

DRIVEABILITY STUDY FOR XL OFFSHORE MONOPILE FOUNDATIONS

JOÃO MARIA FERREIRA DE LEMOS FALCÃO

Dissertação submetida para satisfação parcial dos requisitos do grau de mestre

MESTRE EM ENGENHARIA CIVIL — ESPECIALIZAÇÃO EM GEOTECNIA

Orientador: Professor António Joaquim Pereira Viana Da Fonseca

Coorientadores: Ole Hededal; Kristine Lee Kaufmann

JUNHO DE 2016

Versão Provisória

MESTRADO INTEGRADO EM ENGENHARIA CIVIL 2015/2016

DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446



miiec@fe.up.pt

Editado por

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Rua Dr. Roberto Frias

4200-465 PORTO

Portugal

Tel. +351-22-508 1400

Fax +351-22-508 1440



feup@fe.up.pt



<http://www.fe.up.pt>

Reproduções parciais deste documento serão autorizadas na condição que seja mencionado o Autor e feita referência a Mestrado Integrado em Engenharia Civil - 2015/2016 - Departamento de Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal, 2016.

As opiniões e informações incluídas neste documento representam unicamente o ponto de vista do respetivo Autor, não podendo o Editor aceitar qualquer responsabilidade legal ou outra em relação a erros ou omissões que possam existir.

Este documento foi produzido a partir de versão eletrónica fornecida pelo respetivo Autor.

Aos meus Pais

The best preparation for tomorrow is doing your best today

H. Jackson Brown, Jr.

ACKNOWLEDGMENTS

I would like to express my most sincere gratitude for all those who have accompanied me in this crucial part of my life. In particular, I want to acknowledge:

- Professor Antonio Viana for all the support, suggestions, ideas and guidance offered during my stay in Denmark. But most of all for being responsible for the arrangement with COWI A/S which ultimately made possible, for me to embark on this exiting journey.
- All the colleagues and staff at COWI A/S, for the warm welcoming. During my stay I never felt left out and felt part of the group. Thank you for the experience given on one of the leading consultant groups and for the insight delivered on new and exciting projects.
- Kristine Lee Kaufmann and Ole Hededal my supervisors at COWI A/S, for all the feedback provided, interest demonstrated in the project and for being always available to help.
- All of the Portuguese coworkers, for providing me a wonderful time in Copenhagen, particularly José Calejo. Thank you for receiving me so well and for all the pleasant conversations we had
- To all my friends and family, who shaped me to be the person I am today.
- To my parents, brothers and grandparents for all the support and care given, but most of all, thank you for the encouragement given, this work would not have been possible without your support.

ABSTRACT

Offshore monopiled foundations for wind turbines have reached a size that challenges the basis of the impact driving analysis and the empirical factored knowledge available traditionally for offshore oil and gas jacket piled foundations.

There is a need to perform an appropriate and thorough back analysis of the XL monopile foundations already installed to improve future driving predictions, and to assess if the current used methods are still adequately to use in the offshore industry.

The present work was developed in conjunction with COWI A/S. The company is a leading consultant group which is involved in several offshore windfarm projects developed in the world. COWI A/S are mainly responsible for the design of the foundations of the offshore wind turbine support structures.

After an overview of the main concepts regarding driving, several methods to predict the soil resistance to driving, SRD are presented. The main methods analysed are the Alm and Hamre (2001) approach, the Imperial College Piling (2005) and the Stevens (1982) proposal. The first two methods are CPT based and consider the influence of friction fatigue, a phenomenon known to decrease the shaft resistance as the driving progresses. However, the two designs present different goals. The ICP methodology is more suitable for piles submitted to a static loading, since the piles tested were allowed to age for a period of 10 days and as such it considers set up effects. On the other hand, AH is best suited to analyse pile driveability. It is therefore difficult to clearly compare the two methods as they have different goals during the design stage. Hence, there was a need to remove the consideration of the set up effects in the ICP in order to compare it with AH and to perform an accurate driveability analysis. The method proposed by Stevens (1982) does not consider the effect of fatigue nor is CPT related.

With the CPT borehole report from an undisclosed location in the North Sea, the SRD was calculated as given by the three methods and applied to three sites. After calculating the SRD given by the literature, all of the proposals were applied on the GRLWEAP software. It was seen that the soil resistance given by the software matched the analysed methods, meaning that the software was calibrated and considers the soil resistance as seen in the literature, with all of their underlying aspects.

COWI A/S provided all of the borehole reports as well as driving data, i.e. the piling records for the piles driven in the location. Said records contain information regarding the number of blows and energy that was required to drive the pile. With the provided driving data and the calculated SRD it was possible to back calculate the required number of blows. By comparing the actual number of blows with the predict blow count from the three methods it is possible to evaluate if the methods can still be useful.

It was concluded that the CPT based methods worked fairly well when dealing with clean sands, however when facing more complex soil mixtures both AH and ICP did not prove reliable. Regarding the STV model, it was seen that it clearly underpredicts the number of blows, meaning it may be non-conservative in a driveability point of view.

Further conclusions and developments are described in the thesis itself

KEYWORDS: Pile driving, driving predictions, soil resistance to driving, GRLWEAP, blow count, friction fatigue, setup effects.

RESUMO

Neste momento as fundações de estaca *offshore* para turbinas eólicas chegaram a uma dimensão que desafia as bases da análise de cravação por impacto, assim como o conhecimento tradicional empírico disponível para fundações de estaca *offshore*

Existe uma necessidade de realizar uma análise de cravação das gigantescas estacas instaladas, com o intuito de melhorar as futuras previsões de cravação, e ainda averiguar se os métodos usados hoje em dia ainda são adequados para se utilizar na indústria *offshore*.

Este trabalho foi desenvolvido em conjunto com COWI A/S. A empresa é um grupo de consultoria geotecnia líder no mercado, e que esta envolvida em vários projetos *offshore*. A empresa é responsável pelo dimensionamento das fundações *offshore* que suportam as turbinas eólicas

Depois de uma consideração inicial sobre os principais conceitos relacionados com cravação, vários métodos para prever a capacidade resistente do solo durante a cravação, SRD são apresentados. Os principais modelos analisados são o método de Alm and Hamre (2001), a metodologia do Imperial College Pilling (2005) e a proposta de Stevens (1982). As primeiras duas abordagens são baseadas no CPT e consideram a influência da degradação de resistência lateral, um fenómeno conhecido por reduzir a resistência do fuste enquanto a cravação prossegue. Contudo, os dois métodos apresentam diferentes objetivos. A metodologia ICP é mais adequada para determinar a carga a medio prazo das estacas, uma vez que as estacas testadas foram permitas que envelhecessem durante 10 dias, e, portanto, o método considera efeitos de consolidação. Por outro lado, a teoria de AH é melhor aplicada para analisar a resistência aquando o final da cravação. Devido a este motivo é difícil comparar os dois métodos, pois os dois possuem objetivos diferentes durante o dimensionamento. Portanto, existe a necessidade de remover as considerações de consolidação no ICP para compara-lo com AH e para realizar uma análise de cravação explícita. O método proposto por Stevens (1982) não considera efeitos de fadiga nem é relacionado com o CPT

Com o relatório proveniente do CPT, realizado numa localização não revelada no Mar Norte, foi possível calcular o SRD nas três localizações dado pelos três métodos. Depois de calculado o SRD como visto na literatura, todos os modelos foram aplicados no *software* GRLWEAP onde mais uma vez o SRD foi calculado e comparado com a literatura. Foi verificado que se ajustava com os métodos analisados, o que indica que o *software* foi calibrado e consegue calcular a resistência do solo como na literatura, considerando todos os aspetos sobrejacentes.

Todos os relatórios CPT assim como a informação da cravação, i.e., o relatório de cravação para as estacas cravados na localização, foi fornecida pela COWI A/S. Estes, contêm informação relacionada com o número de pancadas e energia que é necessário para cravar as estacas. Com a informação de cravação e com o SRD calculado foi possível calcular o número de pancadas. Comparando o número de pancadas medido com o número previsto pelos três métodos é possível avaliar se os métodos ainda são adequados.

Foi concluído que os métodos baseados no CPT funcionavam razoavelmente bem quanto lidavam com areias limpas, para misturas de solo ambos AH e ICP não se mostraram adequados. Em relação ao modelo de STV, foi visto que claramente previa por baixo o número de pancadas, o que significa que o método não é conservativo num ponto de vista de cravação

Outras conclusões e desenvolvimentos são descritas na tese propriamente dita.

PALAVRAS-CHAVE: Pile driving, driving predictions, soil resistance to driving, GRLWEAP, blow count, friction fatigue, setup effects.

TABLE OF CONTENTS

ACKNOWLEDGMENTS	I
ABSTRACT	III
RESUMO	V
 1 INTRODUCTION	 1
1.1. PREFACE	1
1.2. OBJECTIVES	1
1.3. STRUCTURE	2
2 PILE FOUNDATIONS	3
2.1. INTRO	3
2.2. CONSTRUCTION INSTALLATION PROCESS	3
2.3. MONITORING	3
2.4. TYPES OF PILE	4
2.4.1. DISPLACEMENT PILES	4
2.4.2. DRILLED PILES	6
2.5. HAMMERS	7
2.6. DROP HAMMERS	7
2.6.1. EXTERNAL COMBUSTION HAMMERS (ECH)	8
2.6.1.1. Air/Steam Hammer	8
2.6.1.2. Hydraulic Impact Hammers	8
2.6.2. INTERNAL COMBUSTION HAMMER (ICH)	8
2.6.3. OPEN ENDED DIESEL HAMMER	9
2.6.4. CLOSED ENDED DIESEL HAMMER	9
2.6.5. VIBRATORY HAMMER	9
2.7. ADDITIONAL EQUIPMENT	11
2.7.1. CUSHION	11
2.7.2. RAM	11
2.7.3. HELMET	12
3 OFFSHORE DESIGN METHODOLOGY	13
3.1. INTRO	13
3.2. SELECTION OF A DESIGN SAFETY FACTOR	14

3.3. AXIAL CAPACITY	14
3.3.1. BROAD CONCEPTS	14
3.3.2. EFFECTS OF THE SOIL PLUG AND UNPLUGGED BEHAVIOUR	16
3.4. PILE DRIVEABILITY AND INSTALLATION	18
3.4.1. FRICTION FATIGUE	19
3.4.2. SETUP AND RELAXATION	20
3.5. DESIGN METHODS	20
3.6. ALM AND HAMRE DESIGN PROPOSAL	21
3.7. ICP (IMPERIAL COLLEGE PILE) DESIGN METHOD	24
3.7.1. SHAFT FRICTION	24
3.7.2. AGEING EFFECTS	28
3.7.3. BASE RESISTANCE	30
3.8. STEVENS MODEL (STV)	30
3.8.1. SHAFT RESISTANCE	31
3.8.2. BASE RESISTANCE	32
3.9. ADDITIONAL METHODS	33
3.9.1. TOOLAN AND FOX	33
3.9.2. API (AMERICAN PETROLEUM INSTITUTE)	34
3.9.3. SCHNEIDER	35
4 WAVE EQUATION APPLIED TO DRIVING - GRLWEAP	37
4.1. INTRO	37
4.2. DYNAMIC FORMULAS FOR STATIC CAPACITY DETERMINATION	37
4.3. LIMITATIONS OF DYNAMIC FORMULATIONS	38
4.4. ALTERNATIVES TO DYNAMIC EXPRESSIONS	39
4.5. FUNDAMENTALS OF WAVE EQUATION FORMULATION/ APPLICATION TO DRIVING	39
4.6. SMITH'S MODEL	39
4.7. GRLWEAP	41
4.7.1. DRIVING SYSTEM	41
4.8. ENERGY IN PILE DRIVING SYSTEMS	44
4.8.1. ENERGY LOSSES	45
4.9. GRLWEAP ANALYSIS OPTION	46
4.9.1. BEARING GRAPH	46
4.9.2. INSPECTOR GRAPH	47

4.9.3. DRIVING ANALYSIS.....	48
4.10. GRLWEAP - SRD	49
4.10.1. GAIN/LOSS FACTORS.....	49
4.11. STANDARD APPROACH USING G/L FACTORS	50
4.11.1. VARIABLE SET-UP	53
4.12. FRICTION FATIGUE METHOD	54
5 CPT AND SOIL PARAMETERIZATION	57
5.1. CONE PENETRATING TEST	57
5.2. CPT INTERPRETATION	58
5.2.1. MEASUREMENT CORRECTIONS AND NORMALISED TIP RESISTANCE	58
5.2.2. SOIL BEHAVIOUR TYPE.....	60
5.2.3. SOIL UNIT WEIGHT.....	61
5.2.4. FRICTION ANGLE.....	61
5.2.5. INTERFACE FRICTION ANGLE	61
5.2.6. RELATIVE DENSITY	62
5.3. COHESIVE SOIL PARAMETERS.....	63
5.3.1. UNDRAINED SHEER STRESS.....	63
5.3.2. SENSITIVITY.....	64
5.3.3. OVER CONSOLIDATION RATIO.....	64
5.4. SITE CONDITIONS	65
6 SRD CALCULATION	71
6.1. INTRO	71
6.2. PILE DESCRIPTION.....	71
6.3. ESTIMATION OF SRD	72
6.4. COMPARISON OF SRD	72
6.5. SET UP EFFECTS	81
6.6. GRLWEAP CALCULATIONS.....	85
6.6.1. ALM AND HAMRE (AH)	85
6.7. IMPERIAL COLLEGE (ICP)	89
6.8. STEVENS (STV).....	92
6.9. FINAL RESULTS FOR LOCATION P02 AND P03	93
7 BACK ANALYSIS.....	95
7.1. INTRO	95

7.2. DRIVING CONDITIONS	95
7.3. METHODOLOGY OF THE BACK ANALYSIS	95
7.3.1. SOIL MODEL	95
7.3.2. PILE AND HAMMER MODEL	97
7.4. CPT RECORDS AND SOIL CONDITIONS	97
7.5. BACK ANALYSIS	98
8 CONCLUSIONS.....	105
8.1. FINISHES	ERROR! BOOKMARK NOT DEFINED.
REFERENCES	109

APPENDIX

INDEX OF FIGURES

Figure 2.1 - Installation procedure of bored piles, seen in Randolph and Gouvernec (2011)	7
Figure 2.2 - Impact Hammer, adapted from vulcanhammer.info	9
Figure 2.3 - ECH hammer	10
Figure 2.4 – Close and open ended diesel hammer	10
Figure 2.5 – Vibratory hammer	11
Figure 3.1 - Close ended pile failure Mechanism	15
Figure 3.2 - Open ended failure mechanism (a) Unplugged (b) Plugged, adapted from Randolph and Gouvernec (2011).....	17
Figure 3.3 - Degradation of Friction; as seen in Heerema et al (1980).....	20
Figure 3.4 - Interface friction angle as seen in Jardine et al. (1992).....	25
Figure 4.1 – Stress Strain diagrams, as seen in Smith (1960).....	40
Figure 4.2 – Smith’s Drivin system, adapeted from Saraiva (2014).....	40
Figure 4.3 – GRLWEAP driving system for an ECH.....	42
Figure 4.4 – Dynamic Resistance Figure 4.5 – Static resistance.....	43
Figure 4.6 – GRLWEAP Bearing Graph.....	47
Figure 4.7 – GRLWEAP Inspector Graph.....	48
Figure 4.8 – Driveability graph	49
Figure 4.9 - Capacity vs Energy/Time for Complete Gain/Loss; as seen in GRLWEAP 2010 background	51
Figure 4.10 - Capacity vs Penetration or Time for Incomplete Gain/Loss	53
Figure 4.11 – GRLWEAP Friction Fatigue Option.....	54
Figure 4.12 – Resistance for different penetration depths; as seen in GRLWEAP 2010 background manual	56
Figure 5.1 – CPTu illustration, Robertson 2015.....	58
Figure 5.2 – Soil type behaviour; adapted from Guide to Penetration testing 2015	60
Figure 5.3 – I_c Table; seen in Guide to Penetration Testing 2015	61
Figure 5.4 - Mean particle size	62
Figure 5.5 - Relative Density.....	63
Figure 5.6 - Over consolidation ratio.....	65
Figure 5.7 – Borehole report for location P01	66
Figure 5.8 – CPT tip resistance; P01	67
Figure 5.9 – CPT tip resistance; P02	68
Figure 5.10 – CPT tip resistance; P03	69
Figure 6.1 - Graphical Representation of estimated SRD evolution.....	74

Figure 6.2 – Calculated unit skin friction at 22m and at 30m; AH	74
Figure 6.3 – Calculated unit skin friction at 22.5m and at 30m; ICP.....	75
Figure 6.4 – Rate of Degradation.....	75
Figure 6.5 – Resistance Distribution for AH and ICP respectively	76
Figure 6.6 – Rate of degradation for AH and ICP respectively	77
Figure 6.7 – Full resistance evolution for AH and ICP respectively	78
Figure 6.8 Increase due to setup for Ah and ICP respectively	79
Figure 6.9. – Logarithmic relation of pile shaft capacity with time.....	81
Figure 6.10. – Chow growth trendline	82
Figure 6.11 – Evolution of Qs with and without setup effects; with $Q_t = 1dayQ_{ICP} = 0.5$	83
Figure 6.12 – SRD comparison between AH and ICP without setup, with $Q_t = 1dayQ_{ICP} = 0.5$	84
Figure 6.13 Evolution of Qs with and without setup effects; with $Q_t = 1dayQ_{ICP} = 0.6667$	84
Figure 6.14 – SRD comparison between AH and ICP without setup, with $Q_t = 1dayQ_{ICP} = 0.6667$	85
Figure 6.15 – Friction fatigue factor	86
Figure 6.16 – Friction Fatigue Factor: Correlated	87
Figure 6.17. – SRD Evolution: AH.....	88
Figure 6.18. – CPT tip resistance.....	89
Figure 6.19– SRD Evolution: ICP	90
Figure 6.20. – CPT tip resistance.....	91
Figure 6.21– Final SRD comparison; P01	92
Figure 6.22 – SRD Evolution: STV	93
Figure 6.23 – Final SRD results; P02	94
Figure 6.24 – Final SRD results; P03	94
Figure 7.1 – CPT tip resistance and soil description for Location P01; P02; P03	98
Figure 7.2- Driveability for Location 01.....	99
Figure 7.3 – Driveability for Location 02	100
Figure 7.4 – Driveability for Location 03	101
Figure 7.5 – Total blows	102
Figure 7.6 – Energy transmitted to the piles	103

INDEX OF TABLES

Table 2.1– Responsibilities of design and construction engineers	5
Table 4.1 – Soil setup factors	51
Table 5.1 - Interface friction angle used by Stevens (1982)	62
Table 6.1 – Calculation table	71
Table 6.3 – SRD results	73
Table 6.4 – SRD comparison between AH upper bound and ICP no setup	91
Table 7.1 – Pile and hammer details	96
Table 7.2 – Soil Model	96

NOMENCLATURE

Capital Latin Characters

A – Dimensionless setup factor

A_{ann} – Pile annulus area

A_{sup} – Embedded surface area

A_{end} – End bearing area

A_{plug} – Plug area

D – Pile diameter

D_{CPT} – Diameter of a CPT probe

D_{int} – Internal pile diameter

D_{out} – External pile diameter

D_R – Relative density

E – Energy

E_r – Rated energy

F – Force

F_p – Pile capacity factor

F_r – Normalized friction ratio

G – Distortion modulus

G/L – Gain Loss factor

I_c – Index of soil behaviour

J – Damping coefficient

K – Lateral earth pressure coefficient

K_c – Coefficient of radial effective stress for shaft after full equalization

K_f – Coefficient of radial effective stress for shaft at failure

L_{Li} – Limit distance

N_c – Dimensionless bearing capacity factor

N_b – Number of blows

N_{kt} – Cone factor

Q_{bf} – Ultimate base resistance

$Q_{bf,p}$ – Shear stress along the length of the soil plug

Q_t – Normalized cone penetration resistance

Q_T – Capacity at time T

Q_0 – Reference capacity

Q_{tn} - Normalized cone penetration resistance where the stress exponent (n) varies with soil type

Q_{sf} – Ultimate shaft resistance

$Q_{sf,ins}$ – Ultimate shaft friction inside the pile

$Q_{sf,out}$ – Ultimate shaft friction outside the pile

R – Pile radius

R^* - Pile effective radius

R_f – Friction ratio

R_{inner} – Inner pile radius

R_{outer} – Outer pile radius

S_t – Sensitivity

S_u – Undrained shear strength

$S_{u,nom}$ – Undrained shear stress if normally consolidated.

P_a – Reference pressure

P_{a2} – Reference pressure in the same units as q_c

V_{ult} – Ultimate Pile axial capacity

W – Ram weight

W'_{pile} – Submerged weight of the pile

Z – Depth

Lowercase Latin Characters

a – Net area ratio

c_0 – Relative density constant

c_1 - Relative density constant

c_2 - Relative density constant

d - Distance

d_{50} – Relative soil size

f_{fi} – Friction fatigue factor

f_l – Fraction of limit length

f_o – Shape factor

f_{rd} – Reduction factor
 f_s – Setup factor
 f_s – Sleeve friction
 f_s^* – Relative pile sensitivity
 f_{sf} – Unit shaft resistance
 $\overline{f_{sf}}$ – Average unit shaft resistance
 $f_{sf,res}$ – Residual shaft resistance
 $f_{sf,max}$ – Maximum shaft resistance
 $f_{s,o}$ – Unit friction outside
 $f_{s,i}$ – Unit friction inside
 h – Height above the pile tip
 k – Shape factor for degradation
 m – Mass
 n – Damping exponent
 n – Stress exponent
 p_a – Atmospheric pressure
 q_{bf} – Unit end bearing resistance
 $q_{bf,w}$ – Unit end bearing resistance acting on the pile annulus
 $q_{bf,p}$ – Unit end bearing resistance acting on the pile plug
 q_c – Corrected CPT tip resistance
 q_t – CPT tip resistance
 s_b – set per blow
 t – Time
 t_0 – Reference time
 u_2 – Pore water pressure
 v – Velocity

Greek Characters

α – Empirical adhesion factor
 β – Dimensionless shaft friction for sand
 Δh_p – Variation of the plug length during one penetration increment
 ΔL – Variation of the pile embedded length that occurs during one penetration increment

Δr – Profile of radial contraction
 Δl_{vy} – Relative void index at yield
 δ_f – Pile-soil interface friction angle at failure
 σ'_{h0} – Effective horizontal stress
 σ'_{v0} – Effective vertical stress
 σ'_{rf} – Effective radial stress
 σ'_{rc} – Equalized effective radial stress
 σ'_m – Mean in situ effective stress
 γ_t – Soil unit weight
 γ_w – Submerged soil unit weight
 η – Hammer efficiency

Abbreviations and Acronyms

AH – Alm and Hamre
API – American Petroleum Institute
CPT – Cone Penetration Test
NC – Normally consolidated
ICP – imperial College Pile
IFR – Increment Filling Ratio
LTSR – Long Term Static Resistance
OCR – Over Consolidation Ratio
STV - Stevens
SRD – Soil resistance during driving

1

INTRODUCTION

1.1. PREFACE

The future global economy is expected to consume ever more energy, particularly with the rising energy demand of developing countries such as China and India. At the same time, the tremendous risk of climate change associated with the use of fossil fuels makes supplying this energy increasingly difficult. New ways of producing energy need to be considered in order to keep up with the development of our society and its exponential population growth of the last decades.

Use of renewable energy alternatives is therefore essential. With the present situation of increasing energy demand, rising energy prices, and global warming, renewable energy sources have taken the spotlight. Renewable energies are the ones that originate from natural processes and are replenished at a faster rate than they are consumed. In 2012, the world relied on renewable sources for around 13.2% of its total primary energy supply. In 2013 renewables accounted for almost 22% of global electricity generation and the IEA Medium-Term Renewable Energy Report 2015 foresees that share reaching at least 26% increase in 2020.

Most recently there has been an increasing interest in offshore wind farms. Wind farms are created when multiple wind turbines are placed in the same location for the purpose of generating large amounts of electric power. Due to rising energy prices and the resultant search for alternatives, there are now thousands of wind farms in many countries around the world. Along the years there has been an increase of the distance from offshore structures to the coast and the environmental conditions have become even harder. Therefore, the offshore industry was required to investigate new pile and structure solutions and new installation method techniques.

In offshore conditions, pile foundations, are commonly favoured as an alternative for shallow foundations. Usually, piled foundations are executed when there are soft soils at the surface and when high horizontal loads are present

Offshore monopiled foundations for wind turbines have reached a size that challenges the basis of the impact driving analysis and the empirical factored knowledge available traditionally for offshore oil and gas jacket piled foundations. Hence there is a need to perform an appropriate and thorough back analysis of the XL monopile foundation already installed to improve future driving predictions, which will be detailed in the present work

1.2. OBJECTIVES

COWI A/S main objective was to ascertain if the current theoretical and semi empiric methods used to predict the soil resistance to driving are still adequate to apply in the monopile foundations currently used in the North Sea. Since the methods are based on small diameter piles, when compared to the ones used in large offshore projects it is vital to understand if they can be indeed used in the large jobs seen today. In order to validate this notion, three largely used methods were chosen and applied in three different locations in the North Sea, Alm and Hamre (2001), Imperial College Pilling (2005) and Stevens (1982). Later, making use of GRLWEAP the number of blows were back calculated from the

SRD values given by the analysed design proposals. Note that the main focus of the work was on sands. The established milestones for this paper were as follows.

- Assessment of soil resistance during driving using various methods, including CPT methods and those proposed by Stevens or others;
- Comparison of obtained soil resistance from various methods;
- Based on derived soil resistance and hammer and pile data, the driving process was modelled in GRLWEAP;
- Comparison of obtained results for driving process and driving logs;
- Tuning of methods for deriving soil resistance for better fit of calculated driving process to driving logs (back-calculation);

1.3. STRUCTURE

The present work is divided in 7 more sections, excluding this initial introduction chapter. Chapter 2 intends to review the used hammers in the industry as well as introducing the types of pile used.

Chapter 3 gives an initial consideration on various important topics to better understand the presented work, however the main focus of the chapter is to introduce and to detail the three main methods used.

Chapter 4 regards GRLWEAP and the wave equation, there the basis of the software are explained as well as other important considerations regarding energy.

In Chapter 5 the soil conditions for the analysed locations are presented as well as explanation regarding the soil parameterization using the CPT unified approach.

Finally in Chapter 6 the three methods were applied, comparison of the methods as well as the results are presented in this section. In Chapter 7 the back analysis was accomplished using the SRD results from Chapter 6. This represents the final purpose of the paper.

Chapter 8 summarizes the main conclusions attained in the work.

2

PILE FOUNDATIONS

2.1. INTRO

Civil engineering piling structures can be divided into two main groups, onshore and offshore, although one could argue the addition of a third group: near-shore structures. What distinguishes these different structures is the area where they are located. The work presented in this paper will focus exclusively on offshore structures.

The offshore industry has been experiencing a fast development in the last few years. Due to progresses being made in the offshore driving situation, the energy industry is taking advantage of the state, and investments are being made in alternative means of energy, such as offshore windfarms. With the favourable conditions presented in the North Sea and the advantage of avoiding onshore environmental and social impacts, this industry may prove to be a lucrative endeavour.

2.2. CONSTRUCTION INSTALLATION PROCESS

Construction of a successful driven pile foundation that meets the design objectives depends on correctly relating the requirements of the static analysis methods presented on the plans to the dynamic methods of the field installation and construction control. A pile foundation must be installed to meet the design requirements for compressive, lateral, bending moments and uplift capacity. This may dictate driving piles for a required ultimate capacity or to a determined length established by the designer. In the same way, it is essential to avoid pile damage or foundation cost overruns by excessive driving. These objectives can all be successfully achieved by means of a wave equation analysis, dynamic monitoring of pile driving and by means of a static loading test.

It is clear that all pile installation restrictions should be considered during the design phase of the project, so that piles presented on the plans can be installed as designed. One example that an experienced engineer should consider is the impact that the driving and installation process have on existing structures. Further limitations on construction equipment access, size, area of operation, or environmental issues may also dictate the pile type that can be cost effectively installed.

2.3. MONITORING

Knowledgeable construction supervision and inspection are the means to a successful pile installation. State-of-the-art designs, detailed plans and other specifications must be coupled with good construction supervision to achieve desired results.

Proper pile installation is as important as rational pile design in order to obtain a cost effective and reliable foundation. Driven piles must develop the prerequisite capacity without sustaining structural damage during installation. Construction monitoring of driven piles is relatively more difficult than for spread footings, since the footing excavation and the footing construction can be visually observed to assure quality. On the other hand, piles cannot be seen after their installation, direct quality control of

the finished product is hard to achieve. As a result, a substantial control of the pile installation must be exercised in order to obtain the desired end product, meaning the required capacity.

Monitoring during construction should be followed in three distinct areas: installation equipment, pile materials, and estimation of static capacity. Understandably all of these areas are inter-laced with each other, since one change in one field will affect the others. Table 2.1 (as seen in FHWA manual Vol II 2006) provides a summarized description of all items that are to be included in the plans and specifications for quality assurance.

2.4. TYPES OF PILE

The selection of a proper pile type depends on a range of different factors, such as the type of structure, required design durability and capacity, ground conditions, among others.

There are two main groups in which pile foundations can be divided: displacement piles and drilled piles (also termed bored piles or replacement piles)

By far, the most common offshore pile is the open-ended driven steel pile, which under ideal circumstances is driven without interruptions to full penetration, except for making add-ons or when changing the hammer. Steel tubular piles provide several advantages when used in offshore conditions: they are robust, cheaper than their concrete counterparts, light to handle and capable of carrying high compressive loads, furthermore they are able to develop a high skin friction.

2.4.1. DISPLACEMENT PILES

Displacement piles are installed by "displacing" the surrounding soil. This type of pile can be either driven or jacked. During the design stage of a displacement pile, some aspects related with driveability and consolidation should be taken into attention.

Driveability concerns are related with the driving stresses, tip damage and refusal. Regarding the pile stresses, the driveability study must ensure that the pile is not overstressed and that the process of driving does not reduce the fatigue capacity of the pile material (not to be confused with the friction fatigue concept detailed in chapter 3 and 4)

If the pile tip is damaged during driving, it might experience buckle and ultimately collapse. Its common practice to execute a thickened wall near the pile tip, this solution to buckling is referred to as "shoe". In hard driving sometimes a stiffer tip is required, in such cases a welded steel plate or a conical tip is adopted. It is important to prevent pile deformation, due to damage induced by driving, as it will lead to the reduction of pile capacity or refusal.

