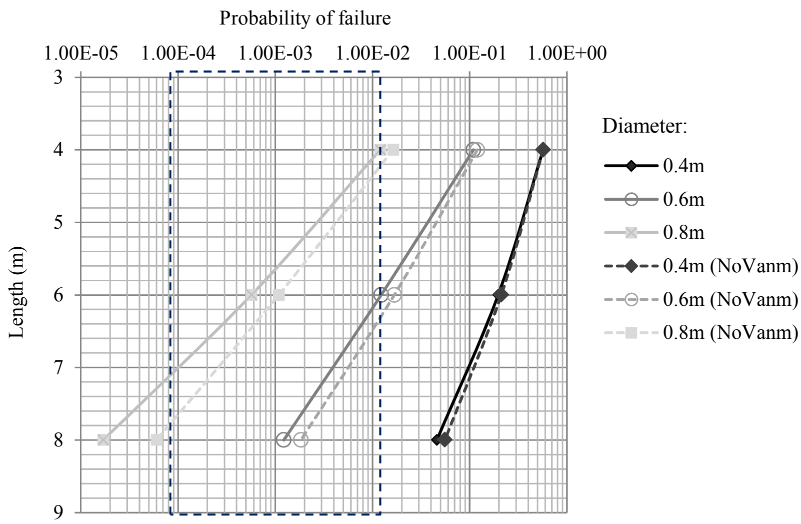


Figure 6.8 – Probabilities of failure (log-scale), using SPT-based calculated COV, for each case and each pile (Case study 1 – FEUP experimental site)

Table 6.8 – Construction costs of the bored piles MU/pile

Length	B = 0.4 m	B = 0.6 m	B = 0.8 m
4 m	200	376	616
6 m	400	564	924
8 m	400	752	1232

For a lower importance construction, the dimensions (D=8m;B=0.4m) or (D=6m;B=0.6m) might also be adequate, leading to the same kind of conclusion, the (D=8m;B=0.4m) would achieve an equivalent reliability for less construction costs (400x4 =1600 MU or 564x4=2256 MU → +41%).

Table 6.9 – Probabilities of failure, using different COV, for each case and each pile (Case study 1 – FEUP experimental site)

a) Very low COV for soil, 2%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.56520	0.84
4	0.6		0.09019	1.34
4	0.8		0.00741	2.44
6	0.4		0.18272	0.91
6	0.6	2%	0.00967	2.34
6	0.8		0.00032	3.41
8	0.4		0.04081	1.74
8	0.6		0.00094	3.11
8	0.8		0.00004	3.94

b) Low COV for soil, 5%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.56447	0.84
4	0.6		0.09043	1.34
4	0.8		0.00769	2.42
6	0.4		0.18337	0.90
6	0.6	5%	0.00922	2.36
6	0.8		0.00039	3.36
8	0.4		0.04209	1.73
8	0.6		0.00094	3.11
8	0.8		0.00002	4.11

c) Low-medium COV for soil, 7%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.56426	0.84
4	0.6		0.09016	1.34
4	0.8		0.00750	2.43
6	0.4		0.18744	0.89
6	0.6	7%	0.00979	2.33
6	0.8		0.00042	3.34
8	0.4		0.04085	1.74
8	0.6		0.00087	3.13
8	0.8		0.00002	4.11

d) Medium COV for soil, 10%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.56441	0.84
4	0.6		0.09281	1.32
4	0.8		0.00857	2.38
6	0.4		0.18504	0.90
6	0.6	10%	0.01068	2.30
6	0.8		0.00038	3.37
8	0.4		0.04402	1.71
8	0.6		0.00106	3.07
8	0.8		0.00002	4.11

e) Medium-high COV for soil, 15%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.56582	0.83
4	0.6		0.09769	1.29
4	0.8		0.00943	2.35
6	0.4		0.19592	0.86
6	0.6	15%	0.01143	2.28
6	0.8		0.00052	3.28
8	0.4		0.04501	1.70
8	0.6		0.00110	3.05
8	0.8		0.00003	4.01

f) High COV for soil, 20%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.57142	0.82
4	0.6		0.10362	1.26
4	0.8		0.01170	2.27
6	0.4		0.20382	0.83
6	0.6	20%	0.01408	2.20
6	0.8		0.00068	3.20
8	0.4		0.05400	1.61
8	0.6		0.00148	2.97
8	0.8		0.00004	3.94

g) Very high COV for soil, 30%

Length (m)	Diameter (m)	Soil COV considered	pf	β
4	0.4		0.56999	0.82
4	0.6		0.12442	1.15
4	0.8		0.01891	2.08
6	0.4		0.22383	0.76
6	0.6	30%	0.02247	2.01
6	0.8		0.00230	2.83
8	0.4		0.07248	1.46
8	0.6		0.00358	2.69
8	0.8		0.00028	3.45

Table 6.10 – Probabilities of failure, using SPT-based calculated COV, for each case and each pile (Case study 1 – FEUP experimental site)

a) Calculated COV for soil, considering spatial autocorrelation

Length (m)	Diameter (m)	Soil COV considered	*pf*	*β*
4	0.4	Calculated with autocorrelation	0.56945	0.83
4	0.6		0.10921	1.23
4	0.8		0.01205	2.26
6	0.4		0.19648	0.85
6	0.6		0.01230	2.25
6	0.8		0.00057	3.25
8	0.4		0.04592	1.69
8	0.6		0.00123	3.03
8	0.8		0.00002	4.14

b) Calculated COV for soil, NOT considering spatial autocorrelation (NoVanm)

Length (m)	Diameter (m)	Soil COV considered	*pf*	*β*
4	0.4	Calculated with NO autocorrelation (NoVanm)	0.57053	0.82
4	0.6		0.11997	1.18
4	0.8		0.01628	2.14
6	0.4		0.21115	0.80
6	0.6		0.01678	2.13
6	0.8		0.00109	3.07
8	0.4		0.05553	1.59
8	0.6		0.00184	2.90
8	0.8		0.00006	3.85

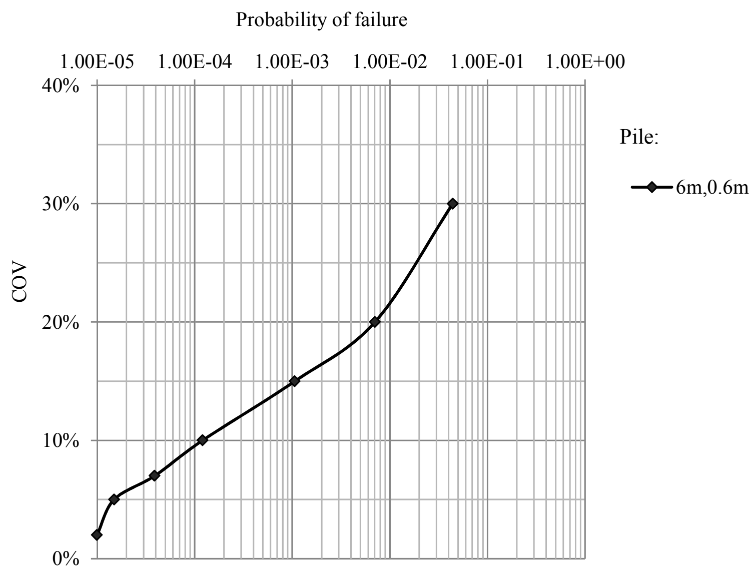


Figure 6.9 – Probabilities of failure (log-scale), using different COV for load test result (1350 kN) for a 6 m pile with 0.6 m diameter (Case study 1 – FEUP experimental site)

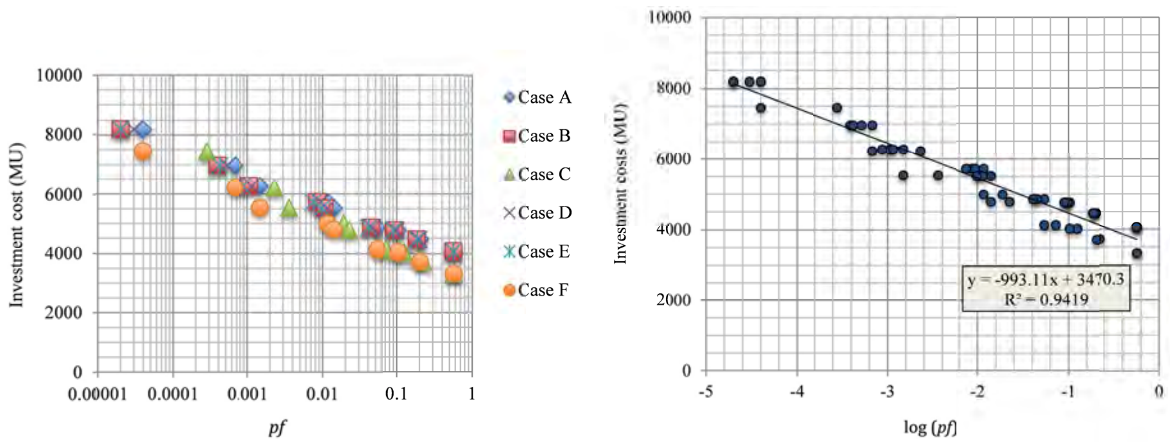
Table 6.11 – Probabilities of failure using different COV for load test result (1350 kN) for a 6 m pile with 0.6 m diameter (Case study 1 – FEUP experimental site)

Length (m)	Diameter (m)	COV		*pf*	*β*
6	0.6	Very low	2%	0.000010	4.26
6	0.6	Low	5%	0.000015	4.17
6	0.6	Low-medium	7%	0.000039	4.03
6	0.6	Medium	10%	0.000121	3.67
6	0.6	Medium-high	15%	0.001064	3.07
6	0.6	High	20%	0.007040	2.46
8	0.6	Very high	30%	0.044159	1.70

Concerning the load test reliability results (Table 6.11 and Figure 6.9), the capacity of 1350 kN was analysed for different COV values, depending on the type of test (more or less reliable) and on the soil information obtained in order to correlate the different piles' capacity. It is proved, as expected, that more reliability can be put into load tests results. Reliability results achieved with load test value (COV<10%) is considerably higher when compared with reliability results achieved with empirical tests (Figure 6.7 or 6.8). However, some care must be taken if the soil mass is not studied or investigated properly leading to wrong correlations between piles' capacity.

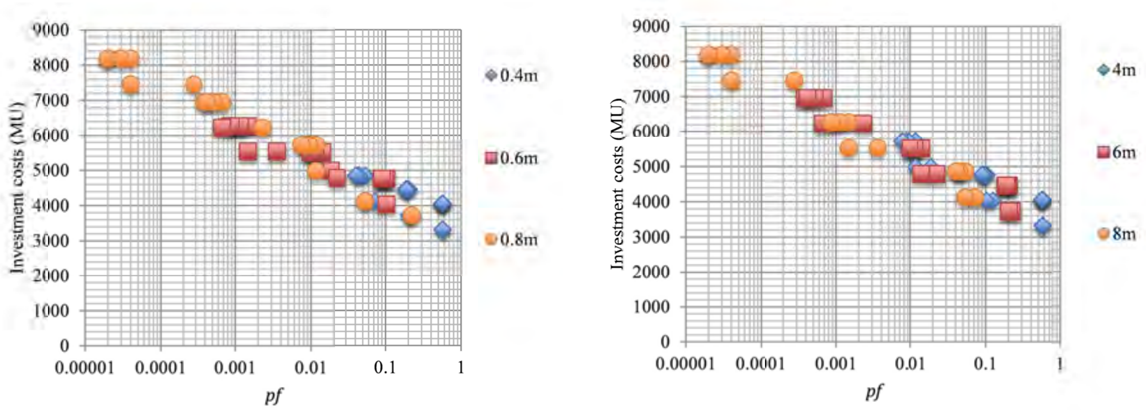
6.4.4 Results cost-reliability

The results are presented in the following figures. For each scenario (S1, S2, S3) plots are depicted showing "Investment Costs" (I$ + C$) versus "Expected probability of failure".



a) S1 per Case b) linear approximation

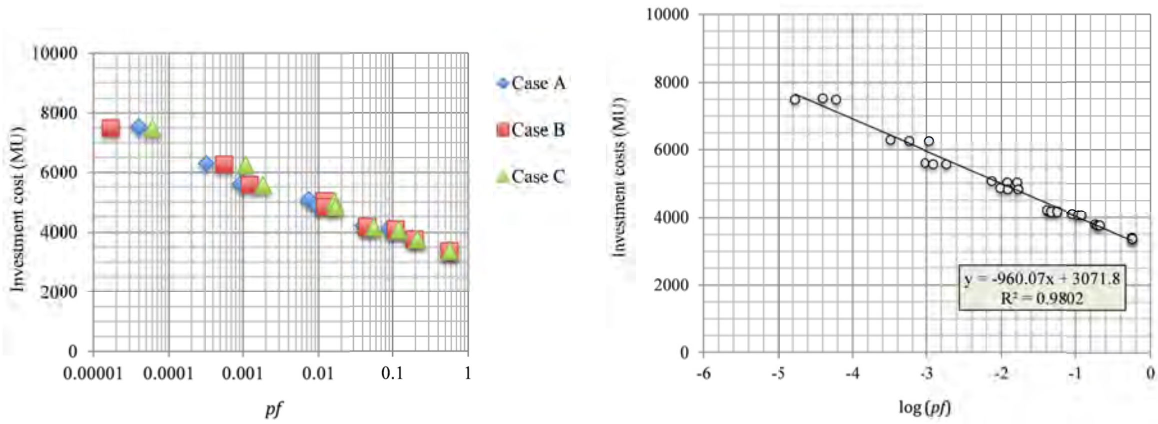
Figure 6.10 – Investment costs *versus* probability of failure (log-scale), using different COV (Table 6.5) for case scenario S1 (Case study 1 – FEUP experimental site)



a) S1 per diameter

b) S1 per length

Figure 6.11 – Investment costs *versus* probability of failure (log-scale), using different COV (Table 6.5) for case scenario S1 (Case study 1 – FEUP experimental site)



a)S2 per Case

b) linear approximation

Figure 6.12 – Investment costs *versus* probabilities of failure (log-scale), using different COV (Table 6.5) for case scenario S2 (Case study 1 – FEUP experimental site)

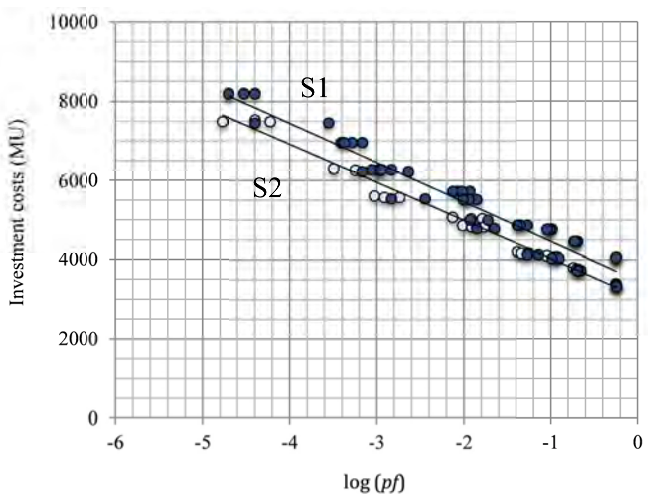
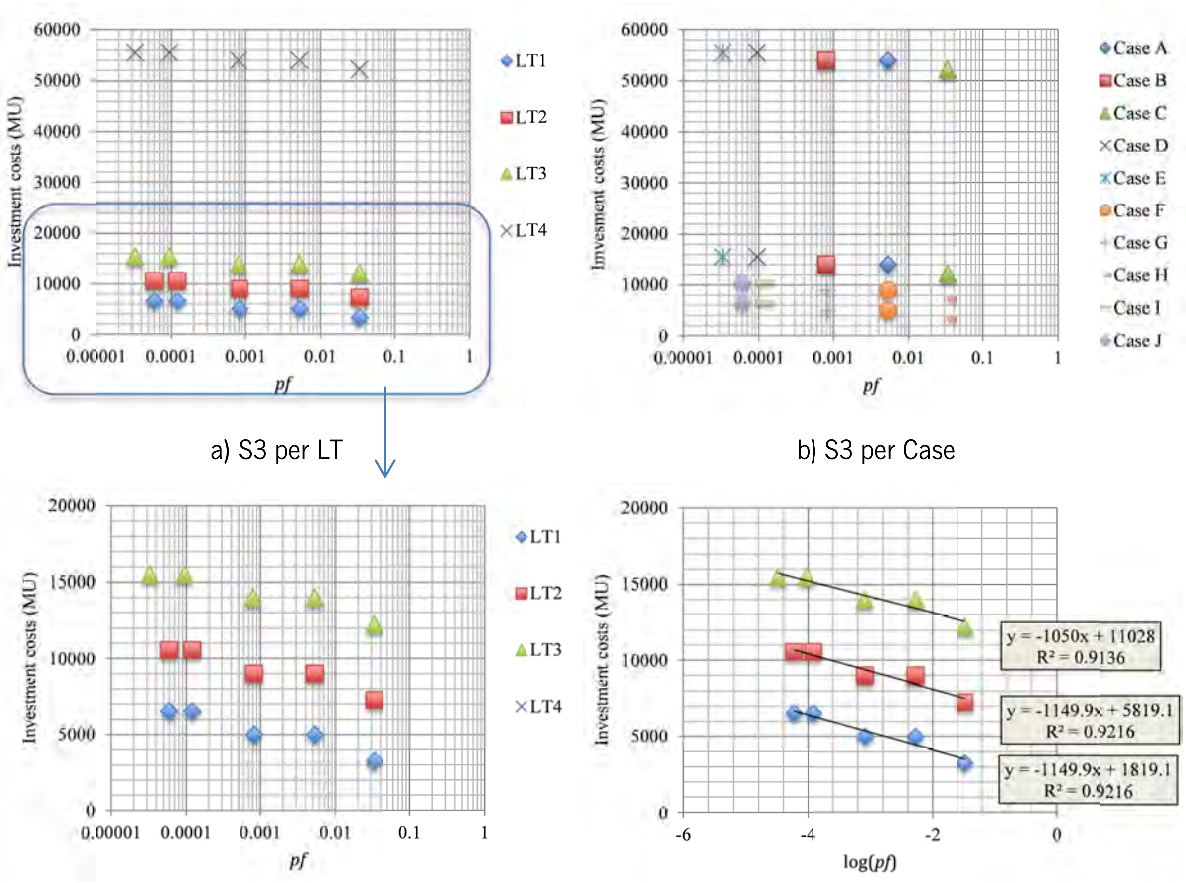


Figure 6.13 – Comparison between the results of case scenario S1 and S2 (Case study 1 – FEUP experimental site)



a) S3 per LT

b) S3 per Case

c) Zoom in (LT3, LT2, LT1)

d) linear approximations (LT3, LT2, LT1)

Figure 6.14 – Investment costs *versus* probabilities of failure (log-scale), using different COV (Table 6.5) for case scenario S3 (Case study 1 – FEUP experimental site)

The analysis of the results for case scenarios S1 and S2 yielded the following outcomes:

- Both cases present a linear relationship of the data, that are independent of the pile dimensions (length and diameter);
- This linear relationship agrees with previous recommendations, such as the empirical rates of failure for civil engineering facilities shown in Figure 2.10 – page 34).
- Results of case scenario S2 present a slight less variability (around the trend line) than S1, this is possibly due to a higher certainty in S2 data, based on more SPT tests;
- The difference between the investment costs for both case scenarios (sum of soil characterisation costs and construction costs) is not as great as expected. For any level of reliability the average increase is 500 MU. For a lower level of reliability (higher probability) it means an increase of approximately 16 %, as for a higher level of reliability (lower probability of failure) means an increase of approximately 7%.

These results are not a surprise, since other studies have also shown similar conclusions. As stated in Goldsworthy *et al.* (2004a,b) study: "The results illustrate a decreasing trend of total foundation cost for an increasing site investigation scope. The results also show that the cost of a foundation, excluding the penalty cost of failure, designed using an increased amount of knowledge regarding the site, does not always result in a less expensive foundation. However, all results suggest that a site investigation scheme with limited testing will result in a more expensive foundation, when the cost of possible foundation failure is included".

Concerning the results of case scenario S3, also trend lines have been drawn and curiously the y-interception coincides with the load test cost and the slope of all trends is the same. The LT3 was not included when zoom in was presented (Figure 6.14c,d) due to a very high cost of load test (50,000 MU) that was not included for the conclusions analyses. In spite of the investment cost in load tests being much higher in comparison with S1 or S2, these costs (pile testing) are normally easily recovered in terms of the total costs of foundations, especially if the project has a high risk associated to its failure. This can be understood from cost-reliability-risk results presented in next section.

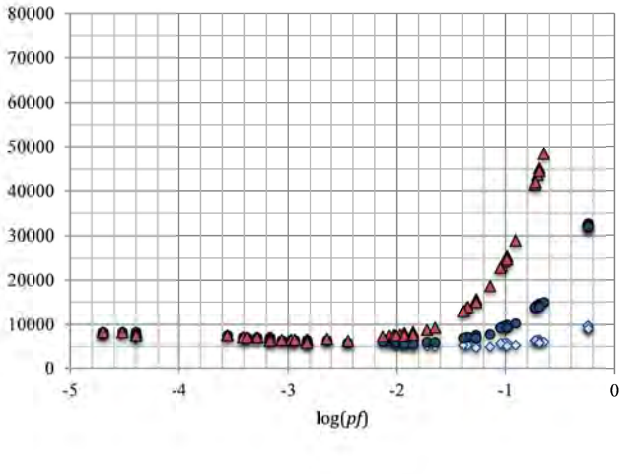
6.4.5 Results cost-reliability-risk

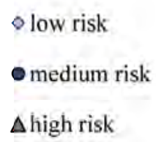
The results of the previous section cannot be properly compared without the consideration of the risk level (low, medium, high). Therefore, for each scenario (S1, S2, S3) plots are depicted next showing "Investment Costs + Risk" ($I\$ + C\$ + F\$ \times pf$) versus "Expected probability of failure".

An obvious exponential trend is detected, that has an increase proportional to the level of risk (Figures 6.15 and 6.16) and also proportional to the probability of failure. In all three case scenarios, the new data considering low risk level does not present a noticeable change to the original data (from previous section).

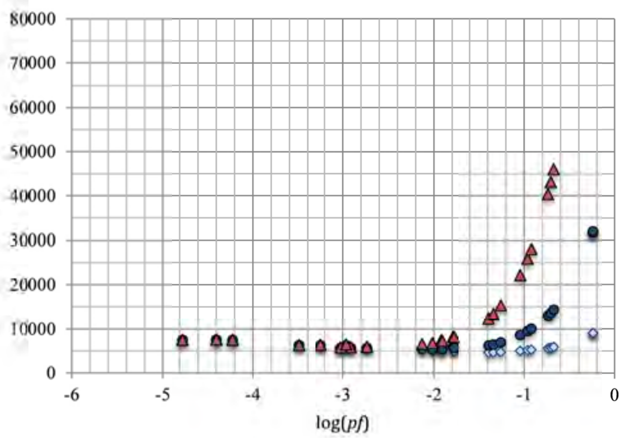
Again, the same behaviour is observed in S1 and S2 results (Figure 6.15) yielding no considerable differences between these two case scenarios, between these two decisions.

Finally, the Figure 6.17 presents the comparison of S1 or S2 with each S3-LT and Figure 6.18 present all case scenarios together. The usual values for the probability of failure are between 0.01 and 0.0001 (log -2 and -4) and a zoom in is presented in Figure 6.19 for examination.



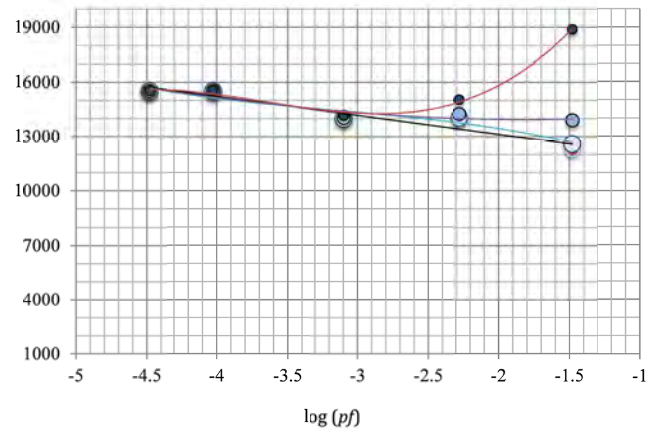


a) Case scenario S1

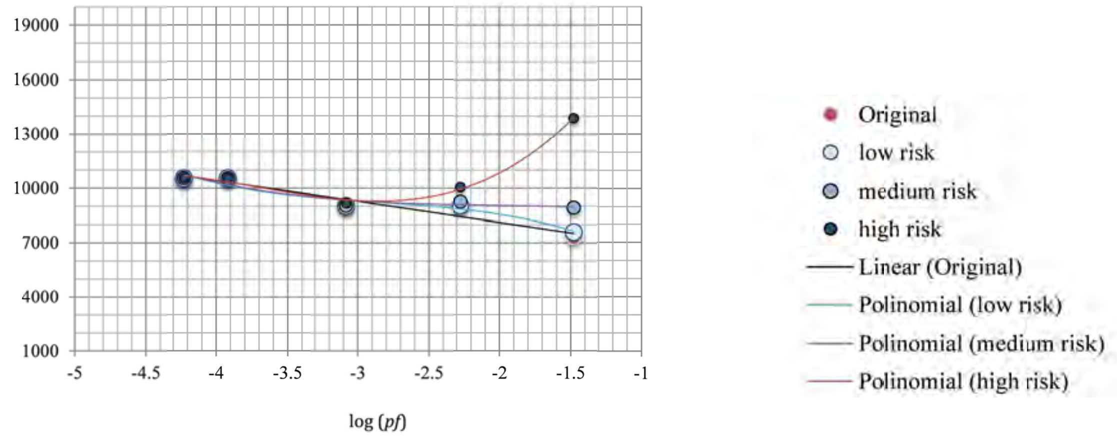


b) Case scenario S2

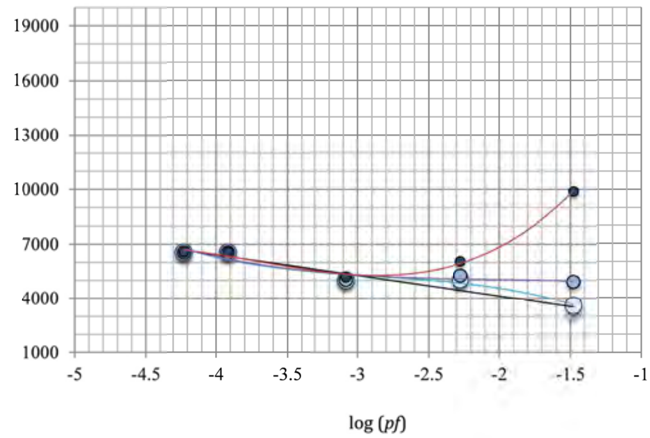
Figure 6.15 – Costs including risk (different levels) *versus* probabilities of failure (log-scale)
(Case study 1 – FEUP experimental site)



a) LT3

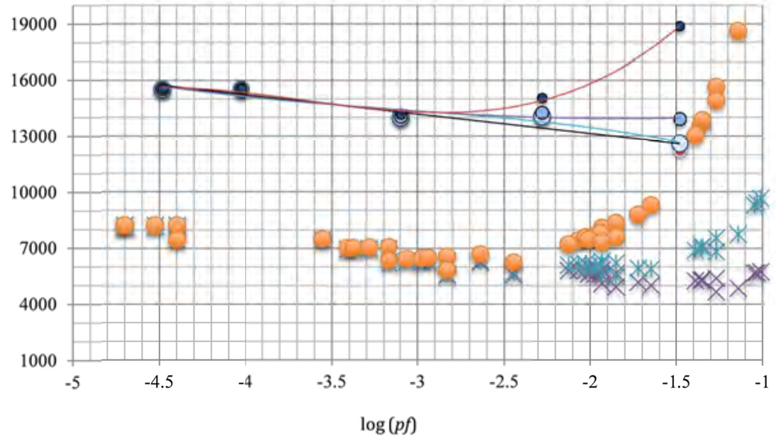


b) LT2

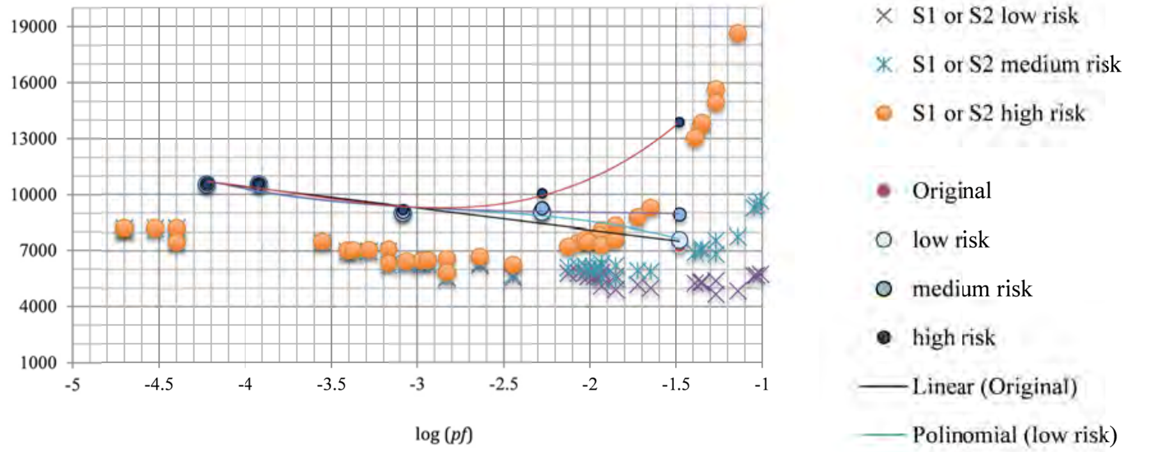


c) LT1

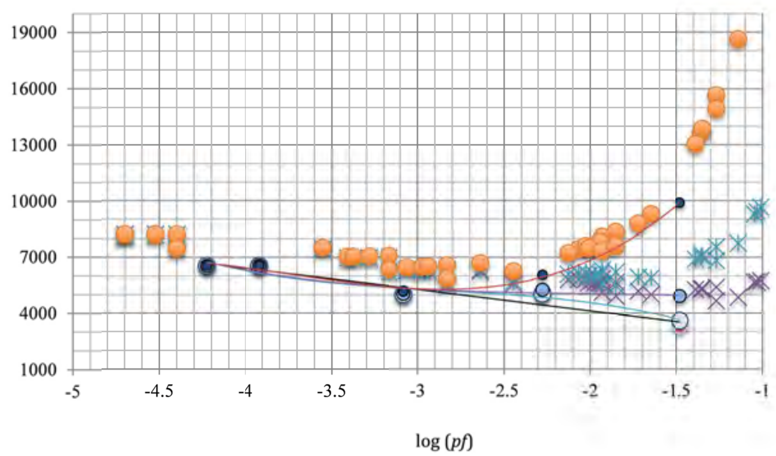
Figure 6.16 – Costs including risk (different levels) *versus* probabilities of failure (log-scale) for case scenario S3 (Case study 1 – FEUP experimental site)



a) LT3



b) LT2



c) LT1

Figure 6.17 – Costs including risk (different levels) *versus* probabilities of failure (log-scale) for comparison between each case scenario S3 and S1-S2 (Case study 1 – FEUP experimental site)

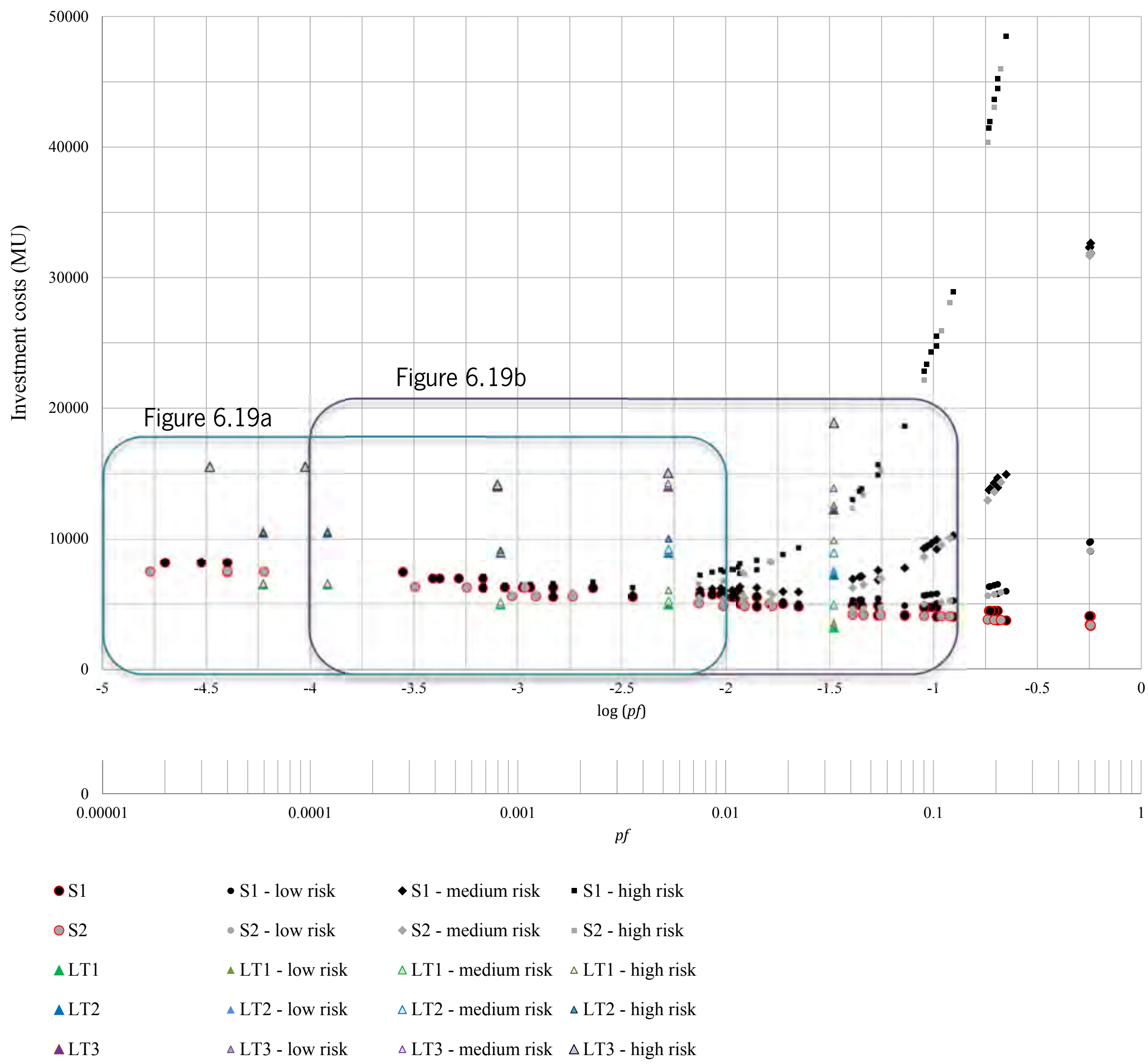
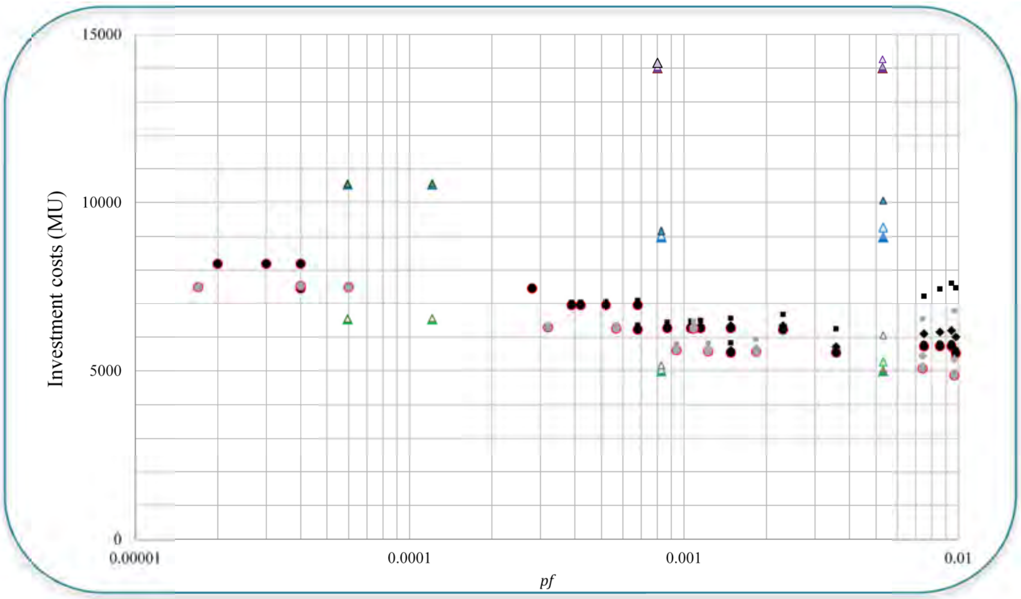
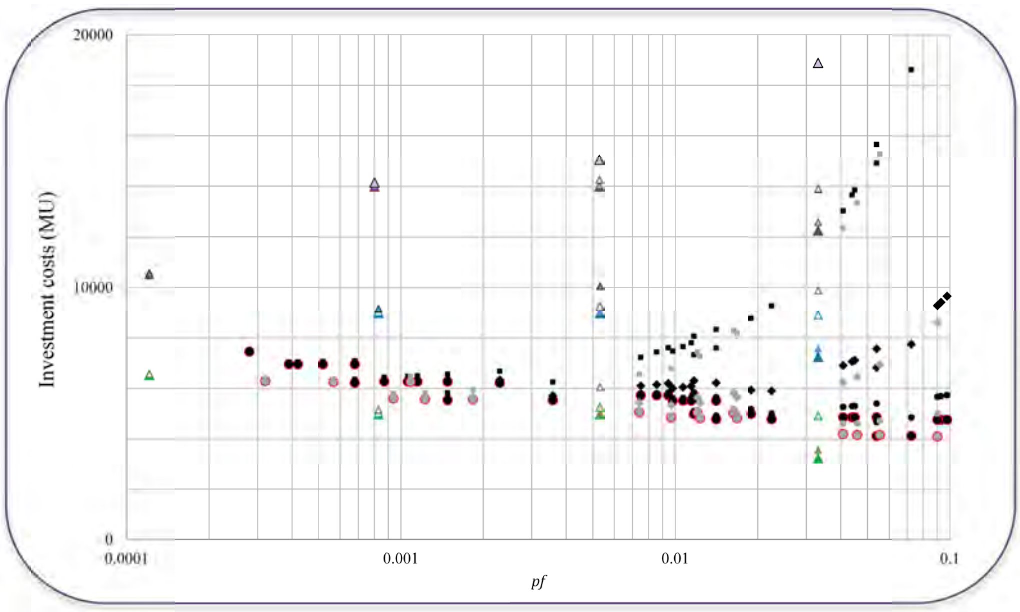


Figure 6.18 – Costs including risk (different levels) *versus* probabilities of failure (log-scale) for comparison between all case scenarios (Case study 1 – FEUP experimental site)



a)



b)

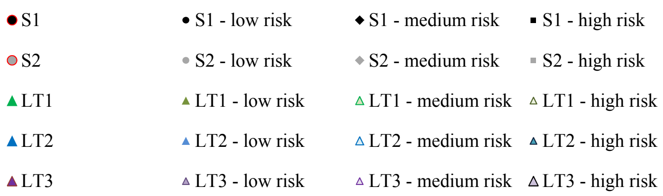


Figure 6.19 – ZOOM IN Figure 6.18

Note that case scenarios S1 and S2 are referred to the 9 combinations of pile dimensions and the case scenario S3 refers only to length 6 m and diameter 0.6 m. However some correlations can be done between the results.

Concluding, it is possible to understand that depending on the reliability level, degree of construction costs and also the level of risk, some strategies may be more advantageous than others. What seems at first a high investment can easily become a safety measure for the future.

Pile testing using static load test, as preliminary trial piles and/or working piles tests, can provide a number of benefits to the project (better understanding of the response of the foundation), forming a basis for the design, and allowing more efficient and less costly designs. Reduced uncertainty, or even reduced safety factors, can be applied when this information is available and studied. On the other hand, dynamic pile load tests can also be used to allow prediction of the pile performance, these tests are quicker and cheaper but care must be taken in the interpretation of the results.

6.5 CONCLUDING REMARKS

Risk analyses answer three basic questions (Robertson, 1998): “What can go wrong?”, “How likely is it?” and “What are the consequences?”. However, the risk is not the kind of knowledge that is easy to quantify or normalise or shared between engineers. Even when such knowledge is available, it is often not structured or offered in ready-to-use format, especially in geotechnical engineering where each project has its specific characteristics.

According to different studies there is a need for improved and informed decisions whereby the effects of risks (level of uncertainties) on the choice and construction of pile foundations are taken into considerations. The geotechnical engineering is one of the most interesting contexts to apply cost-reliability-risk studies and management. The uncertainties in a geotechnical project are of different sources and very broad. They include uncertainty associated with the random nature of the soil, the spatial variability and sometimes an inadequate characterisation of the geotechnical parameters. The reliability-based approach quantifies the penalty or benefit for different levels of information in terms of the required design conservatism (Gilbert & Gambino, 1999; Gilbert, 2003). Moreover, the management of the construction work together with observations and tests is essential to avoid risk and to alter the design if necessary (Abdrabbo & Gaaver, 2012).

Therefore, cost-benefit decisions about the need for an optimal amount of additional site information, such as soil tests, soil samples and/or load test, can be based on such analyses. The engineers (decision-makers) are then able to decide how to achieve a specified level of reliability with a combination of design conservatism and soil information on a project- specific basis.

The risk will generally reduce with increased investment in site investigation (Jaksa *et al.*, 2003; Staveren & Seters, 2004; Sushma, 2009). The potential reductions in construction risks and costs should be considered when budget is available. The cost of investigation should be weighed up against:

- the reduction in uncertainty concerning the ground conditions and corresponding reductions in construction risks (which may carry significant financial consequence);
- the potential savings that can be made in design (avoiding overdesign).

In order to analyse these issues, this chapter presents a cost-reliability-risk analysis procedure applied to a pile foundation. A case study (Case study 1 – FEUP, presented in Chapter 4) is provided to demonstrate its use both in the design of the pile(s) and the strategy of site investigation programs. One of the goals is to explain how to introduce statistical data to estimate the cost associated with certain security/reliability level, reflecting the uncertainties of the variables of the calculation process. This can assist engineers in understanding the relative benefit of the investments in geotechnical projects – correct assignment of resources for a cost-effective strategy.

In spite of all the research published concerning the consequential risk of limited site investigations (Jaksa, 2000; Jaksa *et al.*, 2003, 2005; Moh, 2004; Goldsworthy *et al.*, 2004a,b, 2005), the conclusions that "the practice of recommending lowest tender as the main criteria for site investigation should not be preferred but be discouraged" and that "the risk of a foundation failure is heavily dependent on the quantity and quality of information obtained from a geotechnical site investigation aimed at characterising the underlying soil conditions", are especially true when big volumes of soil need to be analysed. However, it is true to say that the extent and cost of site investigation should be such that the risk is at an established acceptable level to the designer and also comply to the accepted code of practice.

NHBC Foundation report (design guide, 2010) has performed a survey of current practice for low-rise housing in UK, where more than 450 individuals took part. The conclusions were that:

- The design approach generally used to determine acceptable pile working loads is the application of an overall (global) safety factor against ultimate failure, between 2 and 3.
- More than 60% of the respondents to the survey considered that insufficient ground investigation is generally undertaken for low-rise housing

The results achieved from cost-reliability-risk analyses of case study 1 presented in this chapter, demonstrated that the high variability of the design method (how pile capacity is determined), is also a very important factor for achieving reliability in the results and avoiding risks.

The previous two chapters have presented a high influence of the model uncertainty in probability of failure values, and this chapter has again demonstrated that a higher or lower soil variability does not influence that much the results (reliability level). Furthermore, when considering the risk of just using an empirical model for pile capacity or using a proof load test, it is demonstrated that:

- Since the model error is the one that mostly influence the reliability of the pile, the safety factors or the model used for design should always relate to the type of confirmation testing and with sufficient testing to prove the design. When higher design loads are allowed it presents a significant overall saving on the project. The cost of the testing becomes an insignificant cost compared to the potentially great benefit of increased pile loadings.
- Uncertainty should always be quantified and pondered by using both reliability data and expert judgement. The design method or its validation has to be proportional and compatible with the complexity and scale of the construction work. The same applies to the quantity and/or quality of the geotechnical characterisation, minimising the risk of undesirable events along the construction phase but also during its life cycle.

As conclusion, pile test should be carried out, especially when not enough confidence is put into the design method (e.g.: empirical models). It is true that they are sometimes costly and time consuming, and therefore not included in the project due to costs and time restrictions, even though they are a very useful verification tool. However, it is obvious that pile load tests are appropriate for achieving reliability and savings in larger construction projects, being less important on smaller ones.

As example, Frazier *et al.* (2002) study presents cases where the design stresses were considerably increased as a result of additional testing and use of remote dynamic pile testing equipment. This case proven 35% savings in the foundation costs due to a relatively minor amount of additional testing.

Geotechnical characterisation and testing are essential to identify and characterise the foundation, to adopt the most convenient solutions, have the necessary information for reliable design, minimising risks and allowing a time-saving and cost-effective construction work. By failing to prevent this, the project costs will necessarily reflect this mistakes and lack of information. However, also the hypothesis and theoretical models affect the cost and the time required for construction of pile foundations. Field observations (both soil and pile) are essential.

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Chapter 7

CONCLUSIONS

7.1 SUMMARY AND MAIN CONTRIBUTIONS

This dissertation introduces reliability concepts for geotechnical engineering in general and for pile foundations in particular, addressing essentially design aspects. The studies performed in this dissertation identify and clarify the significance of reliability-based methodologies and reliability-based approaches in the design of pile foundations.

For that purpose, in *Chapter 2*, reliability concepts have been reviewed with especial emphasis to geotechnical reliability and pile foundations design. In the past decades there has been a great effort from the geotechnical community to try to implement reliability theory to geotechnical engineering, not only in a theoretical way, but also trying to develop user-friendly tools for geotechnical practice (Tables 2.9 and 2.8; Phoon & Honjo, 2005; Phoon 2008a,b). The various studies in literature show these efforts, defending the reliability theory as an important tool for geotechnical engineering problems, especially for calibration of the reliability-based safety factors. These studies in literature present successful examples, supporting that the geotechnical design can be improved and based on reliability tools. The probabilistic methods, and particularly those based on Bayesian statistics, have proved very useful, internationally and in different contexts, to streamline the incorporation of all the uncertainties, by combining previous information (predictions of ultimate strength) with those derived from experimentation (load tests, for example), allowing a rational updating of the safety indicators.

To this subject, this dissertation aims to give further contribution to the application of reliability methods in axial pile design (vertical bearing capacity), considering probabilistic and statistical information of the variables involved. It is revealed that a reliability-based analysis does not imply significantly more efforts when compared with a traditional analysis. In fact, reliability analyses provide very useful tools for modelling the uncertainties (random variables). They quantify and give information about the variables that mostly influence the behaviour under study, allowing the determination of the possible responsible causes for adverse effects on the structure and risk control.

It is also intended to contribute to preventing the loss of intuitive understanding when applying these tools to design problems, which is an important issue in geotechnical engineering. Also, this dissertation is intended as an aid to pile design decision-makers in assessing the uncertainties associated with the random variables that mostly influence both the probability of failure and the behaviour of the pile.

Characterisation of uncertainty in geotechnical problems is a difficult task, and the values recommended in the literature often cannot be applied to a particular case under study due to high soil variability. Geotechnical practice always depends on the particular site. The engineer has to choose from a wide range of materials, parameters, concerning geology and geotechnical. For this reason, there is the need for more guidance to what is important.

Therefore, this dissertation originates an important contribution. These methodologies are not a current practice in geotechnical area and need some adaptions from structural to geotechnical environment. Additionally, the studies about pile foundation design methods, bearing capacity estimations or behaviour estimations, are wide and diverse. But there are still problems and huge uncertainties involved in these predictions/methods/estimations. These problems, namely the accuracy of the methods and models used and how to combine the existing tools used in practice with the reliability tools, need to be addressed for practical application of geotechnical reliability. It is also important to say that that the pile foundations' section of different design codes were identified to be the most problematic ones during code drafting.

Within reliability analyses (RA), RA level zero and RA level I are traditional approaches to design, while RA level II and RA level III are approaches commonly used for the evaluation of the probability of failure.

After analysing simple problems taking into account the variability, it has been concluded that the deterministic result is definitely not sufficient to infer about the safety or performance. It is necessary to analyse the influence of the variability of parameters for quantifying the probabilistic estimates. However, additional tools must be used to enable better decision making about the performance of the structure.