Refusal is reached when the penetration rate is slower than 250 blows per quarter meter, once refusal is achieved it is no longer practical to drive the pile. Refusal occurs when the soil resistance exceeds the hammer capacity. This phenomenon is the driving specialist worst case scenario, however there are solutions. Firstly, one could augment the hammer stroke (drop height), thus increasing the energy transferred from the hammer to the pile. Though this option might cause overstressed on the pile wall, so it should be used thoughtfully. Another possibility is to remove the soil plug within the pile, hence reducing the penetration resistance

Table 2.1– Responsibilities of design and construction engineers

Item	Design	Construction
Pile Details	Include in plans and specifications: Material and strength: concrete, steel or timber. Cross section: diameter, tapered or straight, and wall thickness. Special coatings for corrosion or downdrag. Splices, toe protection, etc. Estimated pile length. Pile design load and ultimate capacity. Allowable driving stresses.	Quality control testing or certification of materials
Soil Data	Include in plans and specifications: Subsurface profile. Soil resistance to be overcome to reach estimated length. Minimum pile penetration requirements. Special notes: boulders, artesian pressure, buried obstructions, time delays for embankment fills, etc.	Report major discrepancies in soil profile to the engineer
Installation	Include in plans and specifications: Method of hammer approval. Method of determining ultimate pile capacity. Compression, tension, and lateral load test requirements (as needed) including specification for tests and the method of interpretation of test results. Dynamic testing requirements (as needed). Limitations on vibrations, noise, head room, and fish kill Special notes: spudding, predrilling, jetting, set-up period, etc.	Confirm that the hammer and driving system components agree with the contractor's approved submittal. Confirm that the hammer is maintained in good working order and the hammer and pile cushions are replaced regularly. Determination of the final pile length from driving resistance, estimated lengths and subsurface conditions. Pile driving stress control. Conduct pile load tests. Ensure quality control of pile splices, coatings, alignment and driving equipment.

Unfortunately, simply increasing the hammer stroke or removing the soil plug will not always solve the refusal scenario. As such, most of the times when refusal is encountered it is necessary to waste time and money and change to a higher performance hammer.

However, it is also important to refer the impact of consolidation issues on a driven pile. Which are related to the amount of time that takes the soil to reach the full capacity once the driving process stops. This increase in resistance will understandably affect pile driveability. More on this topic will be later discussed.

2.4.2. DRILLED PILES

Where it is thought to be impractical to utilize driven piles, a popular alternative is the use of drilled or replacement piles. There are three basic types of replacement piles currently used in the offshore engineering.

Essentially, a borehole is drilled into the ground, then concrete is placed into the borehole to form the pile. Rotary boring techniques may allow larger diameter piles and permit pile construction through particularly dense or hard strata. Construction techniques naturally depend on the geology of the site;

For end-bearing piles, drilling continues until the borehole has extended a sufficient depth (socketing) into a sufficiently strong layer. Depending on site geology, this can be a rock layer, or hardpan, or other dense, strong layers. Both the diameter of the pile and the depth of the pile are highly specific to the ground conditions, loading conditions, and nature of the project. Pile depths may vary substantially across a project if the bearing layer is not level.

Drilled piles can be tested using a variety of methods to verify the pile integrity during installation.

Figure 2.1 presents visual representation. When compared to their driven counterparts, bored piles are somewhat more expensive since it requires more time, not to mention the fact that sometimes it is necessary to driven through the shallow soft soil that might exist. However there are some reasons one would use drilled piles instead of driven piles, Das (2015) lists a few:

- A single drilled shaft may be used instead of a group of piles and the pile cap.
- Constructing drilled shafts in deposits of dense sand and gravel is easier than driving piles.
- Drilled shafts may be constructed before grading operations are completed.
- When piles are driven by a hammer, the ground vibration may cause damage to nearby structures.
- Piles driven into clay soils may produce ground heaving and cause previously driven piles to move laterally.
- There is no hammer noise during the construction of drilled shafts.
- Because the base of a drilled shaft can be enlarged, it provides great resistance to the uplifting force.
- The surface over which the base of a drilled shaft is constructed can be visually inspected.
- The construction of drilled shafts generally utilizes mobile equipment, which, under proper soil conditions, may prove to be more economical than methods of constructing pile foundations.
- Drilled shafts have a high resistance to lateral loads (larger diameter and longer shafts).

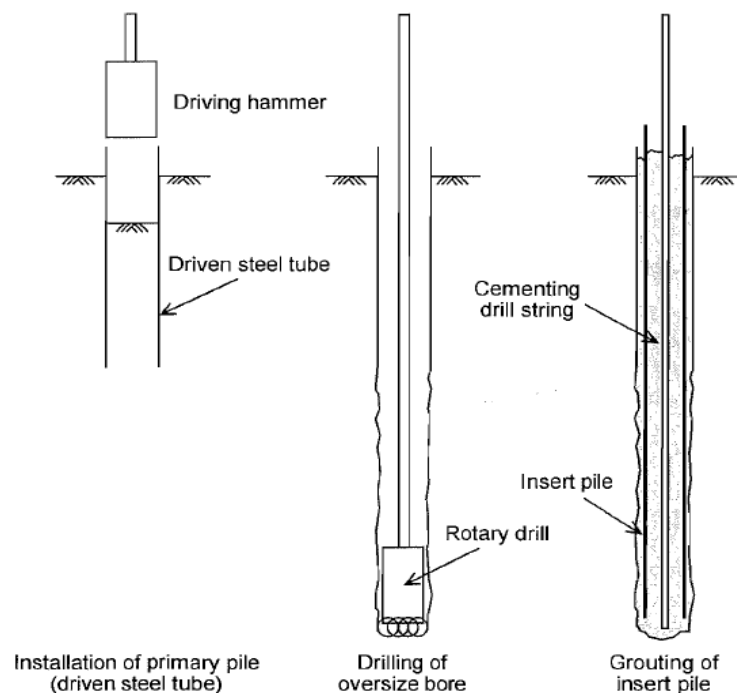


Figure 2.1 - Installation procedure of bored piles, seen in Randolph and Gouvernec (2011)

2.5. HAMMERS

The hammers that are to be analysed in the present section are known as impact hammers. These machines drive the pile by first inducing a downward velocity in a metal ram. Upon impact, the ram crates a force greatly superior than its own weight, which if sufficiently large will move the pile an increment into the ground.

Currently large varieties of different hammers exist. Some are more used than others. The differences in efficiency and variable stroke options are also evident.

2.6. DROP HAMMERS

The drop hammer is the oldest type of hammer in existence. The hammer is connected to a steel cable which is attached to a winch on the crane. The hammer is then raised to the desired stroke height. The winch has a clutch on it that then allows the operator to release the hammer. The hammer will then fall and will strike the pile cap and the pile.

Drop hammers allow a good variation in both weight and speed of blows. It is also one of the cheapest hammer available with the longest service life. Furthermore, it is easy to operate.

However, it must be said, that drop hammers are currently in disuse and are only found in small to very small jobs for unimportant piling, mainly due to its reduced efficiency.

2.6.1. EXTERNAL COMBUSTION HAMMERS (ECH)

External combustion hammers are one of the most currently used hammers in offshore conditions. ECH are hammers that make use of fuel to provide energy for the hammer's operations outside of the hammer itself. Such hammers have external power sources, such as the crane itself, steam boilers, air compressors, or even hydraulic power packs to provide the energy to move the ram upward and in some hammers downward as well.

Two examples of ECH types will now be detailed:

2.6.1.1. Air/Steam Hammer

Air or steam hammers, as the name implies, are operated using steam or compressed air to lift the ram. At a point in the upstroke, the valve is moved and the ram floats to the top of the stroke. Following the raise of the hammer to the desired stroke level, the hammer is allowed to descend using its own weight and makes an impact on the pile. Many air/steam hammers contain a device to control the upstroke valve turnover point, thus enabling the hammer to operate at two different energy levels. This option is especially advantageous when driving and installing concrete piles.

These hammers perform well in most situations and are very reliable, usually in all soil types. Leading to a relatively low blow count, characteristic that is inherent in such hammers. Air/steam hammers have different variation, such as, single acting, double acting or differential acting.

2.6.1.2. Hydraulic Impact Hammers

Hydraulic impact hammers are somewhat recent in the offshore industry and are a new form of impacting the pile driver. One of their biggest advantage is that they are more efficient. As is the case with air/steam hammers, hydraulic hammers can be single or double acting.

These hammers substitute the usage of air or steam for a hydraulic fluid. The hammer is applied to the piston to move the ram. Hydraulic hammers, as with its Air/steam counterpart, have several variations. Most of them employ the use of an electric valve operated with a variable timer, which allows for very flexible control. Figure 2.2 provides a schematic for an impact hammer.

Hydraulic hammers are very efficient both in power pack energy conversion and with impact force transfer. They are also useful when noise attenuation is a requirement. Yet, these hammers are expensive and are difficult to maintain.

2.6.2. INTERNAL COMBUSTION HAMMER (ICH)

These type of hammers also use fuel to power the hammer itself. The only difference is that power needed to move the hammer, takes place inside the pile. The main ICH are the diesel hammers. This type of hammers are also largely popular in the industry however, they do not see much use in Europe, and instead are much more common in North and Central America. Diesel hammers can be divided in two classes.

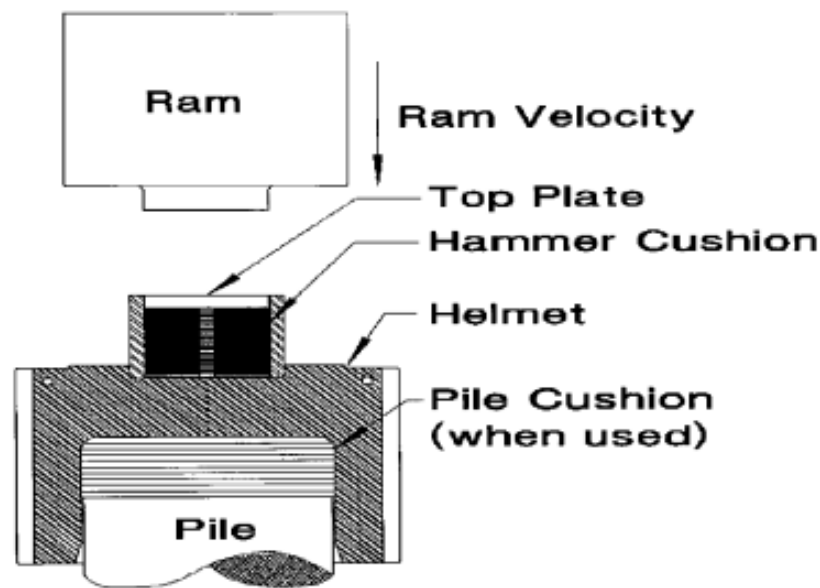


Figure 2.2 - Impact Hammer, adapted from vulcanhammer.info

2.6.3. OPEN ENDED DIESEL HAMMER

The open end diesel hammer operates as follows. The piston with the assistance of the starting device driven either from the winch of the pile driving rig or hydraulically is raised to an upper position, at which point it is released by the starting device and falls down under the effect of gravity. Before the bottom of the ram passes the bottom of the ram passes the exhaust ports, the piston will push the fuel pump lever and thus, fuel from the pump is supplied to the spherical recess of the anvil. At the bottom of the stroke, the piston impacts the anvil. The energy of impact is divided between fuel vaporization and its mixing with heated air and driving of the pile. After a relative short period of time, the air-fuel mixture is ignited and due to the pressure of the expanding exhaust gases the piston is raised up and additional driving impulse is transmitted to the pile.

2.6.4. CLOSED ENDED DIESEL HAMMER

Closed ended hammers are very similar in operating to the already described open ended diesel hammers, with the exception that a compression chamber is engaged on top of the piston to assist the ram in the down stroke. This compression chamber speeds up the blow rate of the hammer, but some of these hammers have a heavier ram relative to the energy than the open ended type. Close ended diesel hammer are usually heavy and are appropriate to use when hard driving is expected.

2.6.5. VIBRATORY HAMMER

A vibratory hammer is a machine that installs the pile into the ground by applying a rapidly alternating force to the pile, i.e. installation by means of vibration. This is generally achieved by rotating eccentric weights about shafts. Each rotating eccentric produces forces that act towards the centerline of the shaft.

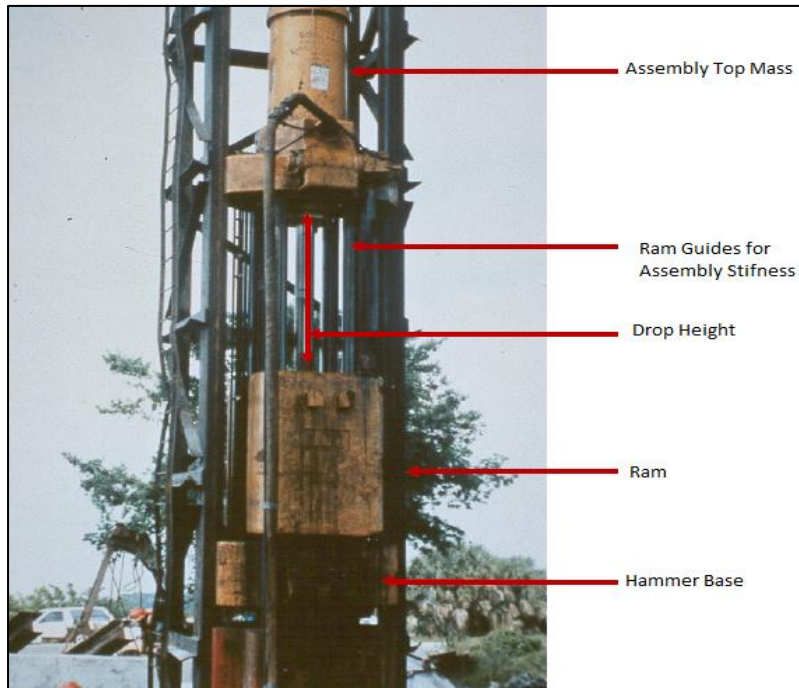


Figure 2.3 - ECH hammer

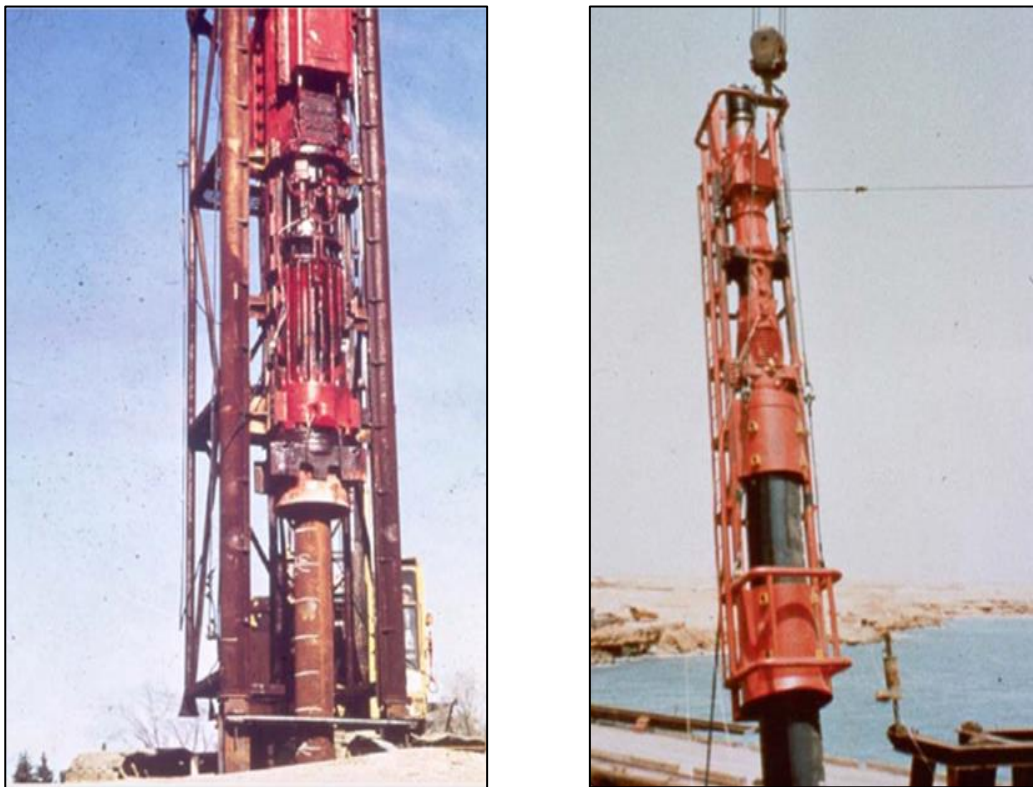


Figure 2.4 – Close and open ended diesel hammer



Figure 2.5 – Vibratory hammer

2.7. ADDITIONAL EQUIPMENT

Driving accessories are most often required in order to achieve an easy driving process. Most of these utensils prevent any damage that might occur to the pile, while others are essential in order to actually drive the pile

2.7.1. CUSHION

Most impact hammers have some kind of cushion under the end of the ram which receives first the striking of the hammer, thus protecting the pile from the full blown impact force delivered by the hammer, preventing any damage that might have occurred. It also modulates the force-time curve of the striking impulse, and can be used to match the impedance of the hammer to the pile, thus increasing the efficiency of the blow. The actual material of the cushion and its arrangement vary, according with hammer configuration. Any hammer cushion should be installed and used in accordance with the recommendations of the hammer manufacturer. Note that pile cushion should only be used when dealing with concrete and plastic piling, when facing with a steel made pipe, no cushion of any kind is required.

2.7.2. RAM

The ram is what actually impacts the pile. In a diesel hammer the initial start-up of the hammer requires the piston (ram) to be raised to a point where the trip automatically releases the piston, allowing it to fall by gravity. As the piston falls, it activates the fuel pump, which discharges a metered amount of fuel into the ball pan of the impact block. The compressed air exerts a pre-load force to hold the impact block firmly against the drive cap and pile. At the bottom of the compression stroke, the piston strikes the impact block, atomizing the fuel and starting the pile on its downward movement. In the instant after the piston strikes, the atomized fuel ignites, and the resulting explosion exerts an even greater force on the already moving pile, driving it further into the ground. The reaction of the explosion rebounding from the resistance of the pile drives the piston upward. As the piston rises, the exhaust ports open, releasing the gases and force of the explosion into the atmosphere. After the piston stops its upward movement, it again falls by gravity to start another cycle.

2.7.3. HELMET

The helmet actually mates the hammer system to the pile, and in doing so, distributes the blow from the hammer more uniformly to the head of the pile, thus minimizing pile damage

3

OFFSHORE DESIGN METHODOLOGY

3.1. INTRO

The design of a pile foundation must consider all aspects of the installation and performance of the system. Thus the offshore pile designer is challenged with three main tasks

- Design of the pile length and section in order to satisfy bearing capacity and structural loading criteria;
- Establishing the response of the pile soil system with respect to structural behaviour;
- The determination of a method and programme of installation;

In the project stage piles must be reliably installed to the desired depth. Also, the resulting foundation needs to have an adequate stiffness and strength to resist the design loads. The foundation should be optimised in order to minimize the number and length of the piles.

All three tasks present an interesting challenge. However, the determination of an adequate method of installation may require the greatest amount of experience and it is this step that will determine the effectiveness of the design and form a basis for the selection of pile type. One of the aspects that render this task in such a complex manner is the fact that the designer is greatly limited in the way that virtually no full scale test data exist for the size of the piles currently used in the offshore industry. Hence most of the design methods are based on small onshore pile load tests

Pile foundation differs in some aspects regarding shallow foundations. The analyses that link soil strength and foundation capacity are less rigorous and more empirical for piles. This is because the failure mechanism of a pile cannot be captured by analytical solutions. In addition, the soil properties and stress states that are considered in this analysis are often modified due to the installation process. Still, the analyses of strength and stiffness of piles, accounting for non-linear response and layering of the soil, often require numerical implementation.

One other aspect that distinguishes driven pile design and construction process is that the driving characteristics are related to pile capacity for most soils, they can be used to improve the accuracy of the pile capacity estimate. In general, the various methods of determining pile capacity from dynamic data such as driving resistance with wave equation analysis and dynamic measurements are considerably more accurate than the static analysis methods based on subsurface exploration information

Further, in pile foundations the interaction between combined loads is less significant than in a shallow foundation. Thus, the application of a horizontal load to the pile has less influence in the axial pile capacity. Also, application of a vertical load has less influence in the horizontal capacity. This occurs because the upper few diameters of the pile resist to the horizontal loads. On the other hand, the vertical load is carried out by the lower part of the pile.

3.2. SELECTION OF A DESIGN SAFETY FACTOR

As with all other constructions, during the design phase, it is essential to select a valid safety factor. Such factor is applied on the design load, which should be carefully chosen as a result of static analyses and consideration of the allowable stresses in the pile material. Depending on the amount of confidence in the analysis method, construction method executed and the information regarding the subsurface exploration program the safety factor may vary from 2.0 to 3.5.

In the construction stage, the ultimate pile capacity to be obtained is the sum of the design load, times the factor of safety, plus the soil resistance from unsuitable layers not counted on long term support or subject to scour. The plans and specifications should state the ultimate pile capacity to be obtained in conjunction with the construction control method to be used for determination of the ultimate pile capacity.

3.3. AXIAL CAPACITY

3.3.1. BROAD CONCEPTS

The aim of this section is to introduce the concept of axial capacity on piles as well as other underlying subjects, so that the reader has a better grasp of the methods used to estimate the driving resistance, which will be presented later on. The main focus will be on sands, hence most of the considerations will regard granular soils.

Axial strength, or axial capacity, of a pile is the maximum vertical load that a given pile can support without failing. In order to assure vertical equilibrium the axial capacity, V_{ult} must be equal to the sum of the ultimate shaft resistance (sometimes called skin resistance), Q_{sf} and the ultimate base resistance, Q_{bf} minus the submerged weight of the pile, which should also be supported by the soil resistance. The formulation and an illustration of this mechanism can be seen below in Equation 3.1 and Figure 3.1

$$V_{ult} = Q_{sf} + Q_{bf} - W'_{pile} \quad (3.1)$$

Figure 3.1 represents the failure mechanism of a close ended pile. However, in open ended piles, shaft friction is observed in both the inner and the external surface of the pile wall. Thus shaft friction should be divided into two different components, the outer shaft friction, $Q_{sf,o}$ and the internal shaft friction, $Q_{sf,i}$.

$$Q_{sf} = Q_{sf,i} + Q_{sf,o} \quad (3.2)$$

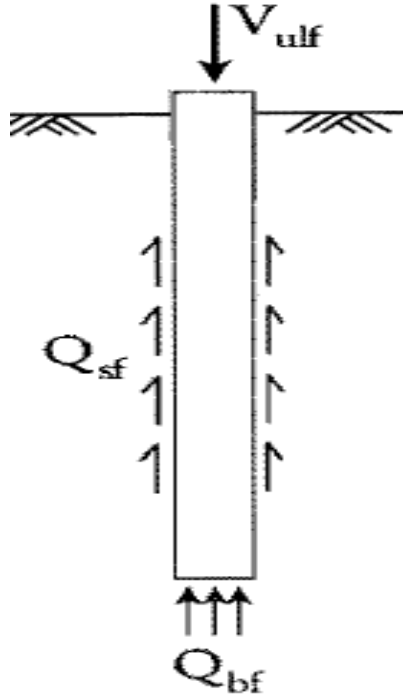


Figure 3.1 - Close ended pile failure Mechanism

Accordingly with Randolph et al (2011) the ultimate skin friction can be obtained by integrating the unit skin friction, f_{sf} over the surface of the pile. It is also advisable to divide the soil stratigraphy into several layers. Therefore, shaft friction comes as:

$$Q_{sf} = \pi D \int_0^L f_{sf} dz \quad (3.3)$$

As for the base resistance, it can be described as the maximum unit stress that can be developed on the pile base multiplied by the pile base area. For close ended piles it is formulated by multiplying the unit end bearing q_{bf} by the pile base area:

$$Q_{bf} = \frac{\pi D^2}{4} q_{bf} \quad (3.4)$$

An important aspect that should be taken into consideration is the fact that the ultimate base resistance is mobilized after high pile settlements. In most cases this high settlement will threaten the safety of the structure that the pile is supposed to support. Due to this issue, the failure base resistance is limited to a mobilized base resistance for an allowable displacement, usually 10% of the pile diameter.

On an open ended pile, the base resistance is comprised as the sum of two components acting on the pile steel cross section, $q_{bf,w}$ and on the soil plug, $q_{bf,p}$. More details on this will be provided later on.

Empirical estimation of the unit base and shaft resistance is demanding. The soil undergoes a change of stress during driving, which is hard to predict.

Due to these underlying issues most methods predict both the skin and base unit friction based on *in situ* conditions prior to the pile installation.

3.3.2. EFFECTS OF THE SOIL PLUG AND UNPLUGGED BEHAVIOUR

In this topic the effect of the phenomenon known as plug will be detailed. Different failure mechanisms are possible to occur in an open ended pile. Randolph et al (2011) described them by:

- Unplugged penetration ('coring') – the soil column within the pile remains stationary during the driving
- Plug penetration – the soil column enclosed in the pile moves downwards at the same rate as the driven pile
- Partial plugging – the soil inside the pile moves down, but in a slower rate than that of the pile.

The ability to predict the soil resistance to driving as well as the axial capacity of an open ended pile is complicated by the soil plug behaviour. The relative amount of soil entering the pile can be quantified, using the incremental filing ratio (IFR).

$$IFR = \frac{\Delta h_p}{\Delta L} \quad (3.5)$$

Where:

Δh_p – Quantifies the variation of the plug length during one penetration increment

ΔL – Defines the variation of the pile embedded length that occurs during one penetration increment.

IFR is translated as the change in plug height for an increment of penetration as compared to the length of the embedded pile. Usually when driving large diameter open ended piles the soil inside the pile tends to remain near to ground level.

An IFR of zero will correspond to a plugged penetration whereas an IFR equal to the unity corresponds to an unplugged penetration. When IFR is between zero and the unity the soil within the pile will plug partially. In order to compute the axial capacity of an open ended pile, two calculations should be made. The first should contemplate an unplugged failure and the second calculation should consider a plugged behaviour. The mechanism with the lowest resistance will govern the pile capacity behaviour.

Hence, if the shear stress along the length of the soil plug exceeds the mobilized base resistance, the pile penetrates in a plugged manner. Since the increase in radial stress during installation of a displacement pile is mainly controlled by the relative level of soil displaced, it is expected that shaft friction reduces as IFR increases.

Figure 3.2 provides a visual representation of an open ended failure mechanism in an unplugged and a plugged manner respectively

It is now made clear that the base resistance in open ended piles depends on the degree of plug that occurs. In an unplugged penetration, the soil column within the pile does not offer resistance at the pile base. The end bearing resistance is mobilized only on the pile steel annulus

When dealing with a fully plugged penetration, the base resistance is mobilized both in the soil plug and in the pile annulus. A reduced base resistance compared to the one estimated in a close ended pile can be adopted. The following equations present the ultimate base resistance for an unplugged and for a plugged penetration. As stated in the previously paragraph $q_{bf,w}$ and $q_{bf,p}$ symbolize the unit base resistance presented by the pile annulus and the soil plug correspondingly.

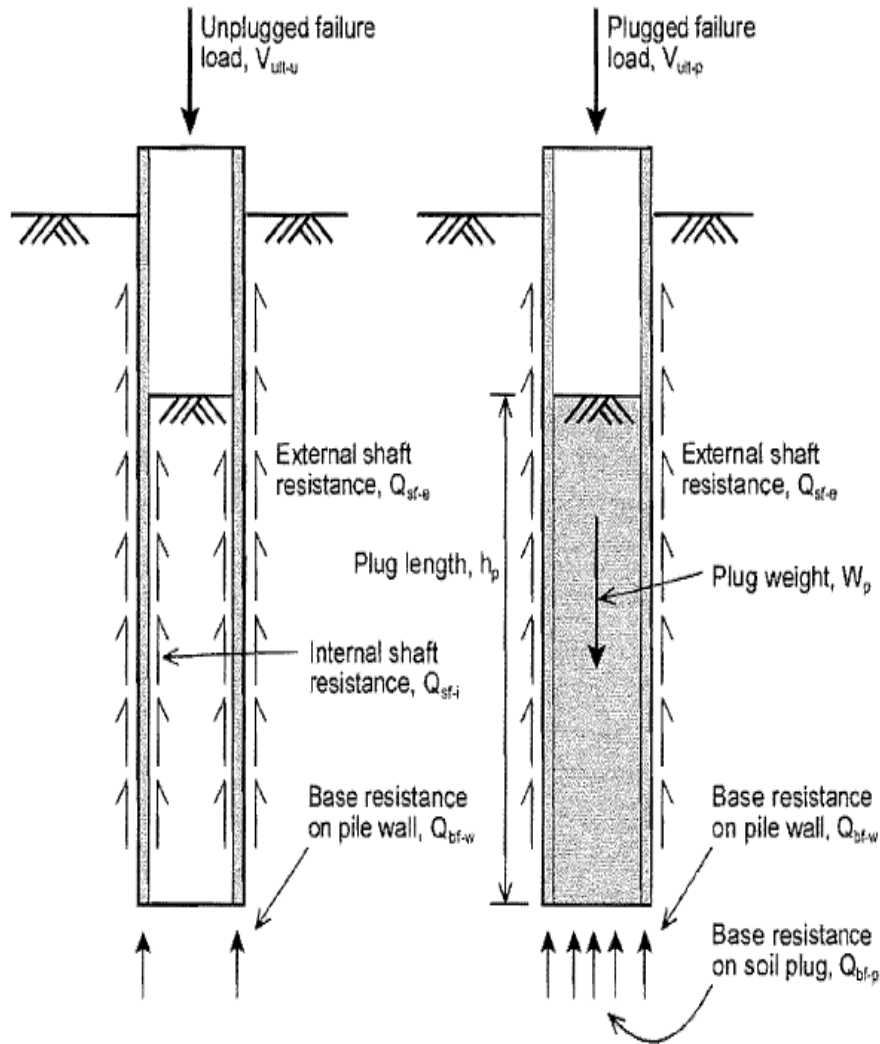


Figure 3.2 - Open ended failure mechanism (a) Unplugged (b) Plugged, adapted from Randolph and Gouvernec (2011)

$$Q_{bf} = \frac{\pi(D^2 - D_t^2)}{4} q_{bf,w} \quad (3.6)$$

$$Q_{bf} = \frac{\pi(D^2 - D_t^2)}{4} q_{bf,w} + \frac{\pi(D_t^2)}{4} q_{bf,p} \quad (3.7)$$

The formation of the soil plug is dependent on several factors, such as the pile diameter, pile length, the installation method, soil type as well as density. The arching effect is one possible effect that could explain the formation of soil plug. The compaction of the soil inside the pile will lead to a rising of the horizontal stresses and high internal shaft frictions, especially around the pile tip.

However plugging is not likely to occur during dynamic driving. Piles usually drive in an unplugged manner, the soil level inside the pile remains approximately at ground level. This is explained due to the inertia of the soil column that creates an additional component of resistance during driving, thus the unplugged penetration resistance is usually lower. However, under static loading piles usually fail in a plugged manner, with the soil level moving downwards with the pile. If plugging is to occur in driving, it is usually when the pile tip passes from a strong material into a weak. Thus, as detailed in the preceding paragraphs, the shear stress along the length of the soil plug exceeds the mobilized base resistance, as seen in Equation 3.8.

$$Q_{sf,i} > Q_{bf,p} - W'_{pile} \quad (3.8)$$

3.4. PILE DRIVEABILITY AND INSTALLATION

In order to achieve a correct design of a driven pile, a detailed driveability study is required. With the driveability analysis it is possible to consider the hammer energy required to drive a pile into a desired depth. Indication on whether or not refusal is encountered is also provided (refusal is achieved when the penetration rate is slower than 250 blows per quarter meter; when refusal is achieved it is no longer practical to drive the pile). With this information a hammer can now be selected and a general prediction of the expected blow counts that will be necessary to finally install the pile is also obtained. Special care should be made to ensure that the pile wall is not overstressed. It should also be mentioned the need to consider setup effects, due to temporary stops in the driving process. To summarize, the driveability study indicates the estimated number of blows and the resulting cycles of stress that will be endured by the pile during installation. The same methods used to assess the static pile strength are also used to predict the soil resistance to driving. SRD can be attained in the same way as the static pile capacity of a pile, i.e. the sum of the shaft capacity with the base resistance, the difference between the two, is that, when calculating the SRD one must consider the effects of consolidation, equalization and fatigue effects.

During installation and throughout the pile's life many changes in the stress field occur due to the radial displacements applied to the sand. As the pile tip advances towards a given element of sand, the stress level rises significantly to push the sand radially away from the pile tip. The stress level will be comparable to the CPT tip resistance.

As the sand passes the tip, it is displaced radially for the pile to penetrate. Thus sand achieves the pile shaft and stresses decrease considerably since the vertical penetration load is no longer acting on that element. At this stage, the stresses behind the pile tip can be comparable with the sleeve friction. This effect is referred to as friction fatigue and will be discussed on the following paragraph

3.4.1. FRICTION FATIGUE

The notion of friction fatigue was first introduced by Heerema (1980). The phenomenon explained why the pile driving at Heather Field was possible to accomplish. Soil conditions of the site consisted of a very heavily consolidated clay with an extremely high undrained shear strength. At the time driveability prediction were mainly prepared by means of ultimate bearing capacity predictions, thus no considerations of consolidation, equalization or time effects were considered. Using axial capacity to predict the driveability pointed to an impossible task. However, the author's research group was optimistic about the possibility of reaching the target depth, this optimism was mainly due to the consideration of the friction fatigue effect. During the driving process it was noted that although the clay's shear strength increased with depth (leading to a supposedly progressive increase in the blow count), what occurred was that the total driving resistance increased only less-than-linearly with depth of penetration, note that if the clays shear strength was constant the total friction should be expected to increase linearly. This reduced increase of resistance meant that the unit skin friction was lost as the pile driving continued, even though the clay's shear strength increased. In order to better attain this concept, several tests were made. It was seen that initial friction was initially high but decreased until a certain low residual value is reached. This decrease in friction is a consequence of a horizontal soil stress decrease. Such decrease can be explained due to elastic expansion and by the vibration caused by the pile pushing the soil outwards. As installation continues, with the pile being hammered numerous times, the soil adjacent to the pile is sheared back and forth. This cyclic shearing leads to contraction of the soil. The contraction leads to the surrounding soil to relax causing a reduction in the horizontal stress acting on the pile shaft. This will form a temporary arching effect around the pile. The exponential function assumed by Heerema (1980) can be seen in Figure 3.3

As formulated, the horizontal stresses are maximum at the pile tip, and will exponentially decrease to a residual value, as penetration increases. It can also be said that friction is minimal in the area that the pile experiences the most work, i.e. shearing due to the consecutive hammer blows and is maximum in the tip where said shearing is not as high.

Currently there is no consensus as to what which is the most suitable method for assessment of friction fatigue. Some designs relate the friction fatigue to the height of a soil layer above the pile tip, Alm and Hamre (2001). Others relate the phenomenon with the normalized height with the pile diameter, such as the ICP formulation. Some methods like the one proposed by White and Lehane (2004) reinforce the notion that the primary mechanism controlling the friction fatigue is the cyclic loading that the pile suffers during installation. Despite this debate all methods agree that friction fatigue can be defined by a reduction of the maximum friction, encountered at the tip to a residual value, as the penetration increases friction will exponentially reduce. A more detailed overview and the application of this noteworthy concept will be given in the following paragraphs, in chapter 4 and in chapter 6.

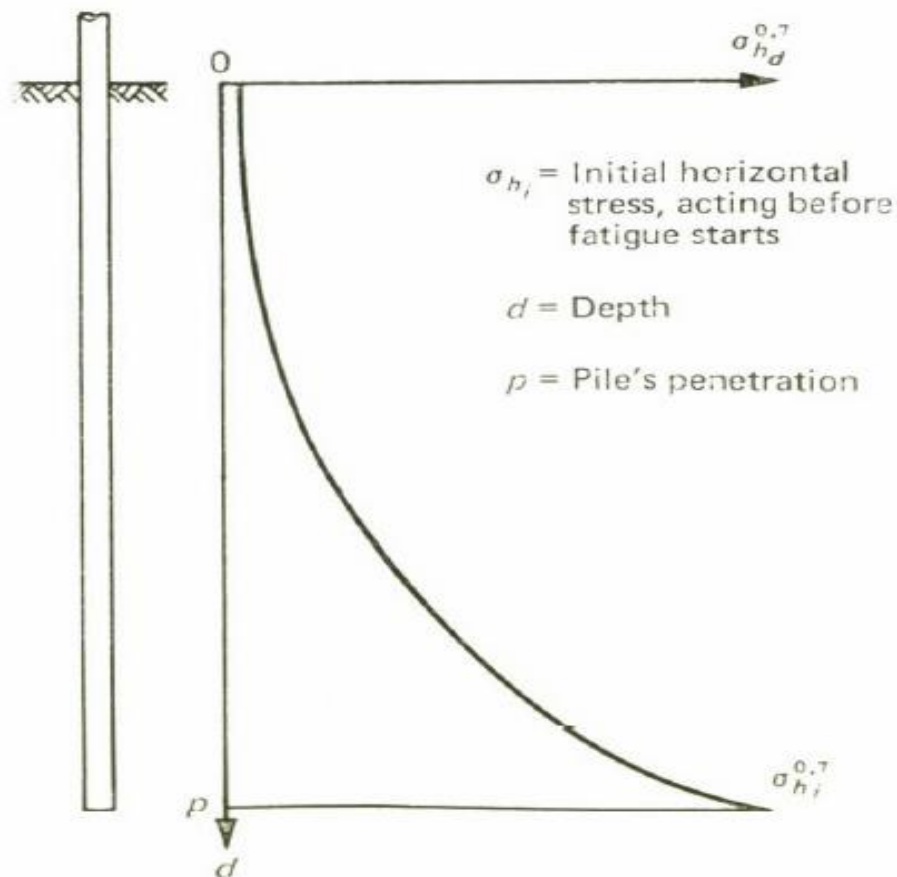


Figure 3.3 - Degradation of Friction; as seen in Heerema et al (1980)

3.4.2. SETUP AND RELAXATION

Pauses in pile driving may result in a significant increase (setup) or decrease (relaxation) in the driving resistance. Generally speaking, setup is more troublesome to deal with in a driveability point of view, since the driving interruption can lead to a premature refusal. Therefore, driving schedules should be carefully planned with as few interruptions as possible, furthermore back up hammers should always be available and ready to use. More detail regarding this effect will be given along the paper.

According to Phillip Gorge in his work in offshore pile foundations, Offshore Pile Mechanics (1976) the setup occurrence may be linked to the dissipation of high water pore pressures in the thin layer of remoulded soil closely adjacent to the pile wall, which then results in an increase adhesion. This effect may be beneficial if the pile has already reached its design penetration, giving the already installed pile a higher resistance. However if the required installation depth is not yet reached the driving process will become considerably more difficult.

3.5. DESIGN METHODS

Driving piles for a foundation is a rough and brutal process. The interactions among the piles and the surrounding soils are not easy to grasp. Generally speaking, when driving a pile into the ground a change of the stress field around the soil is expected and intense strains are set up particularly near the

pile area of influence. Adding to this, the non-homogeneity of soils, along with the effects of the pile group (not discussed in this work) and pile shape hamper the understanding of soil-pile installation.

Broad generalizations about pile behaviour are unrealistic and do not present accurate results. As such, an understanding of the importance of several factors involved is demanded in order to be successful in the design of pile foundations. Due to these inherent complexities, it is necessary to use practical semi-empirical methods of design and to focus attention on significant factors rather than minor or exterior loads. According to the FHWA Vol II, to achieve an optimal project, the foundation designer must have an understanding of foundations loads, subsurface conditions including soil/rock properties and behaviour, the significance of special design events, foundation performance criteria, and current practices in foundation design and construction in the area where the work is to be done. In the following paragraphs a description of the design methods analysed will be discussed.

3.6. ALM AND HAMRE DESIGN PROPOSAL

3.6.1. ALM AND HAMRE - SANDS

There has been an increasing investigation on pile driving records for the purpose of establishing updated soil models for calculation of the static soil resistance, and therefore control of blow counts in a driveability analysis. The offshore industry is designing piles with increasing diameters and penetrations in order to reach at extreme pile capacities. This ramping investigation is motivated by the continuous search for an optimum pile design in terms of a minimum cost.

In this context Alm and Hamre (1998) and Alm and Hamre (2001) developed an improved model to predict the static resistance during driving (SRD). This model is supported in dense to very dense sands from the North Sea as well as clays. The water depth at the different locations lies between 70 and 170 meters. All the piles were driven with hydraulic underwater hammers. The hammer efficiency considered was of 0.85 to 0.95. All the predictions assumed an unplugged pile failure mechanism. The proposed model takes into consideration the friction fatigue concept and is correlated to CPT (following the method update in 2001), thus minimizing additional uncertainty regarding the selection of soil parameters and modelling changes in radial stress due to pile installation.

The total driving resistance can be described as the sum of a static resistance, SRD and a damping constant. The static resistance is mobilized as a function of the pile displacement. SRD, is one of the most important variables in a driveability analyses, and is generally evaluated on basis of pile bearing capacity principles, where the capacity is contributed by pile tip resistance and side friction along the pile surface. Again, SRD is the static component, comprised by the sum of the shaft resistance with the base resistance.

$$R_T = SRD(1 + J \times v^n) \quad (3.9)$$

Where:

SRD – Static Resistance to Driving (kN)

v - Pile segment velocity (m/s)

J – Damping coefficient (s/m)

n - Damping exponent

As stated previously this model includes the effect of friction fatigue both in sands and in clay. The model must define a start and a residual static friction, along with a shape function describing the relative fatigue friction reduction. In other words, the shaft friction is estimated to exponentially decay from a maximum to a residual value. Further, the model for maximum skin friction is based on the Mohr Coulomb failure criteria. The formulation that this method uses is described below:

$$f_{sf} = f_{s,res} + (f_{s,max} - f_{s,res})e^{-kh} \quad (3.10)$$

$$f_{s,max} = K \times \sigma'_{v0} \tan(\delta_f) \quad (3.11)$$

$$f_{s,res} = 0,2 f_{s,max} \quad (3.12)$$

Where:

- h - Height above the pile tip
- σ'_{v0} - Effective overburden pressure
- δ_f - Soil-pile interface friction angle at failure.
- q_t - Cone tip resistance.
- K - Lateral stress coefficient
- k - Shape factor of degradation

Special care should be given to the interface friction angle. It is stated that δ_f should be made equal to the soil friction angle minus 5 degrees. However, in section 5 an alteration to this parameter is proposed.

The lateral stress coefficient is linked to the cone tip resistance from the CPT, using the resulting formulation.

$$K \times \sigma'_{v0} = 0.0132 \times q_t \times (\sigma'_{v0}/p_a)^{0.13} \quad (3.13)$$

In Alm and Hamre (2001) the shape factor for the “rate” of degradation k was also related to the normalized cone tip resistance. It can be described as follows:

$$k = \frac{1}{80} \left(\frac{q_t}{\sigma'_{v0}} \right)^{0.5} \quad (3.14)$$

Using the proposed degradation factor will lead to a rapid degradation for dense sands while in clays, a gentler degradation will occur.

It should be noted that when using the relations established above, one must take into consideration that the formula regarding the lateral earth pressure coefficient has been established under the assumption that friction will only occur on the outside of the pile wall. Therefore, when establishing the soil friction this effect should be taken into consideration. Either by, including simply the outside friction in the calculation or by reducing the unit friction to 50% and applying to both inside and outside of the pile wall. It is true that mathematically this consideration is almost the same as stating that friction will only occur on the outside, however what is actually meant is that the distribution of friction inside and outside is not well known, hence estimating that the ratio between internal and outside friction is equal seems correct.

$$f_{sf,int}/f_{sf,out} = 1 \quad (3.15)$$

In sands unit base resistance offered by the pile tip during driving was formulated by Alm and Hamre (2001) as follows:

$$q_{bf,w} = 0,15 \times q_t (q_t / \sigma'_{v0})^{0,2} \quad (3.16)$$

3.6.2. ALM AND HAMRE - CLAYS

The design proposal also makes considerations on the soil resistance when dealing with clays. However, the main focus of this paper will be on sands. Yet it is still important to describe the formulation presented by Alm and Hamre (2001) for clays. The main difference in clays concerns side friction. Initial skin friction is taken directly from the recorded CPT sleeve friction, since it was found that correlating the skin friction to CPT friction as opposed to standard pile capacity values gave the best fit. Regarding the residual friction, it was first formulated as a function of OCR and was not directly linked to CPT. Following the 2001 update on the method residual friction was found to be a function of the normalized cone tip resistance through the following formulation:

$$f_{s,res} = 0.004 \times q_t (1 - 0.0025 \times q_t / \sigma'_0) \quad (3.17)$$

With increasing cone tip resistance, the residual friction was found to be slightly reduced. Unit tip resistance in clays is simply taken as 60% of the CPT tip resistance. The ultimate shaft capacity and end-bearing capacity are then calculated by the integration of the unit side friction and the unit base resistance, as stated in the beginning of this chapter:

$$Q_{sf} = \pi D \int_0^L f_{sf} \cdot dz \quad (3.18)$$

$$Q_{bf} = \frac{\pi(D^2 - D_i^2)}{4} q_{b,f,w} \quad (3.19)$$

As a final point, this model proposed by Alm and Hamre, is proven to give a reliable prediction of resistance during driving in sand profiles located in the North Sea. The SRD profile can later be used to predict blow count curves by utilizing the Wave Equation Method. By analysing 18 different locations Alm and Hamre found that the proposed method gave good predictions on the number of blows required to drive the pile to the design depth. Best results were found for soft clays and in dense to very dense sands. However, when mixed layers of over consolidated clays and dense sands are encountered the proposed formulation does not perform adequately

Though it should be noted that this model is largely based on relative small diameter piles (1,8m to 2,2m), when compared to the ones that are being implemented in the offshore industry today. Further studies need to be completed to verify if this technique can be applied to larger mono-pile foundations.

3.7. ICP (IMPERIAL COLLEGE PILE) DESIGN METHOD

3.7.1. ICP - SANDS

The ICP method provides an estimation of the axial capacity of a displacement pile based on CPT data. The investigations conducted by Jardine et al (2005) were all conducted on previously un-failed piles. All calculation procedures are intended to be compatible with modern pile testing practice, which includes the consideration of the setup effect of the soil. All the estimation projected aim at predicting the capacities that may be mobilized ten days afterward the driving, since the piles were allowed to age for a period of ten days before being tested.

3.7.2. SHAFT FRICTION

As previously stated, shaft friction can be developed both inside and outside, in open-ended piles. However unlike other approaches the ICP design observed that the inner shaft components are concentrated near the pile end and for simplicity it has considered these components as contributing to the overall base and external shaft components

Shaft capacity is obtained in a similar way to as the one described in Equation 3.18:

$$Q_{sf} = \pi D \int f_{sf} dz \quad (3.20)$$

It was proven by Jardine et al (1992) that, when at failure, the unit shear stress acting on the pile shaft, f_{sf} follows the failure criterion recommended by Mohr Coulomb:

$$f_{sf} = \sigma'_{rf} \times \tan(\delta_f) \quad (3.21)$$

The δ_f term represents the interface angle of friction between the soil and the pile at failure. With sands this is the ultimate value δ_{cv} which is developed when the soil at the interface has ceased dilating or contracting.

The ultimate interface shearing angle depends on the sand's grain size, shape and mineral type, and on the hardness and roughness of the pile's surface, it may also depend on the radial effective stress level. Jardine et al (1992) suggested a relation between the interface friction angle and the size of the particle

of sand, d_{50} . It is advisable to only use this correlation when no other information regarding the soil exists, therefore it is prudent to perform ring shear tests to evaluate the interface angle. Such correlation can be seen in Figure 3.4. With information regarding the mean particle size d_{50} we can easily find the ultimate value of the interface friction angle, δ_{cv} . Correct assessment of this parameter is essential, in order to achieve accurate results.

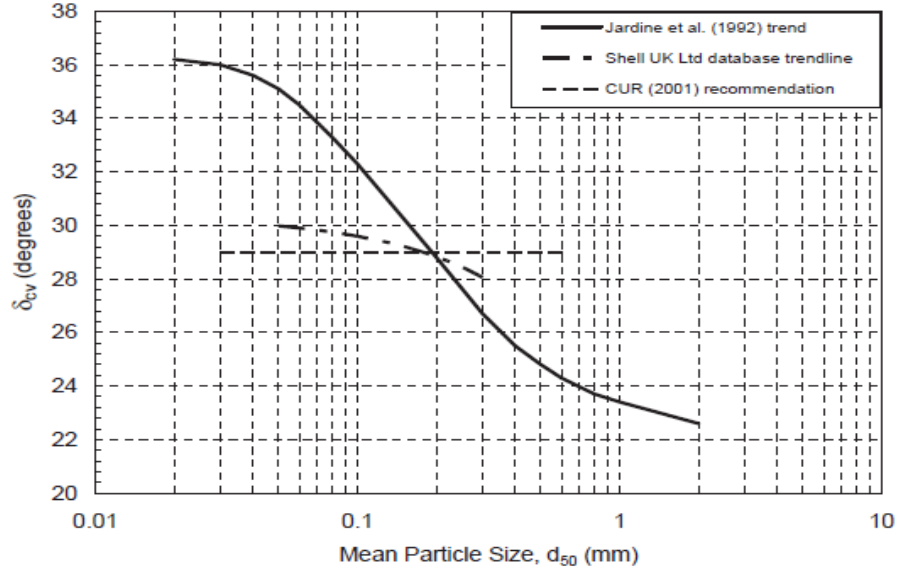


Figure 3.4 - Interface friction angle as seen in Jardine et al. (1992)

The radial effective stress σ'_{rf} can be described as the sum of two components:

$$\sigma'_{rf} = \sigma'_{rc} + \Delta\sigma'_{rd} \quad (3.22)$$

Where, σ'_{rc} represents the equalised radial effective stress and $\Delta\sigma'_{rd}$ denotes the change in radial stress during pile loading. These stresses have a strong variation that is connected to the sand relative density, as revealed by CPT tip resistance. It should be noted that the σ'_{rc} is also dependent on the soil overburden pressure and the pile relative tip depth, h , as shown in Figure 3.5. The radial effective stresses established at any given depth decay suddenly as the pile tip is driven past that depth level and h increases.

This phenomenon, which may vary with different factors including: the installation method, pile shape and sand properties, is the cause of a tendency for the average value of τ_f to reach an almost constant limit once a “critical depth” of around $10D$ has been exceeded (This tendency is only confirmed in uniform sands however). This constant value was found in other approaches and it explains why many methods present limits in the skin resistance.

The friction fatigue effect is considered by the h/R term, where h is the length between a pile element and the pile tip and R is the pile radius (assuming a lower limit of $h/R = 8$). Furthermore, data from test

sites performed by Chow (1996) and Jardine (2005) state that radial stress was almost directly proportional to the CPT base resistance. Hence the ICP methodology recommends that σ'_{rc} can be computed as follows:

$$\sigma'_{rc} = 0,029q_t(\sigma'_{v0}/P_{atm})^{0.13}(h/R)^{-0.38} \quad (3.23)$$

In this expression q_t reflects the combined effects of the soil density and stress level on the value of σ'_{rc} . Jardine et al (2005) suggested that a number of different factors could also contribute to friction fatigue, including the reduction in horizontal effective stress at a given depth due to the relocation of high stress field located in the pile base as it is driven pass the point under consideration. The effects of cyclic loading during installation and the effect of the arching effect also played a major role on the explanation of the friction fatigue.

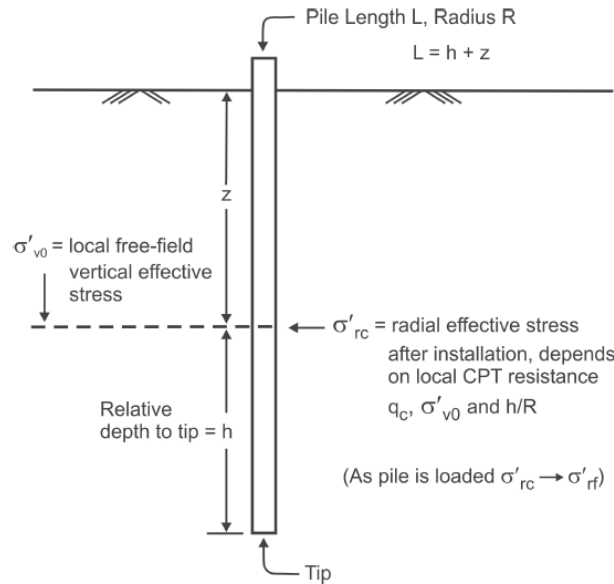


Figure 3.5 – Definitions of parameters for the radial effective stress expression, adapted from Jardine et al (2005)

In open-ended piles, the pile radius is calculated as the equivalent solid pile radius that has the same solid cross-sectional area as the tubular pile:

$$R^* = (R_{outer}^2 - R_{inner}^2)^{0.5} \quad (3.24)$$

The ICP field tests showed that the radial effective stresses, on the shaft experience changes during the course of loading to failure. The key component of change seen in compression tests was interpreted as $\Delta\sigma'_{rd}$ a parameter that is associated with, sand shear stiffness, both pile roughness and radius plus the constrained interface dilation during pile loading. Several studies have shown that cavity

expansion theory can be used to estimate the increase in the horizontal effective stress due to dilation. According to Equation 3.25 it can be expressed by:

$$\Delta\sigma'_{rd} = 2G \times \Delta r / R \quad (3.25)$$

For lightly rusted steel piles, G represents the distortion modulus Δr is equal to a constant value of 0.02.

3.7.3. ICP - CLAYS

As with the Alm and Hamre approach a small overview on the ICP methodology for clays will now be detailed. Based on four sets of testing in clays the shaft failure is governed by the Coulomb criteria with the difference that it is now expanded in terms of a loading factor reduction (about 20%) and the radial stress after equalization:

$$f_{sf} = K_f / K_c \sigma'_{rc} \times \tan(\delta_f) \quad (3.26)$$

Factors that affect the profiles of long-term, fully equalized, radial stresses are expressed in terms of the ratio, $K_c = \sigma'_{rc} / \sigma'_{v0}$. K_c Was found to be dependent on several factors, such as the clay's local Yield Stress Ratio, or apparent OCR, its sensitivity and the relative pile tip depth. ICP reinforces the fact that ring shear tests should be performed in order to attain the interface friction angle, that should fall between the peak and ultimate interface friction. The same should be said to the undrained shear stress and OCR, making use of vane tests or undrained triaxial tests for example. Since these factors affect the profile of σ'_{rc} over the shaft they should be carefully reviewed.

$$K_c = [2.2 + OCR - 0.870 \times \Delta l_{vy}] OCR^{0.42} (h/R)^{-0.2} \quad (3.27)$$

$$\Delta l_{vy} = \log_{10} S_t \quad (3.28)$$

$$K_f / K_c = 0.8 \quad (3.29)$$

The loading factor seems to be constant regardless of the direction of loading or drainage conditions. It should be mentioned that the formulation proposed in Equation 3.21 is only viable when dealing with a compression load. When a tension loading is applied instead of the load, a reduction of the compressive stress regime and the principal stresses leads to a decline of the normal effective stresses. The ICP design routine provides an alternative formulation when dealing with tension loadings. The equation below represents such effect in close ended piles and open ended piles, accordingly.

$$f_{sf} = (0.8\sigma'_{rc} + \Delta\sigma'_{rd}) \tan(\delta_{cv}) \quad (3.30)$$

$$f_{sf} = ((0.8\sigma'_{rc} + \Delta\sigma'_{rd})\tan(\delta_{cv})) * 0,9 \quad (3.31)$$

3.7.3.1. Ageing Effects

One of the main aspects of the ICP design is the consideration of the effect of time on any given driven pile, i.e. ageing effects and the so called setup effect. The consequences of this assumption will now be detailed.

A number of different field tests performed at Dunkirk suggested that the axial capacity of driven piles in sand increased with time, if they were allowed to rest after the driving process. The main focus of the test was to investigate how shaft capacity increased with time in fresh piles which were not tested previously, assess the effects of loading prior to failure on the ageing process and finally to examine how cyclic loading affected piles in contrast to static loading. This increase in capacity is known as setup. In clays the gain in capacities is well understood due to dissipation of excess pore pressures associated with driving. On the other hand, setup effects on sand are not well known, and as such there is a need to reflect on methods that carry out considerations on the setup process.

Chow (1997) considered a number of potential mechanisms to explain the ageing trend. Firstly, it was thought that the development of corrosion in the steel pile and a formation of a welded crust of sand due to physio-chemical factors were the main factors leading to a gain in capacity. However, both theories were discarded. It was observed that setup would occur in steel, concrete and timber piles, thus corrosion was discarded as the primary mechanism. Regarding the formation of the crust, it was indeed observed and measured by a number of different works, the effect of such layer is to cause the failure surface to transfer into the soil mass, increasing the interface friction angle from the soil-pile value to a soil-soil value. However, such change cannot, on its own, explain the capacity gains measured in various tests. Furthermore, it was observed that piles with no crust would also experience an increase of capacity.

Chow, finally theorized that the stresses developed during pile installation, shielded the pile from a zone of high radial stresses. With time, creep stress redistribution lead to an increase in stationary radial effective stress during the setup period causing the increased resistance. The ICP method supports this formulation. In the experiments performed at Dunkirk in piles equipped with radial monitoring sensors, increases in stress were measured during the short equalization period prior to the performance of static load tests.

It is important to refer the fact that only the shaft friction appears to change with the effect of soil setup. Base resistance is therefore not affected by this mechanism.

Ultimately, the effect of ageing was seen to increase the pile capacity based on the load tests performed at Dunkirk, however it was concluded that the rate of growth is quite variable and not well known. Changes in stationary radial stress during set-up and enhanced dilatation during loading were thought to be the principal mechanisms that control the gain in capacity due to setup in sands.

Furthermore, one other major conclusion attained in the experiments was that after performing a range of static and dynamic load tests, a degradation of pile capacity was observed in all tested piles. Jardine et al concluded that any tests, static or cyclic can lead to a brittle load-displacement response. While the pile capacity recovered with time, it would never reach the same resistance as a pile which was allowed to age and had not been previously tested, i.e. fresh piles. This conclusion is related to the friction fatigue proposed by Heerema (1980) where due to the pile installation process the resistance

would be degraded where the pile experienced the most shearing. Figure 3.6 displays the growth trends in shaft capacity obtained in fresh piles (piles that were allowed to age without interference), it is also seen the growth trendline proposed by Chow (1996). The shaft resistance seen in the same figure was normalized to the ICP design approach. Initially the piles gain in capacity is small and quite similar to the one seen in Chow (1996) however, it will soon increase considerably before stabilizing at around 8 months. Note that in point B the pile was tested, leading to a degradation of resistance all the way to point C, which is quite close to the resistance proposed by Chow. This is explained due to the fact that Chow's trendline was obtained not only from fresh piles but from piles which were re-tested and re-stroked, resulting in an under prediction when comparing it to only fresh piles like the one seen in Dunkirk IAC (Intact Ageing Characteristics). On a final note, it is possible to see that for 10 days the ratio $Q_{t=10day}/Q_{ICP} = 1$, meaning that the ICP predicted capacity was finally reached after 10 days.

Some conclusions can be attained from here. Firstly, all of the tested piles suffered an increase in capacity with time, due to the increase of shaft resistance, meaning that the resistance measured during driving will not be the same as those measured in the short to medium term tests. Secondly pile loading (static or cyclic) will lead to a reduction in friction, this reduction is explained by the friction fatigue theory. Furthermore, and based in the final results seen in Figure 3.6 it can be said that the initial gain in capacity measured at Dunkirk and predicted by Chow (1996) are quite similar, yet for more than 10 days Chow trendline will under predict the gains in capacity due to setup, note that the gains in capacity are still not well known and can be quite variable, further study in this area is required. Lastly, it should be noted that the ICP formulation gives a good estimation of the short to medium term (10 days) axial capacity that can be developed on displacement piles.