For a more consistent analysis, the most popular methods to evaluate the probability of failure are the first-order reliability method (FORM) and Monte Carlo simulations (MCS), which correspond to RA level II and RA level III, respectively. MCS is widely used because of its higher level of accuracy and because it is the most straightforward method for reliability analysis, while FORM is very traditional and has been used since when the first studies of structural reliability were conducted. These are the two methods applied throughout the reliability-based analyses in this dissertation.

Based on the basic reliability concepts explained in Chapters 2, the *Chapter 3* presents some proposals describing the main steps for different reliability-based methodologies considering the ultimate limit state criteria: (1) Reliability-based design (RBD), (2) Safety evaluation, (3) Reliability-based safety factors and (4) Reliability-based sensitivity analysis.

The first (RBD) assumes a fixed load value and analyses the differences in the probability of failure for different lengths of the pile, while the second (Safety evaluation) assumes a fixed length and calculates the probabilities for different load values. In both, the selection of the final value for design (minimum dimension or maximum load) is based on a reliability target previously selected.

For the reliability-based sensitivity analyses, it is possible to use both FORM and/or MCS. The former has the sensitivity factors to evaluate the importance of the random variables. Meanwhile, for ordinary MCS the uncertainties influence need to be studied using other methods, like a parametric study. The different reliability-based sensitivity analyses, in both case studies, were conducted in order to understand the role of the variability of each uncertainty in the design of pile foundations using such probabilistic techniques.

The methodologies and approaches presented are believed to be simple and can easily support the design of pile foundations. Also, the proposals presented try to eliminate the possible confusions and difficulties that traditional reliability methodologies used in structures can cause to geotechnical designers in practice. Because even though probabilistic methods are not new, there are still few designers who use these methods in practice.

After a research of pile foundations databases, some data was selected to perform different analyses based on reliability theory and methodologies of Chapter 3. All choices were carefully done based on the information available about each case and site, and also taking into account the dominant design methods and construction processes in Portuguese geotechnical practice. Thus, both sites (case studies) selected are from Portugal.

Two pile foundations examples are presented. Case study 1 (*Chapter 4*) is a pile from the FEUP's experimental site and Case study 2 (*Chapter 5*) is a pile from a bridge project in south of Portugal. Also, a database was aggregated in order to study the model error of SPT-based capacity prediction. This database is part of the PRWI Japanese database, and pertains to bored piles with soil characterisation based on SPT and pile load test results.

The uncertainties considered for the reliability calculations undertaken are: the physical uncertainties of actions, the inherent soil variability, including spatial variability, and the modelling uncertainty (or model error) in the evaluation of vertical/axial resistance of the pile.

After quantification of the uncertainties, the two well-known probabilistic methods FORM and MCS are compared. The approaches RBD and Safety evaluation are presented, expressing the impact on the performance of the pile of its length and applied total load. As well, MCS in combination with design value method are used for the evaluation of the safety factors of resistance and load/action, and reliability-based sensitivity analyses are illustrated, on which the techniques MCS and RBD are used to assess the influence of some variables/uncertainties on the axial pile reliability response.

However, the order of computations is different for case study 1 and case study 2. The uncertainties for case study 2 had different considerations, for both actions (load combination value and uncertainty set) and soil variability (1 SPT or all SPT), so, the results of the different reliability-based methodologies (FORM, MCS, RBD, Safety evaluation and reliability-based safety factors) are presented and compared after a primer comparison of the results of an extensive reliability-based sensitivity analysis. Concerning the results achieved for each case study, it is possible to refer the following findings.

Case study 1:

- The results obtained with FORM present some deviations from the MCS results, although, they can be considered as acceptable approximations.
- The reliability achieved for the bored pile with 6 m length, 0.6 m of diameter and a total load of 800 kN was β_{FORM} = 2.31 and β_{MCS} = 2.24.
- Safety factors values were 0.2-0.4 for resistance safety factors and 0.9-1.2 for actions' safety factors, and they expressed a direct influence with the importance of the uncertainties, namely in the resistance (>>) and actions (<<).

- From the reliability-based sensitivity analyses it was observed that the reliability index is marginally influenced by the soil variability and action's uncertainties, but significant variations are observed influenced by a soil COV of 50% (very high COV) and by the model error.
- Different empirical methods, namely based on SPT, CPT and PMT were considered to determine pile ultimate vertical bearing capacity in reliability analyses. An important finding refers that for all cases (for any empirical model), the model error is the most important uncertainty.

Case study 2:

- In the sensitivity analyses one can see the same kind of behaviour seen in case study 1, the model error has the highest influence in the reliability results, while the combination concerning the actions' uncertainties, it presents an almost insignificant influence in reliability variations, except when load combination load combination 2 is used (40% of permanent load and 60% od variable load).
- The results presented in each combinations' graphs are undeniably consistent, reaffirming the model error as the most influent and important uncertainty in the reliability of the pile.
- Reliability-based sensitivity analyses provided that the most favourable case (higher reliability) is case "all-2-1" (using all SPT for soil variability, load combination 2 and set 1 of uncertainties) and the least favourable case (lower reliability) is case "1-1-2" (using 1 SPT for soil variability, load combination 1 and set 2 of uncertainties).
- This case study also expounded that FORM has limitations concerning the performance function due to its normal and linear approximations. A extensive comparison between FORM and MCS was done and some difficulties were observed in convergence of the reliability indexes. It was also noted that the reliability indexes, although not very far from MCS, presented inconsistent values during the analyses.
- The reliability achieved for the open-ended steel pipe pile with 43.m length, 1.12 m of diameter was, for "all-2-1" β_{MCS} = 3.7 and "1-1-2" β_{MCS} = 2.7.
- Safety factors reflected also this tendencies, expressing a direct influence with the importance of the uncertainties, namely in the resistance (>>) and actions (<<).
- Safety factors values were between 0.2 and 0.4 for resistance safety factors (same magnitude as for case study 1) and 0.9 and 1.3 for actions' safety factors (higher than the case study 1, due to high variability adopted for actions of case study 2).

Furthermore, the following summary statements can be made based on both case studies' results:

- The cases under study expounded that not considering spatial correlation/autocorrelation that allows for the reduction of the soil uncertainty leads to a more reliable result. However, when this reduction is not considered, the result becomes more conservative, therefore, not correct, especially in terms of economy.
- While the reliability index for case study 2 satisfied the recommendations, the reliability index for case study 1 did not. This can be explained by the fact that case study 1 is an experimental field case study in which failure is obviously of minor consequence.
- FORM was only successfully applied to case study 1. FORM cannot incorporate limit conditions that the empirical method requires for calculation of pile bearing capacity demands. These calculations were successfully conducted for case study 1 because the limits were not necessary (due to lower resistance of this pile), but for case study 2 (with a higher magnitude of resistance), the method did not provide realistic results (the resistances predicted were too high) and leading to a possible problem of convergence. Therefore, for case study 1 the sensitivity factors from FORM are considered valid for further analyses, but for case study 2 FORM they have no meaning and cannot be considered.
- MCS was presented as an accurate method, needing only fundamental knowledge of statistics and probability theory. However, MCS can be a time consuming method when numerical analysis are needed. This was not the case since the performance function was based on a design methodology that uses *in situ* geotechnical tests and empirical formulas/models.
- RBD and Safety evaluation results presented that the probability of failure (on a log scale) – or the reliability index – can be seen to have an approximately linear relationship with the length of the pile (on a linear scale) and an approximately exponential relationship with the total load applied to the pile (on a linear scale).
- Also, when comparing the combinations considering all uncertainties and the ones removing tip and side components uncertainties, one can see that the importance of the side component in case study 2 pile, which has a high embedded length, has a much bigger influence than case study 1. The contribution of tip and side uncertainties will depend greatly on the type of pile and the ratio between these two resistances.
- Reliability-based safety factors were evaluated for both cases and for different combinations but in order to make some recommendations to safety factors values, it is necessary to

study a bigger amount of cases. However, it is possible to conclude that the resistance safety factors determined based on reliability have a similar magnitude to the ones recommended by the American codes, being smaller than the ones recommended by the Eurocodes.

- In addition, the results for these two case studies show that soil uncertainties do not exhibit an importance as great as was expected. However, model uncertainties contributed greatly to the probability of failure for both cases studied. Meanwhile, the contribution of toe and side uncertainties depends greatly on the type of pile and the ratio between these two resistances.

Finally, it has been confirmed that the traditional deterministic analyses cannot represent the problem exactly. The traditional way of design (safety factors, mostly empirical) does not present a rational framework to incorporate the different random variables and uncertainties.

It has been highlighted that reliability analyses/tools/applications provide a more rational understanding of: (1) the design, (2) its random variables and (3) the uncertainties that mostly influence pile behaviour.

Also, FORM method can be used as an alternative method and as a first approach, if carefully applied to a simple performance function. However, for more complex analyses, MCS should be used for assessment of the probability of failure. MCS is the most accurate full probabilistic method, normally used as a reference. However, MCS would be time consuming if combined with numerical methods (e.g.: FEM).

The consequences of errors can be controlled through its identification using sensitivity analyses. The variability can result on an unacceptable performance. Therefore, the most desirable design is the one that is the least sensitive to these variabilities, or in other words, is the one that is not excessively compromised by a predictable variation in a certain random variables (known to have effect on its performance).

In order to investigate the relationship between the reliability of a design procedure adopted and the corresponding costs the *Chapter 6* presents cost-reliability-risk analyses for case study 1. Such analyses intend to support cost-benefit decisions, helping and guiding the research of a pile foundation project, and also avoiding spending/investing where it is not appropriate/favourable for the reliability of the pile. As main conclusions it is possible to say that:

- Geotechnical characterisation and testing are essential to identify and characterise the foundation, to adopt the most convenient solutions, have the necessary information for reliable design, minimising risks and allowing a time-saving and cost-effective construction work. By failing to prevent this, the project costs will necessarily reflect this mistakes and lack of information.

- Also the hypothesis and theoretical models affect the cost and the time required for construction of pile foundations. Field observations (both soil and pile) are essential. Therefore, pile test should be carried out, especially when not enough confidence is put into the design method (e.g.: empirical models). However, it is obvious that pile load tests are appropriate for achieving reliability and savings in larger construction projects, being less important on smaller ones.
- And finally, and as for the previous two chapters (4 and 5), a high influence of the model uncertainty in probability of failure values was detected. Once again, this chapter demonstrated that a higher or lower soil variability does not influence that much the results (reliability level).

Overall, it should be stressed that current pile design methodologies need further improvements, using the new tools available, that have been increasingly applied to design in many other engineering areas. Accordingly, this dissertation has considered the areas in axial pile foundation design, where increased efficiency (reliability and economy) can be raised. Of key importance in this study were:

- Provide a practical basis for the subsequent developments of pile reliability-based design studies with understanding of the role of undertaking a thorough site investigation to identify ground/soil hazards.
- Avoid failure due to unknown conditions and provide more appropriate parameters for design, avoiding also overdesign.
- Contribute for the harmonisation and new developments aiming a more rational design methodology, control and understanding of the problem, not only in axial pile design, but also in overall geotechnical designs.

These subjects have caught more and more researchers and engineers' attention, but many engineers are still concerned about what these developments mean and how they can be applied with confidence. Therefore, with scientific and practical value from geotechnical point of view, this dissertation is believed to be necessary to explain the potential of these kind reliability-based methods and developments, in a simple manner, so that they are within reach of the practitioner engineers, leading towards a more appropriate and consistent safety levels in geotechnical designs.

7.2 FUTURE DEVELOPMENTS

It is only recently that reliability tools have been employed to supplement deterministic approaches in geotechnical designs. These tools based on statistical and probabilistic analyses offer additional and important information, allowing reduction of risks and a higher reliability in geotechnical practice.

In different parts of the world, efforts were and are being made, to develop the design codes with reliability-based tools. Therefore, since reliability-based analyses are becoming a central tool, it is desirable that geotechnical Eurocodes, namely the Eurocode 7 that is one of the most important codes in the world, introduces these concepts and adopt the same methodologies. However, according to a survey in UK, less than 30% of the designers currently use the Eurocode 7 (geotechnical design code) for pile design of low-rise housing. They claim that there is a perception that the use of the partial safety factors proposed within the Eurocode 7 can confuse and obscure the really critical issues and is considered by some to be too complex and overly academic.

Accordingly, this dissertation contributes to axial pile foundations design based on reliability. By updating the traditional method and including pile reliability tools, it is aimed to encourage the development of the national and international standards and conformity in assessments systems.

The literature review indicates that this is still a controversial subject, that compatibility of the design codes form different countries needs to be addressed and that further developments is needed based on the correlation between soil parameters or resistance parameters and field tests. So, considering the outcomes of this dissertation and the stage of development of this type of analyses in geotechnical field, and in particular in axial pile foundations design, future research work should provide for the following topics:

- More efforts should be placed on design methodologies based on reliability tools, supplementing or even replacing the traditional deterministic methods in order to obtain the best and most efficient results. This will provide harmonisation between structural and geotechnical design codes. But it will also provide opportunities to update some design methodologies that may be inadequate nowadays. The recent years have brought some new constructive solutions and techniques, more sustainable and economic, and these transformations were not accompanied by an equivalent change in the design methodologies (procedure used to estimate the bearing capacity of a pile has hardly changed). This is especially true for pile foundations, where the resistance models uncertainties are the most influent uncertainty in pile reliability, according to this dissertation results but also other reliability studies.

- Other issues are the recommended target reliability indexes that are mainly or specifically aimed for structures (steel and concrete), which have a very different approach for design. Structural reliability design differs greatly from geotechnical, especially when considering the evaluation of uncertainties and its influence in calculations. The literature review did not provide agreement about which should be the target reliability index for calibration of geotechnical design codes. Therefore, the assessment of the reliability index for each representative group of a geotechnical structure and following recommendations for target reliability index should be studied, since this value depends on many factors and is mandatory for further geotechnical reliability analyses, such as calibration of reliability-based safety factors (Figure 7.1).
- The development of reliability-based safety factors that do not require probabilistic computation in routine pile design would be valuable, although this will require site-specific studies considering large databases and an adequate number of cases for each group/category of geotechnical problem (pile type, soil type, etc.), to adjust to specific needs of a particular project (Figure 7.1).
- Finally, the assessment of sensitivity to variations in the statistics (such as mean and standard deviation of the random variables of the problem) can be an important tool in the decision-making process of pile foundations, as well as any other type of geotechnical structure. This is because the decision-making related to economic and research investments gathering the necessary information to characterise the random variables (uncertainties) that are important in both pile design and its reliability can be facilitated with this type of balanced reliability analysis.

In order to achieve these, and as a first step, it is important to gain knowledge relating to current practice and identify areas of general concern, such as the selection of the type of pile foundation, ground investigation practice, and design and construction methodologies adopted. Surveys about these topics would be needed to understand where additional guidance may be useful.

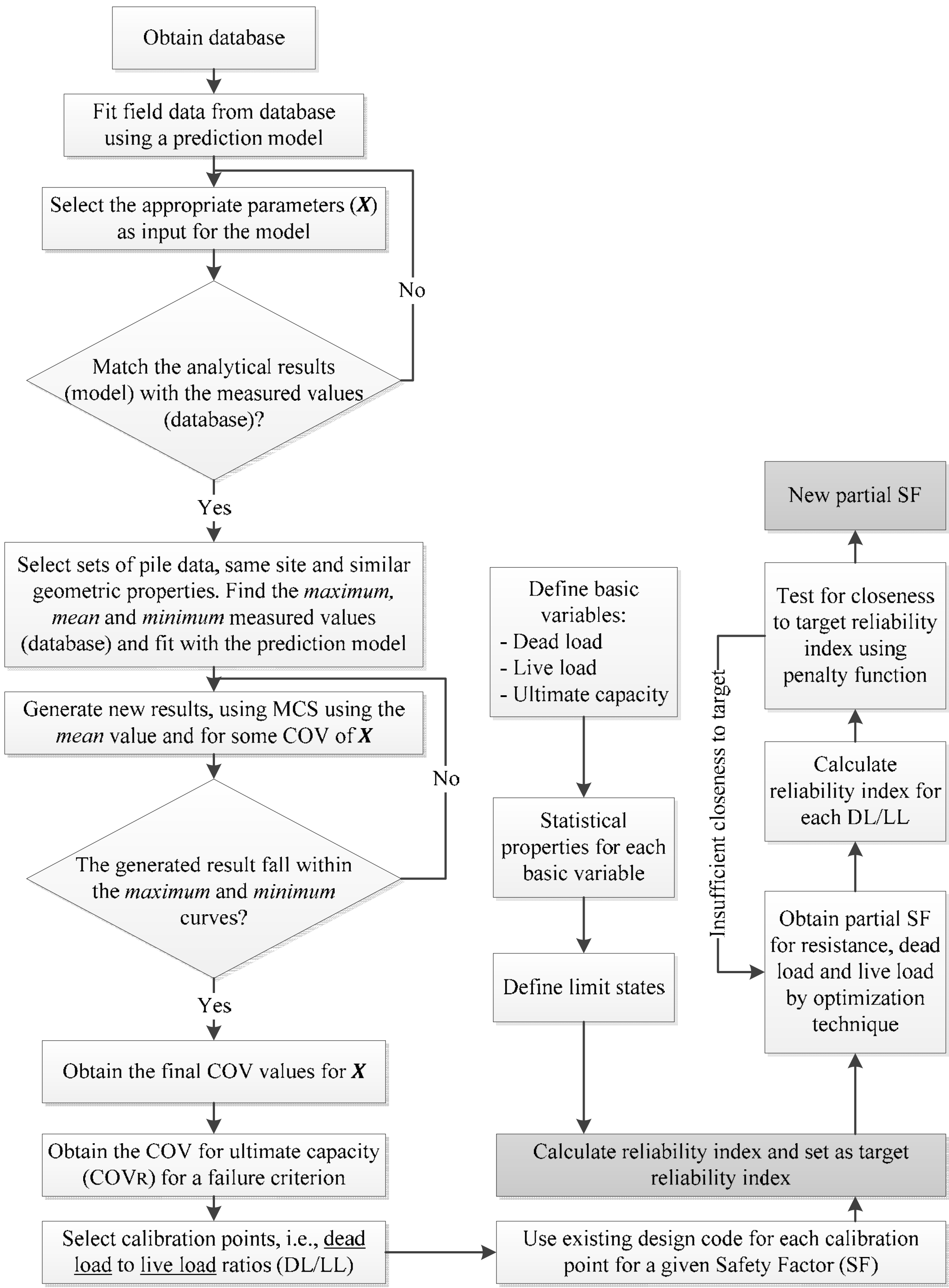

Figure 7.1 – Flowchart for development of reliability-based safety factors and reliability index calibration for axial pile foundations (adapted from Haldar, 2008)

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Annexes

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Annex A – Basic concepts of probability theory and statistics

The definitions presented here are available in any classic book of probability and statistics. The statistical and probabilistic studies focus on random variables (RV) that can be designed as discrete or continuous. Since only continuous RV were used in this dissertation, the following notions and definitions are referred to this type of RV.

Consider the continuous RV x and y:

The MEAN value (μ_x) or first moment of x is given by eq.(A.1). The mean value can also be denominated by E[x], the expected value.

$$\mu_x = \frac{1}{n} \cdot \sum_{i=1}^{n} x_i \tag{A.1}$$

VARIANCE (σ_x^2) and STANDARD DEVIATION (σ_x) of x can be calculated through eq.(A.2) and (A.3).

$$\sigma_x^{\;2} = Var[x] = \frac{1}{n-1} \cdot \sum_{i=1}^{n} (x_i - \mu_x)^2 \tag{A.2}$$

$$\sigma_x = \sqrt{Var[x]} \tag{A.3}$$

COEFFICIENT OF VARIATION of x (COV_x or V_x) is calculated using eq.(A.4):

$$V_x = \frac{\sigma_x}{\mu_x} \tag{A.4}$$

COVARIANCE between x and y (Cov_{xy}) is given by eq.(A.5):

$$\begin{aligned} Cov_{xy} &= E[(x-\mu_x)\cdot(y-\mu_y)] \\ &= E[x \cdot y] - E[x] \cdot E[y] \end{aligned} \tag{A.5}$$

Meanwhile, the COEFFICIENT OF CORRELATION between x and y (ρ_{xy}) is:

$$\rho_{xy} = \frac{Cov_{xy}}{\sigma_x \cdot \sigma_y} \tag{A.6}$$

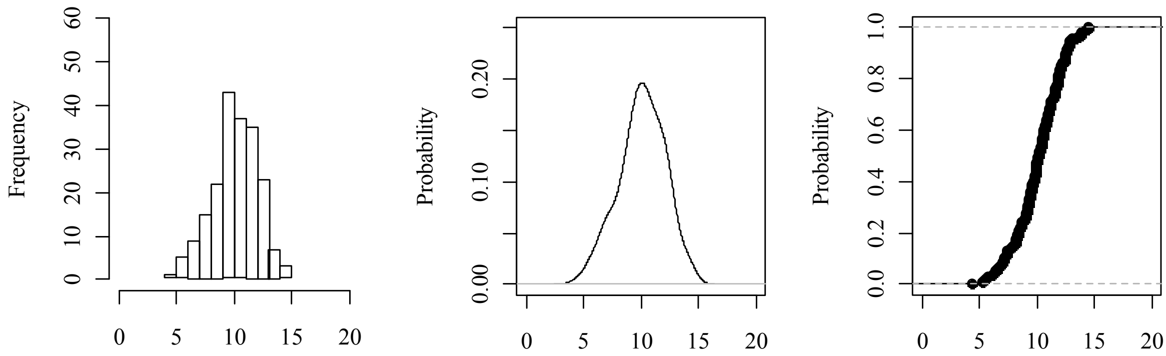
An HISTOGRAM is a graphical representation of the distribution of a RV (x or y), the graph shows the frequencies of discrete intervals, the number of intervals is called number of bins (k) – Figure A.1a.

PROBABILITY DENSITY FUNCTION (PDF or f) or probability distribution, describes the range of possible values of x and gives the probability of a value falling within a particular interval of that range– Figure A.1b.