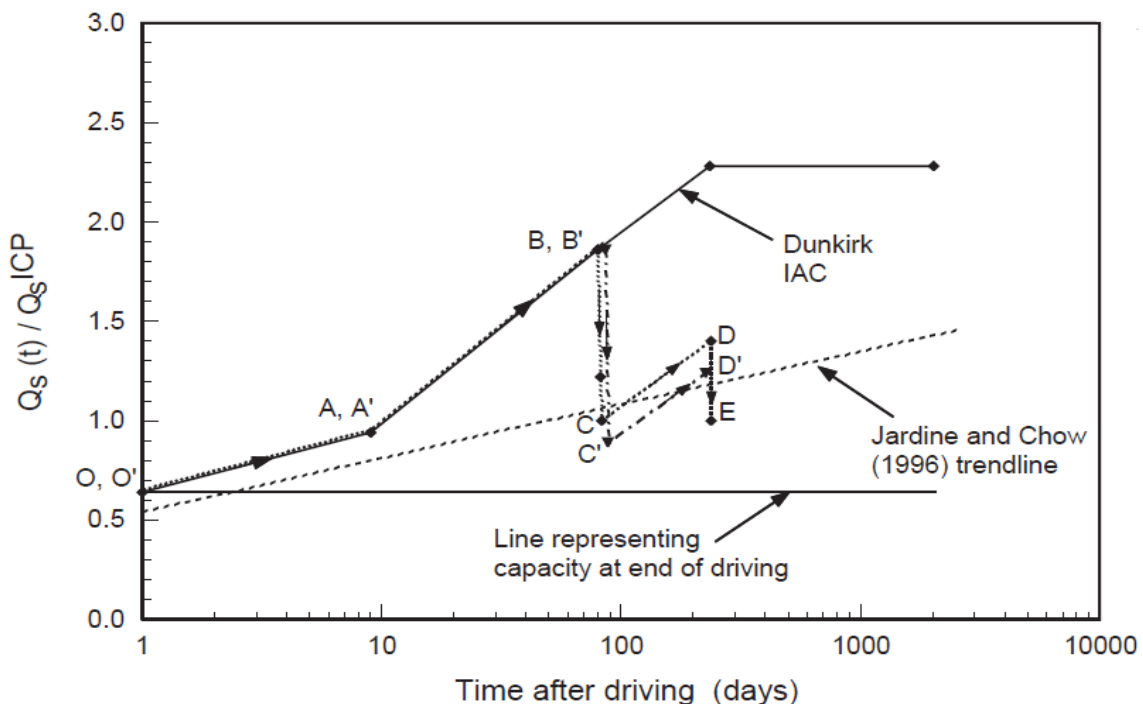


Figure 3.6 – Ageing Trend

- O-A-B – Evidence of pile setup, i.e. increase of capacity with time to the point of
- B-C – loss of capacity due to loading test

- C-D – Partial recovery of setup after letting the pile rest for a while
- D-E – Loss of capacity due to a new loading test

3.7.4. BASE RESISTANCE

Now regarding the pile base resistance, for open ended piles, it is known that these types of piles develop their base capacities through a relation of the internal skin friction transferred over to the solid column enclosed in the inside of the pile. The other factor that influences the base resistance is, understandably, the resistance beneath the annular area of the pile as well as the arching effect. Based on his and others studies (Brucy et al., 1991) Jardine et al concluded that the arching effect is unlikely to be effective in large diameter piles. Thus a simplified alternative was proposed. ICP provides two principles which provide the minimum relative density, D_R to achieve a full arching effect, given a pile diameter.

$$D_{int} < 0.02 \times (D_R - 30) \quad (3.32)$$

$$D_{int}/D_{CPT} < 0.083 \times q_c/p_a \quad (3.33)$$

The base resistance developed by a fully arching plugged pile is lower than that of a comparable closed-ended pile. We can, therefore, assume a smaller base resistance. According to this Jardine et al theorized that a fully plugged pile develops 50% of the base resistance of a similar close ended-pile with the same diameter, for a pile settlement of $D/10$. As a safety precaution it is advisable to respect the following lower limits proposed by ICP.

- The fully plugged capacity should be no less than the unplugged capacity
- q_b should be larger than $0.15 \times q_t$

Below, the equation for the unit base resistance, in an open ended fully plugged pile is presented.

$$q_b = q_t [0.5 - 0.25 \log(D/D_{CPT})] \quad (3.34)$$

For unplugged piles it is assumed that they sustain end bearing on the annular pile base area only and it is expected that the unit base resistance, q_b should be equal to the cone tip resistance, q_c .

3.8. STEVENS MODEL (STV)

The model proposed by Stevens et al (1982) was presented firstly in the 14th annual OTC (Offshore Technology Conference) and remains to this day one of the most used methods to predict driveability in the North Sea. The design approach gives estimation to the SRD by correlating measured and predicted blow counts.

Several other methods that try to predict the SRD or axial capacity of a given driven pile were based on this proposal since it was one the first well accepted theory's regarding driveability, such as the ones presented in Alm and Hamre (1998) and Alm and Hamre (2001).

In the paper several case histories of driven piles in the Arabian Gulf are presented. The pile diameters are approximately of 1 meter with a wall thickness of 3.81 cm and were all driven by hammers with

rated energies ranging from 16300 J to 44100 J. To greatly enhance the correlations made in his paper, a careful monitoring of the driveability of the pile was made. Note however that Stevens design approach does not depend on CPT test results, nor it makes considerations on friction fatigue.

Stevens followed a certain methodology when it comes to the pile driveability study. Firstly, the driving resistance that can be overcome was calculated by a wave equation analysis. Secondly, the SRD is estimated from the in-situ soil properties. Lastly these results are compared and an estimation of the pile driveability is then made. As expected it is always wise to include in the estimations past experiences and sound judgments of similar regions.

Again, the estimation of the SRD is similar to the computation of the axial capacity of the pile. Thus, SRD can be described as the sum of the shaft and base resistance. Shaft resistance and base resistance will be overview in the next sections. Formulation regarding clays and sands will be presented.

3.8.1. SHAFT RESISTANCE

Shaft resistance is described by Stevens et al (1982) by:

$$Q_{sf} = \overline{f_{sf}} \times A_{sup} \quad (3.35)$$

Where:

$\overline{f_{sf}}$ – Average unit side friction during driving

A_{sup} – Embedded surface area of the pile

For piles driven in clays the unit skin friction, or side friction, during driving is estimated by correlating it to the clay stress history in terms of OCR. The friction is also adjusted using an additional aspect, the pile capacity factor F_p .

$$f_{sf} = F_p \times \alpha \times S_u \quad (3.36)$$

$$F_p = 0.5OCR^{0.3} \quad (3.37)$$

The Over Consolidation Ratio, OCR is estimated by Equation 3.38

$$OCR = \frac{S_u}{S_{norm}} \quad (3.38)$$

S_u – represents the undrained shear stress

S_{norm} - Undrained shear stress if normally consolidated.

When dealing with granular soils, the skin friction during driving can be formulated as the basic static capacity procedures. It can be described by:

$$f_{sf} = K \times \sigma'_v \times \tan \delta \quad (3.39)$$

While:

K – Coefficient of lateral earth pressure

σ'_v – Effective overburden stress

In the article Stevens et al suggested that the coefficient of lateral earth pressure should be $K = 0.7$. However new studies conducted by Schneider et al (2010) proposed relating the earth pressure coefficient with CPT results, as follows:

$$K \times \sigma'_{vo} = \frac{q_t}{76} \left(\frac{\sigma'_{vo}}{p_{ref}} \right)^{0.13} \quad (3.40)$$

The model also made a few predictions when encountering rocks. It is anticipated that the driveability will fracture the rock layers reducing the rock to a granular material. Therefore, the calculations made to predict unit skin friction for piles driven in rocks are made assuming sand parameters.

3.8.2. BASE RESISTANCE

Now dealing with the base resistance, it was formulated as follows:

$$Q_{bf} = q_{bf,w} \times A_{End} \quad (3.41)$$

Where:

$q_{bf,w}$ – Unit end Bearing

A_{End} – End bearing Area

For cohesive soils, unit end bearing is computed by:

$$q_{bf} = S_u N_c \quad (3.42)$$

Whereas N_c is the dimensionless bearing capacity factor. Unit end bearing is calculated in an identical manner when facing with granular soils with N_c made equal to 40. Thus, q_{bf} can be theorized by:

$$q_{bf} = \sigma'_v N_c \quad (3.43)$$

It is important to refer the fact that Stevens proposed limitations to the unit end bearing and unit skin resistance. In sands these limits are of 95 kPa to skin friction and 9575 kPa to unit end bearing.

Stevens et al (1982), in the name of safety, advised calculating both a lower and upper bound values of the SRD for both “coring” (unplugged) and plugged conditions.

It is known that when the pile cores, the soil within the pile remains stationary, thus skin friction is developed both in the outside and the inside of the pile wall. In this situation the lower bound value is calculated assuming that the skin friction developed on the inside of the pile wall is 50% of the outside shaft friction. The upper bound value is made assuming that the internal and the outside shaft friction are equal. In other words $f_{sf,i}/f_{sf,o}$ ranges from 0.5 to 1.0.

On the other hand, when the pile is plugged, the soil contained within the pile moves downwards with the pile. Skin friction will therefore only be mobilized on the outside of the pile wall. Stevens et al (1982) commented that, for the upper bound plugged case and when dealing with granular soils, the unit skin friction and unit end bearing are developed by increasing their values by 30% and 50% of their respective lower bound values. It is worth mention that when computing the SRD for a plugged pile, end bearing should not be limited to the frictional resistance developed by the soil plug.

For cohesive soils a parallel increase should be considered. Stevens recommends a 67% increase in the unit end bearing. Regarding the unit skin friction, it should not be increased.

3.9. ADDITIONAL METHODS

Correct assessment of pile driveability is vital to reach a safe design. As such it does not come as a surprise that several other theories besides the ones described earlier attempt to give their estimates to calculate the SRD.

The aim of this section is to give a summarized view on other design methods, however only the approaches described earlier are to be analysed in the following chapters.

3.9.1. TOOLAN AND FOX

Another method that manages to predict the static soil resistance at the time of driving is the one presented by Toolan and Fox (1977). The model is based on the successful installation of several piles in the Forties Oil Field, the largest oil field in the North Sea.

The methodology proposes estimation for unit skin friction for both granular and cohesive soils. In clays, unit skin friction is equal to the remoulded undrained shear strength, however accurately assessing this parameter is quite difficult, as such a portion of the measured undistributed strength is often assumed, expressed by the factor α . Unit skin friction can be expressed by:

$$f_{sf} = \alpha S_u \quad (3.44)$$

On the other hand, unit tip resistance is set equal to the CPT tip resistance.

Regarding granular soils skin resistance is calculated as a fraction of the recorded cone tip resistance, usually 1/300 when dealing with dense sands. Tip resistance is also assumed as a fraction of the CPT tip resistance, one third to be exact. On a final note, it is commonly accepted that large diameter piles behave in a fully coring manner, meaning that unit skin friction is applied to the external and internal pile wall and unit end bearing to the cross sectional area of the pile.

Hence SRD will be the least of:

$$V_{ult} = \sum f_{s,o} A_{s,o} + \sum f_{s,i} A_{s,i} + q_p A_w \quad (3.45)$$

$$V_{ult} = \sum f_{s,o} A_{s,o} + q_p A_{plug} \quad (3.46)$$

Where:

$f_{s,o}$ – Unit shaft friction (outside pile)

$A_{s,o}$ – Shaft area of the pile (outside pile)

$f_{s,i}$ – Unit shaft friction (inside pile)

$A_{s,i}$ – Shaft area of the pile (inside pile)

q_p – Unit end bearing capacity

A_w – Cross-sectional area of steel wall

A_{plug} – End bearing area

Driving piles will sometimes produce changes in the soil and pore water. These underlying effects will affect the degree of SRD. The method proposed by Toolan and Fox (1977) does not consider such effects in their formulations. However, it was found, that these equations give a reasonable estimation of the static soil resistance to driving.

3.9.2. API (AMERICAN PETROLEUM INSTITUTE)

The method provided by the American Petroleum Institute, serves to give a simplified estimation of the axial capacity of a pile. It is not unusual to apply such method in the offshore industry. However, this model presents one large limitation. It does not provide any information about pile displacement. Serviceability assessments are therefore very limited without any information regarding displacement. This method is noticeably conservative. The model has its basis on the static and monotonic application of axial loads. It does not make any considerations regarding the mechanisms that occur in the pile soil interface during installation. One could describe this model as a β – approach.

$$f_{sf} = \beta \cdot p'_0(z) \leq f_{sf,lim} \quad (3.47)$$

The values for β and $\tau_{s,lim}$ vary depending on the type of soil

When dealing with the bearing capacity, a formulation such as the one described in Equation 3.48 can be used, the values of $q_{bf,lim}$ and the end bearing capacity are also dependent on the soil type and density

$$q_{bf} = N_c \times p'_{0,TIP} \leq q_{bf,lim} \quad (3.48)$$

3.9.3. SCHNEIDER

The model proposed by James A. Schneider in Schneider et al (2010), serves to analyse driveability of open ended piles in very dense sands. This modern method takes into consideration the reduction in local friction (using the same model as the one detailed in Alm and Hamre), with increased pile embedment, also known as friction fatigue. It also correlates CPT test data to link soil properties to the driving resistance. The paper back calculates the installation resistance of open ended piles (diameter ranging from 0.356m to 2m).

As with the models described previously, pile driving resistance results from the sum of a static and a dynamic component. The dynamic component is related to an increase in resistance due to inertial and viscous rate effects. The static component is already known as the SRD.

As we know SRD is the sum of the pile skin resistance and the pile base resistance.

Some methods present the internal unit shaft friction as a fraction of the outer shaft resistance. Schneider et al (2010) proposes that the following ratio $f_{s,i}/f_{s,o} = 0,5$, this portion is the same as the lower bound used by Stevens et al (1982). Thus SRD will come as:

$$SRD = \pi D \sum_{Z_{tip-L_{emb}}^{Z_{tip}}} 1.5f_{s,o} \times \Delta Z + q_{ann}A_{ann} \quad (3.49)$$

With the calculated SRD a comparison between the measured and the estimated blow counts, needed to drive the pile was made. The model gave the best results when dealing with larger diameter piles.

To conclude, all CPT and empirical based methods adopt similar formulations for shaft and base resistance, with some minor differences in the mechanisms taken into account. With the increasing size of offshore monopile foundations, there has been a need to validate some methods that aim to predict then axial pile capacity. Therefore, the use of these newer methods may involve extrapolations from existing databases, such as driving logs of previously driven piles and by means of a one dimensional wave equation model back calculate the required number of blows.

4

WAVE EQUATION APPLIED TO DRIVING - GRLWEAP**4.1. INTRO**

Pile driveability is an important topic of study and research since the early 20th century. The problem can be summarized in a longitudinal wave transmission that is covered in a general way by the wave equation analysis. The models used at that time were all based on empirical formulations, such as dynamic equations.

In the mid 1920's dynamic formulae were all established in the driven pile industry. Without any truly consistent methods of static capacity assessment, and given the conservative nature of most foundation designs, dynamic formulations were never seen as anything but robust.

The event that brought their weakness to the forefront was the development of precast concrete piles. Dynamic formulae assume, wrongly, that the pile is a rigid mass, or at best a simple spring. In the process of driving these relatively new concrete piles, tension cracking were developed in the mid-section of the pile.

Dynamic formulas, together with observed and experimental penetration resistances, do not produce reasonably predictions of actual pile capacities. Additionally, they do not provide information on compression and tension stresses in the piles during driving. Hence the so-called "wave equation analysis" of pile driving has eliminated many inadequacies associated with dynamic formulas by realistically simulating the hammer impacts and pile penetration process. The wave equation refers to a partial differential equation, meaning a comprehensive approach to the mathematical representation of a system consisting of a hammer, cushions, helmet, pile and soil. It describes the mechanics of force transmission along the pile.

Fortunately, it is unnecessary for the piling engineer to solve this calculation; this is done, in a fairly accurate method, by means of Smith's lumped mass model. Still, the important contribution of Smith was not proposing the wave equation itself, but developing a complete analysis routine, including recommendations for hammer, driving system and pile and soil considerations, all done with the assistance of a supplementary computer program for the appropriate calculation of the dynamic motions and forces that occur in the described system after ram impact. This structure composes the wave equation analysis. The final purpose of the present chapters is to attempt to give an explanation of what is a wave equation analysis, how it works and what problems it can solve

4.2. DYNAMIC FORMULAS FOR STATIC CAPACITY DETERMINATION

Ever since piles have been used to support structures, there has been an attempt to find rational methods for determining the pile's load carrying capacity. Several methods designed for predicting capacities were then proposed, using pile penetration observations obtained from the driving process. At the time the only realistic measurement that could be obtained during driving was the pile set per blow (how far the pile moves in one blow). As a result, energy concepts connecting the kinetic energy of the hammer to the resistance on the pile as it penetrates the soil were developed to determine pile capacity. Equation 4.1 represents said concept.

$$W \times h = R \times s_b \quad (4.1)$$

Where:

- W - Ram Weight
- h - Ram stroke
- R - Soil Resistance
- s_b - Set per blow

Expressions such as Equation 4.1 are known as dynamic formulas. Because of their relative simplicity they have been widely used for many years. More comprehensive formulation includes several other aspects such as pile weight, energy losses among others. The modified Gates formulation, seen in Equation 4.2 is a more refined dynamic expression and its one of the few dynamic formulas that currently, is still somewhat in use to calculate the ultimate resistance.

$$V_u = [6,7\sqrt{W \times h} \log(10N_b)] - 445 \quad (4.2)$$

Where:

- V_u – Ultimate pile capacity (kN)
- $W \times h = E_r$ – Rated hammer energy (J)
- N_b – Number of blows

Dynamic formulae often thought of driven piles into tested piles, as the resistance of the pile could be estimated by the blow count of the hammer, which is readily monitored during driving. However, such interpretations were shown to have poor correlations and wide scatter when statistically compared with static load tests. Therefore, with the exception of when well supported empirical correlations under a given set of physical and geological conditions are available (or where a site specific correlation has been obtained for a hammer-pile-soil system with a static load test result) dynamic formulas should not be used.

4.3. LIMITATIONS OF DYNAMIC FORMULATIONS

The main issue with dynamic formulas is that in the most part they are fundamentally incorrect. The problem can be traced to the modelling of each component within the pile driving process, the driving system. A correct representation of the driving system and its energy losses can be difficult to achieve using dynamic formulas. Furthermore, such formulations assume a rigid pile behaviour, thus ignoring pile axial stiffness effects on driveability. Lastly it is also assumed that the soil resistance is constant and instantaneous to the impact force, hence neglecting the characteristics of a real soil behaviour. The dynamic soil resistance is the resistance of the soil to rapid pile penetration produced by the hammer blows. This resistance is not equal to the static soil resistance. However most dynamic formulas assume, incorrectly, that the resistance during driving is identical to the static resistance. The rapid penetration of the pile during driving is not only resisted by static friction, quake, but also by the soil viscosity, damping.

4.4. ALTERNATIVES TO DYNAMIC EXPRESSIONS

Most shortcomings of dynamic formulations can be overcome by a more realistic analysis of the pile driving process. A one-dimensional wave equation analysis such as the one GRLWEAP is based on, is a realistic method to simulate the process of pile driving. Such analysis might have been difficult to use, in routine projects twenty years ago, but with the advances made with technology, today one-dimensional wave equation programs are commonly used. The basis of the wave equation model is presented by Smith.

4.5. FUNDAMENTALS OF WAVE EQUATION FORMULATION/ APPLICATION TO DRIVING

There has long been a recognition in how the wave equation as impacted driven piles, and is now commonly used to calculate, adapt and confirm driving stresses and ultimate capacity of driven piles. The present and following sections will be focused on how waves are propagated in piles and in turn, how can this give information as to the performance of the foundation, not only during installation, but also during the piles service life.

The influence of the stress wave theory to piles is based on the statement, that due to the impact of the hammer, a stress wave propagates downwards along the pile, producing a dynamic interaction and possible changes in the pile impedance. The pile driving process readily provides information regarding the soil resistance.

4.6. SMITH'S MODEL

This method was first considered by E.A.L Smith (1960) and provides a sensible basis for several other researches. In the one-dimensional model offered by Smith the hammer, pile and cap-block are divided by segments or nodes (with approximately one meter in length) and replaced by several distinct masses and weightless springs. On a side note, Smith stated that when dividing the pile into unit lengths, for the purpose of an accurate calculation, they must be considerable shorter than the length of the wave produced by the impact of a hammer, fortunately, a pile driving impact usually produces a fairly long wave form. Spring stiffness and mass values are calculated from the cross sectional area, modulus of elasticity, and specific weight of the corresponding pile section. These elements are used to emulate the pile stiffness and weight. The motion of each weight and each spring is then calculated as though each were a distinct object. Below in Figure 4.2 visual representation of Smith's model is provided.

As it can be seen, side resistance is modelled by a group of point forces at each pile segment, whilst base resistance is modelled by a single point resistance applied to the final pile segment, both tip and shaft. The soil resistance to driving is a function of a parameter named, soil quake, Q . Figure 4.1 and the following paragraph provides more insight on the matter.

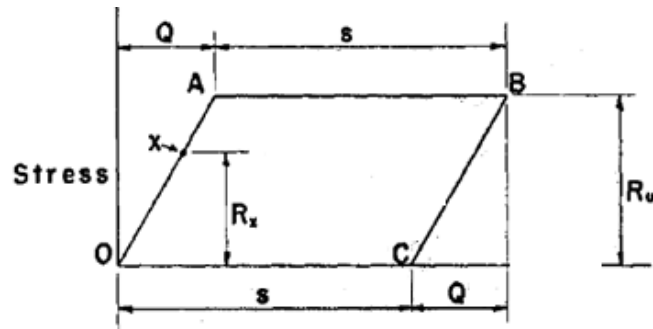


Figure 4.1 – Stress Strain diagrams, as seen in Smith (1960)

The displacement at which the soil changes from elastic to a plastic behaviour is stated to as the soil quake. Compressing the soil beyond point A will make the soil reach its ultimate capacity R_u . At this point plastic failure occurs and ground resistance will remain constant, until point B is reached. Elastic rebound equal to Q is then expected and motion ceases at point C. However, this static component does not take into consideration the element of time and velocity. Thus Smith introduced an additional factor, of viscous damping. The damping resistance is temporary and does not entirely contribute to the bearing capacity of the pile.

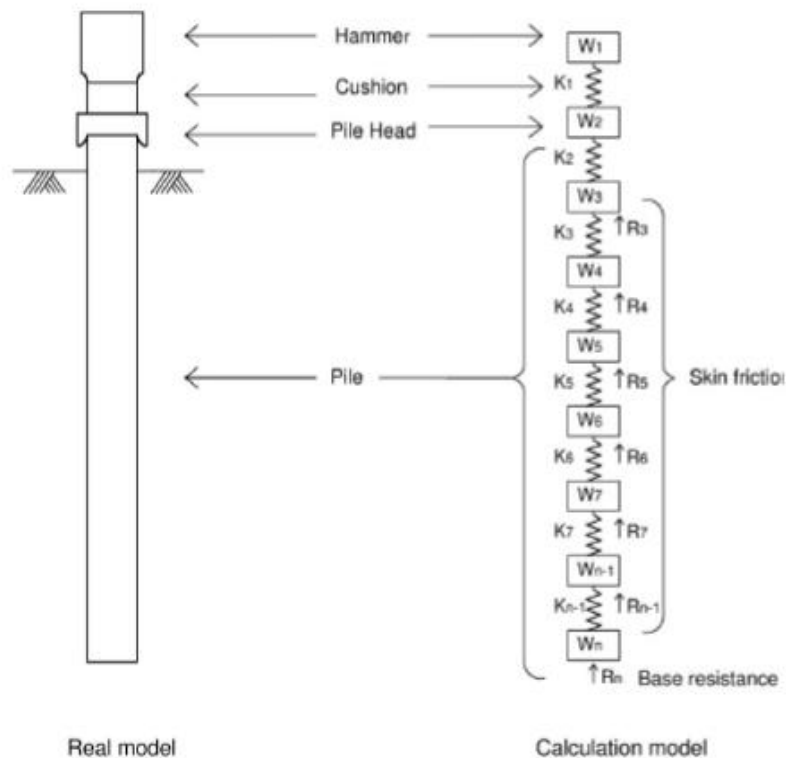


Figure 4.2 – Smith's Drivin system, adapted from Saraiva (2014)

Despite having been one of the most used and accepted methods it is not without limitations. In a driveability analysis, the modelling of the soil performance is one of the most important issues in order to get a reliable prediction on the SRD. Consequently, Smith's model is only as good as the certainty of the values used for soil parameters. The soil factors used in the model cannot be associated with soil properties; also they cannot be measured using common geotechnical investigation methods. Thus numerous researches are carried out to provide accurate quake and damping values to develop a better representation of the soil reaction.

Furthermore, the wave equation is a theoretical solution rather than the empirical solutions of most pile formulas. It describes only the structural dynamics of soil, pile and hammer. It does not solve the associated soils mechanics problems.

Finally, such as other foundation theories the wave equation will assist but cannot replace the sound judgment developed from experience in the field of pile driving by engineers and contractors.

4.7. GRLWEAP

GRLWEAP was developed out of the WEAP (Wave Equation Analysis of Pile driving) program back in 1976 and its first appearance was around the 1980s; the software suffers several major updates until 2010 and is continuously revised and kept up to date with current driving systems. In addition to being one of the most used programs to select and optimize pile driving equipment it is also capable to compute the pile static capacity. The software is a powerful tool that is able to accurately simulate motion's and forces in a foundation pile when driving by either an impact hammer or a vibratory hammer. In general, the software is able to generate essential information regarding the pile driveability, which can be summarized in the following way:

- The blow count of a pile under one or more assumed ultimate resistance values and other dynamic soil resistance parameters given a hammer and driving system, or from a static analysis
- The axial stresses in a pile, both tension and compression, average over the cross section for a certain pile penetration and associated ultimate capacity values. Consideration of bending stresses is also possible
- Energy transferred by the hammer to the pile for certain pile penetration and associated capacity values.
- The pile velocity and displacements along the pile for certain pile penetration and associated capacity values.
- Residual stresses remaining in the pile between hammer blows

For a number of different simulations by varying the hammer type, driving system parameters and pile properties an optimal system can be selected, assuming the soil properties are accurately accessed.

4.7.1. DRIVING SYSTEM

GRLWEAP relies on the wave equation method as a means to accurately represent the driving system. The model is heavily based on Smiths proposal, where by dividing the system into segments and using springs and dashpots an accurate representation of the soil dynamic and static response is delivered. Figure 4.3 represents the one of the driving system used by GRLWEAP. The system consists of a ram, assembly (only applicable in ECH hammers) hammer cushion, helmet and an additional pillow. The

additional pillow is only used in concrete piles since there is an additional need to control stresses. It is evident that the presented model is based in Smith (1960). Similar to Smith in order to represent the different elements of the system, they were replaced and divided by masses, springs and dashpots. Again, the pile springs and segment masses represent the pile stiffness and weigh. The same can be said for the assembly, ram, helmet and pile cushion.

4.7.2. SOIL MODEL

Again, GRLWEAP's soil model is essentially a Smith approach. Regarding soil resistance along the embedded portion of the pile, and at the pile toe. Both are represented by both static and dynamic components, i.e. quake and damping. Therefore, both of these components are mobilized on every embedded pile segment. The static soil resistance forces are represented by elastic-plastic springs. Dynamic soil resistance forces are modelled by dashpots which simulates the resistance to movement via viscous friction as well as the energy loss. Figure 4.4 and Figure 4.5 represent the static resistance and the dynamic resistance respectively.

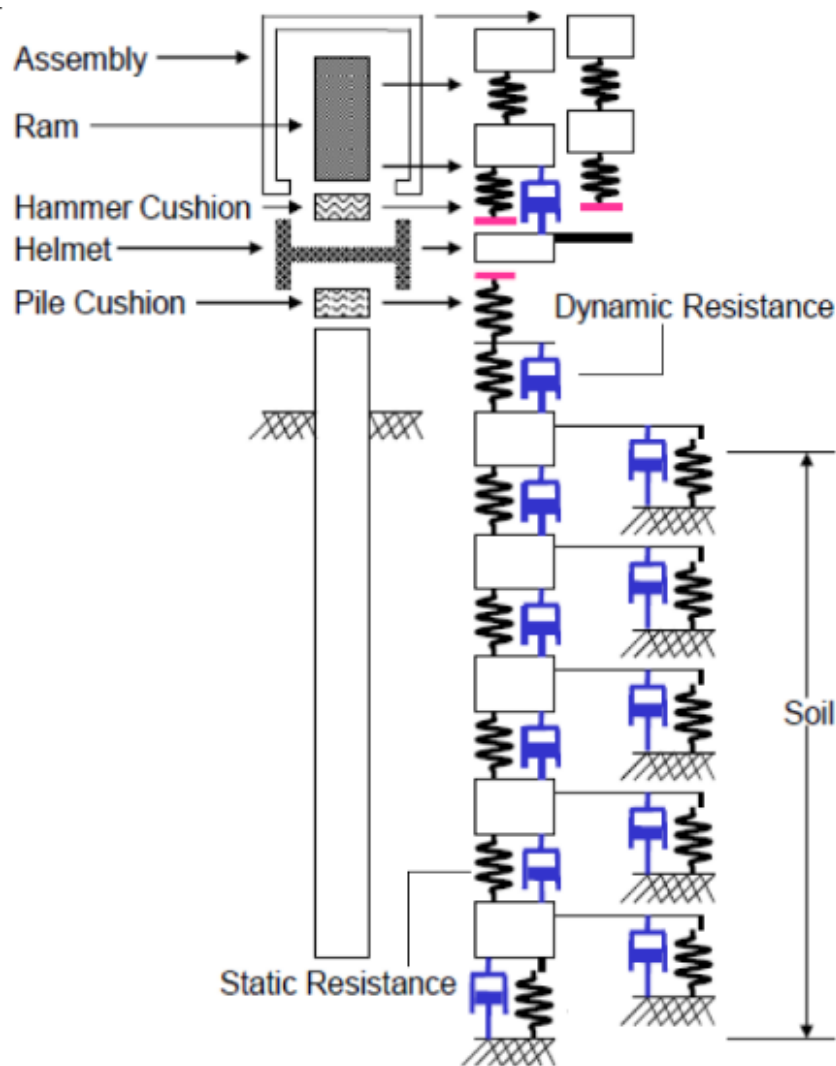


Figure 4.3 – GRLWEAP driving system for an ECH

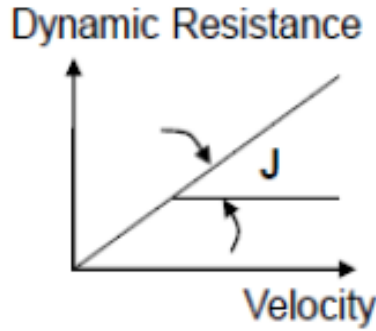


Figure 4.4 – Dynamic Resistance

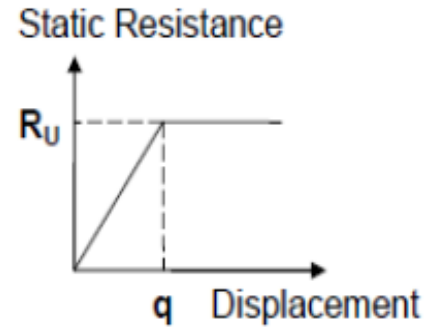


Figure 4.5 – Static resistance

4.7.3. BLOW COUNT COMPUTATION

For blow count calculations, the difference between the maximum toe displacement, u_{mt} , and the average quake is calculated. The average quake is:

$$q_{avg} = \sum [R_{ui}(q_i)] / R_{ut} \quad (4.3)$$

Where R_{ui} and q_i are the individual ultimate resistance values and quakes, respectively, and R_{ut} is the total ultimate capacity. A summation is made over all elements from $i = 1$ to $N+1$ (N is the number of pile segments). Resistance number $N+1$ represents the end bearing. The predicted permanent pile set and blow count are computed as follows:

$$set = u_{mt} - q_{av} \quad (4.4)$$

$$B_{ct} = \frac{1}{set} \quad (4.5)$$

It should be noted that for strongly variable quakes a residual stress analysis may be a more accurate method of blow count computation.