CUMULATIVE DENSITY FUNCTION (CDF or F) or distribution function is the area so far of the PDF, in other words, the CDF gives the probability of x being less than or equal to a specific value – see eq.(A.7), eq.(A.8) and Figure A.1c.

$$F_x(x) = \int_{-\infty}^{x} f_x dx \qquad \text{(A.7)}$$

$$P[a \le x \le b] = F_x(b) - F_x(a) = \int_{a}^{b} f_x dx \qquad \text{(A.8)}$$



a) Histogram b) PDF c) CDF

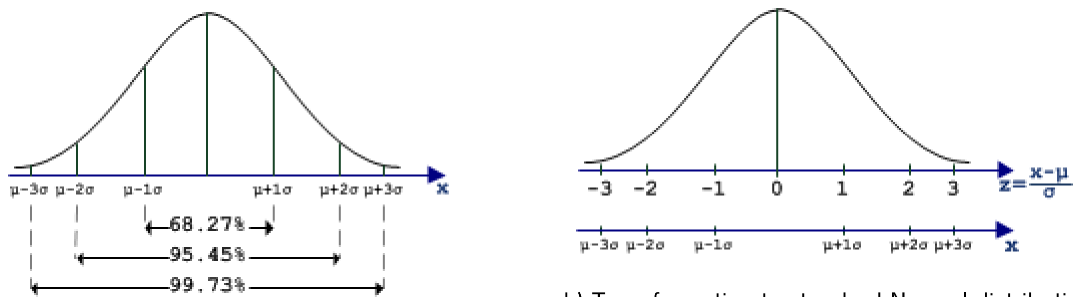
Figure A.1 – Graphical representation of a continuous random variable

RV with Normal distribution (~N)

The Normal (or Gaussian) distribution is the most important in probability and statistics. One of the reasons is the central limit theorem and also because most of the statistical theory is based on the normality assumption. The Normal PDF formula is given by eq.(A.9).

$$f(x) = \frac{1}{\sigma \cdot \sqrt{2 \cdot \pi}} \cdot e^{-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2} \qquad \text{(A.9)}$$

Consider that x is a Normal distributed random variable, $x \sim N(\mu;\sigma^2)$, with mean μ and standard deviation σ, the next figure shows its particularities (Figure A.2).



a) Fractiles of Normal distribution

b) Transformation to standard Normal distribution ~N(0,1), see eq.(A.10)

Figure A.2 – Particularities of Normal distribution

A RV can be normalised using eq.(A.10). When normalising x, this variable will have a new mean equal to zero and standard deviation equal to 1. The values of the CDF of the standard normal RV can be consulted in Table A.1.

$$Z = \frac{X - \mu}{\sigma} \qquad \text{(A.10)}$$

When RV are Normal it is possible to rewrite the eq.(A.8) in a normalised form:

$$P[a \le x \le b] = \Phi\left(\frac{b - \mu}{\sigma}\right) - \Phi\left(\frac{a - \mu}{\sigma}\right) \qquad \text{(A.11)}$$

Table A.1 – Standard Normal distribution values with mean 0 and variance 1

Area under the Normal Curve from $-\infty$ to z

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.50000	0.50399	0.50798	0.51197	0.51595	0.51994	0.52392	0.52790	0.53188	0.53586
0.1	0.53983	0.54380	0.54776	0.55172	0.55567	0.55962	0.56356	0.56749	0.57142	0.57535
0.2	0.57926	0.58317	0.58706	0.59095	0.59483	0.59871	0.60257	0.60642	0.61026	0.61409
0.3	0.61791	0.62172	0.62552	0.62930	0.63307	0.63683	0.64058	0.64431	0.64803	0.65173
0.4	0.65542	0.65910	0.66276	0.66640	0.67003	0.67364	0.67724	0.68082	0.68439	0.68793
0.5	0.69146	0.69497	0.69847	0.70194	0.70540	0.70884	0.71226	0.71566	0.71904	0.72240
0.6	0.72575	0.72907	0.73237	0.73565	0.73891	0.74215	0.74537	0.74857	0.75175	0.75490
0.7	0.75804	0.76115	0.76424	0.76730	0.77035	0.77337	0.77637	0.77935	0.78230	0.78524
0.8	0.78814	0.79103	0.79389	0.79673	0.79955	0.80234	0.80511	0.80785	0.81057	0.81327
0.9	0.81594	0.81859	0.82121	0.82381	0.82639	0.82894	0.83147	0.83398	0.83646	0.83891
1.0	0.84134	0.84375	0.84614	0.84849	0.85083	0.85314	0.85543	0.85769	0.85993	0.86214
1.1	0.86433	0.86650	0.86864	0.87076	0.87286	0.87493	0.87698	0.87900	0.88100	0.88298
1.2	0.88493	0.88686	0.88877	0.89065	0.89251	0.89435	0.89617	0.89796	0.89973	0.90147
1.3	0.90320	0.90490	0.90658	0.90824	0.90988	0.91149	0.91308	0.91466	0.91621	0.91774
1.4	0.91924	0.92073	0.92220	0.92364	0.92507	0.92647	0.92785	0.92922	0.93056	0.93189
1.5	0.93319	0.93448	0.93574	0.93699	0.93822	0.93943	0.94062	0.94179	0.94295	0.94408
1.6	0.94520	0.94630	0.94738	0.94845	0.94950	0.95053	0.95154	0.95254	0.95352	0.95449
1.7	0.95543	0.95637	0.95728	0.95818	0.95907	0.95994	0.96080	0.96164	0.96246	0.96327
1.8	0.96407	0.96485	0.96562	0.96638	0.96712	0.96784	0.96856	0.96926	0.96995	0.97062
1.9	0.97128	0.97193	0.97257	0.97320	0.97381	0.97441	0.97500	0.97558	0.97615	0.97670
2.0	0.97725	0.97778	0.97831	0.97882	0.97932	0.97982	0.98030	0.98077	0.98124	0.98169
2.1	0.98214	0.98257	0.98300	0.98341	0.98382	0.98422	0.98461	0.98500	0.98537	0.98574
2.2	0.98610	0.98645	0.98679	0.98713	0.98745	0.98778	0.98809	0.98840	0.98870	0.98899
2.3	0.98928	0.98956	0.98983	0.99010	0.99036	0.99061	0.99086	0.99111	0.99134	0.99158
2.4	0.99180	0.99202	0.99224	0.99245	0.99266	0.99286	0.99305	0.99324	0.99343	0.99361
2.5	0.99379	0.99396	0.99413	0.99430	0.99446	0.99461	0.99477	0.99492	0.99506	0.99520
2.6	0.99534	0.99547	0.99560	0.99573	0.99585	0.99598	0.99609	0.99621	0.99632	0.99643
2.7	0.99653	0.99664	0.99674	0.99683	0.99693	0.99702	0.99711	0.99720	0.99728	0.99736
2.8	0.99744	0.99752	0.99760	0.99767	0.99774	0.99781	0.99788	0.99795	0.99801	0.99807
2.9	0.99813	0.99819	0.99825	0.99831	0.99836	0.99841	0.99846	0.99851	0.99856	0.99861
3.0	0.99865	0.99869	0.99874	0.99878	0.99882	0.99886	0.99889	0.99893	0.99896	0.99900
3.1	0.99903	0.99906	0.99910	0.99913	0.99916	0.99918	0.99921	0.99924	0.99926	0.99929
3.2	0.99931	0.99934	0.99936	0.99938	0.99940	0.99942	0.99944	0.99946	0.99948	0.99950
3.3	0.99952	0.99953	0.99955	0.99957	0.99958	0.99960	0.99961	0.99962	0.99964	0.99965
3.4	0.99966	0.99968	0.99969	0.99970	0.99971	0.99972	0.99973	0.99974	0.99975	0.99976
3.5	0.99977	0.99978	0.99978	0.99979	0.99980	0.99981	0.99981	0.99982	0.99983	0.99983
3.6	0.99984	0.99985	0.99985	0.99986	0.99986	0.99987	0.99987	0.99988	0.99988	0.99989
3.7	0.99989	0.99990	0.99990	0.99990	0.99991	0.99991	0.99992	0.99992	0.99992	0.99992
3.8	0.99993	0.99993	0.99993	0.99994	0.99994	0.99994	0.99994	0.99995	0.99995	0.99995
3.9	0.99995	0.99995	0.99996	0.99996	0.99996	0.99996	0.99996	0.99996	0.99997	0.99997
4.0	0.99997	0.99997	0.99997	0.99997	0.99997	0.99997	0.99998	0.99998	0.99998	0.99998

RV with Lognormal distribution (~*LN*)

The Lognormal distribution is of particular interest in geotechnical reliability, because it has certain properties similar to some commonly geotechnical RV (US Army Corps of Engineers, 1999):

- it is a continuous distribution with a zero lower bound and an infinite upper bound;
- as the log of the value is normally distributed, rather than the value itself, it provides a convenient model for random variables with relatively large coefficients of variation (>30%) for which an assumption of normality would imply a significant probability of negative values.

A random variable X is said to have a Lognormal distribution if y=ln(X) is normally distributed with mean μ and standard deviation σ. The Lognormal PDF formula is given by eq.(A.12).

$$f(x) = \frac{1}{\zeta \cdot x \cdot \sqrt{2 \cdot \pi}} \cdot e^{-\frac{1}{2} \cdot \left(\frac{\ln(x) - \lambda}{\zeta} \right)^2} \qquad \text{(A.12)}$$

Consider that x is a Lognormal distributed random variable, $x \sim LN(\lambda; \zeta^2)$, it has mean value λ and standard deviation ζ (see eq.(A.13) and eq.(A.14)).

$$\mu = e^{\left(\lambda + \frac{1}{2} \cdot \zeta^2 \right)} \ ; \ \sigma = \mu \cdot \sqrt{e^{\zeta^2} - 1} \qquad \text{(A.13)}$$

$$\lambda = E[\ln(X)] \ ; \ \zeta = \sqrt{Var[\ln(X)]} \qquad \text{(A.14)}$$

Next figure presents the differences in Normal and Lognormal PDF.

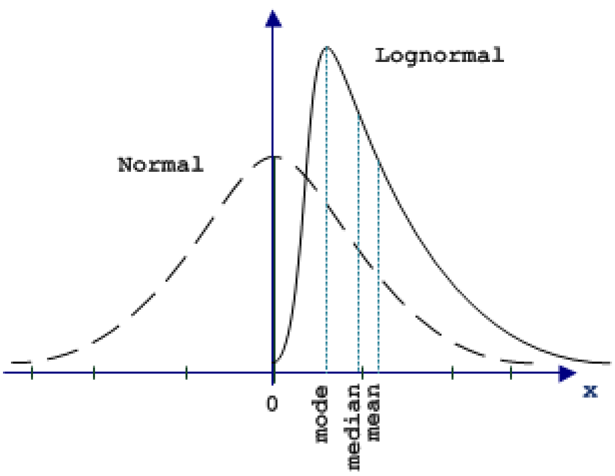


Figure A.2 – Frequency curves of the Normal and Lognormal distributions

Annex B – Coefficients of variation and spatial correlation for geotechnical parameters

The Tables B.1 and B.2 present some recommendations for the values of coefficients of variation (COV) for soil parameters/properties and the Tables B.3 and B.4 present examples of autocorrelation values for soil parameters/properties.

Table B.1 – Coefficients of variation for geotechnical parameters adapted from Flores (2008), Ribeiro (2008) and US Army Corps of Engineers (1999)

Parameter	COV (%) min	COV (%) Max	Reference(s)
Unit weight (γ)	3	7	[1], [2]
	4	8	[3]
Unit weight of residual soils (γ)	1.5	9.4	[4]
Unit weight of sedimentary clays (γ)	2	7	[4]
Effective stress friction angle of sands ($\varphi´$)	2	13	[5], [2], [6]
	5	15	[7], [8], [9]
	3.7	9.3	[3]
Friction angle of clays (φ)	12	56	[7], [9]
	7.5	10.1	[10]
Coef. of friction of residual soils (tg $\varphi´$)	2.4	16.1	[4]
Coef. of friction of sedimentary clays (tg $\varphi´$)	3	6	[4]
Effective cohesion of sedimentary clays ($c´$)	8	14	[4]
Undrained strength (Su)	13	40	[5], [2], [11]
	20	50	[7], [9]
Undrained strength - effective stress ratio ($Su/\sigma_v´$)	5	15	[6], [12]
Compression index	10	37	[6], [1], [2]
Liquid limit (WL)	2	48	[7], [9], [13], [14]
Plastic limit (WP)	9	29	[7], [9], [13], [14]
Plasticity index (PI)	7	79	[7], [9], [13]
Consolidation coefficient	33	68	[6]
Preconsolidation stress ($\sigma_p´$)	10	35	[1], [6], [12]
N of SPT	15	45	[5], [2]
	10	70	[15]
q_c of electric CPT	5	15	[2]
q_c of mechanic CPT	15	37	[1], [2]
q of DMT	5	15	[2]
Su by Vane test	10	20	[2]
	10	40	[15]

References from Table B.1:

[1] Harr (1984)

[2] Kulhawy (1992)

[3] Wolff (1996)

[4] Guedes (1997)

[5] Harr (1987)

[6] Duncan (2000)

[7] Lumb (1974)

[8] Hoeg & Murarka (1974)

[9] Singh (1971)

[10] Wolff (1985)

[11] Lacasse & Nadim (1996)

[12] Lacasse & Nadim (1997)

[13] Kuhn (1971)

[14] Mitchell (1993)

[15] Phoon & Kulhawy (1999a)

Table B.2 – Coefficients of variation for geotechnical parameters adapted from Phoon (2008a)

Test	Property	Soil type	Mean	Units	COV(%)
CPT	q_T	Clay	0.5 – 2.5	MN/m^2	< 20
CPT	q_c	Clay	0.5 – 2	MN/m^2	20 – 40
CPT	q_c	Sand	0.5 – 30	MN/m^2	20 – 60
VST	Su	Clay	5 – 400	kN/m^2	10 – 40
SPT	N	Clay and Sand	10 – 70	Blows/ft	25 – 50
DMT	A reading	Clay	100 – 450	kN/m^2	10 – 35
DMT	A reading	Sand	60 – 1300	kN/m^2	20 – 50
DMT	B reading	Clay	500 – 880	kN/m^2	10 – 35
DMT	B reading	Sand	350 – 2400	kN/m^2	20 – 50
DMT	ID	Sand	1 – 8		20 – 60
DMT	KD	Sand	2 – 30		20 – 60
DMT	ED	Sand	10 – 50	MN/m^2	15 – 65
DMT	p_L	Clay	400 – 2800	kN/m^2	10 – 35
PMT	p_L	Sand	1600 – 3500	kN/m^2	20 – 50
PMT	EPMT	Sand	5 – 15	MN/m^2	15 – 65
Lab index	Wn	Clay and Silt	13 – 100	%	8 – 30
Lab index	WL	Clay and Silt	30 – 90	%	6 – 30
Lab index	WP	Clay and Silt	15 – 15	%	6 – 30
Lab index	PI	Clay and Silt	10 – 40	%	[3-12] / mean
Lab index	LI	Clay and Silt	10	%	[3-12] / mean
Lab index	γ, γ_d	Clay and Silt	13 – 20	kN/m^3	<10
Lab index	Dr	Sand	30 – 70	%	10 – 40* ; 50 – 70**

*direct method of determination.

**indirect determination by SPT values.

Table B.3 – Spatial correlation lengths, adapted from Alonso (1976)

Type of soil	Direction	Parameter	θ (m)
Sand	Vertical	Tip resistance of CPT	2.2
		Friction ratio of CPT	1.3
Clay	Vertical	Tip resistance of CPT	1.1
Silty Clay	-	Water content	12.7
		Clay %	8.7
		Silt %	6.5
		Unit weight	7.9
		Void content	10.5
		Liquid limit	8.7
Gravel	-	Porosity	14.7

Table B.4 – Spatial correlation lengths, adapted from Vanmarcke (1977)

Type of soil	Direction	Parameter	θ (m)
Sand	Vertical	Water content	2.7
		Void content	3.0
		SPT N value	2.4
	Horizontal	Compression index	55.0
Clay	Vertical	Tip resistance of CPT	1.2

Annex C – Determination of autocorrelation graph

Consider a set of data, for example the qc (numb) parameter of a CPT with n points (index). The autocorrelation graph is calculated based on the correlation between the residuals after determining the trend of the data (Figure C.1 and C.2, where the index corresponds to the depth). The correlation of those residuals can be calculated, like shown on Table C.1.

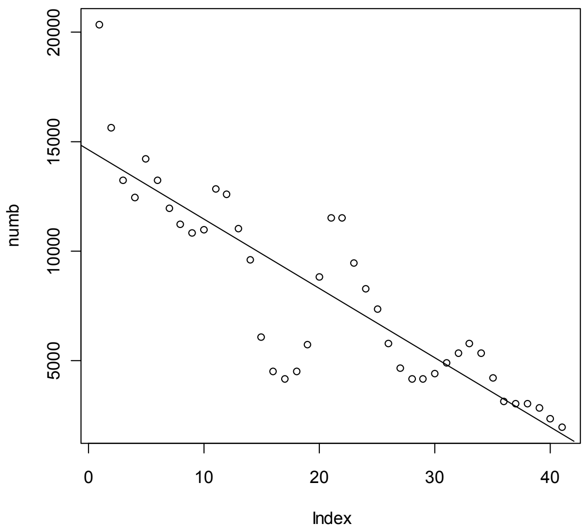


Figure C.1 – Data representation and trend

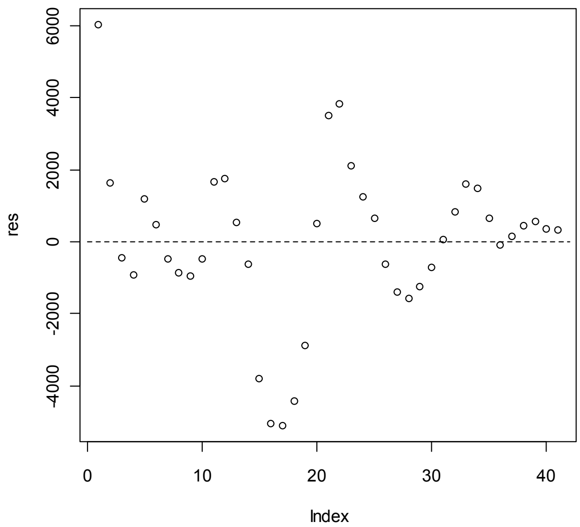


Figure C.2 – Residuals of the data

Table C.1 – Correlations for computing and plotting autocorrelation graph

Lag distance	Correlation (x,y)	x data	y data	Total of points for correlation
1	(x_i,x_{i+1})	1:(n-1)	2:n	n-1
2	(x_i,x_{i+2})	1:(n-2)	3:n	n-2
3	(x_i,x_{i+3})	1:(n-3)	4:n	n-3
4	(x_i,x_{i+4})	1:(n-4)	5:n	n-4
5	(x_i,x_{i+5})	1:(n-5)	6:n	n-5
...	...	...	...	...
n-2	$(x_i,x_{i+(n-2)})$	1:3	76:78	3
n-1	$(x_i,x_{i+(n-1)})$	1:2	77:78	2

The correlations between points are computed, like between first (x_i) and second (x_i+1), first (x_i) and thirds (x_i+2), and so on. Individual graphs for each correlation can be seen on Figure C.3. Then, with each correlation the autocorrelation graph can be drawn like depicted in Figure C.4.

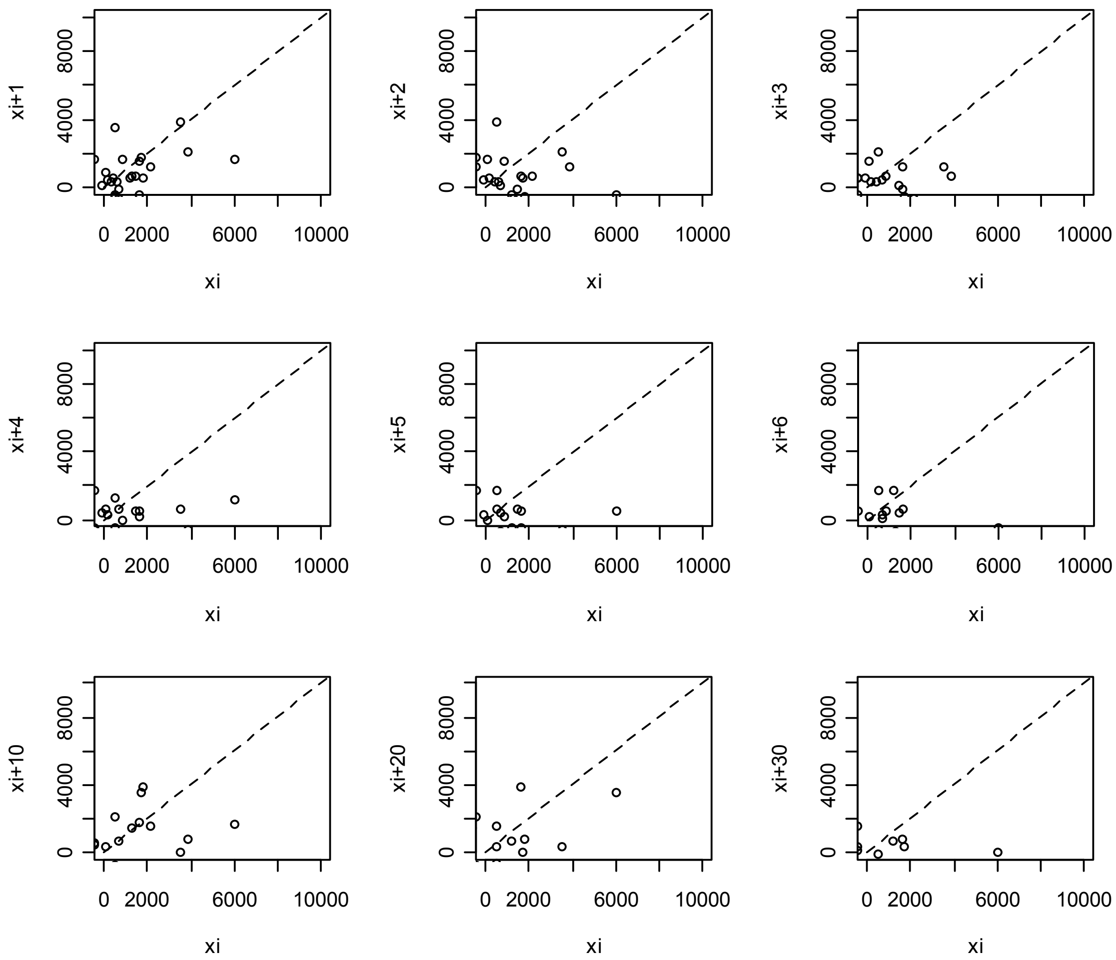


Figure C.3 – Graphical representation of each correlation

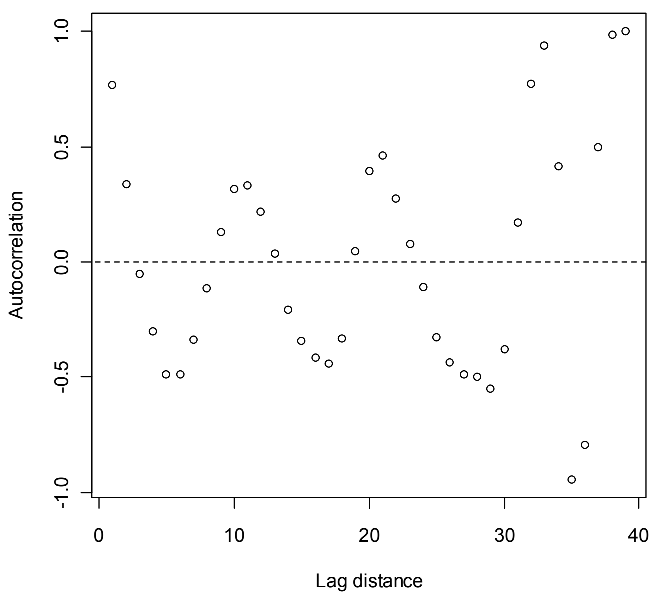


Figure C.4 – Autocorrelation distance graph for data in Figure C.1

Annex D – Pros and Cons of the partial factor approaches: MFA and RFA

Material factor approach (MFA) and (multiple) resistance factor approach (RFA or MRFA) are two different approaches for RA level I that uses deterministic formulas, characteristic values and safety factors (SF). Next, are presented the pros and cons of these two approaches, adapted from Honjo & Amatya (2005) and Kieu Le (2008).

MFA		
	Pros	Some may find this approach more intuitive and reasonable, because it treats the uncertainties on their sources. Also with MFA is easier to accommodate the development of construction techniques and design methods. Having the uncertainties treated at their origin, it allows flexibility to update the SF related to new developments that may appear along the way, with time (e.g.: improvement of construction techniques).
	Cons	MFA may cause a difficult evaluation of the reliability of the structure for the following reasons: (1) not all sources of uncertainties are known quantitatively, (2) some of the sources are correlated and that correlation becomes difficult to consider, (3) when overall uncertainty of the structure can be estimated it is difficult to break down the result into each source. Furthermore, many of the partial SF have non-linear effects on the resulting uncertainty, thus, it is difficult to control the reliability of the structure at the source of the uncertainties. And finally, since MFA modifies the material property, the actual behaviour of the structure based on this modified property can be far from reality (or far from the most likely behaviour). Also this methodology has too many factors.

RFA and **MRFA**		
	Pros	The calculation by RFA/MRFA, especially for the resistance side, predicts the most likely behaviour of the structure to its last stage of design. In geotechnical design, where the interaction soil-structure is so high and complex, it is impossible to know if the reduction of the material parameter value will result on a safe or unsafe modification. This is especially true for sophisticated design calculation methods like FEM.

(...)

RFA **MRFA**	(...) Pros	Also, in many code calibration situations uncertainties of a design method are provided as a result of many loading tests (databases), the given uncertainties include all aspects of design, and it is impossible to decompose to its various sources. Therefore, in practice, it is possible to calibrate codes based on RFA but not on MFA. Lastly, RFA/MRFA has a design verification formula that is more close to the traditional safety factors method (WDS/ASD) than the MFA. Finally, RFA/MRFA achieves relatively uniform levels of safety based on the strength of soil and rock for different limit states and foundation types.
	Cons	The pros of MFA. The resistance SF vary with design methods chosen.

Annex E – Techniques to improve computational time in MCS

In Monte Carlo simulations (MCS) the random basic variables are generated and then the performance function is computed n times. This n is the number of simulations and its value can be from 100 up to 1,000,000 depending on the problem.

For example, when the performance function takes 1 minute to compute, the MCS would take from 1.7 hours (n=100) up to 695 days (n=1,000,000). These computational times are not acceptable in practice; therefore, techniques to improve MCS, called reduction of variance techniques, are implemented to diminish the time spent with computations (Phoon, 2008a).

These techniques allow the determination of an adequate result with a lower number of simulations. The most used techniques are presented next.

Importance sampling

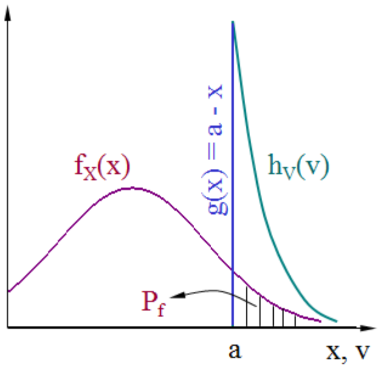
The idea of this technique is to concentrate the sample points on the most significant zone for the structural analysis. The selection of those points depends on the specific problem and is based on a probabilistic criteria (Figure E.1 - h_V is the Importance sampling function) – Chan & Low (2009).