For 2-pile analyses, u_{mt} is the lesser of the two pile toe maximum displacements. However, this does not guarantee that the blow count is calculated accurately for a 2-pile analysis. For this analysis option, the user must carefully review the relative motions of both piles.

For two pile toe resistance values, only the primary pile toe is considered, however, q_{av} considers the resistance effect of the second pile toe.

For vibratory analysis the average pile penetration per unit time corresponds to set per blow for impact hammers. The inverse of this value is the penetration time per unit time (e.g. per second) and this is a value that is often recorded for construction control.

4.8. ENERGY IN PILE DRIVING SYSTEMS

Energy is one of the most discussed aspects of pile driving and of the wave equation, especially in the matter of efficiency and transfer of energy. Hammer efficiency represents the ratio of the actual energy generated or transferred through any point of the driving system to an ideal energy stage. A more accurate definition of the ideal energy stage as well as the actual energy will be provided later on.

As it is knowing the amount of energy applied to the pile will drive it to a certain depth, naturally energy drives piles. Therefore, it is vital to understand how energy is generated.

Energy is the product of a force moving or moved through a distance. The force can be constant or variable with time (or distance). If dealing with a constant force through the same distance, like a single action hammer, energy can be formulated by:

$$E = F \times d \quad (4.6)$$

With a single acting impact hammer the force of gravity acts on the ram mass moving it downwards over a given stroke height. The ram travels the stroke distance and impacts the pile head. Such impact will produce a force with a high intensity but through a short distance. Moving through such a short distance, the force imparts energy and will move the pile. It is important to reinforce the notion that a force is moving over a distance, thus effecting a transfer of energy. On one hand the gravity force acts on the ram through its stroke. On the other hand, the impact force moves over the initial compression of the pile top and this leads to another transfer of energy. If both forces are constant all over the travelled distances and if by admitting that the resistance of the pile is simply the resistance of the soil, this process could be formulated by:

$$R_u = \frac{W \times Stroke}{set} \quad (4.7)$$

This oversimplification was the basis of pile driving, despite being over simplistic it serves to illustrate the concept of energy in driving as well as the relation between energy, set, and the resistance given by the soil.

Note that all of the previous considerations were made in a “static” manner. As it is known pile driving is not a static phenomenon. Hence there is still a need to identify how energy is transferred from the ram to the pile. The answer is kinetic energy. As the ram moves downwards due to the force of gravity the potential energy from that field is progressively converted to kinetic energy. Without further losses, Equation 4.5 can cover such effect.

$$E = W_s \times Stroke = \frac{1}{2} m \times v^2 \quad (4.8)$$

Upon impact this kinetic energy is converted into a force which moves through a distance, in this example, such distance represents the movement of the pile, known as *set*.

To sum up, the concept of pile driving and energy can be described as

- Conversion of potential to kinetic energy due to the movement caused by gravity.
- Conversion of the kinetic energy from the ram to the impact force, leading to pile movement.

4.8.1. ENERGY LOSSES

The wave equation analysis of an impact hammer requires the calculation of ram impact velocity. During the fall of the hammer ram, the pile does not experience dynamic forces. However, the weight of anvil, helmet or the assembly itself can create a static force in pile and soil. Hence the dynamic analysis only has to cover the time period during and after the impact.

During the driving process energy losses are to be expected, i.e. the energy transmitted by the hammer will not be the energy the pile actually receives. There are a number of different factors that can slow a ram during its descent to impact the pile. Friction on the ram guides, back pressure losses, inefficient combustion (for the diesel hammers) are one of the many factors that can reduce the energy. Said losses are considered in the hammer efficiency parameter. Usually the more modern hammers present impact velocity monitoring, and according to the GRLWEAP background manual reasonable results are achieved with an efficacy value of 0.95. Naturally, most losses, such as hammer friction, are considered by the impact velocity measurement. Only losses occurring during the impact occurrence have to be considered. It is heavily recommended to carefully review and modify the efficiency values used. If for instance a stress analysis of the pile is required, it would be wise and conservative to increase the efficiency used. On the other hand, when analysing resistance capacities or blow counts a lower hammer efficiency value should be used, since it would lead to higher soil resistance and higher blow counts, in other words a more conservative driveability analysis.

It is now clear that the energy produced by the hammer may not be the same energy seen in the pile. ENTHRU refers to the energy that actually arrives at the pile top without consideration of rebound from the pile. The efficiency rating associated with this important quantity is known as system efficiency.

In the previous paragraphs review of energy losses near the pile top were made. However, once the energy passes through the pile head, it will travel the pile and into the soil. Thus, the pile acts as a transmission line to direct the energy obtained through the pile top and later transmitted to the surrounding soil. Pile material have internal dynamic components, damping, and can dissipate the energy in the pile itself, however these losses are not that great when compared to the losses occurring in the surrounding soil.

As detailed above, the soil acting against the pile can be described as a spring (representing the elastic resistance of the soil) and as a viscous dashpot (representing the linear dissipation of energy into the soil mass). The point of pile driving is to advance the pile into the soil, this means overcoming the elastic limit of the soil to actually penetrate the pile into the ground. Once the elastic limit is overcome, soil resistance represents dissipation of energy into the soil.

Finally, a small note on energy losses for the hammer and for cap system, particularly for the cushion. Most driving systems make use of a cushion in order to protect the hammer from excessive stress and loads during driving and to protect the pile from the same excessive loads (particularly true for concrete piles). During compression the cushion will experience a rebound which takes place after

each hammer blow. To account for the cushion plasticity a coefficient of restitution, C.O.R is usually applied. Usually, the lower the C.O.R, the higher the energy losses are for the cushion material.

To conclude, in a driveability analysis and when making use of GRLWEAP it is essential to know how the energy is generated, how it travels along the pile and how it is lost along the driving system. Armed with this knowledge it is possible to model an accurate driving system.

4.9. GRLWEAP ANALYSIS OPTION

In this section, an overview of the software many uses will be detailed. Special focus will be given in the driveability analysis as well as its underlying aspects, such as friction fatigue and setup effects. The driveability analysis is the main point of the presented work, and as such it is important to understand how GRLWEAP functions, since this option was used to back calculate the required number of blows.

4.9.1. BEARING GRAPH

One of the most commonly used applications of the GRLWEAP wave equation analysis is the option to calculate a bearing graph. The so called bearing graph provides the user with relevant information such as the expected blow count needed to achieve a certain capacity. It starts with establishing a relationship between ultimate capacity and penetration resistance. The user is required to input data of the resistance values along shaft and at the toe, so that the software estimates the permanent set in mm under one hammer blow. Specifying several different ultimate capacity values yields a relationship between ultimate capacity and penetration resistance in blow per meter, or 0.25 meters. With this tool it is also possible to relate the driving stresses in the pile with penetration resistance. Figure 4.6 displays the plotted capacity vs calculated blow count on two different soils.

To sum up, the analyst assumes a total ultimate capacity and an input of the distribution of shaft and toe resistance is made. The blow count is calculated. A higher ultimate capacity value is chosen next and shaft and toe resistance are proportionally increased to match the capacity. The program is able to analyse up to ten capacity values.

The geotechnical analyst usually develops a bearing graph for different pile penetration lengths and uses these graphs in the field, with the observed penetration resistance, to determine when the pile has been driven to the required ultimate capacity. It is a useful tool to have on the field. However, when dealing with diesel hammers or ECH an inspector's chart may be the best option.

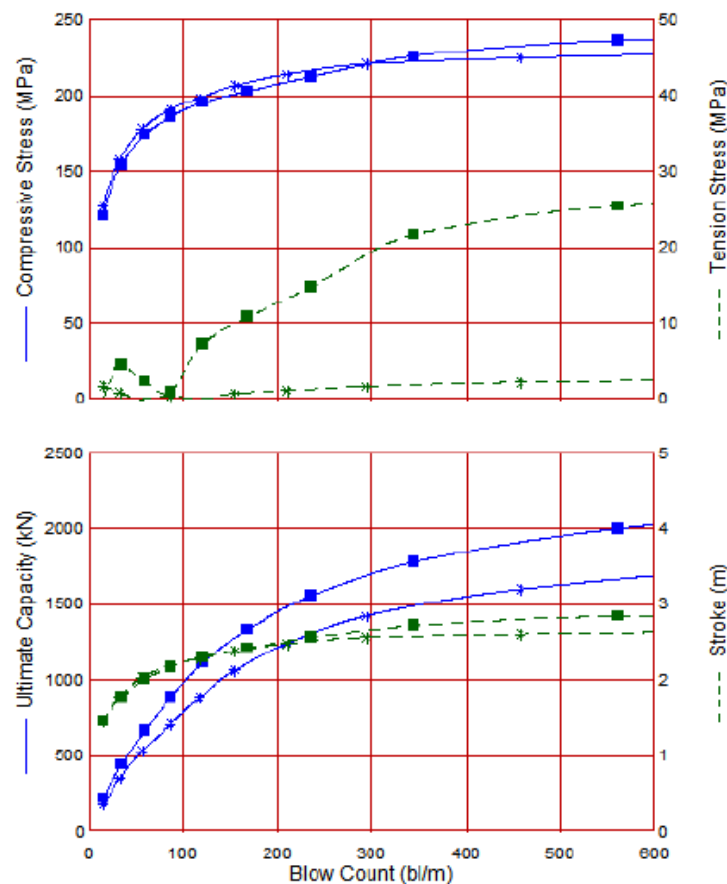


Figure 4.6 – GRLWEAP Bearing Graph

4.9.2. INSPECTOR GRAPH

For the more modern hammers it is frequently required to adjust the driving criterion according to the apparent stroke. Stroke of open ended diesel piles can be monitored using appropriated instrumentation. Therefore, it may be required to run individual analysis with several fixed strokes in order to provide the field inspector with a driving criterion for each apparent stroke. Once more GRLWEAP provides such analysis in the form of an Inspector Graph. The software automatically produces ten different analyses with increasing strokes from a user specified minimum value to the hammer's rated stroke. The inspector chart provides a relationship between stroke and blow count required to reach one ultimate capacity value, in Figure 4.7 it can be see an example of an inspector's graph. On the other hand, this type of analysis is not viable when dealing with traditional mechanical hammers, since they are only capable of operating with a single fixed stroke. However, the more modern hammers such as the hydraulic or the diesel hammers are capable of operating at different stroke levels. Thus, this versatility is quite convenient since it gives the chance to uphold control of the stress levels acting the pile.

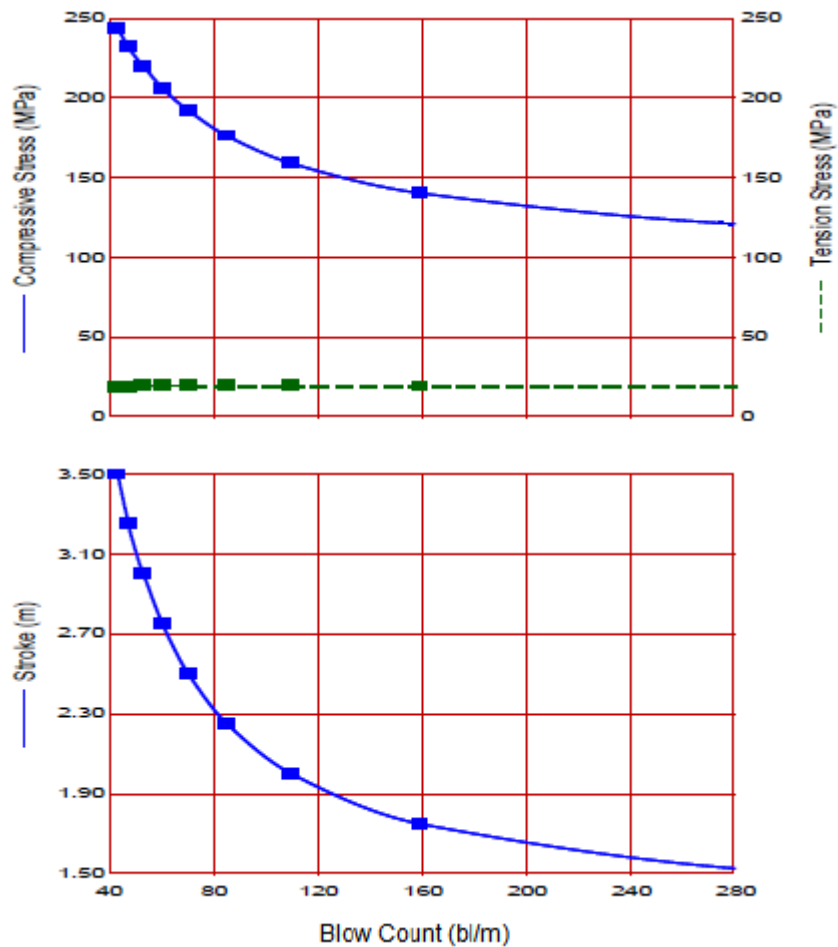


Figure 4.7 – GRLWEAP Inspector Graph

4.9.3. DRIVING ANALYSIS

For a more detailed analysis a driving analysis should be considered. This option calculates blow count, stress and transferred energy with pile penetration. This alternative can be more convenient since it calculates all of the above mentioned parameters without having the need to perform several bearing graph analyses for each penetration depth. One could say that a driving analysis consists of several automatically computed bearing graphs given the pile tip penetrations, in fact GRLWEAP is capable of analysing up to 100 different depths in this option. In Figure 4.8 an example of a driveability graph is provided. As it can be seen it provides the evolution of the resistance to driving and the required number of blows to drive the pile. In order to perform such driving analysis, the user needs to input some information. As with the other options, input consists of unit shaft resistance and end bearing usually estimated by the means of a static analysis or by using the software different tools to access soil resistance. To perform a driveability analysis, it is essential to use an additional ratio, named Gain/Loss factors. In a general way these values are a means of calculating the final resistance value, SRD. Further explanation relative to G/L will be given in the following paragraphs.

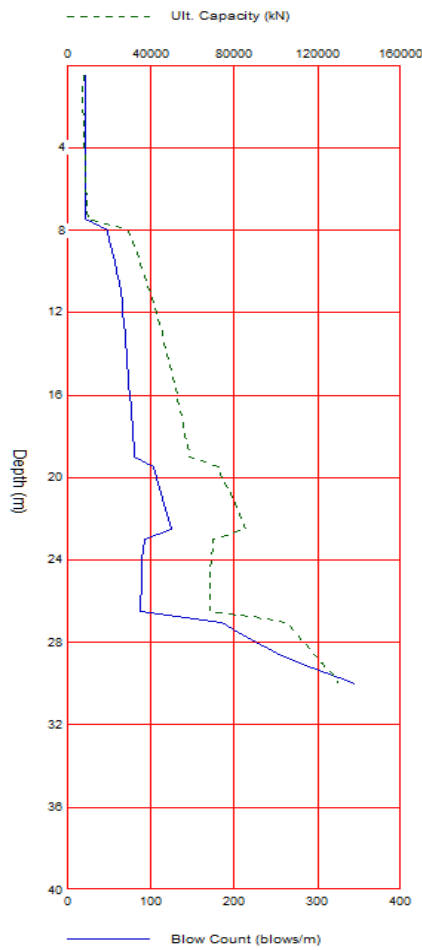


Figure 4.8 – Driveability graph

4.10. GRLWEAP - SRD

4.10.1. GAIN/LOSS FACTORS

In order to perform the driveability analysis and to estimate the SRD, generally the static pile capacity is estimated and applied to GRLWEAP which in turn will reduce it and will give the reduced resistance, simulating the degradation of friction encountered through driving or, otherwise increase said resistance, transforming it to the unreduced long term resistance, LTSR due to setup effects. In GRLWEAP this convergence is accomplished by means off Gain/Loss factors and setup factors. Naturally it is essential to use the gain loss factors for the overall soil profile as well as assigning individual soil setup factors for the different layers. G/L factors control the absolute variation of static soil resistance. Whilst the setup factors, f_s governs the relative change of soil resistance among the different soil layers. Again, gain loss factors are to be applied to all soil layers in proportion to their assigned soil setup factor.

The main focus of this section is to provide insight on the friction degradation used by GRLWEAP. There are two different approaches which allow for the calculation of SRD from LTSR or vice versa. The standard GRLWEAP approach, that will be discussed in the present section and the Friction Fatigue approach, which is frequently more used in offshore.

4.11. STANDARD APPROACH USING G/L FACTORS

For a particular soil type, SRD can be calculated by:

$$SRD = LTSR \times \frac{1}{f_s} \quad (4.9)$$

Again, SRD, as detailed, represents the soil resistance to driving, on the other hand LTSR, the long term static resistance, represents the unreduced soil resistance that can be achieved due to setup, in other words the maximum resistance, finally $1/f_s$ represents the gain or loss that can be applied to SRD, in GRLWEAP this proportion is known as the gain/loss factor, G/L. Further explanation will be provided.

As the driving progresses and the pile penetrates deeper the resistance will commence to be reduced due to consideration of friction fatigue. Hence, if the driveability progresses for long enough and no interruptions are made, soil resistance could reach a stage where it becomes fully reduced. Figure 4.9 represents the evolution of resistance for a driven pile, as it can be seen R_{uf} , the full ultimate capacity, or in other words, the LTSR will evolve into a fully reduced value as penetration increases. Such degradation can be explained due to the friction fatigue influence. It can also be seen that there is an interval where, despite the pile being driven deeper, no further reduction is applied, demonstrating that the influence of friction fatigue has a limit. It is easy to understand why such limit was imposed, otherwise if driving an extremely large pile there would be a penetration depth where the resistance to driving would be null. The driving distance required to fully degrade the LTSR is referred as limit distance and is of extreme importance in the GRLWEAP offshore option.

In the above paragraph the reduction of resistance was detailed. However, there are also situations where the fully reduced resistance can develop into the LTSR, such situation can occur due to interruptions in the penetration. As seen in the same figure, there was a time when driving stopped. Due to the effects of soil setup, shaft resistance will increase with time and if enough time is passed said resistance can reach a stage where it will evolve into LTSR. A limitation is also imposed as to impede resistance to increase indefinitely. The time needed for complete soil setup to be gained is referred as setup time

For Any Segment I

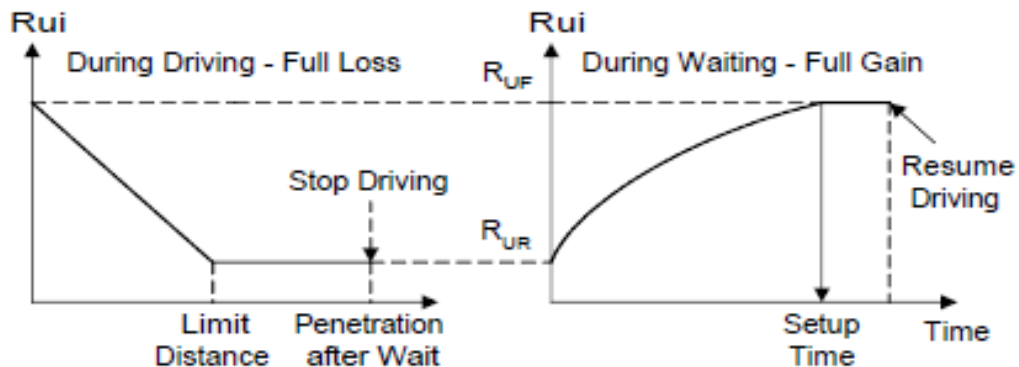


Figure 4.9 - Capacity vs Energy/Time for Complete Gain/Loss; as seen in GRLWEAP 2010 background

As stated above the G/L factor controls the distribution and evolution of resistance. This factor allows the user to model total or partial loss of soil setup, relaxation effects or the long term resistance. GRLWEAP gives the option to analyse up to five different G/L factors, thus providing up to five different capacity values at every analysed depth. The variation of the G/L has the following implications. Essentially G/L is an analysis option for the user to simulate resistance gains or losses:

- $G/L = 1$ No change in soil strength during driving and thus no resistance gain/loss will be analysed. Meaning that a consideration of no loss in friction is made and therefore the full LTSR is analysed
- $G/L < 1$ Proportionally reduces the resistance values under consideration of their relative setup factors and thus reflects that the soil resistance is lower during driving and increases after pile installation (i.e. soil setup, pile gains strength after installation)
- $G/L > 1$ Proportionally increases the resistance values and thus reflects the soil relaxation scenario. Soil resistance is greater during driving (i.e. relaxation, pile loses strength after installation)

G/L is calculated based on the soil setup factor, f_s . In a normal analysis, G/L should be made equal to the inverse of the biggest soil setup value, $1/f_s$.

Current soil setup factors used for different soil types are displayed on Table 4.1

Table 4.1 – Soil setup factors

Soil Type	Setup Factor
Clay	3
Silty Clay	2.5
Silty Sand	1.5
Sand	1.2
Fine sand	1
Sand - gravel	1

Let us consider, for example, a single soil layer composed entirely by clay with a setup factor of $f_s = 2.5$. The reduction factor in this theoretical exercise, would therefore be $1/2.5$. If the user intends to evaluate the reduction of LTSR to SRD the G/L factor used should be 0.4. However, if needed, we could analyse the restrike situation with full setup, meaning that no reduction in friction was considered, hence a G/L factor of 1.0 would be required. There might also be a need to analyse incomplete setup, in that situation the friction reduction factor should be any value intended by the user as long it is between the minimum value of 0.4 and the maximum value of 1.0

However, in most cases, there are more than just one soil type. When using GRLWEAP to analyse multiple layers this might present a problem, since there would be several different setup factors and therefore multiple gain loss factors. When facing, for instance with a layer of sand, $f_s = 1.2$ and a clay layer, $f_s = 2.5$ and analysing full loss of resistance, there would be two different reduction factors of 0.8 and 0.4 for sand and clay respectively (meaning the sand would lose 20% of the static resistance whilst the clay would lose 60%). GRLWEAP solves this problem by considering only one G/L factor specified by the user to be consistent with the most sensitive layer, i.e. the maximum setup factor of the two layers. Therefore, if we were to analyse a gain loss factor of 0.4, 0.7 and 1.0 the sands reduction factor would not be the same as the clays. Instead GRLWEAP applies capacity multipliers for each layer:

First a relative pile sensitivity, f_s^* is calculated from the setup factors f_s

$$f_s^* = (1 - 1/f_s)/(1 - 1/f_{s,max}) \quad (4.10)$$

In the provided example using the same sand and clay setup factors, we get a sensitivity of 0.333 meaning that the sand has 1/3 of the clays sensitivity since it only loses 20% compared to the clays 60%

With the relative pile sensitivity GRLWEAP can estimate the reduction factor, f_{RD}

$$f_{RD} = (1 - f_s^* + f_s^* \times G/L) \quad (4.11)$$

Hence and using the above formulation, the applied reduction factors to the sand layer would be 0.8, 0.9 and 1 as opposed to the reduction factors of 0.4 0.7 and 1 seen in the clay layer

So far only shaft resistance setup was considered. However, it is also possible to vary the end bearing as the shaft resistance is varied at a particular analysis depth. For example, the pile is driven through clay into a silty sand. Since the silty sand layer is very dense, it has the potential to build up negative pore water pressures and therefore high end bearing values during driving, say 50% higher than the long term value. When driving is over, the pore water pressures dissipate and then the toe capacity goes back to the value which is known from static calculations. For example, if the pile is driven through clay (set-up factor 2.5 in the example) into a very dense, fine sand and silt, it may be reasonable to perform an analysis with shaft gain/loss factors of 0.4 and 1.0 and respective toe gain/loss factors of 1.5 and 1.0. The resulting two analyses at each depth would consider the temporary dynamic (expected) and the long-term static (restrike or worst-case driveability) situation,

respectively. This option can also be interesting to use when analysing a pile that was driven with some degree of plug.

4.11.1. VARIABLE SET-UP

In the previous paragraphs a relationship for complete gain or total loss of set-up were described. On the other hand, if the setup period is interrupted by renewed driving or if the loss of capacity due to driving is interrupted due to a new setup period, then capacity values, either gains or losses, will as a result commence from an intermediate value, illustration provided in Figure 4.10. As it can be seen the driving was stopped before the SRD could reach its full residual value. Likewise, the waiting time was not enough to reach the long term resistance. When driving resumes, naturally the resistance will start to decay due to the influence of friction fatigue. To model this behaviour in GRLWEAP the capacity reduction factor as a function of setup time and as a function of the driving distance was established. Regarding the setup time, the reduction was thought to behave in a logarithmic manner, whilst for the loss of resistance due to pile driving a simple relationship is established between SRD and the driven distance.

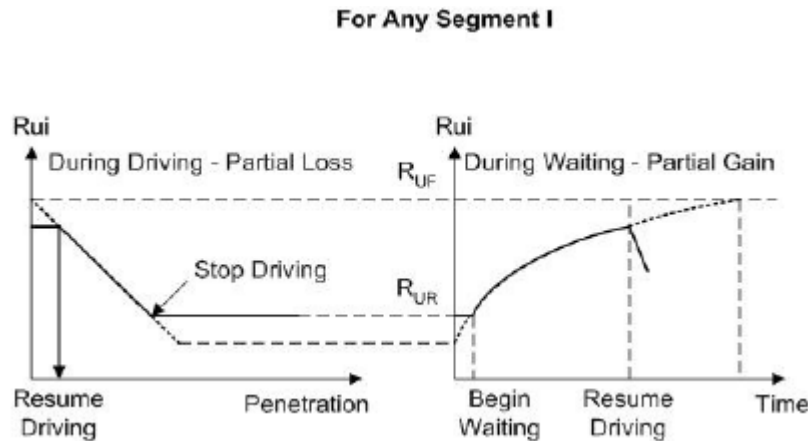


Figure 4.10 - Capacity vs Penetration or Time for Incomplete Gain/Loss

To conclude, it is important to reinforce the fact that GRLWEAP works with the setup Factor, f_s which is used to describe the characteristic setup or regain of static soil resistance which happens after driving for each soil layer; it is a soil property. Gain/Loss Factor, G/L specifies the relative level of static soil resistance change that will be analysed for all soil layers. It is an analysis option. In other words, the set-up factor is based on site specific knowledge and, in conjunction with the resistance Gain/Loss factors, determines for each soil layer the soil resistance to driving. Again, G/L factor is a means to analyse the evolution of the SRD and it is not a soil property.

- $G/L = G/L_{min}$ Full loss of setup (SRD)
- $G/L_{min} \leq G/L \leq 1$ Partial loss of setup
- $G/L = 1$ No loss of setup (LTSR)

4.12. FRICTION FATIGUE METHOD

Based on the approach given by Heerema (1980), GRLWEAP offers a second way to calculate the SRD, in offshore conditions. The following method is known as “Friction Fatigue Method”. It combines some features of the standard approach with those proposed by Heerema (1980). A familiar assumption is made regarding the pile driving. It is stated that driving causes a minimal loss of shaft resistance near the toe, but a considerably higher resistance loss is encountered closer to the seabed, where the pile shaft has already done much more work, on the soil between the pile toe and a certain distance. This distance is to be mentioned as limit distance, or sometimes referred as, limit length; it refers to the driving distance needed for soil to reach the SRD value from the LTSR.

Overall whilst the pile is being driven deeper it will experience a bigger loss until it has reached a point of total loss, meaning it has now reach the SRD baseline. Below, Figure 4.11 is provided in order to better grasp the friction fatigue concept used in GRLWEAP. It is seen that the exponential degradation of friction is clearly similar to the Heerema proposal. The main factors that control the shape of degradation are the limit length L_{Li} , shape factor f_0 and the undegraded distance $f_i L_{Li}$.

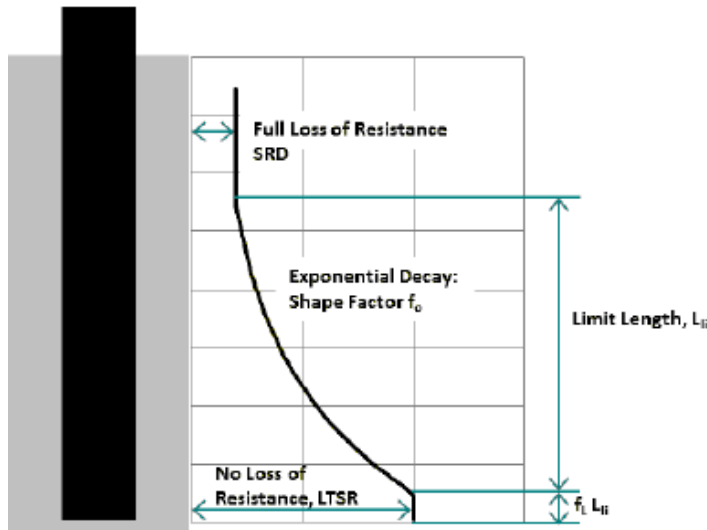


Figure 4.11 – GRLWEAP Friction Fatigue Option

The program calculates the resistance to driving by applying a reduction factor in the long term resistance, in a similar way to what was already described:

$$SRD = LTSR_i \times f_{fi} \quad (4.12)$$

Again LTSR refers to the long term static resistance to driving and is determined by means of a static approach, the sub letter i denotes a given pile segment. f_{fi} , represents the friction fatigue factor and it can be defined by the following formulations:

$$f_{fi} = 1/f_{si} \quad \text{For} \quad Z \geq (1 + f_L)L_{Li} \quad (4.13)$$

$$f_{fi} = 1 \quad \text{For} \quad Z \leq f_L L_{li} \quad (4.14)$$

$$f_{fi} = 1/f_s - f_0 + X_1 e^{(\alpha_i z^*)} \quad \text{For} \quad f_L L_{li} \leq Z \leq (1 + f_L L_{li}) L_{li} \quad (4.15)$$

As stated above the reduction factor represents the loss of resistance that can be encountered. Equation 4.10 can be described as a fully reduced resistance for a pile segment above the limit length plus the bottom section, which is usually expected near the seabed. Equation 4.11 expresses full resistance over a distance equal to $f_L \times L_{li}$, and is close to the pile tip, where most of the shearing was lost. Regarding Equation 4.12 it represents a partial loss of resistance, naturally it is applied in between the previously described equations. The factor f_0 defines the shape of the exponential function and should be the same for all soil layers. However the limit distance, L_{li} and setup factors, f_s can and should be made different depending on the different soil layers.

In GRLWEAP the following limits are applied:

$$f_0 \leq 0,9 \times (1/f_s) \quad (4.16)$$

$$f_0 \geq 0,001 \quad (4.17)$$

To further reinforce this idea, an example provided by the GRLWEAP manual will be given. Consider a pile with 100 m in length driven into a single uniform sand layer where the LTSR of each 1 m long pile segment is 500 kN. The exponent for degradation factor f_0 was set to 0.01; this factor controls the shape of the degradation curve. Also consider the undegraded distance fraction, f_L equal to zero, meaning that the loss of friction will commence immediately (if we had given f_L a value different than zero we would have a friction value constant and equal to the LTSR over a distance of f_L times the limit length, L_l). Figure 4.12 shows the calculated resistance distribution for pile toe depths of 25, 50 and 75 meters for a given 1m pile segment.

It can be seen a different distribution for each penetration depth. In all penetrations the resistance will exponentially decrease from a high resistance value to a reduced resistance. Note that the plotted results show for the 1m depth bellow mudline a small resistance above the fully reduced resistance for the penetration depth of 25m. This is due to the L_l which is greater than the actual penetration meaning that, when the pile tip is at 25m it will never reach the fully reduce resistance, it will need to penetrate deeper so it can reach the final residual value. For the deeper penetrations of 50m and 75m the resistance at 1m is exactly equal to the fully reduced value, i.e. SRD. It should be noted however that the bottom segment resistance is never exactly equal to the full resistance value i.e. LTSR, since the finite pile segment length of 1 m above the bottom resistance is already reduced. In other words, each pile segment is 1 m long and as f_L was set to 0 the degradation will start from the pile toe, however what is seen in Figure 4.12 corresponds to the centre of the final segment and as such the resistance will already be reduced.

On a final point, both the standard approach and the friction fatigue option are usually utilized to calculate the reduced resistance value, by degrading the LTSR by means of a degradation factor. The main difference between the two is related to the ratio and shape of degradation. In section 6 a more detailed comparison of the two methods will be provided, when applied to the analysed design proposals which consider the influence of friction fatigue.

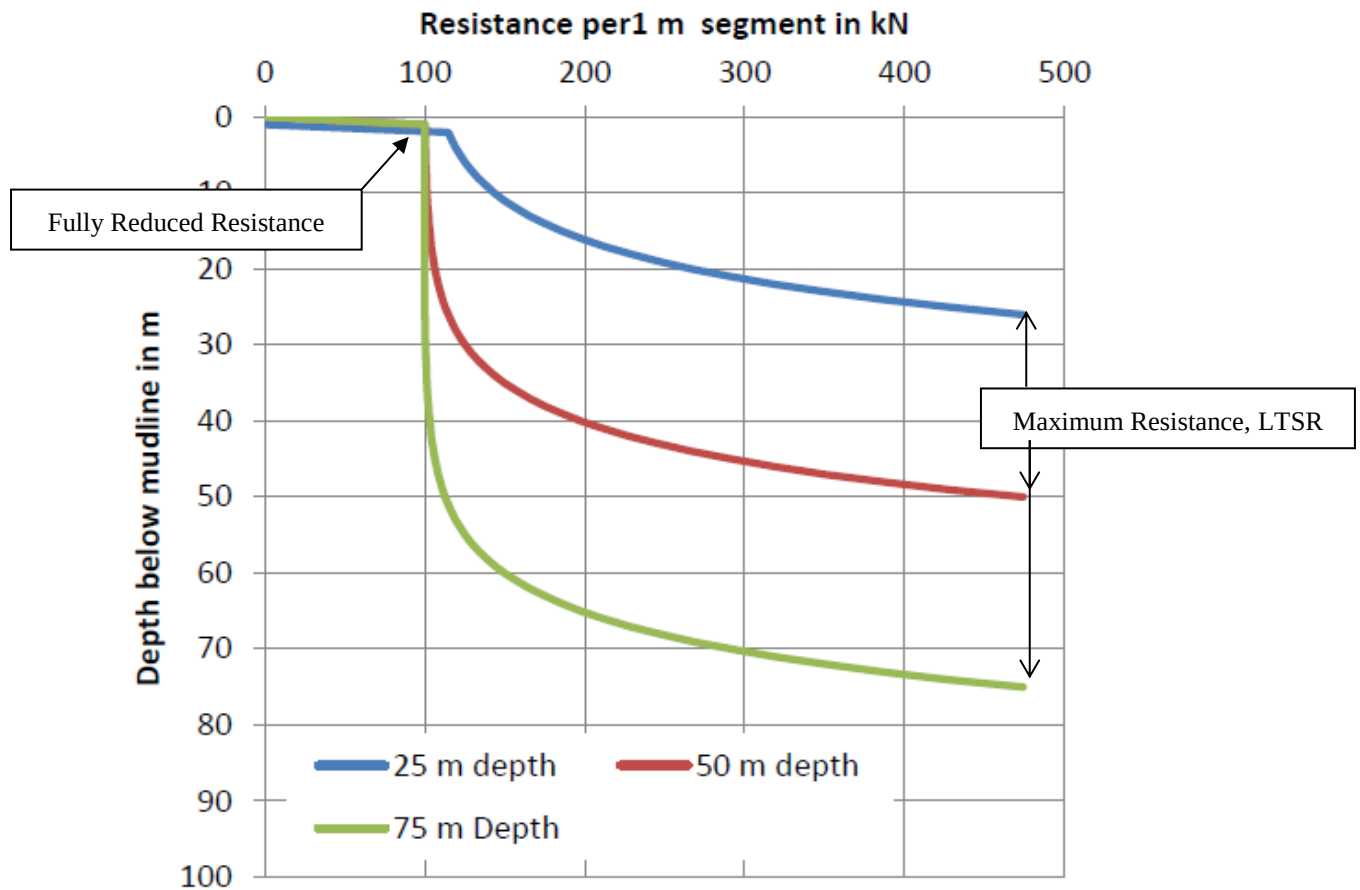


Figure 4.12 – Resistance for different penetration depths; as seen in GRLWEAP 2010 background manual

5

CPT AND SOIL PARAMETERIZATION

5.1. CONE PENETRATING TEST

Characterization soil methods compose an important part in the design of any offshore foundation, in order to determine the seabed stratigraphy and to estimate soil parameters. In contrast with onshore pilling, offshore foundations present an additional challenge, all of the investigation procedures are carried out in harsh conditions. Although the way that information is processed is essentially the same, both onshore and offshore.

One of the most used exploration method currently used in the geotechnical industry is the CPT (Cone Penetration test) detailed for geotechnical use in Robertson et al (1997) and its enhanced versions such as the piezocone (CPTu) and the seismic (SCPT). All have extensive applications in a wide range of soils. CPT is best used in soft soils, although with modern large pushing equipment and more robust cones, the CPT can be performed in almost any soils with the exception of rocks.

The objectives of using CPT as a subsurface investigation are to determine the following topics:

- Nature and sequence of the subsurface strata
- Groundwater conditions
- Physical and mechanical properties of the subsurface stratification

Geotechnical studies are of extreme importance in any offshore project and are usually accomplished in the design stage. The main goal is to access information regarding the soil stratigraphy and its properties. In an offshore project, geotechnical investigation contains sampling and *in situ* testing such as the cone penetration test, CPT as well as some laboratory testing. This investigation method provides a fast and continuous profiling and is known to give reliable and accurate information. Furthermore, it has a large theoretical basis for interpretation. However, CPT does not allow collection of *in situ* soil samples, despite this disadvantage it continues to be one of the most used geotechnical test in the industry. It should be said that although it is not possible to retrieve soil samples during a CPT, it is possible to obtain soil samples using a CPT pushing equipment. Due to the CPT continuous penetrating nature it is recommended to first perform several CPT soundings in order to define soil stratigraphy profile as well as estimating geotechnical parameters, only then should one precede with sampling.

CPT is performed by pushing a cone on the end of a series of rods into the ground at a constant rate whilst continuous measurements of the resistance to penetration of the cone, q_c and surface sleeve friction resistance, f_s are performed. For offshore conditions there is a large variety of pushing equipment available depending on the water depth. In shallow water, floating or jacked-up barges are common, on the other hand, in deep water it is common practice to place the CPT pushing equipment on the seafloor using especially designed subaquatic systems.

The total force acting on the cone, Q_c divided by the projected area of the cone, A_c will give the cone resistance. Side friction is given by the division of the total acting force on the sleeve, Q_s by the surface area, A_s . In 1974 cone penetrometers that could also measure pore pressure were introduced.

The measurement was made possible using a pore pressure filter, located behind the cone. In Figure 5.1 a schematic of a CPTu is provided.

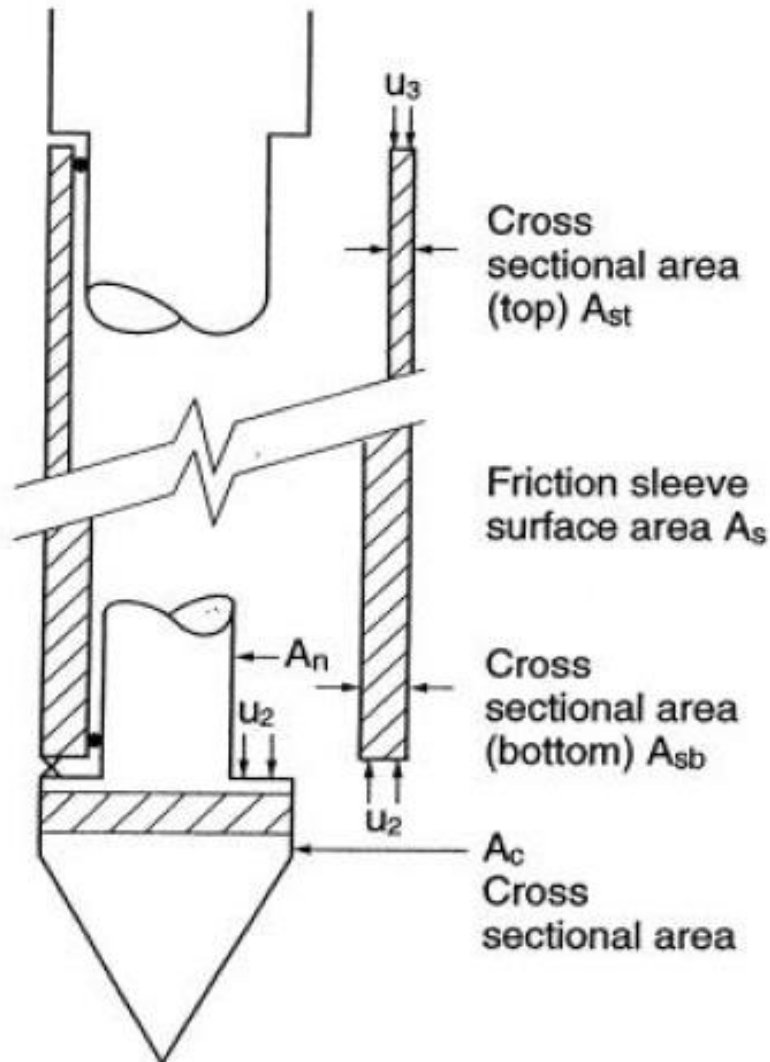


Figure 5.1 – CPTu illustration, Robertson 2015

5.2. CPT INTERPRETATION

Numerous semi empirical correlations, intended for calculating soil properties from the CPT have been developed. Hence, allowing determination of soil stratigraphy. This procedure was done in order to ascertain the soil conditions of the soil surrounding the piles in question. A fully drained cone penetration was considered.

5.2.1. MEASUREMENT CORRECTIONS AND NORMALISED TIP RESISTANCE

Several factors can have a significant impact on the measurements of CPT and thus may influence them. Pore water pressure around the cone, temperature changes, inclination and calibration errors are some of the factors that should be corrected in order to achieve a correct interpretation of the CPT.

The influence of the pore water pressure is due to high hydrostatic pressures applied in the cone due to its geometry. As seen in Figure 5.1 pore water pressures act in the section behind the cone where the filter is located. Thus in tests performed where pore pressures may influence the results, such as offshore, the CPT tip resistance, q_c should be corrected by a parameter representing the net area ratio.

$$q_c = q_t + u_2 \times (1 - a) \quad (5.1)$$

The net area ratio determined from laboratory calibration with a typical value between 0.7 and 0.85 can be assumed equal to the unity in soils where sand is the predominate layer. A similar correction presented in Equation 5.1 should be applied to the sleeve friction, however modern CPT cones have an equal end area friction sleeve reducing the need to apply such correction.

In the offshore industry, where CPT can be carried out in very deep water, cones are sometimes compensated (filled with oil) so that the pressure inside the cone is equal to the hydrostatic water pressure outside the cone. For compensated cones the correction applied to the tip resistance is slightly different from the above, however the difference is not great.

It is also common to normalize the tip resistance and friction ratio so that it can better correlate to other CPT based formulations. Normalized cone penetration resistance expressed in a non-dimensional form and taking account of the in-situ stresses is formulated by:

$$Q_t = \left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \right) \quad (5.2)$$

The term Q_t represents the simple normalization with a stress exponent of 1.0, when taking account for the in situ stresses and variances in the stress with soil type and stress level an exponent n is used, making Equation 5.3:

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{P_{a2}} \right) \left(\frac{P_a}{\sigma'_{v0}} \right)^n \quad (5.3)$$

Normalized friction ratio is calculated in a similar way:

$$F_r(\%) = \left(\frac{f_s}{q_t - \sigma_{v0}} \right) \times 100\% \quad (5.4)$$

Robertson formulated the variation of the stress exponent by:

$$n = 0.381(I_c) + 0.05(\sigma'_{v0}/p_a) - 0.15 \quad (5.5)$$

5.2.2. SOIL BEHAVIOUR TYPE

Normalized cone parameters, Q_t and F_r can be combined into one soil behaviour type index, denoted as I_c . Representing the soil type behaviour. Accurately assessing soil behaviour index leads to an accurate soil stratification and a more accurate representation of the soil reality. Profiles of I_c provide a simple guide to the continuous variation of the soil behaviour type in a given soil profile. Robertson et al (2005) provides normalized CPT behaviour representation, seen in Figure 5.2 and in Figure 5.3.

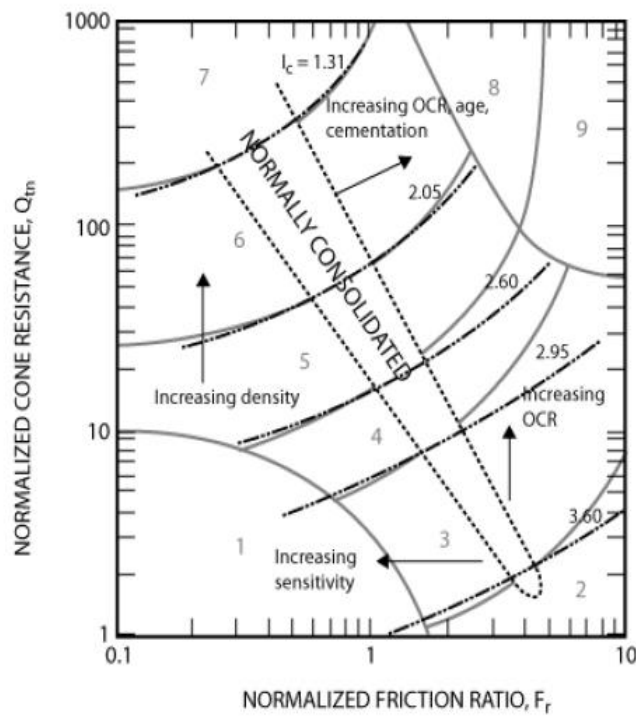


Figure 5.2 – Soil type behaviour; adapted from Guide to Penetration testing 2015

Zone	Soil Behavior Type	I_c
1	<i>Sensitive, fine grained</i>	N/A
2	<i>Organic soils – clay</i>	> 3.6
3	<i>Clays – silty clay to clay</i>	$2.95 - 3.6$
4	<i>Silt mixtures – clayey silt to silty clay</i>	$2.60 - 2.95$
5	<i>Sand mixtures – silty sand to sandy silt</i>	$2.05 - 2.6$
6	<i>Sands – clean sand to silty sand</i>	$1.31 - 2.05$
7	<i>Gravelly sand to dense sand</i>	< 1.31
8	<i>Very stiff sand to clayey sand*</i>	N/A
9	<i>Very stiff, fine grained*</i>	N/A

Figure 5.3 – I_c Table; seen in Guide to Penetration Testing 2015

5.2.3. SOIL UNIT WEIGHT

Soil properties, such as soil unit weight, were calculated using Robertson's unified approach, detailed in Guide to Penetration Testing for Geotechnical Engineers (2005). Soil unit weight is best obtained from relatively undisturbed samples and weighting a known volume of soil. When no sample is provided, total unit weight can be obtained by the following formulae.

$$\frac{\gamma_t}{\gamma_w} = 0.27[\log R_f] + 0.36[\log q_t/p_a] + 1.236 \quad (5.6)$$

5.2.4. FRICTION ANGLE

It controls the shear strength of uncemented grain soils and is usually expressed in terms of a peak secant friction angle, ϕ' . A number of means to calculate the friction angle are available. The method used was a relationship with the equivalent normalized cone resistance, Q_{tn} provided by Kullawy and Mayne (1990)

$$\phi' = 17.6 + 11 \log Q_{tn} \quad (5.7)$$

5.2.5. INTERFACE FRICTION ANGLE

As described in section 3 the described methods require the interface friction angle in order to correctly estimate the resistance to driving. It was recommended using detailed laboratory tests, such as ring shear tests, to obtain the interface friction between the pile and the soil, since an accurate value of the interface friction is vital to obtain the best results. However, no tests were performed to attain δ , and as such the correlation already described in section 3.7.1. was used. For all sites analysed when facing with sands a constant value of 29 degrees was used. Figure 5.4 provides information on how the mean particle size was estimated for sands, whilst Figure 3.4 represents the used correlation. Notice that in Alm and Hamre, the same relation was used. The proposed $\phi' - 5^\circ$ was not used to obtain the

interface friction, since in very dense soils it would lead to a large value of SRD which in turn would severely overestimate the blow count seen in section 7.

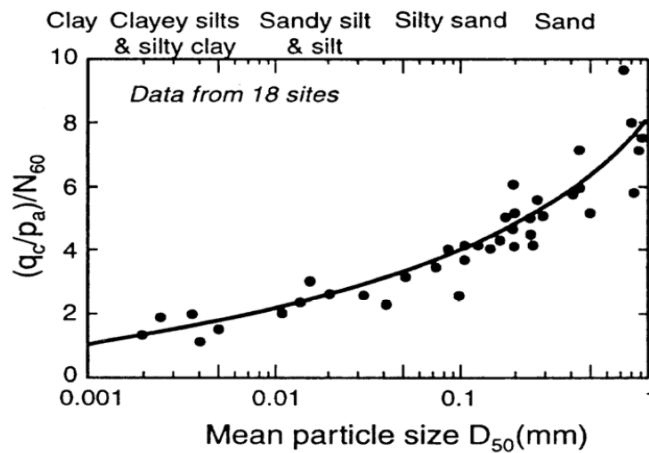


Figure 5.4 - Mean particle size

Note that this correlation was only used in the CPT based methods, since the model proposed by Stevens makes use of their own proposed interface friction angle, such proposal can be seen in Table 5.1

Table 5.1 - Interface friction angle used by Stevens (1982)

Soil type	Interface Friction Angle
Sand	30°
Silty Sand	25°
Sandy Silt	20°
Silt	15°

5.2.6. RELATIVE DENSITY

Relative density was used as an intermediate soil parameter to assess stress-strain-strength characteristics of sands. Figure 5.5 and Equations 5.8 and 5.9 taken from Robertson et al (1997) illustrates the correlation between cone tip resistance and relative density used to estimate the relative density for normally (NC) and overly consolidated sands (OC). C_0 , C_1 and C_2 represent soil constants, and σ'_m represents the mean effective stress. Sand density can be categorized by: loose sands if D_r is between 20% and 60%, dense if D_r ranges from 65% to 85%, for values higher than 85% sands are classified as very dense sands, while in some cases D_r can be larger than 100%.

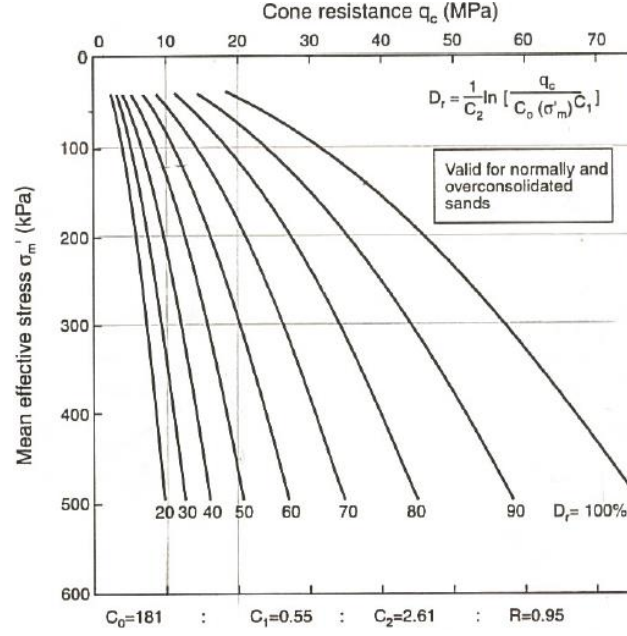


Figure 5.5 - Relative Density

$$D_r = \frac{1}{C_2} \ln \left[\frac{q_c}{C_0 (\sigma'_m) C_1} \right] \quad (5.8)$$

$$\sigma'_m = (\sigma'_{v0} + 2\sigma'_{h0})/3 \quad (5.9)$$

5.3. COHESIVE SOIL PARAMETERS

As already explained, the main focus of this work will be on sands. However some of the analysed locations presented small layers of cohesive soils and as such it is important to correctly assess their characteristics so that the final results can attain a certain degree of accuracy. Hence some of the parameters used in the CPT interpretation will now be detailed.

5.3.1. UNDRAINED SHEAR STRESS

It is impossible to attain a single value of undrained shear stress, since the response of undrained soils depends on the actual direction of loading, soil anisotropy, strain rate and stress history. Usually the value of S_u depends on the design problem and is taken as the average undrained strength. It is highly recommended to define this parameter with some adequate testing, such as vane test for example. However it is also possible to calculate S_u by relating it with the corrected tip resistance.

$$S_u = \frac{q_t - \sigma_v}{N_{kt}} \quad (5.10)$$

N_{kt} Represents a theoretical cone factor usually varying from 10 to 18, in the calculations the average value of 14 was used. The cone factor tends to increase with increasing plasticity whilst decreasing with great soil sensitivities.

5.3.2. SENSITIVITY

Soil sensitivity is an important characteristic of a clay since it defines the ratio of undisturbed peak undrained shear stress to totally remoulded undrained shear stress. CPT tip and sleeve friction are a function of the soil sensitivity, therefore they can be used to estimate it:

$$S_t = \frac{q_t - \sigma_v}{N_{kt}} \times \frac{1}{f_s} \quad (5.11)$$

5.3.3. OVER CONSOLIDATION RATIO

Over consolidation ratio (OCR) is defined as the ratio of the maximum past effective consolidation stress and the present effective consolidation stress and the present overburden effective stress.

$$OCR = \frac{\sigma'_p}{\sigma'_{v0}} \quad (5.12)$$

However, the above ratio is only viable for mechanically over consolidated soils where the only changes have been the removal of overburden stress. In most soils OCR may represent the ratio of the yield stress, undrained shear stress and the present effective overburden stress and can be related by:

$$S_u/\sigma'_{v0} = [S_u/\sigma'_{v0}]_{NC} OCR^{0.85} \quad (5.13)$$

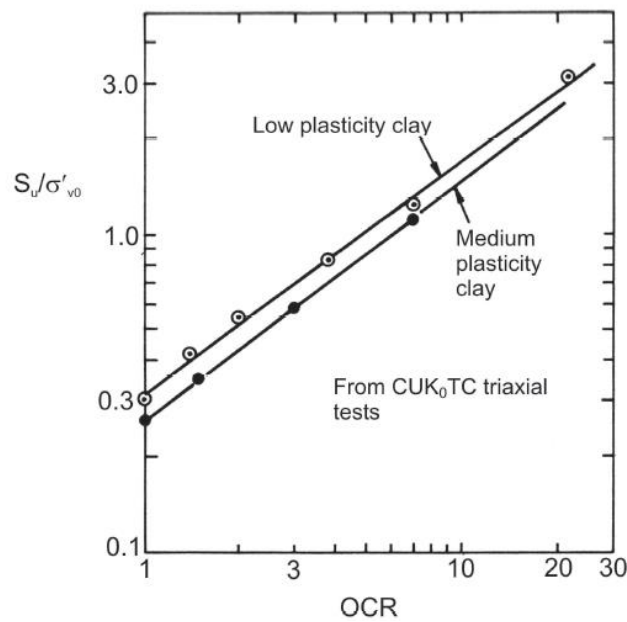


Figure 5.6 - Over consolidation ratio

The ratio $[S_u/\sigma'_{v0}]_{NC}$ falls between 0.25 and 0.35 for a wide variety of soils found in compression tests on normally consolidated samples. Figure 5.6 illustrates the above trend for two clays, with low and high plasticity values. As it can be seen there is not a great disparity between the two clays, and as such the range for $[S_u/\sigma'_{v0}]_{NC}$ appears to be adequate.

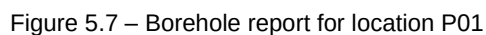
5.4. SITE CONDITIONS

As detailed, the present work is based on an undisclosed location in the North Sea, three locations were analysed, P01, P02 and P03. Based on extensive geotechnical, geological and geophysical logging data from ground investigations, performed by Fugro (a geotechnical company specialized in survey, subsea and geosciences services), it was assessed that the site consists mainly of quaternary granular soils. Figure 5.7 represents the borehole report for the location P01. As it can be seen it is largely composed by fine and medium sands, there is however a small 4m silt layer at around 23m

Regarding P02 and P03, their CPT log reports can be seen in the attached files.

Location P02 as a more diverse stratigraphy, it appears that exists a very dense layer considerably close to the mudline. Further, a large 12m layer of cohesive soil was encountered up until 17m depth. The remaining soil is again composed of clean sands. Lastly site P03, as with the other locations, is composed of clean granular soil, with the exception of a small silty sand layer near the mudline.

Using all the above relations and definitions as well as the CPT borehole report made by Fugro a plot of the cone tip resistance versus depth is presented in Figure 5.7. Note that the CPT tip resistance used in the calculations was simplified and smoothen in order to better understand the complex mechanisms that occur during driving and to simplify calculations. Furthermore, since the site conditions appear to be dominated by sands, no correction was applied to neither CPT cone resistance nor its sleeve resistance meaning that, $q_t = q_c$.