Stratified sampling

The stratified sampling is based on the principle that all portions of the domain of the variables should be represented. This technique considers the range of values and divides it in different intervals with same probability of occurrence. Then, only one point in each interval is selected for computing the performance function (see Figure E.2), each point corresponds to the centroid of the distribution function of its interval.

Latin hypercube

The Latin hypercube technique is similar to the stratified sampling; the difference is in the representation of the intervals. As depicted in Figure E.3, only one interval in each line and row are selected for representation and computation of the performance function. This way each interval intervenes only once during the sampling process, resulting in a much smaller number of samples.



$$P_f = \int \ldots \int I\left[g\left(x \le 0\right)\right] \frac{f_X\left(x\right)}{h_V\left(x\right)} h_V\left(x\right) dx$$

$$P_f \approx \frac{1}{N}\left\{\sum_{j=1}^{N} I\left[g\left(v_j \le 0\right)\right] \frac{f_X\left(v_j\right)}{h_V\left(v_j\right)}\right\}$$

Figure E.1 – Schematic representation of the importance sampling technique

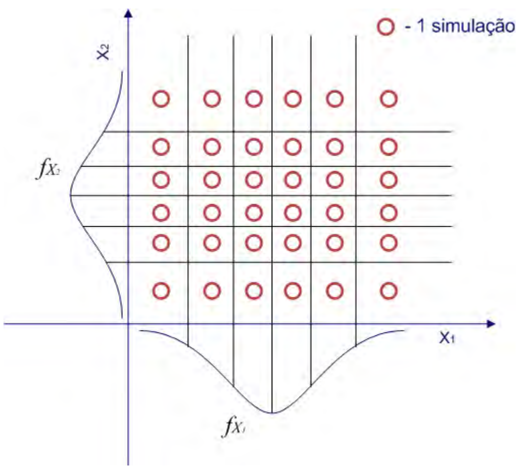


Figure E.2 – Schematic representation of the stratified sampling technique

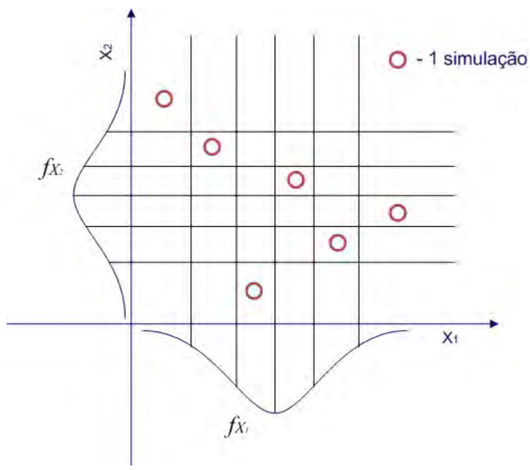


Figure E.3 – Schematic representation of the Latin hypercube technique

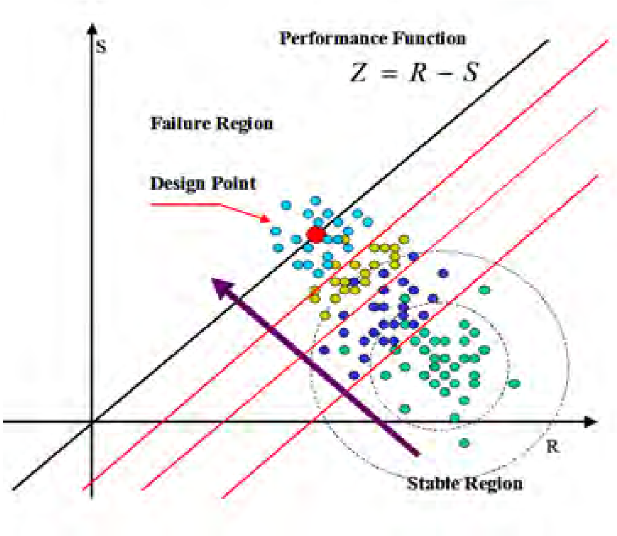
Markov chain Monte Carlo

The basic idea consists in: a small failure probability can be indicated as a product of larger conditional probabilities of some intermediate failure events. A simulation problem of a rare event is converted into the problem of a sequence of more frequent events (Figure E.4) – Kieu Le & Honjo (2007); Zhang *et al.* (2010, 2012).

<u>Subset MCMC algorithm</u>

1. For k=1, generate N_t samples for X_i based on given PDF and calculate $g(X)$ for each generated sample;
2. Determine F_k and choose the N_s seeds (samples that are closer to the limit state function $g(X)$ = 0);

3. For $k=k+1$, for each seed of X_i generate N_t/N_s samples (using M-H algoritm[1]), defining subset F_{k+1};
4. Evaluate $g(X_{k+1})$ at each consecutive state of X_i:
   - If $g(X_{k+1}) < F_k$ accept the state X_{k+1}
   - If $g(X_{k+1}) > F_k$ reject the state, i.e. $X_{k+1}= X_k$
5. Repeat steps 2 to 4 until $F_k = F_m$
6. And finally estimate failure probability of the target failure event.



$$F_{k+1} = \left\{ x \mid g(x) < \frac{g_{Ns} + g_{Ns+1}}{2} \right\}$$

$$P_f = P\left(g(x) \leq 0\right) = \left(\frac{N_s}{N_t}\right)^{m-1} \frac{N_f}{N_t}$$

Figure E.4 – Schematic representation of the MCMC (Kieu Le & Honjo, 2007)

[1] Metropolis–Hastings algorithm is a Markov chain Monte Carlo (MCMC) method for obtaining a sequence of random samples from a probability distribution for which direct sampling is difficult. M-H algorithm constructs a Markov Chain by proposing a value for from the candidate distribution, and then either accepting or rejecting this value (with acertain probability).

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Annex F – Safety factors' and characteristic values' recommendations by some design codes for pile design

In the past decades civil engineering has been experienced the change of its design codes to performance-based design methodology. The reliability accepted by each one of the codes depends on the value given to human life by society, the material costs, loss of services, among others, which is not linear. It varies from location to location, conditions, culture, mentality, economy, among others. Before these new methodologies based on probabilistic and statistics, RA level I was the one recommended. Research is underway to elaborate codes that are RA level II and level III. RA Level IV is only used for specific projects or advanced-level structures and/or critical conditions. To compare levels of reliability given for the different codes, this annex reviews the recommendations for characteristic values and safety factors of codes from Europe, North America and Japan.

Safety factors

Eurocode 7 has been under development since 1980 and it replaces working stresses and overall safety factors (global SF) for the limit state design (LSD) approach and respective partial SF. For geotechnical engineering and according to Eurocode 7, the ultimate limit states can be divided in 5 categories: EQU, STR, GEO, UPL and HYD, each category has a particular set of partial SF.

In foundation design the main categories are GEO and STR limit states. UPL and HYD should only be checked if buoyancy and hydraulic gradients are of concern, while EQU is mainly relevant to structural design, and limited to rare cases such as rigid foundation bearing on rock. For GEO and STR, one needs to verify that the limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Design Approach 1 Combination 1: A1 + M1 + R1 (*)

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

Design Approach 2 Combination: A1 + M1 + R2 (***)

Design Approach 3 Combination: (A1 or A2) + M2 + R3 (****)

(*) partial factors are applied to actions and to ground strength parameters

(**) partial factors are applied to actions, to ground resistances and sometimes to ground strength parameters. set M1 is used for calculating resistances of piles or anchors and set M2 for calculating unfavourable actions on piles owing (e.g.: to negative skin friction or transverse loading)

(***) partial factors are applied to actions or to the effects of actions and to ground resistances

(****) set A1 is used for structural actions and set A2 used for geotechnical actions

The partial factors proposed by Eurocode for pile design are presented in Table F.1.

Table F.1 – Partial SF sets for pile's ULS resistance to axial load (GEO and STR) based on Eurocode 7 (CEN, 2007)

Actions or effects of actions:		Symbol	Set:	A1	A2		
Permanent	Unfavourable	γ_G		1.35	1.00		
	Favourable			1.00	1.00		
Variable	Unfavourable	γ_Q		1.50	1.30		
	Favourable			-	-		
Material property/soil parameter:				M1	M2		
Angle of shearing resistance (tan φ')		$\gamma_{\varphi'}$		1.00	1.25		
Effective cohesion		$\gamma_{c'}$		1.00	1.25		
Undrained shear strength		γ_{cu}		1.00	1.40		
Unconfined compressive strength		γ_{qu}		1.00	1.40		
Weight density (γ)		γ_γ		1.00	1.00		
Resistance of driven piles:				R1	R2	R3	R4
Base		γ_b		1.00	1.10	1.00	1.30
Shaft (compression)		γ_l		1.00	1.10	1.00	1.30
Total combined (compression)		γ_t		1.00	1.10	1.00	1.30
Shaft in tension		γ_m		1.25	1.15	1.10	1.60
Resistance of bored piles:				R1	R2	R3	R4
Base		γ_b		1.25	1.10	1.00	1.60
Shaft (compression)		γ_l		1.00	1.10	1.00	1.30
Total combined (compression)		γ_t		1.15	1.10	1.00	1.50
Shaft in tension		γ_m		1.25	1.15	1.10	1.60
Resistance of CFA* piles:				R1	R2	R3	R4
Base		γ_b		1.10	1.10	1.00	1.45
Shaft (compression)		γ_l		1.00	1.10	1.00	1.30
Total combined (compression)		γ_t		1.10	1.10	1.00	1.40
Shaft in tension		γ_m		1.25	1.15	1.10	1.60

*continuous flight auger

In North America there are the Canadian codes and the United States (USA) codes, that considers dead and live loads (same as permanent and variable respectively). Dead load is a permanent load due to the weight of building and other components, and live load is a variable load due to intended use and occupancy. The following tables (Tables F.2 and F.3 from Canadian practice and Tables F.4 and F.5 from USA practice) show the factors adopted by each code in these countries.

Table F.2 – Partial SF sets for pile's ULS resistance to axial load, based on Canadian Code for buildings, NBC[2] (NRC, 2005) and CFEM (CGS, 2006)

Actions:	Comb:	1	2
Dead load		1.40	1.25* or 0.90
Live load		0.00	1.50
Resistance factors (multiplying factor):		Factor	1 / Factor
Semi-empirical analysis using laboratory and *in situ* test data		0.4	2.50
Analysis using static loading test results		0.6	1.67
Analysis using dynamic monitoring results		0.5	2.00
Uplift resistance by semi-empirical analysis		0.3	3.33
Uplift resistance using loading test results		0.4	2.50

* The load factor 1.25 for dead load, for soil, superimposed earth, plants and trees shall be increased to 1.50, except that when the soil depth exceeds 1.2 m, the factor may be reduced to [1 + 0.6/*hs*] but not less than 1.25, where *hs* is the depth of soil in metres supported by the structure.

Table F.3 – Partial SF sets for pile's ULS resistance to axial load, based on Canadian Code for bridges, CHBDC[3] (CSA, 2006)

Actions:	Comb:	1	2	3
Dead load		α_D	α_D	α_D
Live load		1.70	1.60	1.40
maximum and minimum values of α_D:		Max	min	
Factory-produced components, excluding wood		1.10	0.95	
Cast-in-place concrete, wood, and all non-structural components		1.20	0.9	
Wearing surfaces, based on nominal or specified thickness		1.50	0.65	
Earth fill, negative skin friction on piles		1.25	0.80	
Water		1.10	0.90	
Geotechnical resistance (multiplying factor):		Factor	1 / Factor	
Static Analysis - Compression		0.40	2.50	
Static Analysis - Tension		0.30	3.33	
Static Test - Compression		0.60	1.67	
Static Test - Tension		0.40	2.50	
Dynamic Analysis Compression		0.40	2.50	
Dynamic Test Compression (field measurement and analysis)		0.50	2.00	

Table F.4 – Partial SF sets for actions in ULS based on USA Codes ACI[4], AISC[5] and API[6] (Foye *et al.*, 2004)

Actions or effects of actions:	Code:	ACI	AISC	API
Permanent loads		1.40	1.20	1.10 -1.30
Live load		1.70	1.40	1.10-1.50

[2] *National Building Code of Canada - National Research Council of Canada*
[3] *Canada Highway Bridge Design Code – Canadian Standards Association*
[4] *American Concrete Institute (Building Code Requirements for Structural Concrete)*
[5] *American Institute of Steel Construction*
[6] *American Petroleum Institute*

Table F.5 – Partial SF sets for pile's ULS based on USA Code for bridges, AASHTO[7] (2007)

Actions:	Comb:	1	2	3	4	5
Permanent		γ_P	γ_P	γ_P	γ_P	γ_P
Variable		1.00 to 1.75	1.00 to 1.35	1.00 to 1.40	1.00	0.40 to 1.35

maximum and minimum values of γ_P:	Max	min
Component and attachments	1.25	0.90
Downdrag	1.80	0.45
Wearing surfaces and utilities	1.50	0.65
Horizontal earth pressure: active	1.50	0.90
Horizontal earth pressure: at rest	1.35	0.90
Locked-in erection stresses	1.00	1.00
Vertical earth pressure: overall stability	1.00	-
Vertical earth pressure: retaining walls and abutments	1.35	1.00
Vertical earth pressure: rigid buried structure	1.30	0.90
Vertical earth pressure: rigid frames	1.35	0.90
Vertical earth pressure: flexible buried structures other than metal box culverts	1.95	0.90
Vertical earth pressure: flexible metal box culverts	1.50	0.90
Earth surcharge	1.50	0.75

Ultimate bearing resistance of single drilled shafts:	Factor	1 / Factor
Side resistance (clay)	0.65	1.54
Base resistance (clay)	0.55	1.81
Side resistance (sand)	0.55	1.81
Base resistance (sand)	0.50	2.00
Side resistance (rock)	0.55	1.81
Base resistance (rock)	0.65	1.54
Side resistance and end bearing (load test)	0.80	1.25

Ultimate bearing resistance of single driven piles:		Factor	1 / Factor
Skin friction (clay)	α method	0.70λ	$1.43 / \lambda$
	β method	0.50λ	$2.00 / \lambda$
	λ method	0.55λ	$1.81 / \lambda$
End bearing (clay)		0.70λ	$1.43 / \lambda$
End bearing (rock)		0.50λ	$2.00 / \lambda$
Skin friction and end bearing	SPT (sand)	0.45λ	$2.22 / \lambda$
	CPT (sand)	0.55λ	$1.81 / \lambda$
Wave equation analysis with driving resistance		0.65λ	$1.54 / \lambda$
Load test		0.80λ	$1.25 / \lambda$

Method of controlling the installation:	λ
Pile Driving Formulas, e.g., ENR, equation without stress wave measurements during driving	0.80
Bearing graph from wave equation analysis without stress wave measurements during driving	0.85
Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g.:	
- the pile driving analyzer (PDA)	0.90
- PDA and static load test to verify capacity	1.00
- PDA and CAPWAP analyses to verify capacity	1.00
Stress wave measurements on 10% to 70% of piles, capacity verified by simplified methods, e.g., the PDA	1.00

[7] *American Association of State Highway and Transportation Officials (Bridge Design Specifications)*

In Japan considerable amount of work is being done to revise the major Japanese structural and geotechnical design codes from the traditional descriptive specifications to performance-based specifications, and from working (or allowable) stress design codes to the limit state design codes (Honjo & Kusakabe, 2000, 2002; Honjo, 2003; Honjo *et al.*, 2000b, 2003b, 2005, 2009, 2010d; Okahara *et al.*, 2003; Honjo & Nagao, 2007; Watabe *et al.*, 2009). Table F.6 shows some values of the partial factors adopted by the Japanese codes for port facilities - open type pier - in the design of pile foundations.

Table F.6 – Partial factors sets for pile's ULS based on Japanese codes for port facilities

Actions or effects of actions:	Action #: 1	2	3
Dead load Vertical load γ_q	1.00	1.00	1.00
Live load Horizontal force γ_{pH}	1.30	1.00	-
Live load Seismic action γ_{kh}	-	-	1.23
Resistance factors:			
Factor for cohesion γ_c	1.00	1.00	1.00
Factor for SPT-N value γ_N	1.00	1.00	1.00
Pull out resistance γ_{apull}	0.33	0.40	0.40
Compression γ_{acomp}	0.40	-	-
Compression (End bearing pile)	-	0.66	0.66
Compression (Friction pile)	-	0.50	0.50

Action #1 is for the effect of ship at berthing and during mooring.

Action #2 is for the effect of strong wind.

Action #3 is for strong seismic motion called level 1 earthquake.

Characteristic values

The definition of the characteristic value is still very discussed, especially in geotechnical engineering (Orr, 2000). In structural engineering problems, the value is normally defined as the lower fractile for resistances and upper fractile for loads (see Figure F.1).

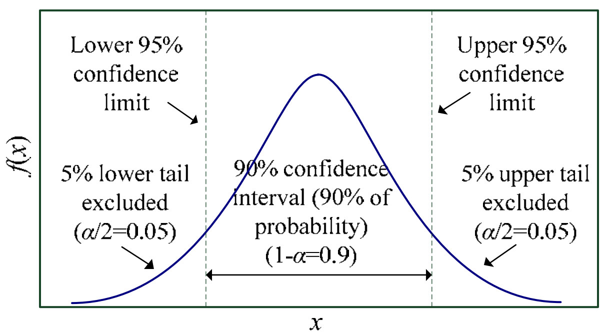

Figure F.1 – 90% Confidence interval

The characteristic value associated to *J%* fractile is calculated according to eq.(F.1):

$$
\begin{aligned}
& P[\text{variable value} \leq \text{characteristic value}] = J\% \\
& \Leftrightarrow \Phi\left[\frac{\text{characteristic value} - \text{mean value}}{\text{std deviation}}\right] = J\% \\
& \Leftrightarrow \text{characteristic value} = \text{std deviation} \cdot \left(\frac{\Phi^{-1}(J\%) + \text{mean value}}{\text{std deviation}}\right)
\end{aligned}
\tag{F.1}
$$

The Eurocode (CEN, 2002a) defines the characteristic value as the "value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series". As referred, this value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property. Also, a nominal value is used as the characteristic value in some circumstances.

Regarding other recommendation for the characteristic values, design codes say to choose it based on an assessment of the material actually in the ground and the way that material will affect the performance of the ground and structure in relation to a particular limit state (Eurocodes) or that it should be chosen based the value that is expected in principle, but not the mere average. The statistical errors in association with the testing method, the inhomogeneity of the soil itself, and limited number of the test data are taken into account in this method (Japanese design codes). For geotechnical engineering this decision will always depend on the engineer's judgement and experience. Other proposals are based on statistical methods, choosing fractiles of 5% (t student distribution), half of SD below mean (Schneider's) or based on the Bayesian approach, choosing characteristic values based on comparable experience.

Annex G – Relationship between reliability index and probability of failure

This annex presents the relationship between the reliability index (β) and the probability of failure (pf) through Table G.1 and Figure G.1, considering a Normal distribution. Remember that:

$$pf = \Phi(-\beta) = 1 - \Phi(\beta) \qquad \text{(G.1)}$$

Table G.1 – Relationship between reliability index and probability of failure (Normal distribution)

Reliability index, β	Probability of failure, pf		Reliability, $1 - pf$
0.00	0.5	5.00E-01	0.5
0.50	0.309	3.09E-01	0.691
1.00	0.159	1.59E-01	0.841
1.28	0.100	1.00E-01	0.900
1.50	0.0668	6.68E-02	0.9332
2.00	0.0228	2.28E-02	0.9772
2.33	0.0100	1.00E-02	0.9900
2.50	0.00621	6.21E-03	0.99379
3.00	0.00135	1.35E-03	0.99865
3.72	0.000100	1.00E-04	0.99990
3.50	0.000233	2.33E-04	0.999767
3.72	0.0001000	1.00E-04	0.999900
4.00	0.0000317	3.17E-05	0.9999683
4.26	0.00001	1.00E-05	0.99999
4.75	0.000001	1.00E-06	0.999999
5.20	0.0000001	1.00E-07	0.9999999
5.61	0.00000001	1.00E-08	0.99999999
6.00	0.000000001	1.00E-09	0.999999999

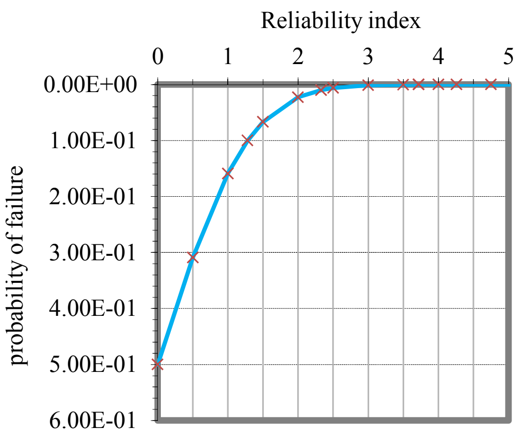

Figure G.1 – Relationship between reliability index and probability of failure (Normal distribution)

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Annex H – Empirical models based on SPT, CPT and PMT for pile vertical bearing capacity prediction

The resistance of piles can be estimated based on empirical methods (models) that are based in classic type of bearing capacity formulas and empirical factors. There are countless methods for different types of soils, different types of piles and different types of conditions. The ones selected are based on most known *in situ* tests SPT[8], CPT[9] and PMT[10] applicable to piles under vertical load in different types of soils and with information about the model error (bias) in its prediction. All methods have the same basic formula; the pile ultimate bearing capacity (R_u) of a single pile is calculated according to:

$$R_u = R_{tip} + R_{side} = A \times q_{tip} + U \times f_{side} \qquad \text{(H.1)}$$

Where R_{tip} is the tip resistance of the pile, R_{side} is the side resistance of the pile, q_{tip} is the predicted unit tip resistance, f_{side} is the predicted unit side resistance, A is the area of the tip of the pile ($\pi \cdot B^2$) and U the area of the pile in contact with the soil for side resistance ($\pi \cdot B \cdot D$).

So, the unit tip and side unit bearing capacities (q_{tip}, f_{side}) can be predicted by the following methods:

Table H.1 – Formulas for unit vertical bearing capacity of pile foundations

Method ID:	Unit tip resistance, q_{tip} (kPa)	Unit side resistance, f_{side} (kPa)	Reference(s)
SHB	$100N < 3{,}000$	$\begin{cases} 5 \times N < 200 & ,sand \\ 10 \times N < 150 & ,clay \end{cases}$	Honjo *et al.* (2002b); JRA (2001)
AIJ	$100N < 10{,}000$	$\begin{cases} 3.3 \times N & ,sand \\ Cu & ,clay \end{cases}$	Honjo *et al.* (2002b); AIJ (2000)
FRc	$k_c \cdot \dfrac{1}{b+3a}\displaystyle\int_{D-b}^{D+3a} q_c\ dz$	$\displaystyle\int_0^D \alpha \cdot [(a' \cdot q_c + b') \cdot (1 - e^{-c' \cdot q_c})]\ dz$ $q_c(z)$ in MPa	AFNOR (2012); Burlon (2011)
FRp	$k_p \cdot \dfrac{1}{b+3a}\displaystyle\int_{D-b}^{D+3a} pl\ dz$	$\displaystyle\int_0^D \alpha \cdot [(a' + b' \cdot pl) \cdot (1 - e^{-c' \cdot pl})]\ dz$ $pl(z)$ in MPa	AFNOR (2012); Burlon *et al.* (2012)

(...)

[8] Standard Penetration Test
[9] Cone Penetration Test
[10] Menard Pressuremeter Test

(...)

Method ID:	Unit tip resistance, q_{tip} (kPa)	Unit side resistance, f_{side} (kPa)	Reference(s)
S&F	$\begin{cases} 100 \times N_1 & , sand \\ 150 \times N_1 & , sandy\ gravel \end{cases}$	$\frac{(N_1 \times z)_{sands} + (5 \times N_1 \times z)_{clays}}{L}$	Shioi & Fukui (1982); Pereira (2003)
A&V	$\frac{K \times N_1}{F_1}$	$\frac{\alpha' \times K \times N_1}{F_2}$	Aoki & Velloso (1975); Schnaid (2000)

Where:

N: average value of SPT test, around pile base for tip resistance or around pile embedment length.

Cu: average value of undrained shear strength or undrained cohesion.

k_c: factor related to pile dimensions (D, B) and also type of soil. This factor assumes values between 0 and 1.

a,b: factors depending on pile diameter and length: $a = Max\left\{\frac{B}{2}, 0.5\right\}; b = min\{a, D\}$.

α: factor depending on the type of pile and type of soil, assuming values between 0 and 4 for PMT method and between 0 and 3 for CPT method.

pl: value of the limit pressure of the PMT test, around pile base for tip resistance or around pile embedment length.

a',b',c': factors depending on the type of soil, assume values between 0 and 4 for PMT method and between 0 and 0.5 for CPT method.

k_p: factor related to pile dimensions (D, B) and also type of soil. This factor assumes values between 1 and 4.

q_c: value of the unit tip resistance of the CPT test, around pile base for tip resistance or around pile embedment length.

K: factor related to the type of soil. This factor assumes values between 200 and 1000 kPa.

N_1: corrected average value of SPT test, around pile base for tip resistance or around pile embedment length. The N values were corrected using 100 kPa stress as reference and the method proposed by the Eurocode 7 (CEN, 2007).

F: factors related to the type of pile. For driven piles F_1=1.75 , F_2=3.50 and for bored piles F_1=3.0 to 3.5 , F_2=6.0 to 7.0.

α': factor related to the type of soil. This factor assumes values between 1 and 6 %.

For more details about how to calculate or which values to assume for the empirical factors, please refer to the works in table above (references).

Annex I – Japanese PWRI database data

To determine the model error of another two empirical methods based on SPT (Standard Penetration Test) a total of 12 vertically loaded bored piles were collected and used from the database of Japan, by Public Works Research Institute (PWRI) (Okahara *et al.*, 1991).

The bored piles were selected because it is the type of pile most used in Portugal, but also because they had information about the SPT data and load static test results. A data summary can be consulted in Table I.1.

Table I.1 – Information of the pile shafts (reinforced concrete) from Japanese database (PWRI)

#	Pile id.	Type of load	Cross section dimention (m)	Length (m)	Embedded length (m)	Ultimate bearing capacity (kN)
1	5108	Compression/Bearing pile	1.50	26.00	26.00	11,279
2	5110	Compression/Bearing pile	1.50	20.00	18.00	18,345
3	5111	Compression/Bearing pile	1.20	23.50	22.50	15,700
4	5112	Compression/Bearing pile	1.50	44.90	43.90	19,991
5	5113	Compression/Bearing pile	1.00	51.50	49.50	13,364
6	5114	Compression/Bearing pile	2.00	38.00	37.00	29,484
7	5128	Compression/Bearing pile	1.20	37.85	37.05	7,840
8	5129	Compression/Friction pile	1.00	34.14	33.64	4,260**
9	5130	Compression/Friction pile	1.00	18.90	17.90	6,034
10	6511	Compression/Friction pile	2.00	38.10	37.00	30,112
11	6528	Compression/Bearing pile	1.20	23.50	22.50	5,364
12	6529	Compression/Bearing pile	1.20	31.00	30.00	7,687

** Pile 5129 has few data points, therefore the fitting is considered as not reliable.

The following figures show the results of the SPT and the static load tests performed on each pile of the Japanese PWRI database. Also, to determine the ultimate bearing capacity (criterion - 10% of diameter), hyperbola model were used to approximate the load tests not performed to failure.

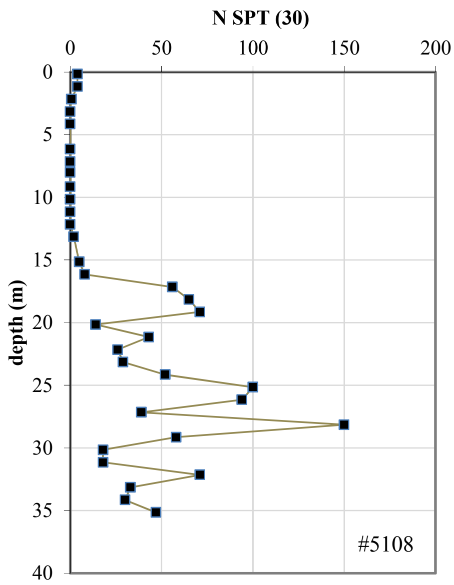

Figure I.1 – SPT data of pile #5108

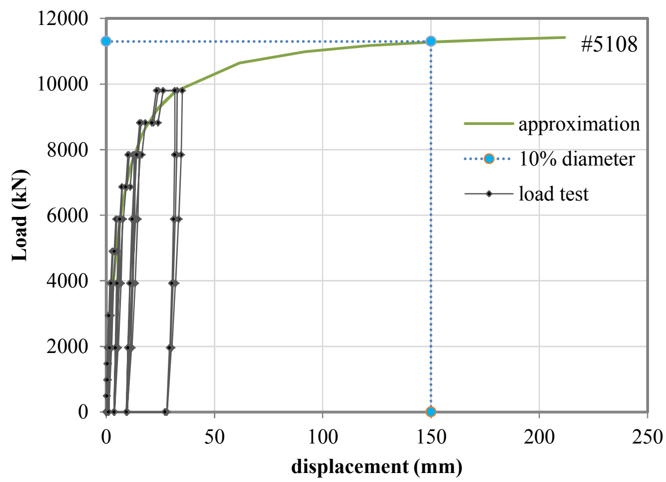

Figure I.2 – Load test result and approximation for pile #5108

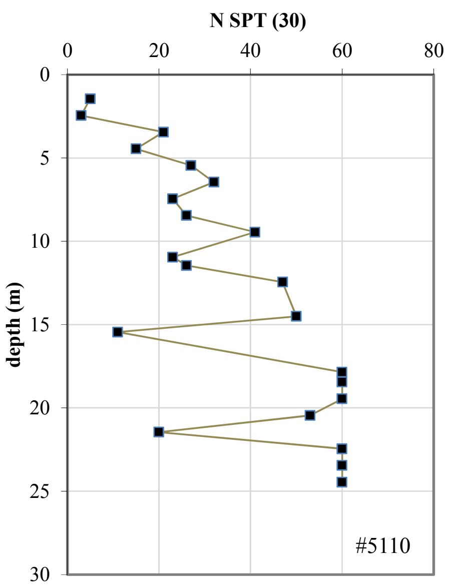

Figure I.3 – SPT data of pile #5110

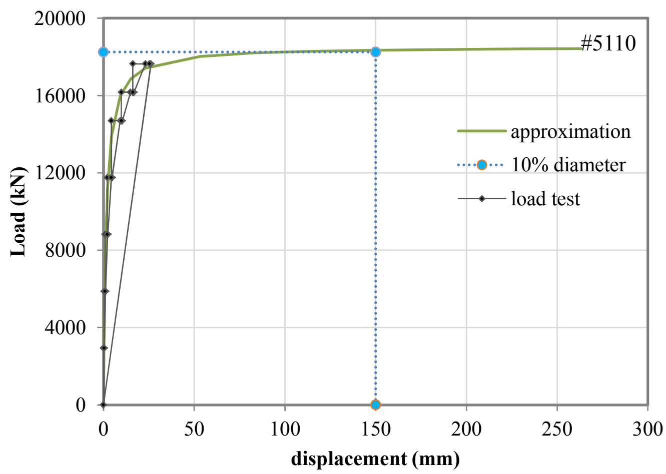

Figure I.4 – Load test result and approximation for pile #5110

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Annex J – Model error for SPT-based empirical models SHB, S&F and A&V, using the Japanese PWRI database

In order to determine the model error of the three methods chosen: SHB, S&F and A&V, the prediction of the vertical bearing capacity of the 12 piles of the Japanese PWRI database compiled was done (Annex I). The results of the predictions are shown in Table J.1 for each pile, also showing the bias (eq.(J.1)) for each method. Finally the model error, Mean[δ], and its standard deviation (SD) were determined.

$$\text{bias} = \frac{R_{u,\text{load test}}}{R_{u,\text{predicted}}} \qquad \text{(J.1)}$$

Table J.1 – Results of the prediction of pile vertical bearing capacity to determine model error

#	Pile ID:	SHB	bias	S&F	bias	A&V	bias
1	5108	13,344	0.85	9,617	1.18	5,556	2.03
2	5110	10,902	1.53	5,067	3.29	8,756	1.90
3	5111	14,149	1.29	7,984	2.29	14,526	1.26
4	5112	12,720	1.57	9,410	2.13	16,352	1.22
5	5113	7,347	1.82	3,453	3.88	6,227	2.15
6	5114	18,182	1.62	16,187	1.82	30,015	0.98
7	5128	12,403	0.63	2,810	2.79	1,716	4.57
8	5129	10,595	0.40	2,670	1.59	3,480	1.22
9	5130	5,491	1.09	1,431	4.19	2,505	2.39
10	6511	20,452	1.47	8,315	3.61	14,161	2.12
11	6513	6,151	0.87	1,563	3.42	1,046	5.12
12	6528	9,620	0.80	1,872	4.11	1,635	4.71
		Mean[δ]	1.16	-	2.86	-	2.47
		SD[δ]	0.45	-	1.04	-	1.48
		COV (%)	39	-	36	-	60
			SHB		S&F		A&V

The following Figures (J.1, J.2 and J.3) display the graphical representations of the bias for each pile and each method.

The pile number 8 (ID 5129) was considered for the calculations, although the value of the test was said to be unreliable. This is justified by the fact that this pile did not show different behaviour in the prediction, not an outlier.

When comparing the SHB method's uncertainties form the reference and here calculated, one can conclude that the results can be considered as exactly the same (Chapter 4, section 4.4).

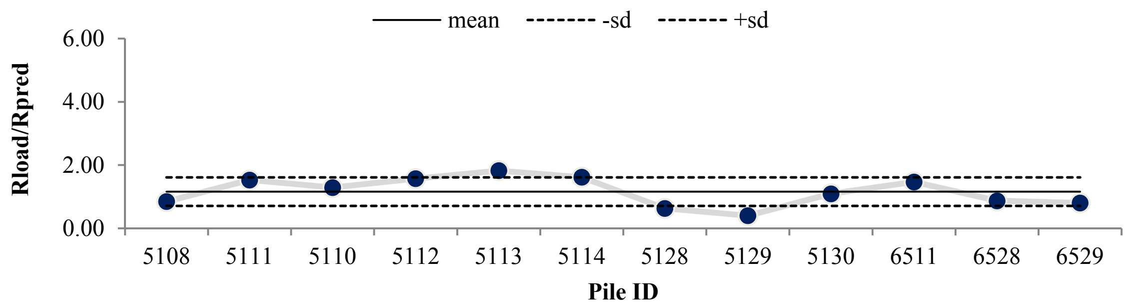


Figure J.1 – Predictions and bias for SHB method, using Japanese PWRI database

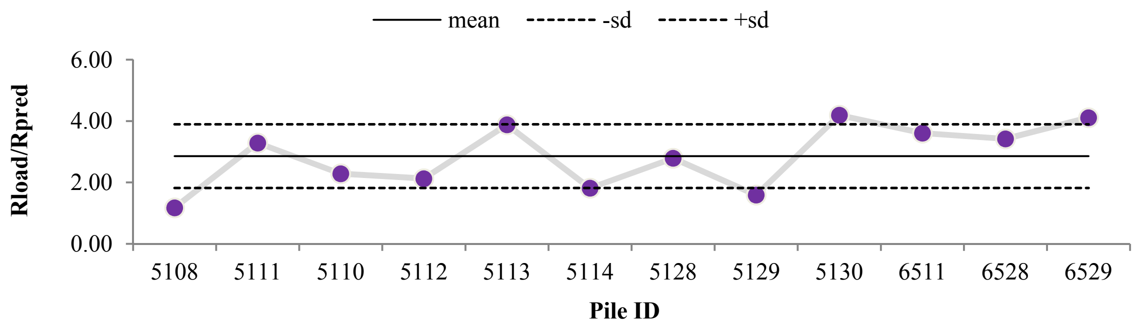


Figure J.2 – Predictions and bias for S&F method, using Japanese PWRI database

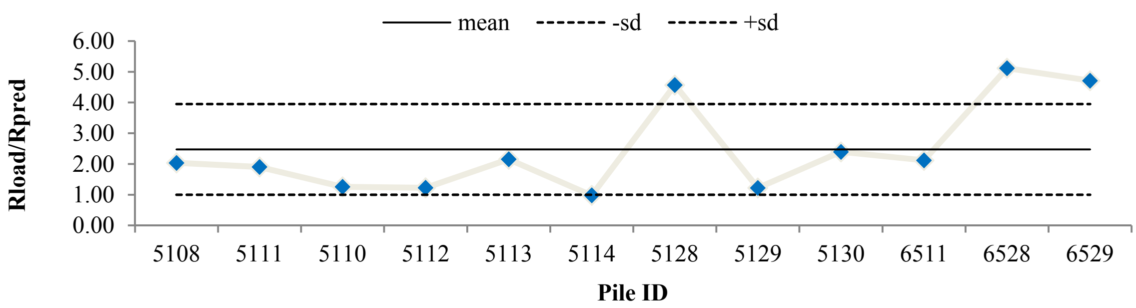


Figure J.3 – Predictions and bias for A&V method, using Japanese PWRI database

Annex K – Study of the soil variability from CPT data (example)

The data used for this study belongs to the experimental site of FEUP in Porto, Portugal (Viana da Fonseca & Santos, 2008). A great amount of tests were performed on this site (especially SPT and CPT tests – see Figure K.1), allowing the analysis of soil variability. This annex presents the study of soil uncertainty from CPT data, with total of 9 *in situ* CPT tests – Figures K.2 and K.3.

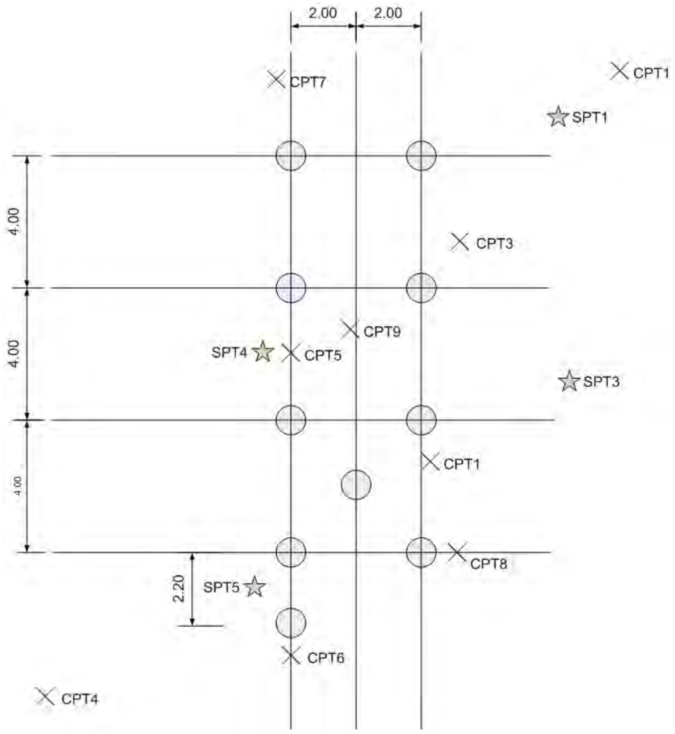


Figure K.1 – CPT tests location from experimental site of FEUP
(adapted from Viana da Fonseca & Santos, 2008)

As one can see (Figure K.2) the variability is big when considering the 9 CPT tests (low R^2, 0.25 for q_c and 0.40 for f_s), so based on geotechnical judgment some tests were removed from the analysis. They were assumed to have large measurement errors, since they differed considerably from the others, some had just few data so they were also removed. With the new data set, after removal of the outliers, the results were the following – Figure K.3.

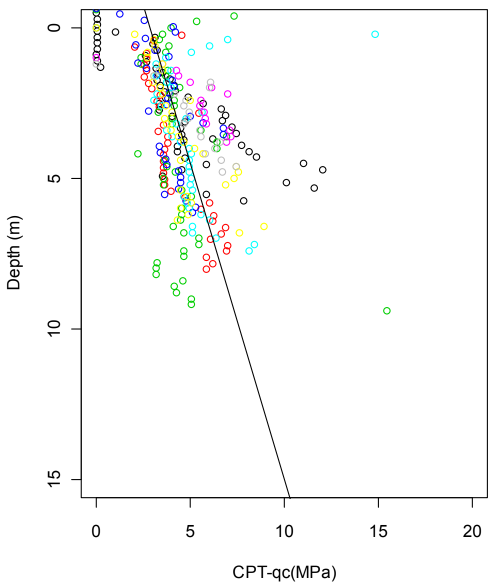
The trends obtained for this data set, after removal of the test considered as misleading (outliers), are clearly better and with bigger R^2. The histograms show that the q_c and the f_s residuals have a distribution similar to Normal and also, a slightly better fit to Q-Q plot for Normal distribution are obtained for this set.

Concerning the autocorrelation (refer to Annex C), the graphs show the relationship between the different points in depth of the CPT test, so, lag distance show the intervals of depth (CPT test is 20 cm). For both, q_c and f_s, the autocorrelation distance is about 20 to 40 cm (between 2nd and 3rd points – refer to section 2.2.3, Tables 2.2 and 2.3). This value is important for the control of the error in estimation of prediction of intermediate parameters and is also used for reducing the variance of the parameter of the test when local average of the parameter is considered in design (Vanmarke, 1977).

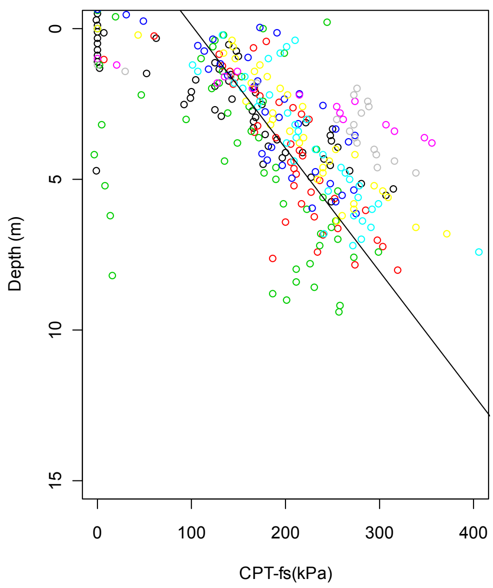
From these autocorrelation graphs is possible to comprehend that the qc parameter has a higher correlation between points than fs parameter. As such, the autocorrelation value (correlation length) will be in agreement with this.

Figure K.2 – Results obtained for soil variability study with all 9 CPT tests

q_c resistance	f_s resistance
Data and Trend	



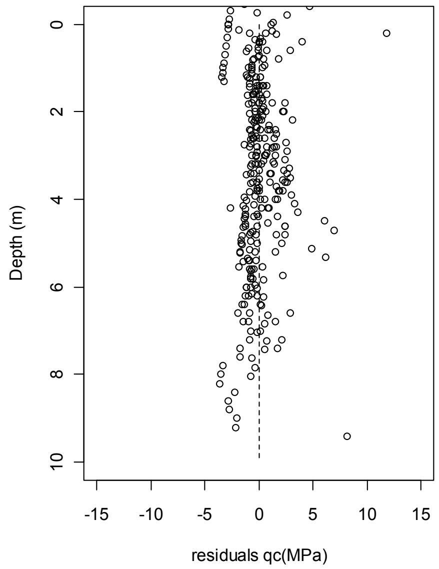
	trend	R^2
q_c (MPa)	2.8+0.5z	0.2536

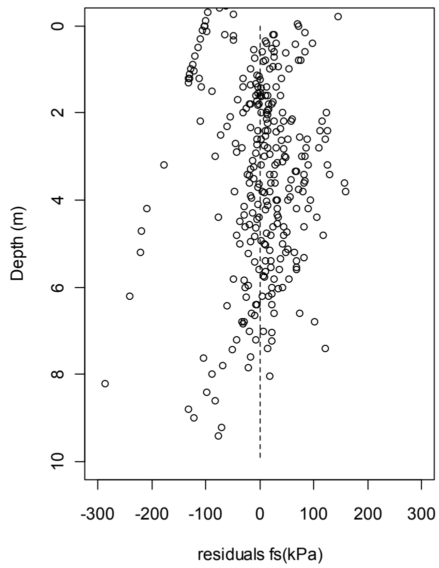


	trend	R^2
f_s (kPa)	103+24.5z	0.4043

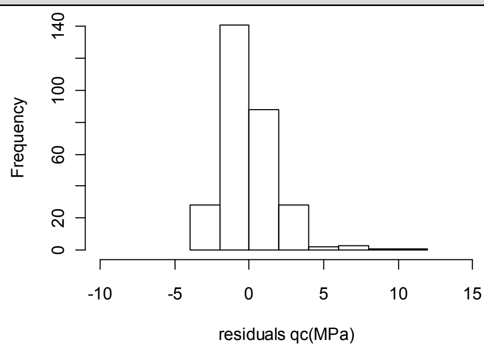
q_c resistance	f_s resistance

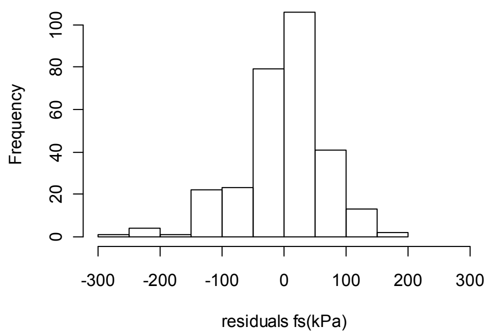
Residuals



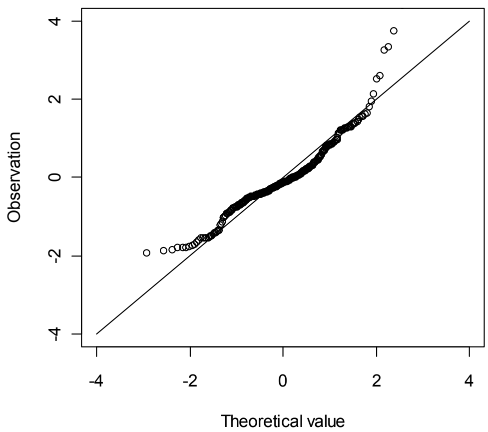


Histograms





Standard Normal Q-Q plot



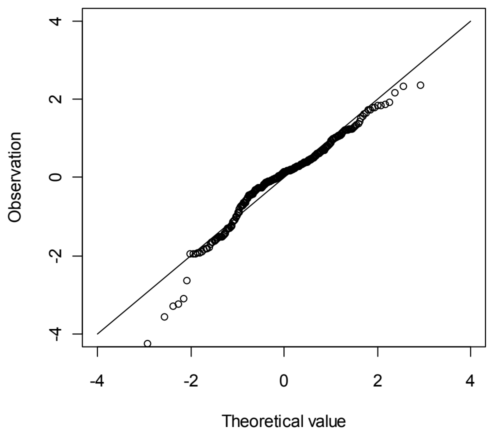
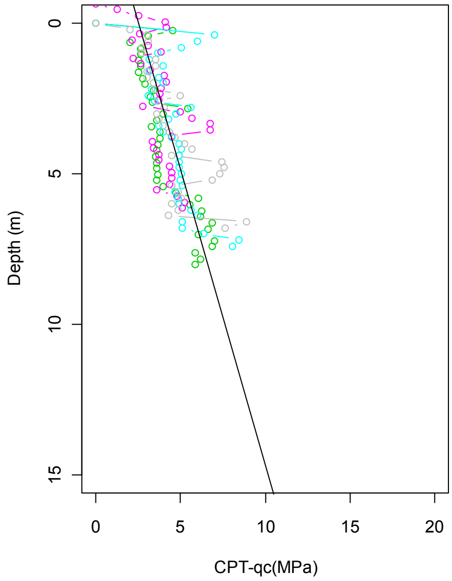


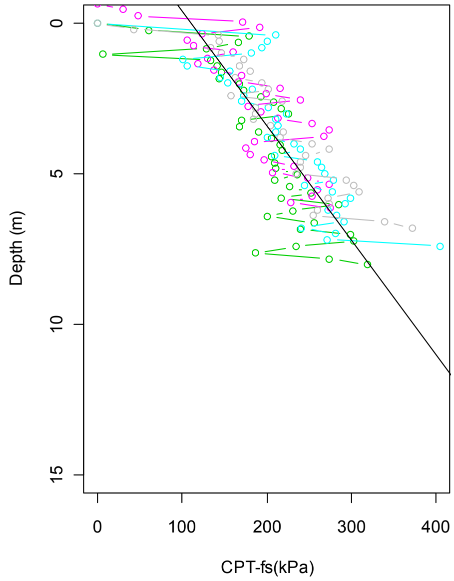
Figure K.3 – Results obtained for soil variability study with CPT tests # 3,6,5 and 8

q_c resistance	f_s resistance

Data and Trend

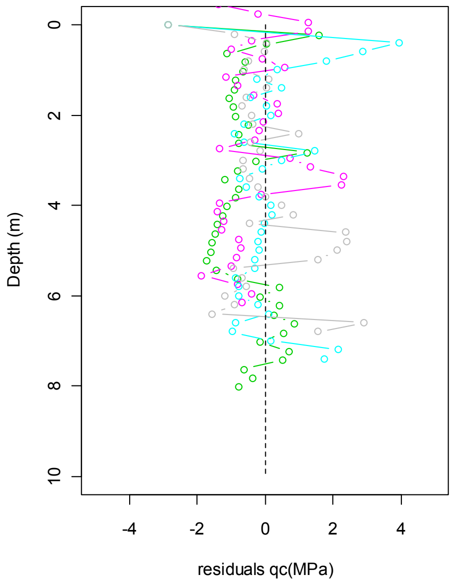


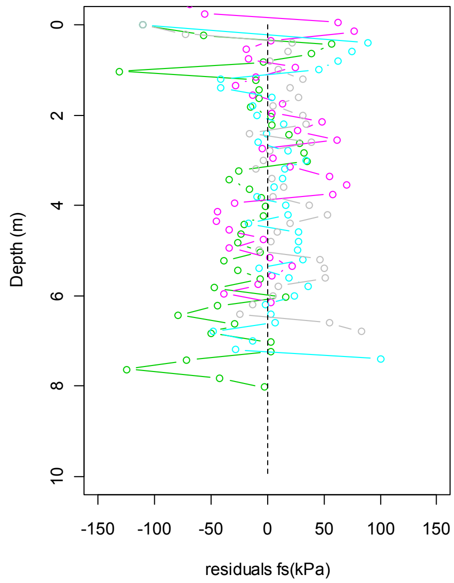
	trend	R^2
q_c (MPa)	2.5+0.5z	0.5038



	trend	R^2
f_s (kPa)	110.3+26.3z	0.6693

Residuals

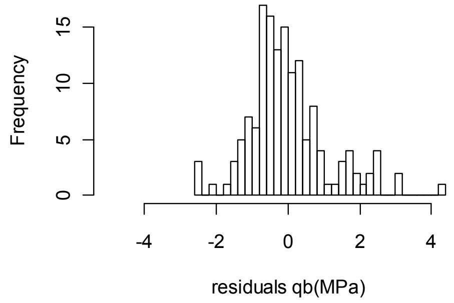


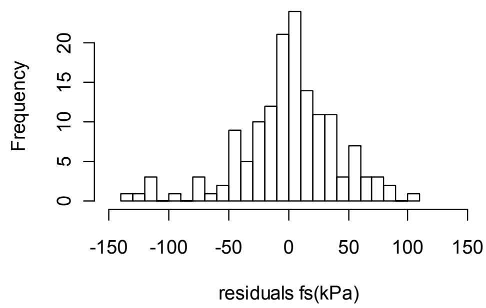


(...)