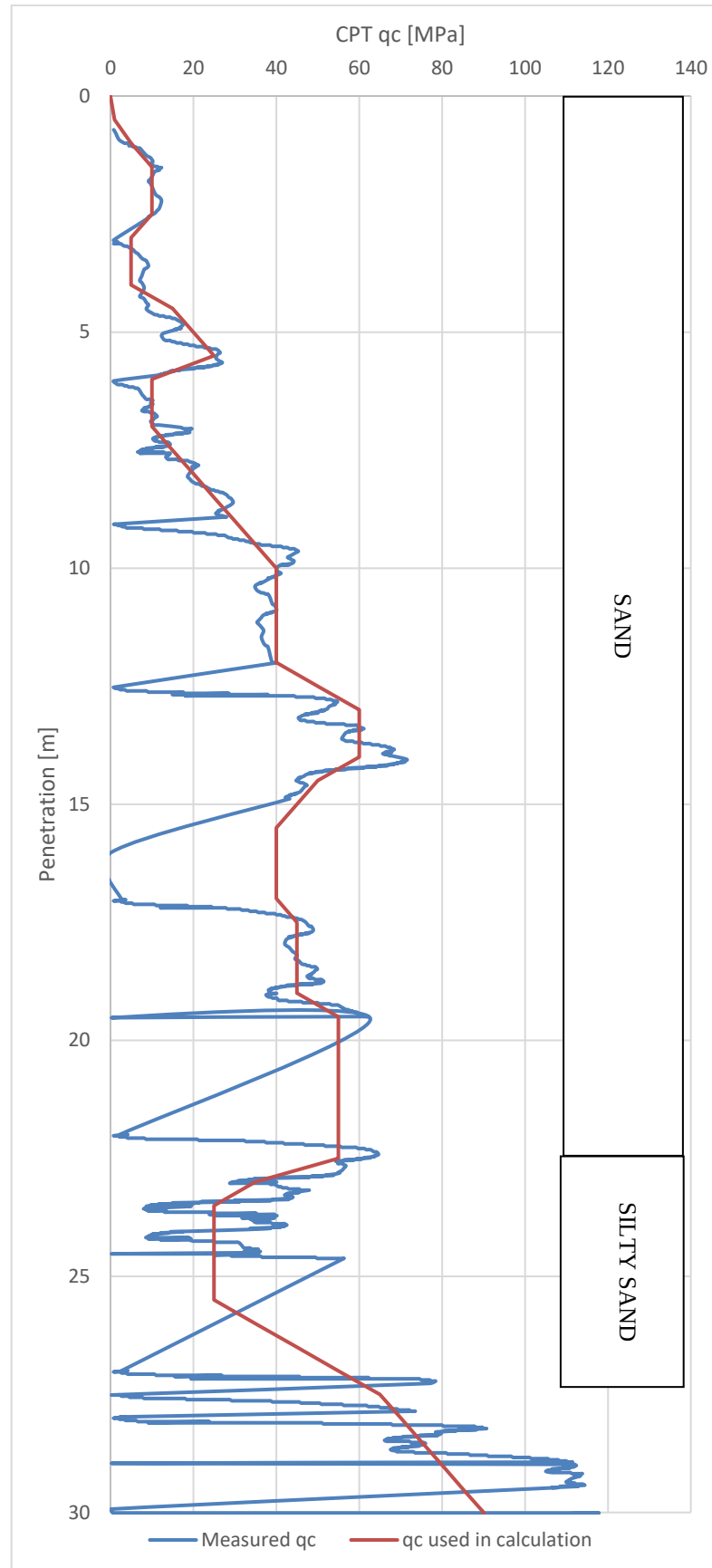


Figure 5.8 – CPT tip resistance; P01

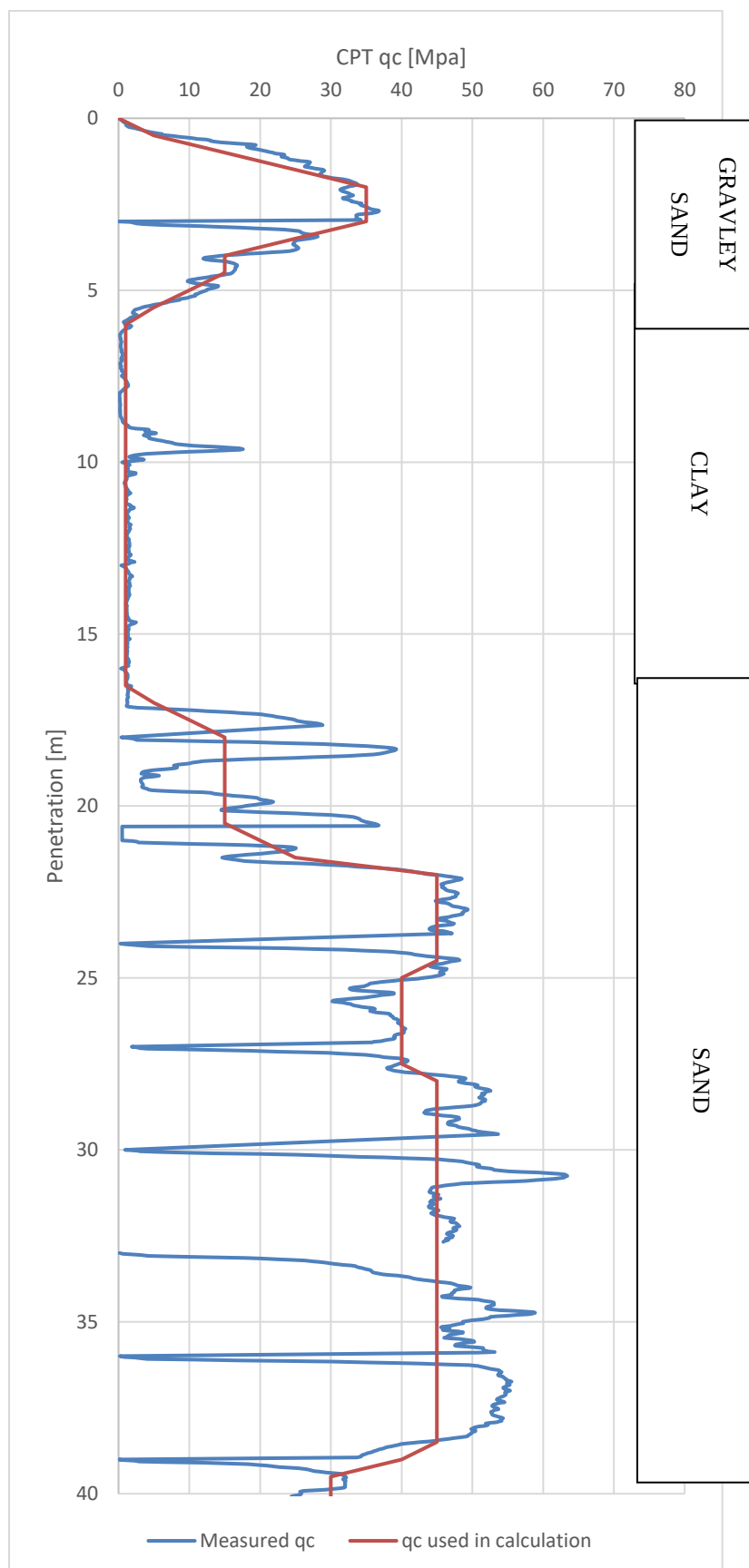


Figure 5.9 – CPT tip resistance; P02

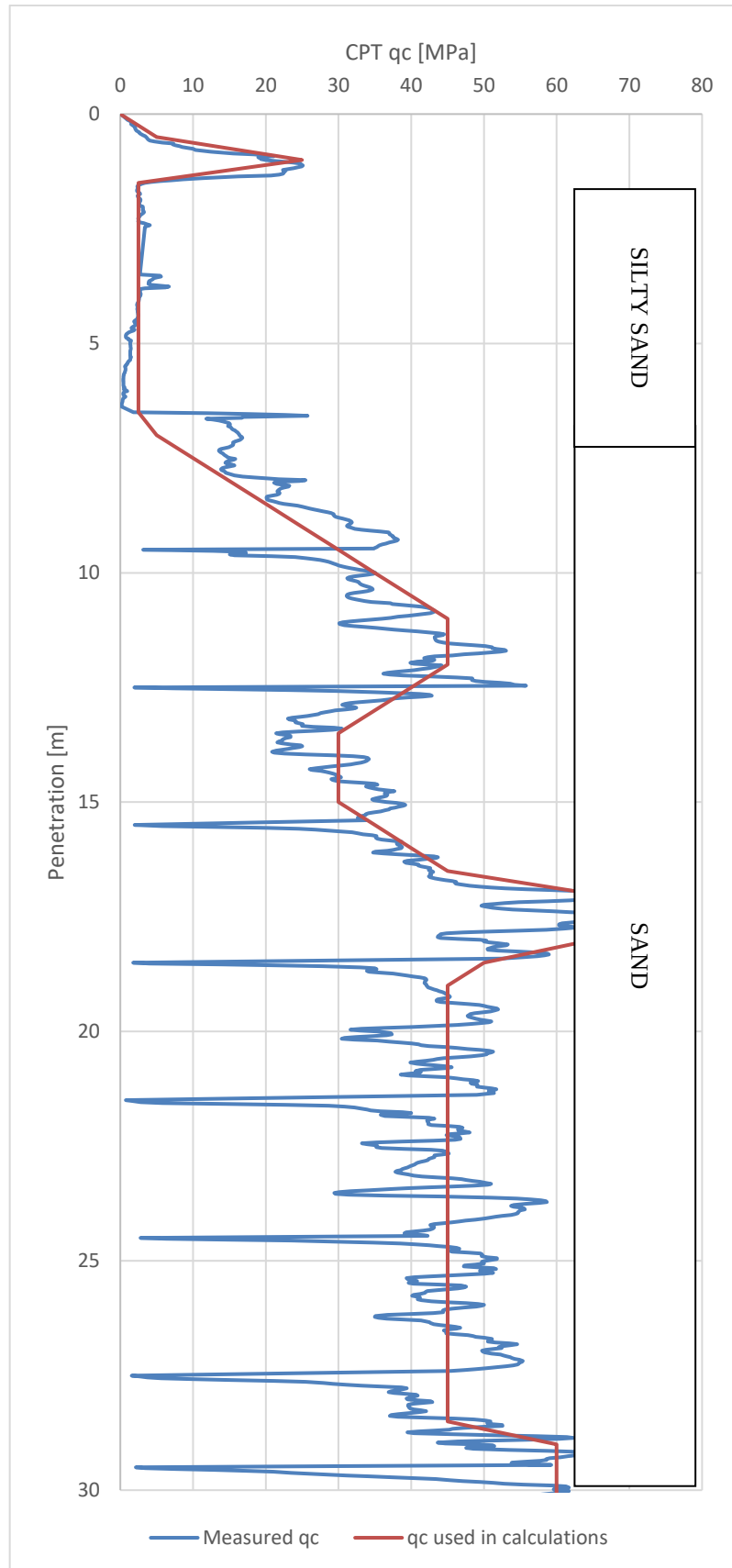


Figure 5.10 – CPT tip resistance; P03

6

SRD CALCULATION

6.1. INTRO

One of the main goals of this paper is to assess if the three main methods described in chapter 3 are still suitable to analyse the SRD and blow count of large diameter tubular piles installed by impact driving, in sands. As discussed earlier all three methods were based on small diameter jacket piles, when compared with the piles used currently, in the offshore industry.

In order to prove if the different approaches are still viable to apply in large offshore piles, there is a need to estimate the SRD given by the literature and, later, on chapter 7, back calculate the number of blows required to drive the pile. This section will present the evaluation of the estimated resistance to driving, SRD given by the literature and applied to GRLWEAP.

6.2. PILE DESCRIPTION

The pile in study is one of the many piles designed by COWI/AS for a project in the North Sea, named "Offshore Wind Farm DanTysk". The pile itself will serve as the foundation of an offshore wind turbine. In order to meet the design criteria, the 62.80 m long, open ended tubular pile, was driven to a final depth of 30 m. The pile presents an outer diameter of 5.94 m and the inner diameter varies from 5.77 m to 5.82 m. The pile is entirely composed of steel. All calculations were made assuming the pile drives in an unplugged manner, with the annulus area of 11083.5 cm^2 . More detail will be given in section 7.2

Table 6.1 – Calculation table

	q_b	f_{sf}	Q_b	Q_s
AH	$0.15 \times q_t (q_t / \sigma'_{v0})^{0.2}$	$f_{s,res} + (f_{s,max} - f_{s,res}) e^{-kh}$	$\frac{\pi (D^2 - D_i^2)}{4} q_b$	$\pi D \int_0^L f_s \cdot dz$
ICP	q_t	$\sigma'_{rf} \times \tan(\delta_f)$	$\frac{\pi (D^2 - D_i^2)}{4} q_b$	$\pi D \int_0^L f_s \cdot dz$
STV	$\sigma'_v \times N_q$	$K \times \sigma'_v \times \tan \delta$	$\frac{\pi (D^2 - D_i^2)}{4} q_b$	$\overline{f_{s,ins}} \times A_{ins} + \overline{f_{s,out}} \times A_{out}$

6.3. ESTIMATION OF SRD

As described earlier, SRD represents the soil resistance to driving, and can be computed as the sum of the total shaft friction with the base resistance, while considering ageing, consolidation or fatigue effects. The three methods that are analysed differ on the considerations regarding both the base and shaft resistance, therefore the SRD results will understandably not match. Table 6.1 provides a very brief overview of all designs. In the following paragraphs the results achieved using the three different approaches are shown. Comparison of the diverse methods is also presented.

6.4. COMPARISON OF SRD

Note that only one of the three pile locations was analysed in this section, i.e. P01. For the other two locations similar relations were utilised leading to similar conclusions and as such there was no need to present them in this section. The final results for all location. P01; P02 and P03 will be presented later on section 7. The main focus of this section will be the comparison of the three methods and special attention will be given to aspects such as friction fatigue and setup effects. With the soil properties calculated from the CPT, the SRD can now be estimated using the recommendations proposed in the literature. Table 6.2 provides the final results of calculations made to access the soil resistance for different penetration depths and Figure 6.1 presents its graphical representation.

Since friction changes with the advance of the pile tip it was necessary to calculate resistance values for several different depths. Penetration depths were chosen every time there was a sudden change in the CPT tip resistance and where more accuracy was needed.

Based on these results it is made clear that all three methods give different estimations for the SRD, particularly the ICP and AH approach. It is important to reinforce the idea that the method proposed by Alm and Hamre (2001) offers an estimation of the driving resistance without consideration of any ageing factor, i.e. the theory proposes an SRD in the short term, during driving. Whilst the ICP methodology was based on test piles that were tested at least ten days after driving, allowing ageing of the surrounding soil. Therefore, all estimations in the ICP should aim at predicting the capacities that may be mobilized ten days after driving the pile, i.e. it is best used to predict the long term capacity.

One must also make reference of the fatigue influence on AH and ICP designs. Both methods take into consideration the friction fatigue impact, but in slightly different manners. The ICP method considers friction reduction by relating it to the normalised height, h/R^* . Whilst Alm and Hamre (2001) state that the friction loss is mainly related, to the distance from a given soil layer already passed through during driving to the pile tip.

Regarding the STV model, no consideration of the friction fatigue or setup effects were made. Further studies are needed to assess if indeed this design can still be applied, as most driveability methods considers the degradation of friction along the pile shaft as a widespread fact. However, it is still interesting to compare the different methods and analyse their main differences.

In Figure 6.2 and 6.3 unit shaft resistance, as calculated by the AH and ICP procedures was plotted for two different penetrations. Whereas in Figure 6.4 illustration, on how both theories consider the friction degradation, is provided.

It is observed, in Figures 6.2 and 6.3 that the shaft friction distribution based on a pile tip depth of 32 m will underpredict pile shaft friction when the pile tip is at for example 24m depth. This is a clear evidence of the phenomenon, already described in chapter 3, known as friction fatigue.

Table 6.2 – SRD results

AH		ICP		STV	
Penetration (m)	SRD (MN)	Penetration (m)	SRD (MN)	Penetration (m)	SRD (MN)
0.5	0.06	0.5	1.2	0.5	0.1
5.5	18.73	5.5	33.6	5.5	4.5
7.5	13.07	7.5	25.4	7.5	7.2
8.0	16.83	8.0	32.2	8.0	8.0
10.0	35.02	10.0	62.9	10.0	11.6
12.5	48.69	12.5	87.9	12.5	17.2
14.0	61.69	14.0	111.2	14.0	21.0
16.0	51.82	16.0	100.3	16.0	26.7
19.0	60.02	19.0	121.4	19.0	36.5
19.5	67.74	19.5	135.7	19.5	38.3
22.5	76.72	22.5	154.7	22.5	49.9
24.0	56.68	24.0	126.13	24.0	48.43
26.5	67.07	26.5	155.08	26.5	63.96
27.0	74.99	27.0	168.97	27.0	65.72
32.0	129.29	32.0	252.07	32.0	83.27

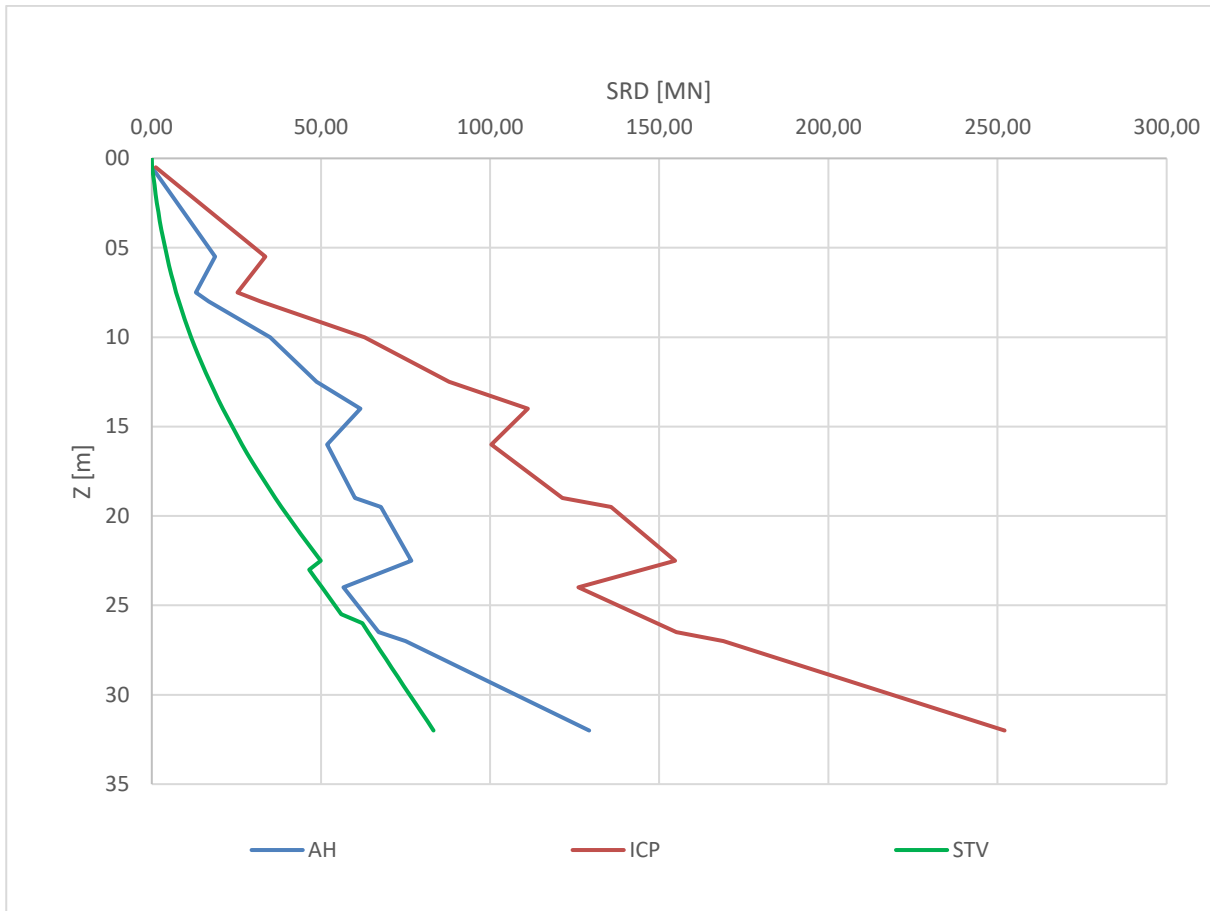


Figure 6.1 - Graphical Representation of estimated SRD evolution

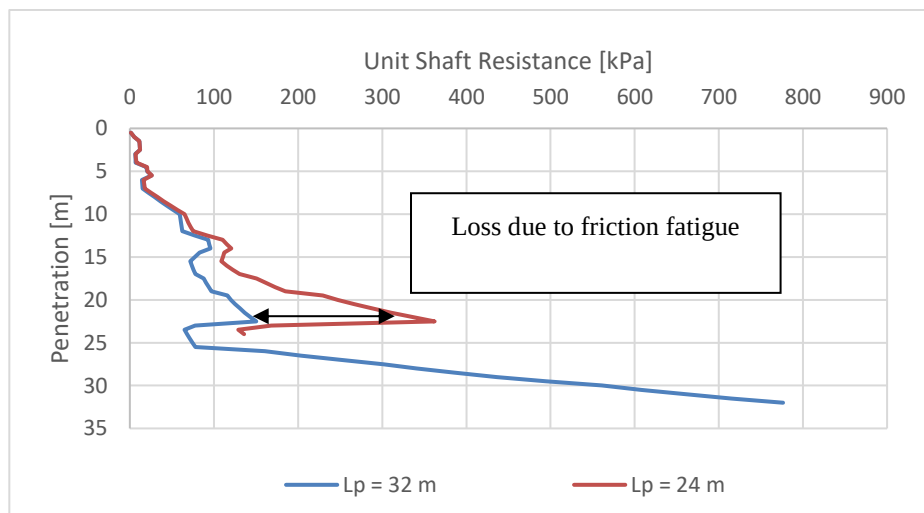


Figure 6.2 – Calculated unit skin friction at 22m and at 30m; AH

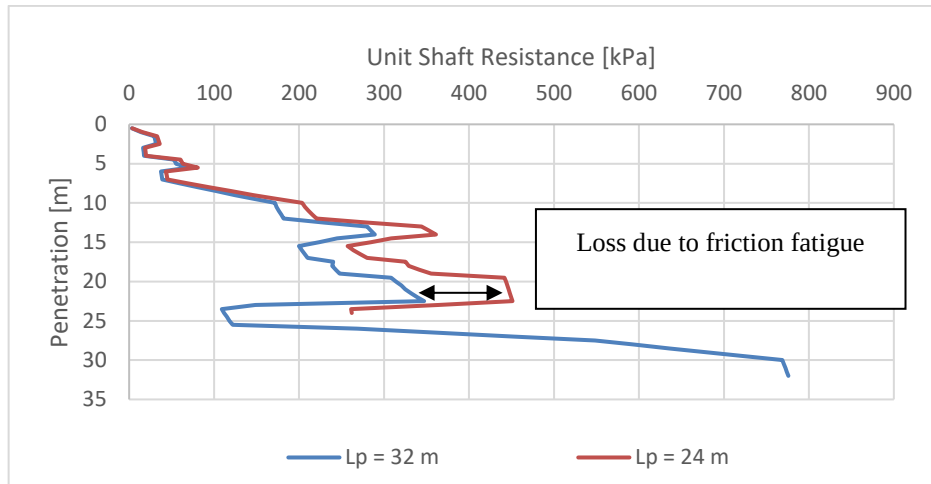


Figure 6.3 – Calculated unit skin friction at 22.5m and at 30m; ICP

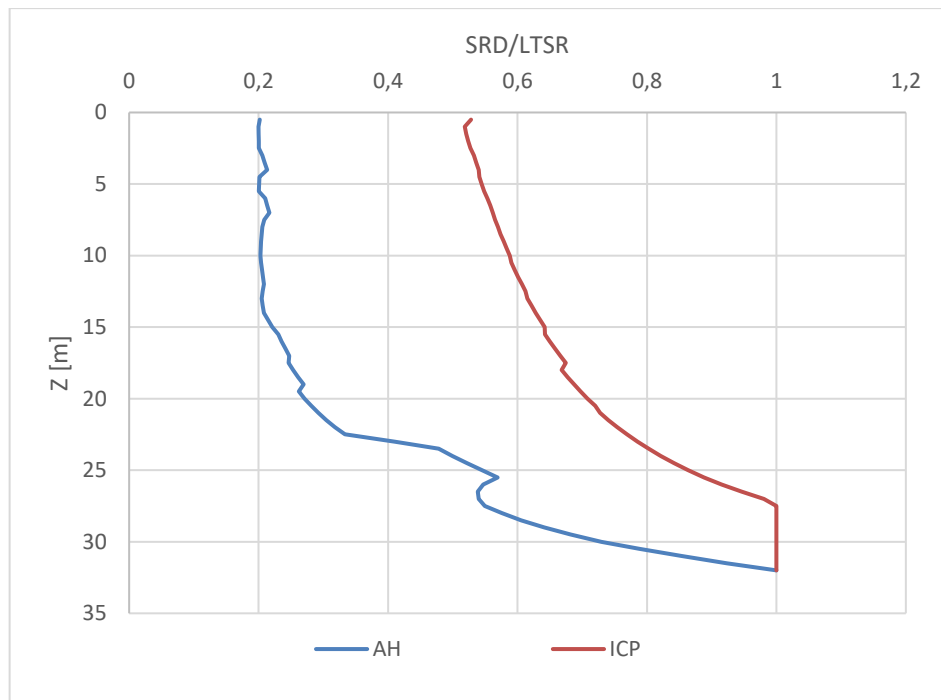


Figure 6.4 – Rate of Degradation

Furthermore, by analysing Figure 6.4 it is evident that both manners consider the degradation of friction in different manners. Alm and Hamre method considers a more severe degradation whereas the imperial college theory proposes a more light degradation. It is also noticeable that ICP considers a non-degraded region (corresponding to the region where $h/R^* < 8$ and as such the lower limit of $h/R^* = 8$ should be used) on the other hand, AH proposes an instantaneous reduction of friction.

Given the importance of this subject and for further illustration on the matter let us consider a similar example already given in section 4 and consider the effects of friction fatigue in both Alm and Hamre and the Imperial College theory. Suppose that a uniform pile was driven in a homogenous layer of sand. Let us consider a 5.94m diameter pile similar to the one analysed in this section, with the same

length of 62.8m, driven to the same target depth of 32m bellow the mudline, however for simplification it was assumed that internal diameter remained constant, resulting in a uniform pile. Using the two design proposals unit shaft resistance was calculated, which lead to the calculation of the resistance/.5m, the results were plotted in Figure 6.5

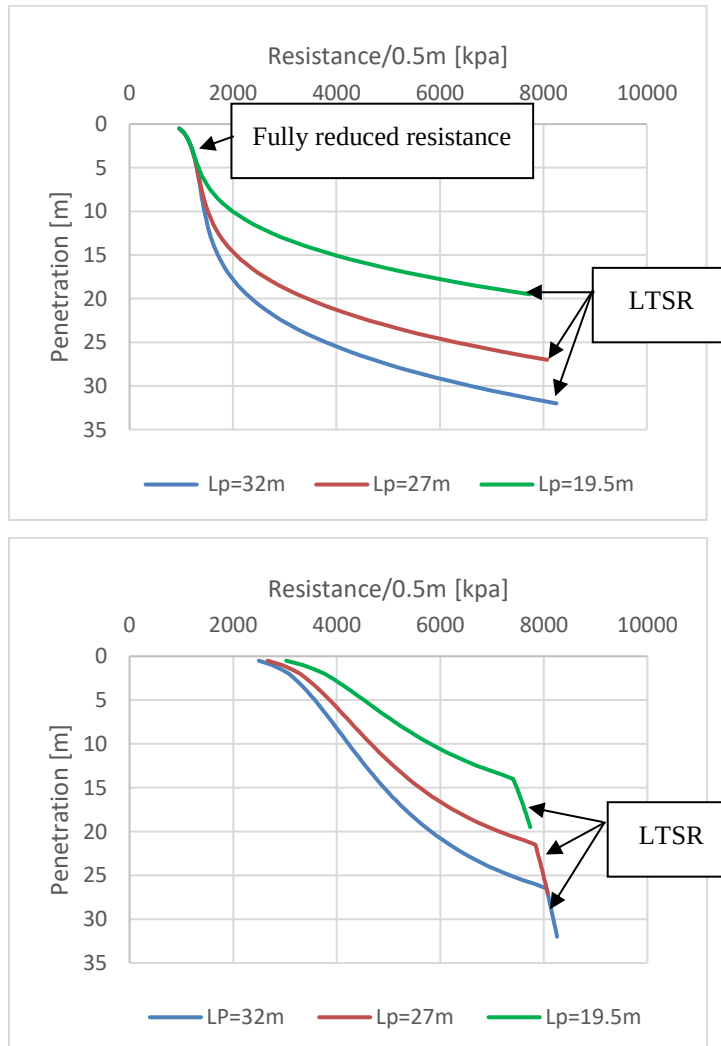


Figure 6.5 – Resistance Distribution for AH and ICP respectively

Figure 6.5 presents for different penetration depths the encountered resistance. Parallel to Figure 6.3 it is possible to see more clearly, that as the pile tip advances the distribution of resistance will suffer changes, the resistance to driving at 15m when the pile tip is at 30m is not the same as when the pile tip is at 27 or at 19.5m, it is in fact lesser than for the other penetration depths. It is possible to conclude that resistance is much higher at the pile tip and it will decrease as it moves away from the tip. Again, this is effect is due to the influence of friction fatigue. Moreover, it is also possible to see in the AH resistance distribution (similar to the example in Figure 4.12) that the resistance at around 5m is always the same for all three penetrations. As explained the resistance at this depth is totally reduced and cannot be any lower. On the other hand, the resistance is maximum at the pile tip and is referred to LTSR (long term static resistance). Interestingly the same is not seen in the ICP distribution, it is not

possible to see for the different penetration methods the same fully reduced resistance value, this can be explained based on the fact that both theories consider friction in different manners. In the ICP methodology friction degradation is highly sensitive to the relative pile tip depth h , and as it is known that the radial effective stresses developed at any given depth sharply decline as the pile tip is driven past that depth level. Hence the reduced resistance value was not yet achieved, in the ICP design methodology it is said that the fully reduced value is usually obtained for driving distances equal or greater than ten times the pile diameter. Lastly, if the resistance distribution is not the same for different penetration depths, then for each penetration there will be a different rate on how resistance evolves, such rate can be seen in Figure 6.6. On a final note, the resistance distribution seen in AH is lower than the distribution seen in the ICP except near the pile tip where the resistance is at its maximum, LTSR.

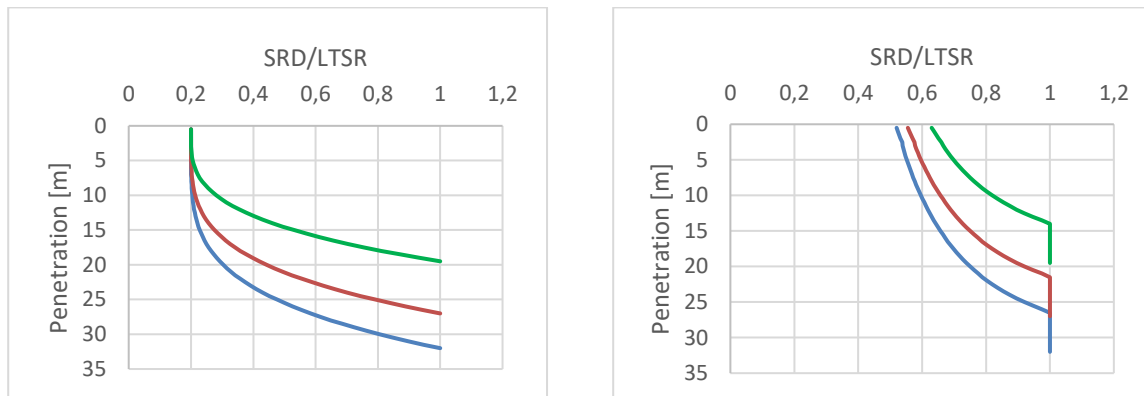


Figure 6.6 – Rate of degradation for AH and ICP respectively

Note that the AH rate of degradation considers that the lowest resistance value is 0.2 times the maximum resistance value for all depths. This ratio matches the one seen in Equation (3.12), i.e. the AH proposal.

Additionally the sudden “break” seen in the ICP resistance distribution is again due to the Jardine (2005) interpretation of the fatigue influence, as explained this effect is considered in the h/R^* ratio. When the lower limit $h/R^* = 8$ is in effect the resistance in ICP is non-degraded, meaning that in that range no friction fatigue is applied and the resistance is maximum and equal to LTSR.

In the above paragraphs the consideration of the friction fatigue and its implications in AH and ICP were analysed. However, comparison between the two when no degradation is considered should also be detailed, i.e. when the SRD has had time to develop into the long term resistance.