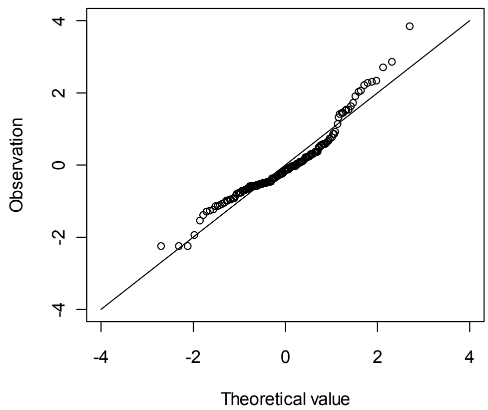
q_c resistance	f_s resistance

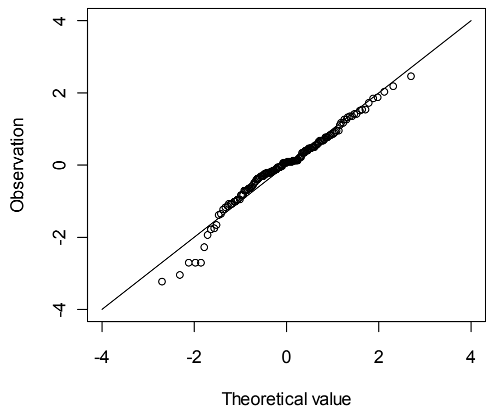
Histogram



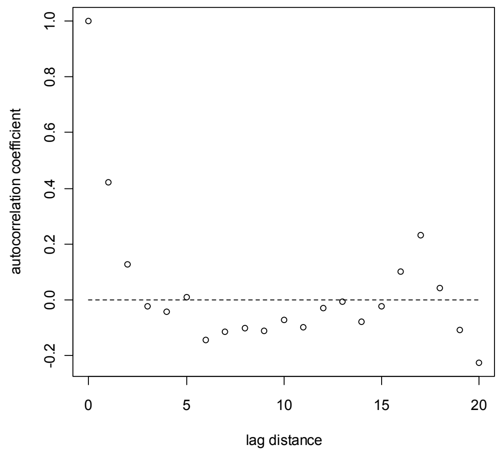


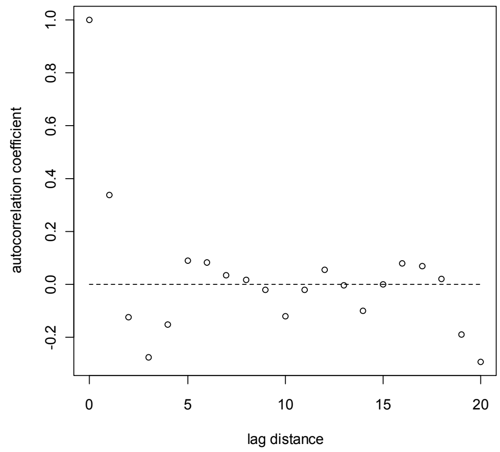
Standard Normal Q-Q plot





Autocorrelation graph





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Annex L – Application example input and output for FORM software

Input parameters

```
# # # Title of the problem
Analyse reliability of theoretical pile D=10m and B=1m
# # # Number of random variables
    3
# # # Definition of the coefficients of the performance function
   # Coefficient (independent) A:
        0.00
   # Coefficients (linear) Bi
        3.14   31.42 -1.00
   # Coefficients (quadratic) Cij
        0.00   0.00   0.00
        0.00   0.00
        0.00
# # # Characterisation of variables' uncertainties
   # Types of available laws of probability PDF:
   #      designation    parameter 1    parameter 2
   # 1) - normal           average    standard deviation
   # 2) - log-normal       average    standard deviation
   # 3) - uniform       lower limit     upper limit
   # 4) - Gumbel max.    u-central     alpha-deviation
   # 5) - Gumbel min.    u-central     alpha-deviation
   # 6) - Weibull min.   k-central      beta-deviation
 # variable#   PDF    parameter 1    parameter 2
         1      1       92.3           18.5
         2      1       19.4           5.8
         3      1       550            55
# # # Linear correlation coefficients between the variables (n lines)
   1.00  0.00   0.00
   1.00  0.00
   1.00
# # # Maximum number of iterations
      1000
# # # Tolerance for calculating Beta (Toler = | b(i) - b(i-1) |)
      1E-04
```

Output

```
# # # Title of the problem
Analyse reliability of theoretical pile D=10m and B=1m

***  Number of variables =   3

*** coefficients of the performance function:
g(X) = A + Sum (Bi.Xi) + Sum (Cij.Xi.Xj)

    Coefficient (independent): A =    .000
    Coefficients (linear) Bi:
      3.140  31.420  -1.000
    Coefficients (quadratic) C 1j:
       .000    .000    .000
    Coefficients (quadratic)  C 2j:
       .000    .000
    Coefficients (quadratic)  C 3j:
       .000                                                     (…)
```

```
*** Characterisation of variables' uncertainties

   Types of available laws of probability PDF:
         designation    parameter 1    parameter 2
    1) - normal           average    standard deviation
    2) - log-normal       average    standard deviation
    3) - uniform        lower limit     upper limit
    4) - Gumbel max.    u-central     alpha-deviation
    5) - Gumbel min.    u-central     alpha-deviation
    6) - Weibull min.   k-central      beta-deviation

    variable#   PDF    parameter 1    parameter 2
         1      1       92.3           18.5
         2      1       19.4           5.8
         3      1       550            55

*** Correlation coefficients between the variables
    Correlation between variable 1 and the others:
  1.0000   .0000   .0000
    Correlation between variable 2 and the others:
  1.0000   .0000
    Correlation between variable 1 and the others:
  1.0000

*** Maximum number of iterations: 1000
*** Tolerance for calculating Beta (Toler = | b(i) − b(i-1) |): .100E-03

>>>>>>>>>>>> R E S U L T S  <<<<<<<<<<<<<

  ---------->  interaction    1

               - reliability index (Beta):     1.75544
               - value of the performance function :  .34937E+03
               - residual | b(i) − b(i-1)|       :  .17554E+01

               - sensitivity factors for each variable:
                      variable #       alfa value
                           1           .291879E+00
                           2           .915662E+00
                           3          -.276353E+00

               - design point coordinates :
                      variable #       coordinate
                           1           .828210E+02
                           2           .100771E+02
                           3           .576682E+03

  ---------->  iteraction    2

               - reliability index (Beta):     1.75544
               - value of the performance function :  .00000E+00
               - residual | b(i) − b(i-1)|       :  .00000E+00

               - sensitivity factors for each variable:
                      variable #       alfa value
                           1           .291879E+00
                           2           .915662E+00
                           3          -.276353E+00

               - design point coordinates:
                      variable #       coordinate
                           1           .828210E+02
                           2           .100771E+02
                           3           .576682E+03
```

Annex M – Information of DFLT database

The following figures (Figures M.1 to M.3) show the data of DFLT (Deep Foundation Load Test) database, such as the number of pile types, number of load tests and number and type of soil tests.

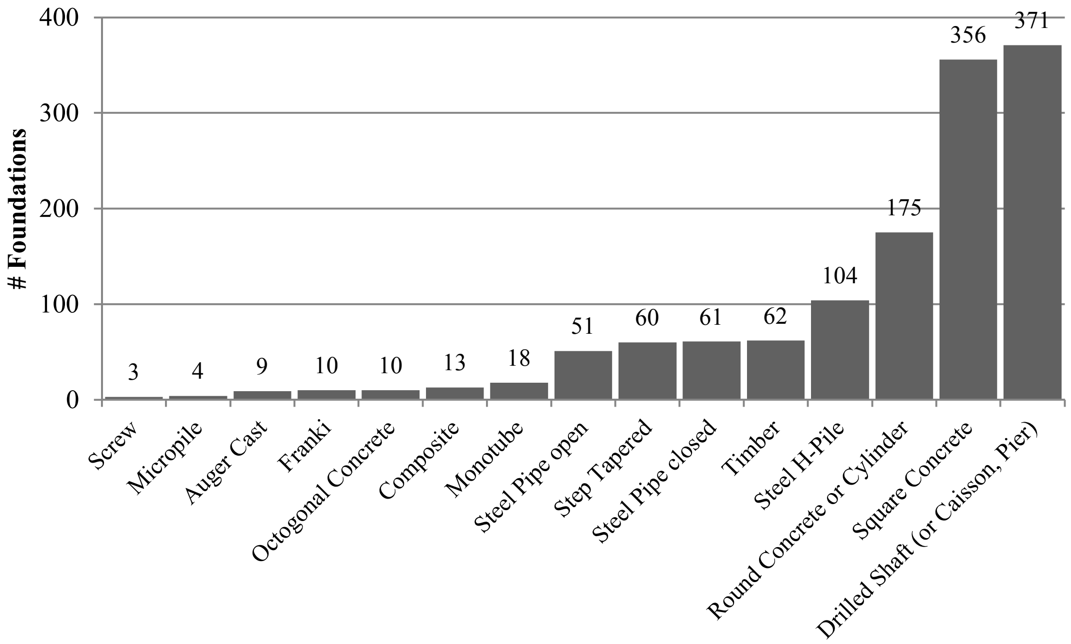


Figure M.1 – Type and number of pile foundations (DFLT database)

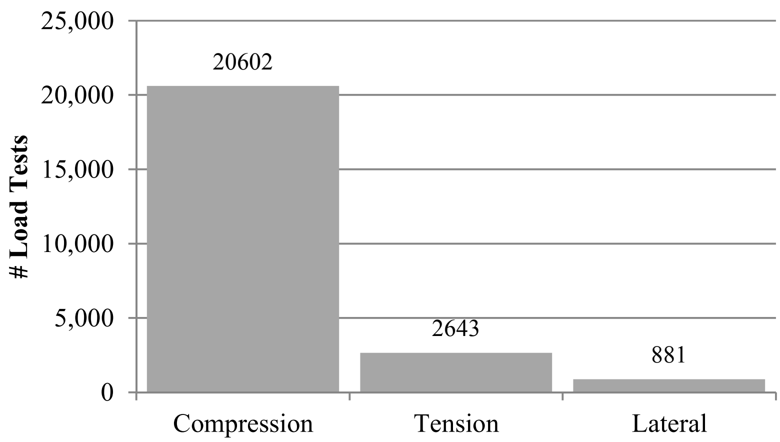
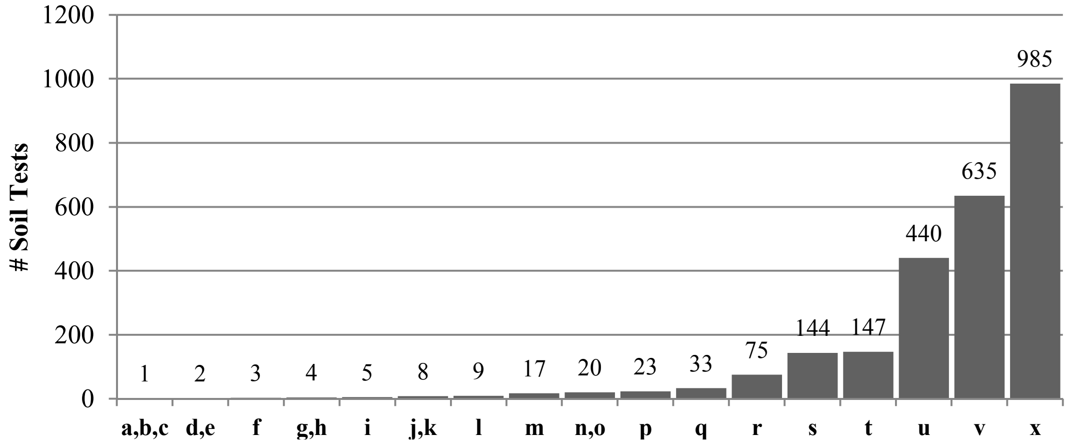


Figure M.2 – Type and number of static load tests (DFLT database)



Legend:

a	Miniature Cone	**m**	Texam Pressuremeter
b	Split Spoon Sample	**n**	Drirect Shear Test
c	Standard Penetration Test (automatic hammer)	**o**	Driven Texam Pressuremeter
d	Falling Head Permeability Test	**p**	Pocket Penetrometer
e	Field Vane Test	**q**	Triaxial Shear Test
f	Shelby Tube	**r**	Electric Penetromer Tip
g	Resonant Column	**s**	Consolidometer
h	Triaxial/Shelby Tube	**t**	Mechanical Cone
i	Self Boring Pressuremeter	**u**	Unconfined Compression Test
j	Dutch Mantle Cone	**v**	Physical Properties
k	Not Available	**x**	Standard Penetration Test
l	Step Blade Vane		

Figure M.3 – Type and number of soil tests (DFLT database)

Annex N – Stability of MCS for case study 1 (FEUP)

Here are presented the calculations carried out to determine the number of simulation necessary to undertake Monte Carlo Simulation (MCS) analyses. Normally one can start to determine the number of simulations (n) needed based on the probability of failure (pf) required. This number can be achieved with formulas, like eq.(N.1), or with a simple definition of how many failures one wishes to obtain with the simulations. For example, for a $pf = 10^{-3} = 1/1{,}000$, if one would want 100 failures, n should be 100/1 x 1,000 = 100,000.

According to Henriques (1998), citing Broding in 1964, it reports that the value of n, necessary to achieve a confidence level c in a pf, is calculated as follows:

$$n > \frac{-ln(1-c)}{pf} \qquad \text{(N.1)}$$

Where n is the number of MCS, c is the confidence level on the result and pf the probability of failure.

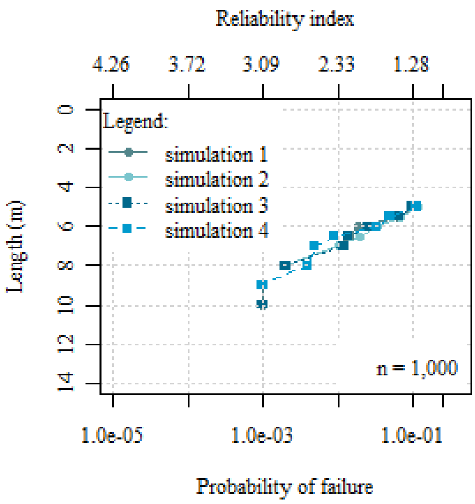
According to Henriques (1998), Bjeranger in 1990 suggested that n should be in the range between $1/pf$ and $10/pf$. Therefore, n should be between the values depicted in Table N.1. The interval considered was 6×10^{-3} and 3×10^{-5} [corresponding to β of 2.5 and 4.0] as previously discussed in Chapter 3, section 3.3.

Table N.1 – Values recommended for the number of MCS (n)

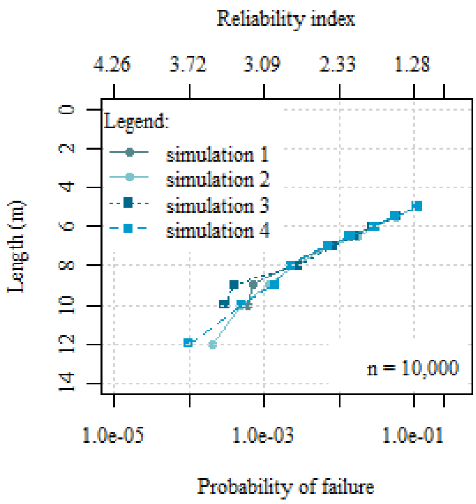
Method / Reference	β	pf	n
Based on number of failures Broding (1964) (c=0.95) Bjeranger (1990)	2.5 to 4.0	6×10^{-3} to 3×10^{-5}	16,667 to 3,333,333 500 to 100,000 167 to 333,333

The first method is the most conservative. Based on these intervals the stability of pf was studied for each case. The following figures present the calculations carried out. As one can see, the problem of stability increases with higher probabilities. It is seen that, for a low value of n, pf has a high deviation and sometimes low values of n also makes it impossible to obtain a result (pf=0).

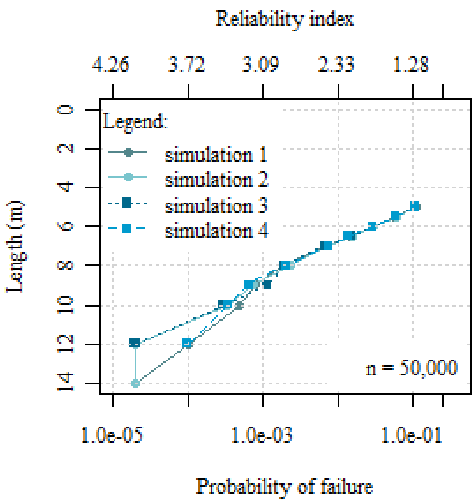
Therefore, for case study 1 it was concluded that n could be 200,000 simulations (Figure N.1) taking into account the values in Table N.1, assuming a minmum pf of 10^{-4} (β of 3.7) and especially considering the time consumed (see Figure N.2 for computational time).I



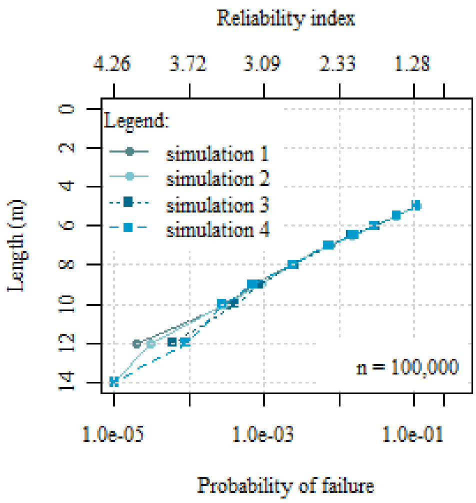
a) 1,000 (1.7 seconds)



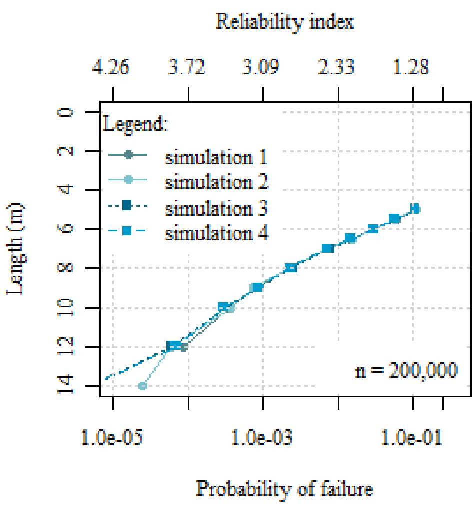
b) 10,000 (15 seconds)



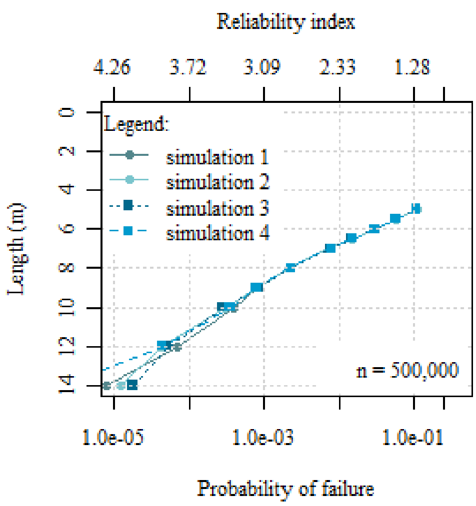
c) 50,000 (1.2 minutes)



d) 100,000 (2.5 minutes)

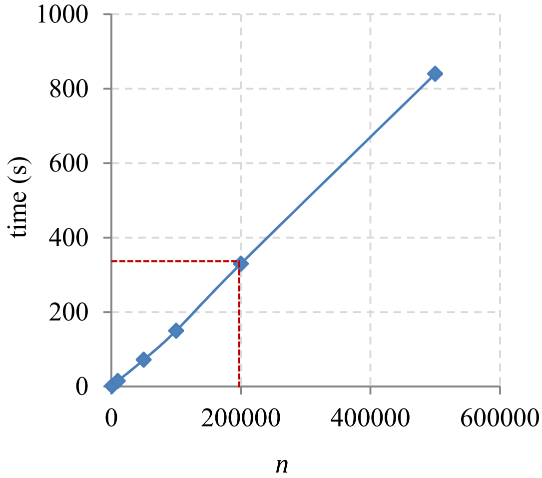


e) 200,000 (5.5 minutes)

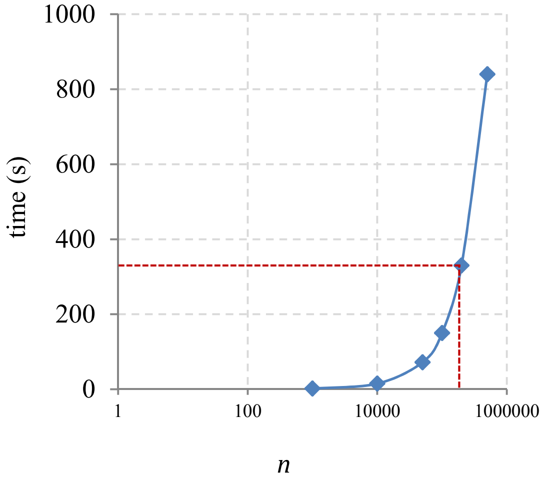


f) 500,000 (14 minutes)

Figure N.3 – Results of the stability study of the number of MCS for case study 1 (FEUP)



a) Normal scale



b) Logarithmic scale

Figure N.2 – Computational time of each n MCS (using Windows 7, Core 2 Duo CPU T9300 @ 2.50GHz, 4.00 GB RAM)

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Annex O – Results of dynamic load testing and signal matching of pile PPR1-B - Case study 2 (bridge)

The figures in this annex show the results obtained by dynamic load test and signal matching of steel pipe pile PPR1-B of case study 2 (bridge). These results were prepared by GeoDrive Technology BV.

RESULTS FOR REDRIVE 1

The energy, peak force at sensor level and driving resistance versus the applied blow number is shown below. After about 20 blows the fuel pressure was increased and the hammer delivered more energy.

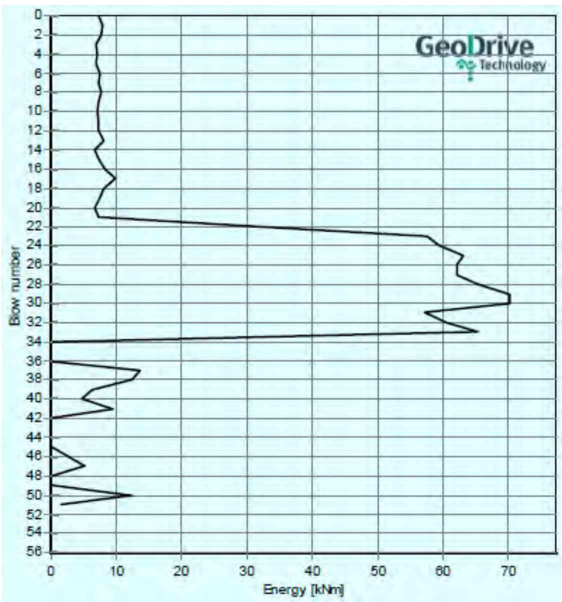

Figure O.1 – Transferred energy as function of blow number (PPR1-B)

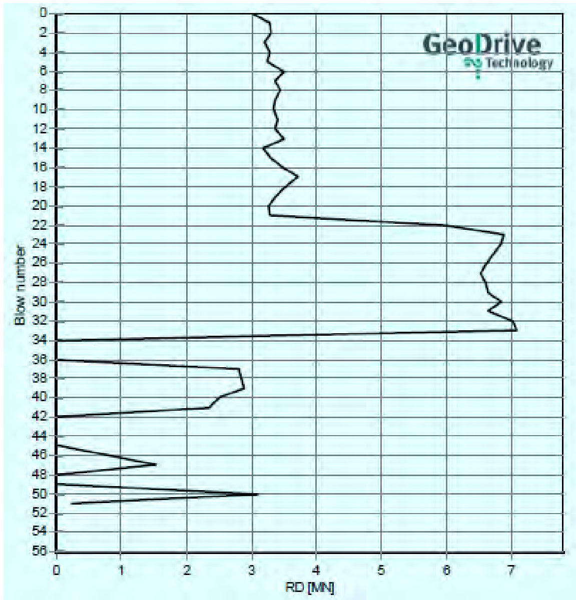

Figure O.2 – Resistance of driving as function of blow number (PPR1-B)

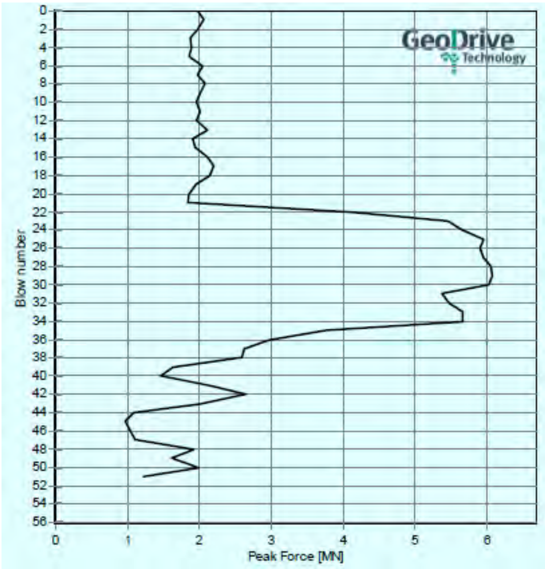

Figure O.3 – Peak force as function of blow number (PPR1-B)

Results for blow 22:

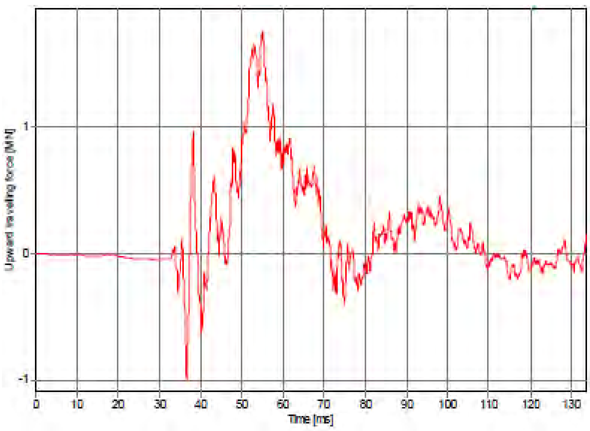

Figure O.4 – Upward travelling force for blow 22 (PPR1-B)

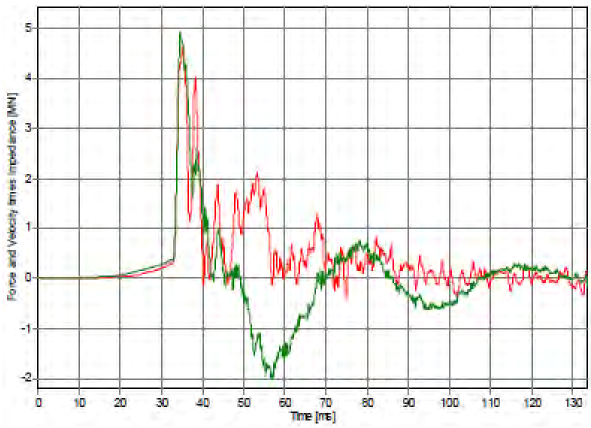

Figure O.5 – Force and velocity times impedance for blow 22 (PPR1-B)

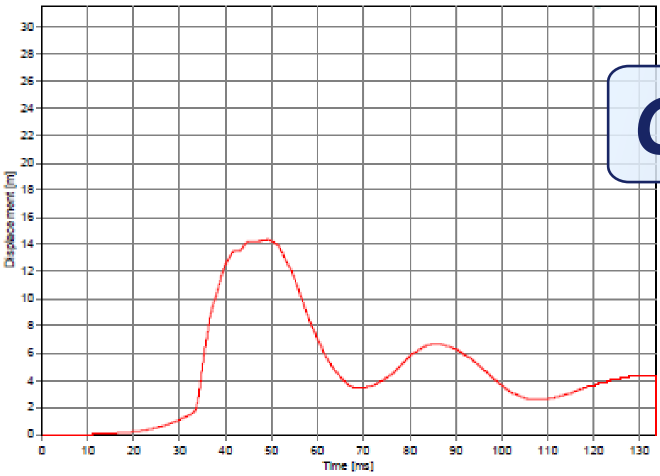

Figure O.6 – Displacement for blow 22 (PPR1-B)

```
MRF results for 22
------------------------------
Date   [illegible]
Time       22:32:40

Maximum Case Method
Static resistance 6.009 [MN]
Dyn    resistance 0.000 [MN]
TOTAL  resistance 6.009 [MN]
```

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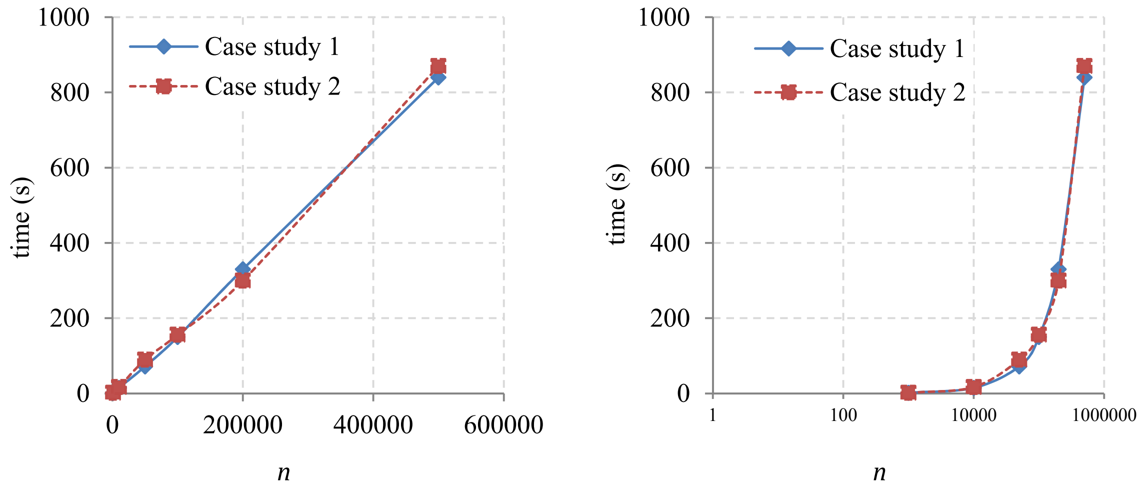
Annex P – Stability of MCS for case study 2 (bridge)

As presented for case study 1 (FEUP) in Annex N, here is presented the stability study for case study 2 (bridge). In other words, here are presented the calculations carried out to determine the number of simulation necessary to undertake stable analyses using MCS.

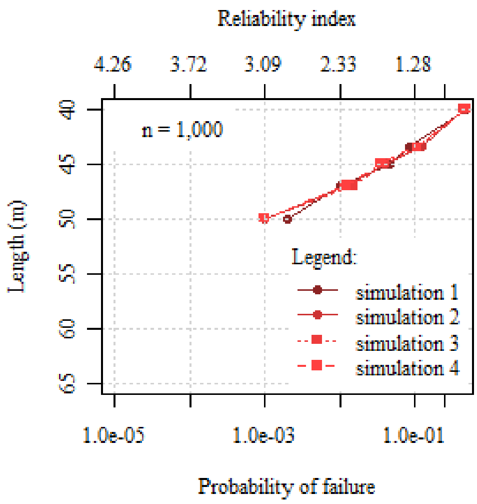
The values in Table N.1 (Annex N) are also appropriate to this case. Considering the interval6×10^{-3} and 3×10^{-5} [corresponding to β of 2.5 and 4.0] – Annex N, is concluded that case study 2 achieves stability for n = 500,000 simulations (Figures P.2). The time consumed for the calculation of these simulations is indicated in each figure, and a comparison between all simulations and also a comparison with case study 1's simulations is presented in Figure P.1.



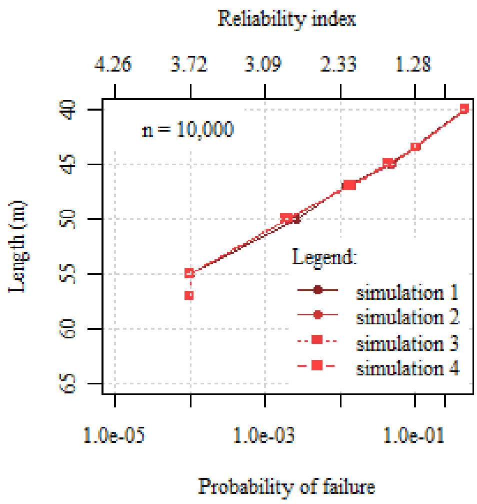
a) Normal scale

b) Logarithmic scale

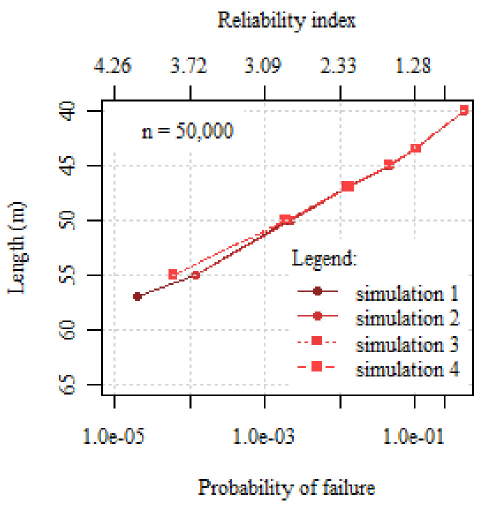
Figure P.1 – Computational time of each n MCS (using Windows 7, Core 2 Duo CPU T9300 @ 2.50GHz, 4.00 GB RAM)



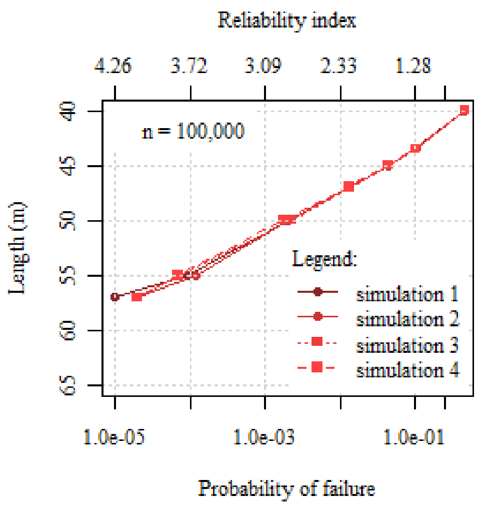
a) 1,000 (1.7 seconds)



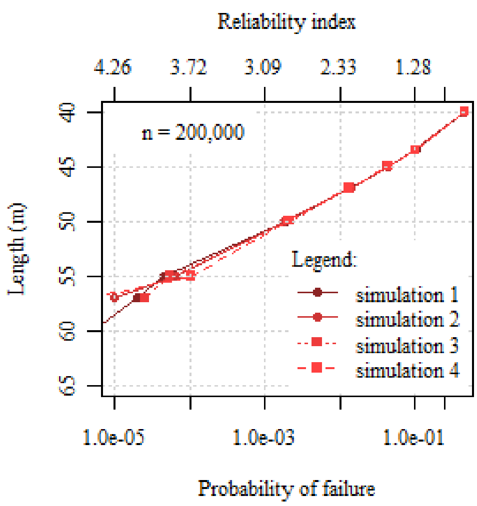
b) 10,000 (17 seconds)



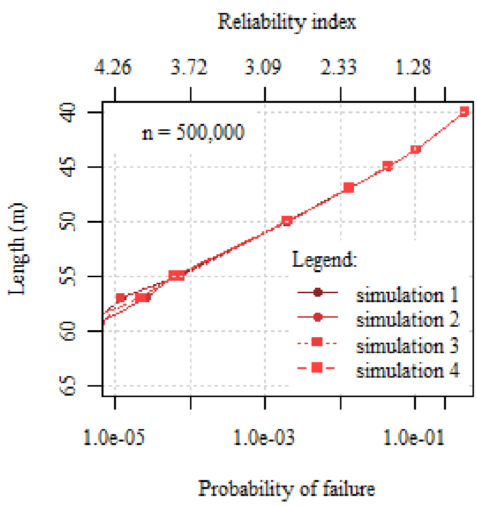
c) 50,000 (1.5 minutes)



d) 100,000 (2.6 minutes)



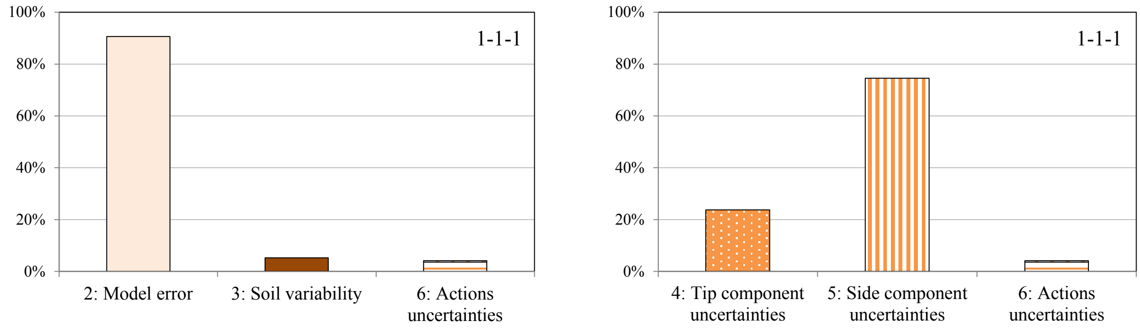
e) 200,000 (5 minutes)



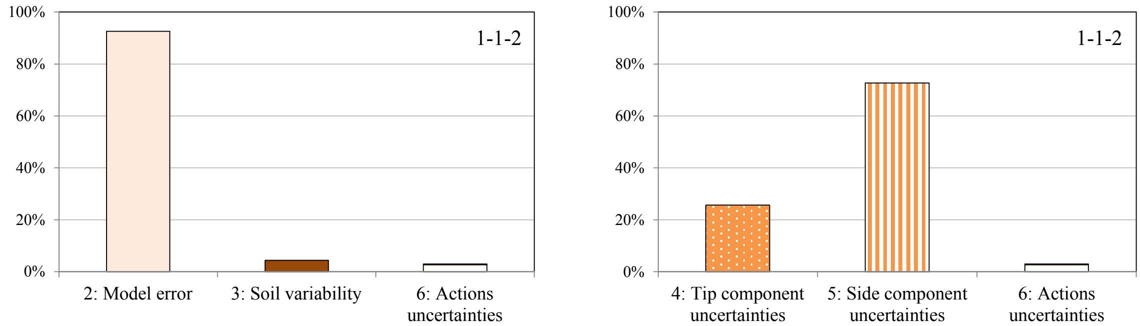
f) 500,000 (14.5 minutes)

Figure P.3 – Results of the stability study of the number of MCS for case study 2 (bridge)

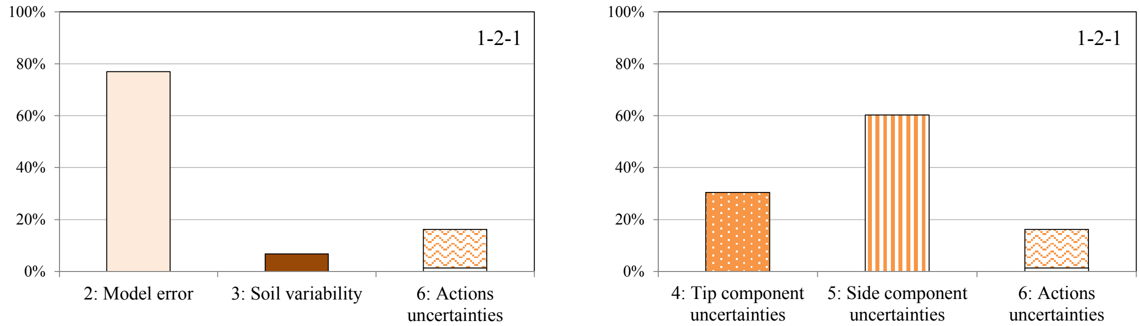
Annex Q – Individual results of the relative influence of the uncertainties (reliability-based sensitivity analyses) for case study 2



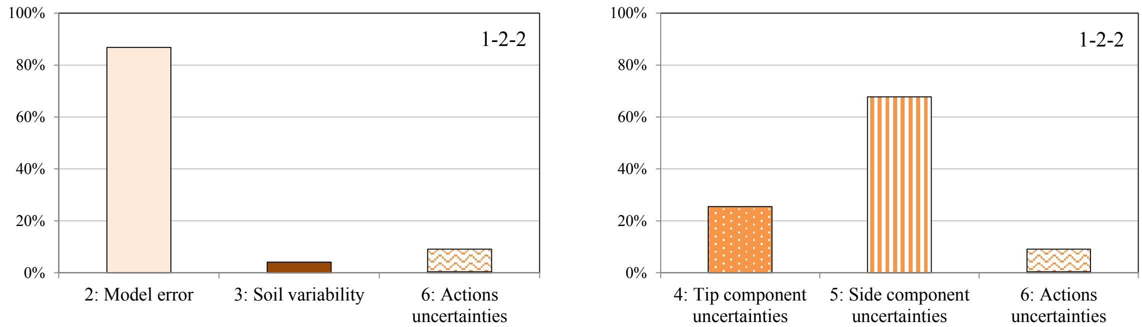
a) LC-1 and uncertainties Set 1



b) LC-1 and uncertainties Set 2

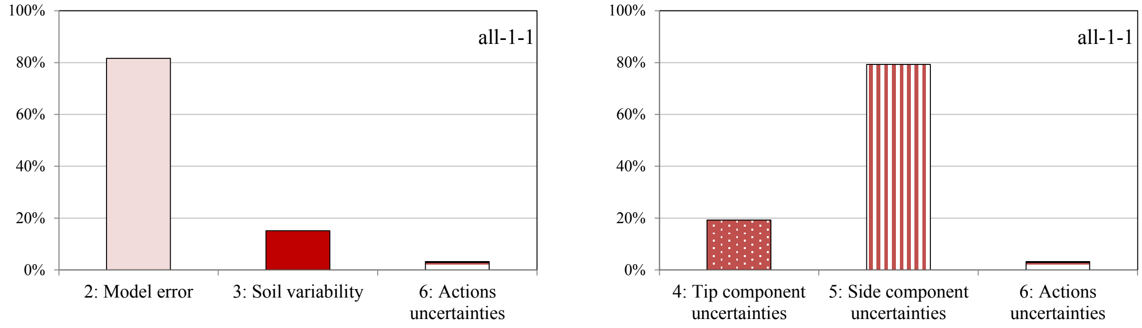


c) LC-2 and uncertainties Set 1

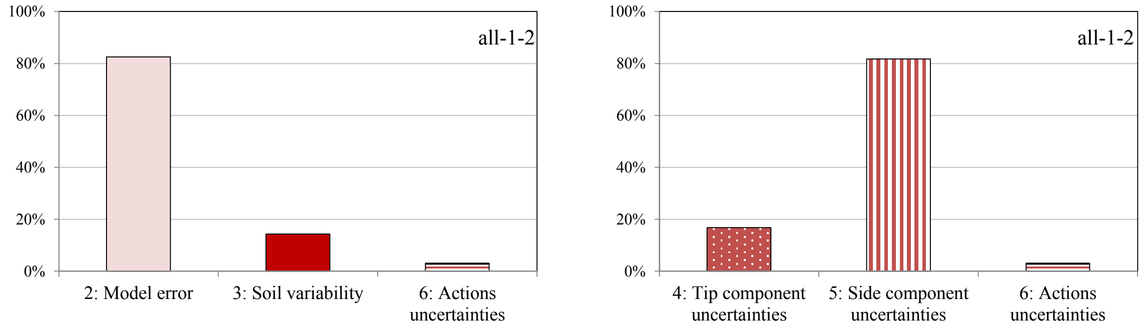


d) LC-2 and uncertainties Set 2

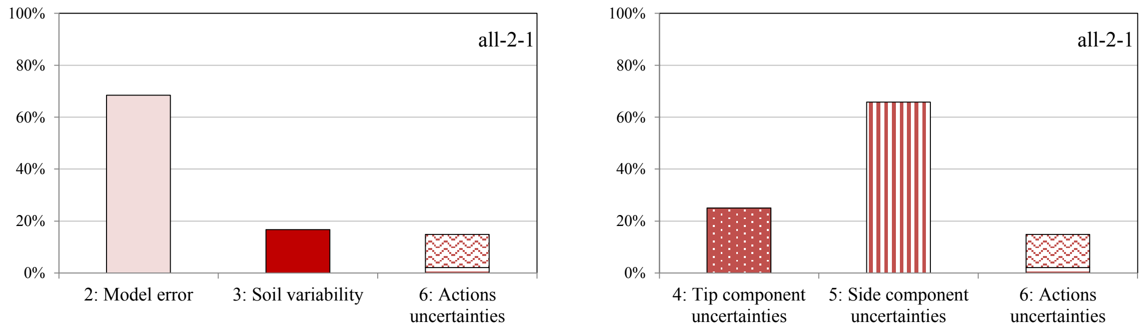
Figure Q.1 – Relative influence results from reliability-based sensitivity analyses, using MCS and soil variability from 1 SPT, case study 2 (bridge)



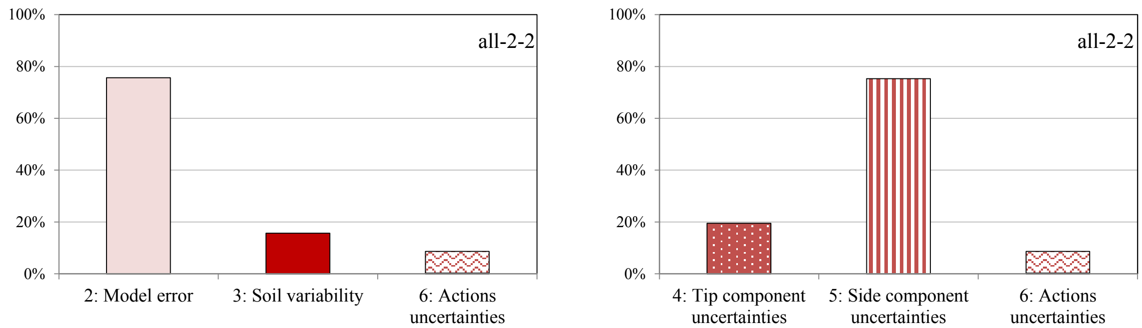
a) LC-1 and uncertainties Set 1



b) LC-1 and uncertainties Set 2



c) LC-2 and uncertainties Set 1



d) LC-2 and uncertainties Set 2

Figure Q.2 – Relative influence results from reliability-based sensitivity analyses, using MCS and soil variability from all SPT, case study 2 (bridge)