Figure 6.7 illustrates the evolution of the fully non-degraded resistance for both AH and ICP. Immediately it is possible to see that the evolution of resistance to driving for all penetrations depths is exactly the same, further supporting the fact that friction fatigue plays a major role in the distribution of resistance. Also note that the values of resistance for the pile tip (LTSR) seen in Figure 6.5 are the same for the resistance values analysed in Figure 6.7 thus showing that indeed the resistance is maximum at the pile tip. Furthermore, resistance is greater in all depths when compared to the evolution of the reduced resistance (except at the pile tip where it is equal). Hence, driving in this situation would be immensely more difficult and it would most likely lead to refusal. Lastly note that the full shaft resistance/0.5m is almost the same for both AH and ICP, again this can be explained due to the fact that the two methods are based on the same principle. In order to calculate the skin resistance

both methods rely on the Mohr Coulomb failure criteria (Equation 3.11 and Equation 3.21 both follow the same principle), further since the resistance is equal to the LTSR and as such the friction fatigue plays no role it is possible to understand that why the resistance is identical for both methods.

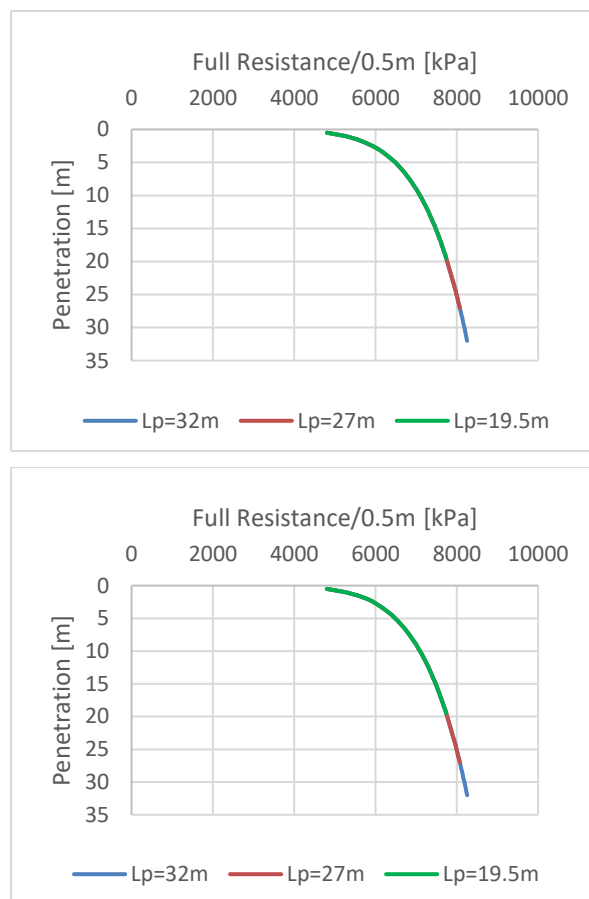


Figure 6.7 – Full resistance evolution for AH and ICP respectively

Now let us analyse the intermediate situation, driving with fully reduced resistance and due to soil setup, driving with no reduction in resistance. This is particularly interesting for piles that were continuously driven but for some reason the driving process was interrupted and in that time soil setup occurred, mobilising the reduced resistance to the LTSR

Analysing Figure 6.8 it is clear that the “interruption” in driving was performed at around 20m, the “waiting time” was long enough so that full setup could be achieved. Thus when the pile tip is at 19.5m no alteration in resistance is seen, since the interruption has yet to occur, and so the driving is performed as seen in Figure 6.5. On the other hand, when the pile tip is at 27m or at 32m, driving was “stopped” when the pile reached 20m, thus allowing for setup to occur, leading to an increase in resistance. When driving recommences it will be immensely more difficult to recommence the process due to the large increase in driving resistance, however as driving progresses the LTSR will naturally return to the reduced resistance evolution seen in Figure 6.5. It is important to mention the fact that in this example the transformation from the reduced resistance to LTSR and vice versa occurs almost instantaneously (meaning that both the limit distance and setup time were extremely low leading to an

instant gain and subsequently loss in friction), evidently this might not occur in a real driving scenario. Figure 6.9 represents a more realistic scenario where the resistance is gradually increased after the stop and will gradually reduce as driving commences.

It is therefore recommend, in actual pile driveability projects, that the driving process should be made with the least amount of interruptions, of course such recommendation is not always possible to follow, since it is often necessary to stop the process due to changes in the hammer, malfunctions, workers schedule and so on.

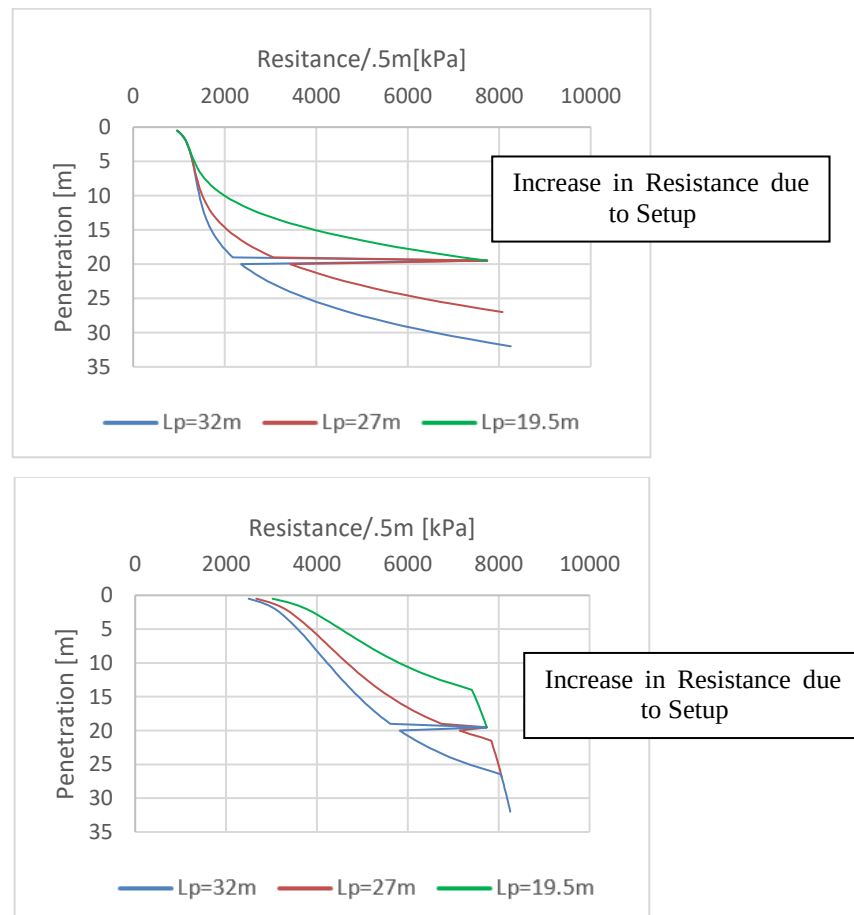


Figure 6.8 Increase due to setup for Ah and ICP respectively

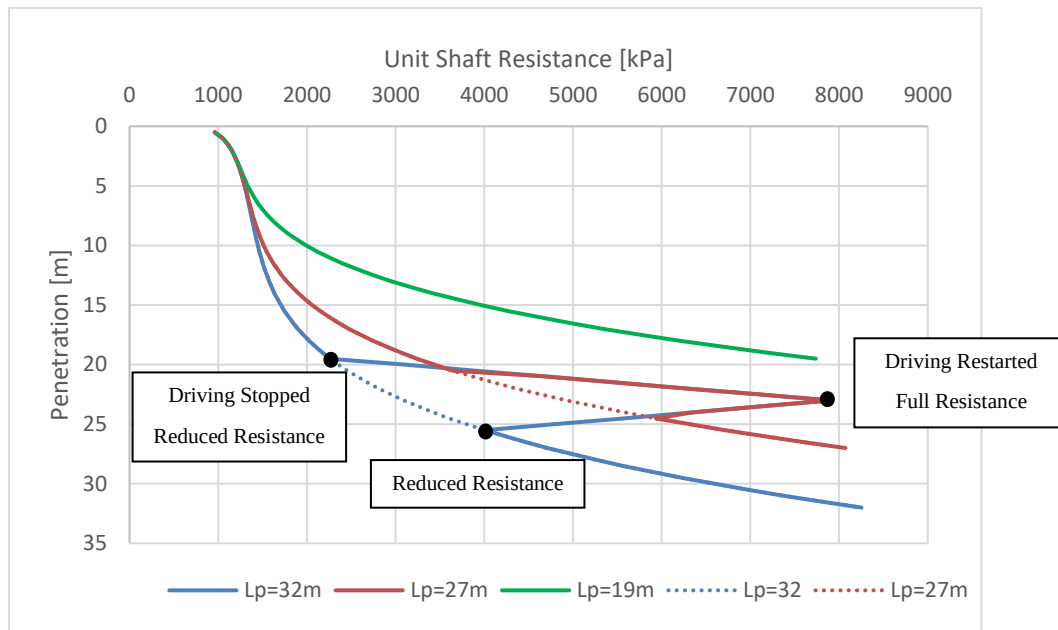


Figure 6.9 Increase due to setup for Ah and ICP respectively

Most importantly, note that the amount of resistance gained for both methods is not the same, in fact it is lesser in ICP than it is for AH. This is explained, based on the fact that ICP already considers a degree of setup in their formulations, since their experiments were conducted in piles that were allowed to age for at least 10 days and as such some degree of setup will have had time to develop. Hence, since the formulation used in ICP already considers an increase in resistance it is not difficult to understand why the effect of the interruption does not produce the same effects as in AH. If it starts with more resistance understandably it will not take such a high increment in resistance to achieve the LTSR.

This presents one of the major differences between the two methods and could explain the differences seen in Table 6.2

Hopefully now the effects of friction fatigue are now made clearer and it is possible to see that driving a pile when resistance is fully reduced is much easier than driving a pile when considering a non-reduced resistance. The consideration of friction fatigue in AH and ICP and how these predict the required number of blows could be immensely interesting to use in the offshore industry, since it could mean the selection of an hammer that might lead to an adequate and cost effective driving, instead of an hammer that would vastly over perform resulting in an overpriced driving.

However the manner in which both methods take into consideration the friction fatigue does not explain, on its own why the final SRD results presents such a great difference. It is therefore concluded that the main issue that explains the huge discrepancy in the SRD results is the consideration of setup effects in the ICP method. As already explained in chapter 3 the consideration of soil setup implies a significant increase in soil capacity, therefore the large value of SRD obtained when using ICP formulations is to be expected.

This method is best suited for piles submitted to a static loading. On the other hand AH is best suited to analyse pile driveability. It is therefore difficult to clearly compare the two methods as they have different goals during the design stage.

6.5. SET UP EFFECTS

It would be interesting to relate the two when removing the consideration of the setup effects in the ICP theory. Figure 6.9 reproduced from Gavin et al 2015, shows a compilation of several case histories of setup effects for piles driven in sandy soils. It was proven by Skov and Denver (1988) that the increase in pile capacity varies with the logarithm of time after driving. Formulation is provided in Equation 6.1 It was also proven that it has the same pattern as other viscous behaviour such as creep deformation or stress relaxation.

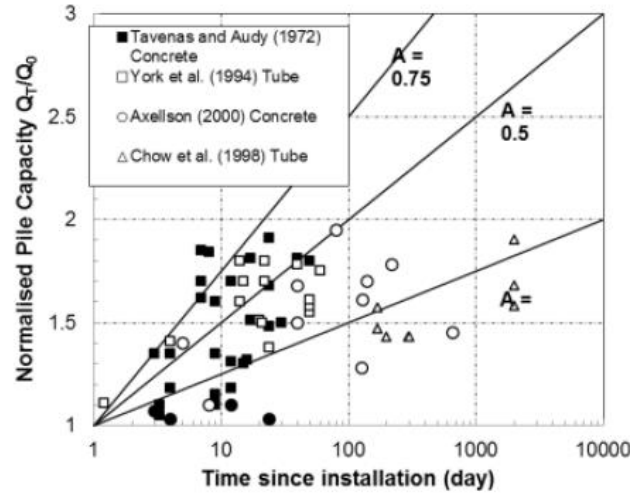


Figure 6.9. – Logarithmic relation of pile shaft capacity with time

$$\frac{Q_t}{Q_0} = 1 + A \times \log \frac{t}{t_0} \quad (6.1)$$

Where:

Q_t – Capacity at time t

Q_0 – Reference capacity at time t_0

A – Dimensionless setup factor

t – Time after driving

t_0 – Initial reference time

Despite the large increase seen in the tests sites, it is seen in the above figure that the gains in pile capacity at a given time are highly variable. For instance Tavernas and Audy (1972) measured ratios of Q_t/Q_0 between 1.1 and 1.9 for the ninth day. Such inconsistency results in a wide range for the setup factor A . Lim and Lehane (2015) have shown that the value of the dimensionless setup factor, A is critically dependent on the reference time and that the quantification of setup is better attained by normalising capacity with respect to a reference capacity such as given by a particular design method, for instance, the ICP method.

Using Equation 6.1 and with a recommended growth trendline it is possible to back calculate the normalized pile capacity Q_t/Q_0 . The trendline used was the one proposed by Chow et al. (1996), seen in Figure 6.10 meaning that the normalization was achieved with a setup factor of 0.25 and a reference

time of 100 days. Since the relation between the dimensionless setup factor and the reference time was not well known, it was decided to adopt this trendline to calculate the absolutely reduced resistance, more detail will be provided now.

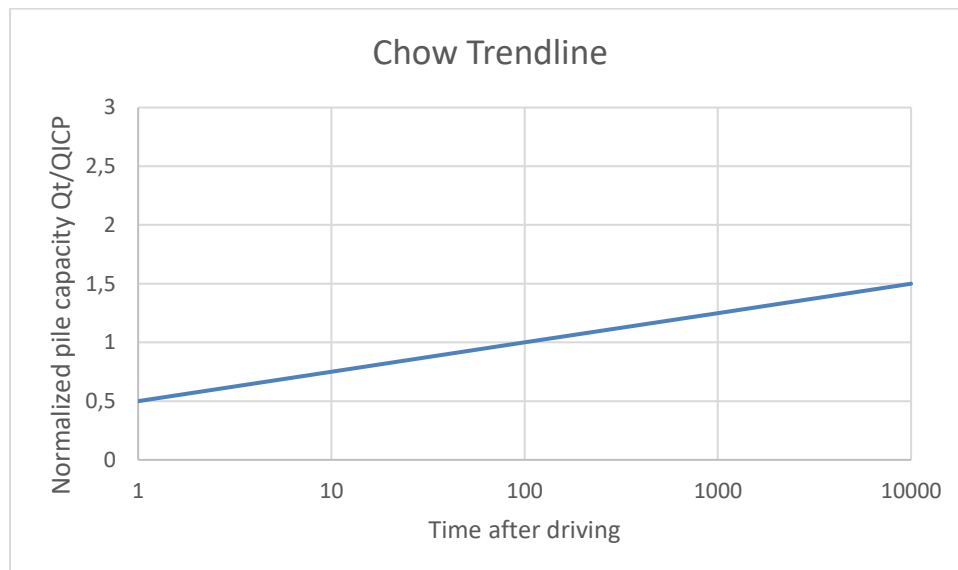


Figure 6.10. – Chow growth trendline

With the following growth evolution, it is possible to calculate the resistance at any time, however the point of this exercise is to back calculate the resistance at the end of driving.

At the time of 1 day after driving, the normalized pile capacity is $Q_{t=1day}/Q_{ICP} = 0.5$. Hence, for a penetration depth of 32m the shaft capacity (at 1 day after driving) is equal to half of the normalized shaft capacity (capacity achieved 10 days after driving). With this calibration the ICP method can be calculated at 1 day after the end of driving instead of 10 days later, meaning that most of the consideration of setup effects is now gone. Still, Chow trendline is in a logarithmic scale and as such 1 day after driving is the closest time span to the end of driving that can be correctly assessed, during that day some increase in capacity is likely to occur however it is negligible and in the following calculations and back analysis 1 day after driving is to be considered the end of driving itself.

It must be said that the trendline seen is normally applied to calculate the gains in capacity with time, instead of the fully reduced value. Yet the time span between 10 and 1 days is small enough and it is believed that the final results are adequate. Also the following relation is best used in sands, since the tests which lead to this trendline were performed in a site mainly compressed by sands.

Finally, it was chosen to use the above trendline instead of the Dunkirk IAC (Intact Ageing Characteristics), growth, seen in Figure 3.6, to calculate the fully reduced resistance due to the good results achieved when comparing it to the Alm and Hamre (2001) approach and in the final blow count prediction. It is important to reinforce the idea that it was intended to calculate the resistance after one day of driving and not to calculate any gains in capacity. Hence the relation of $Q_{t=1day}/Q_{ICP} = 0.5$ seen in Chow trendline was used instead of the ratio $Q_{t=1day}/Q_{ICP} = 0.667$ seen in the Dunkirk IAC trendline. However, it should be said, that if we were to calculate the gains in capacity the IAC trendline should be used instead of Chow's, since it is known that in her test records not all piles were

fresh and thus Chow (1996) trendline will underestimate the gains in resistance and is therefore conservative when calculating the long term axial capacity.

With the setup effect reverted, it is possible to estimate the SRD given by Jardine et al. (2005) and compare it with the AH proposal. Such comparison can be seen in Figure 6.11 and 6.12. It is possible to see that AH and ICP consider unit tip resistance in almost the same way. On a side note, if the relation, $Q_{t=1day}/Q_{ICP} = 0.667$ seen in the IAC trendline were to be used the unit tip resistance would not match as well and the final SRD result would become even larger, which would in turn lead to a not so accurate back calculation of the blow required to drive the pile. Figure 6.13 and 6.14 represent the skin friction and final SRD when using said relation.

As stated using the $Q_{t=1day}/Q_{ICP} = 0.5$ relation means that a good match in the unit skin friction for the AH and ICP is achieved. This should not come as a surprise since both methods are now on the same level, meaning they both consider no setup effects, also the two proposals have been based on identical formulations, the Mohr Coulomb failure criteria. Despite the good match observed in the skin friction, the final SRD results are still somewhat dissimilar. This is due to the consideration of the base resistance, since the both ICP and AH provide different estimations for it and setup effects do not affect the base resistance whatsoever, furthermore it was chosen not to rearrange their considerations regarding the base resistance since it would not be productive analysing two virtually identically methods.

The main differences in the unit base resistance will now be described. AH states that the unit tip resistance will be in the range of about 0.35 to 0.55 times the CPT base resistance. These values are lower than what its expected from current bearing capacities relations. On the contrary, Jardine et al. (1995) proposes that the unit tip resistance should be directly equal to the CPT base resistance, despite the different shapes of the cone and pile annulus. This over predicts the unit resistance. The authors numerical analysis have shown that q_b/q_{CPT} should be around 0.7. The additional estimation in capacity for open piles, by allowing $q_b = q_{CPT}$ compensates the internal friction that is not explicitly considered in the ICP formulation. In a way, internal friction is considered on the base. According to Jardine et al (2005), The additional ‘end bearing’ component is approximately equivalent to applying over the internal pile wall the (maximum) external shaft shear stresses (applying at the tip) up to a level above the toe of 30 to 40 times the wall thicknesses. As a result, AH design will often underestimate SRD results, whilst ICP will tend to over predict.

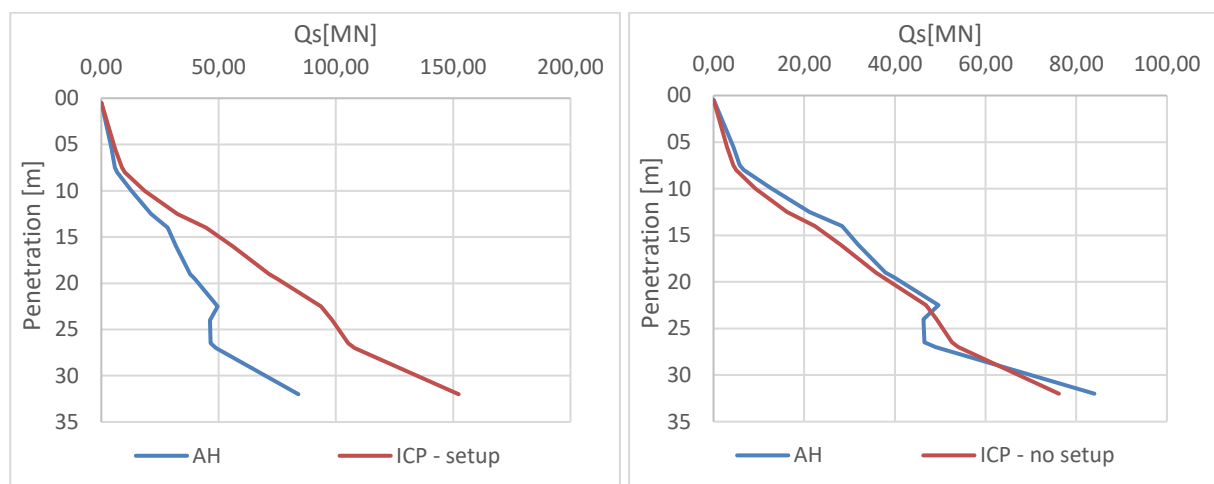


Figure 6.11 – Evolution of Q_s with and without setup effects; with $Q_{t=1day}/Q_{ICP} = 0.5$

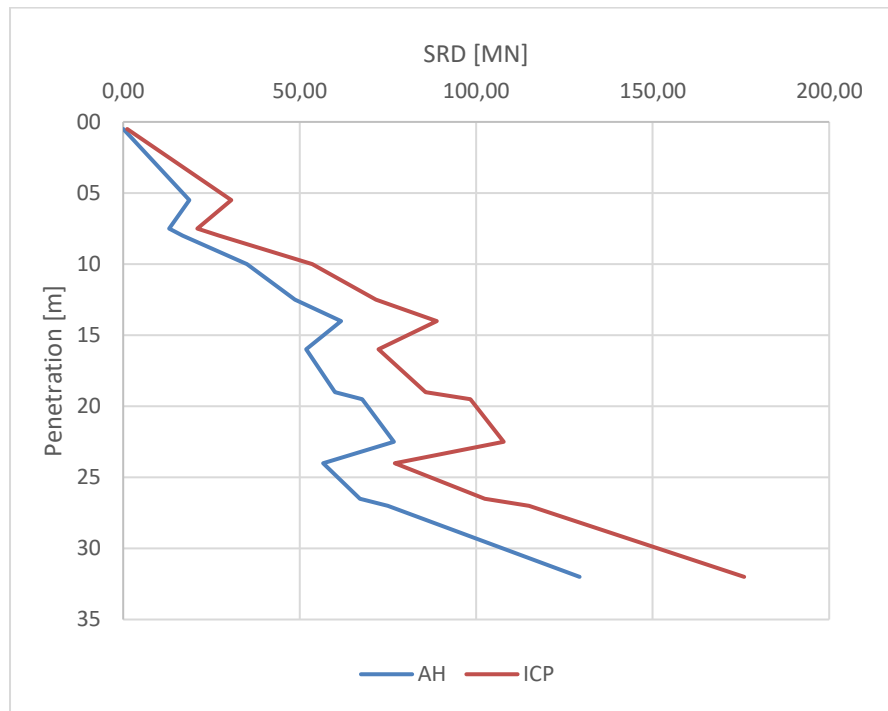


Figure 6.12 – SRD comparison between AH and ICP without setup, with $Q_{t=1day}/Q_{ICP} = 0.5$

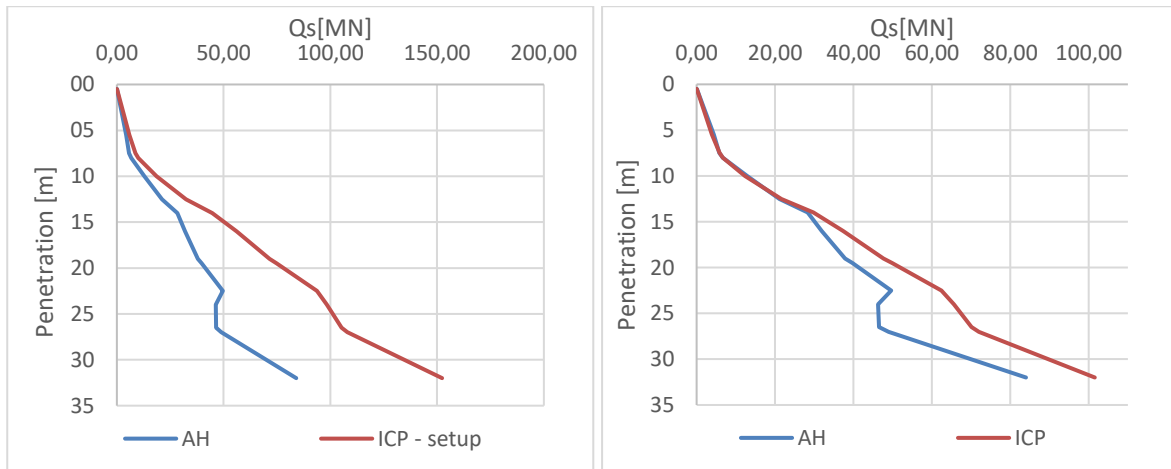


Figure 6.13 Evolution of Qs with and without setup effects; with $Q_{t=1day}/Q_{ICP} = 0.6667$

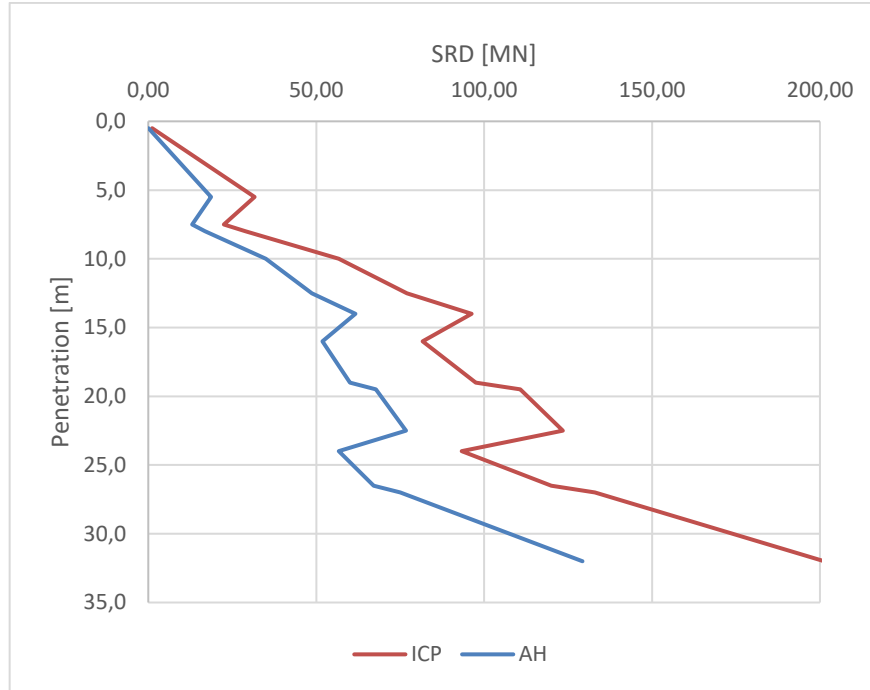


Figure 6.14 – SRD comparison between AH and ICP without setup, with $Q_{t=1day}/Q_{ICP} = 0.6667$

6.6. GRLWEAP CALCULATIONS

GRLWEAP makes use of the wave equation to calculate the SRD. With a given input it is possible to simulate the different methods and all their considerations on the GRLWEAP. However, it is necessary to understand all of the underlying aspects of the design methods (such as the consideration of fatigue) and to manipulate the input accordingly to achieve a correct correlation between the literature and the output from GRLWEAP. Although, it should be noted that it is problematic comparing the previously discussed methodologies within GRLWEAP, since shaft friction distribution changes each time the pile tip is advanced. Again this is why different penetration values were considered. Regarding the base resistance, no difficulty was encountered in any of the methodologies used within GRLWEAP, since friction fatigue does not have a significant effect on it, and it is calculated in a similar way in all methods.

6.6.1. ALM AND HAMRE (AH)

In order to calculate the SRD in GRLWEAP as given by AH, it was necessary to use the offshore option available in the software, using the interpretations already described in chapter 4. With the option turned on the program calculates SRD taking consideration friction fatigue as seen in the Heerema (1980) approach. This presents a problem since, the main point is that GRLWEAP considers the evolution of friction in the same way as Alm and Hamre (2001). Hence it is necessary to simulate the Alm and Hamre approach in the GRLWEAP offshore option.

As described in section 4 GRLWEAP requires the user to input the shape factor, f_0 , which controls the shape of the degradation factor as well as the limit distance, this factor is of extreme importance in this approach since it will also control the rate of degradation. It is also required the input of the fraction of the limit length, f_l .

Some considerations of the friction fatigue factor. In the AH approach this factor is mainly controlled by a shape factor denoted as k . This value is one of the methods Alm and Hamre (2001) used to relate their proposal to CPT (as well as the reformulation of the earth lateral stress coefficient K), thus this fatigue shape factor is heavily dependent on the CPT cone resistance and as such will vary with depth and soil type, as the name implies it controls the ratio of the degradation in friction.

However, as explained the two main factors that control the way that friction is considered in GRLWEAP, are the limit distance and the shape factor. In order to match the friction fatigue factor of the two procedures both the limit distance and shape factor have to be correlated to the AH degradation, k . Again, limit distance refers to the driving distance needed for soil setup to be lost, i.e. the distance required so that LTSR can be fully reduced and it is recommended to vary it with depth, so that a more robust correlation can be achieved.

To sum up, in order to match the Alm and Hamre (2001) degradation in GRLWEAP there is a need to relate the described factors in a correlation system.

The correlation procedure was developed with an initial deduction of the two factors, f_0 and L_l used in a simple scenario, a homogenous sand layer. The friction fatigue factor f_{fi} was calculated using the formulations seen in chapter 4 (Equation 4.10; 4.11; and 4.12) thus, by varying the limit distance with depth based on the surrounding soil and speculating on the shape factor value in an iteration process, a correlation between the degradation given by AH and GRLWEAP can be attained. Note that an empirical relation between the limit distance and the shape factor provided in the literature was not used. This would prove a difficult and time consuming process, as such the iterative process based on trial and error was adopted to achieve the final match between the degradation seen in Alm and Hamre (2001) and GRLWEAP. It was discovered that when dealing with a simple sand layer GRLWEAP friction fatigue curve was matched to the AH degradation curve for a shape factor $f_0 = 0.01$ and for a limit distance of around 20m with a small increment of 0.15m for every 0.5m driven. Figure 6.15 provides the attained relation.

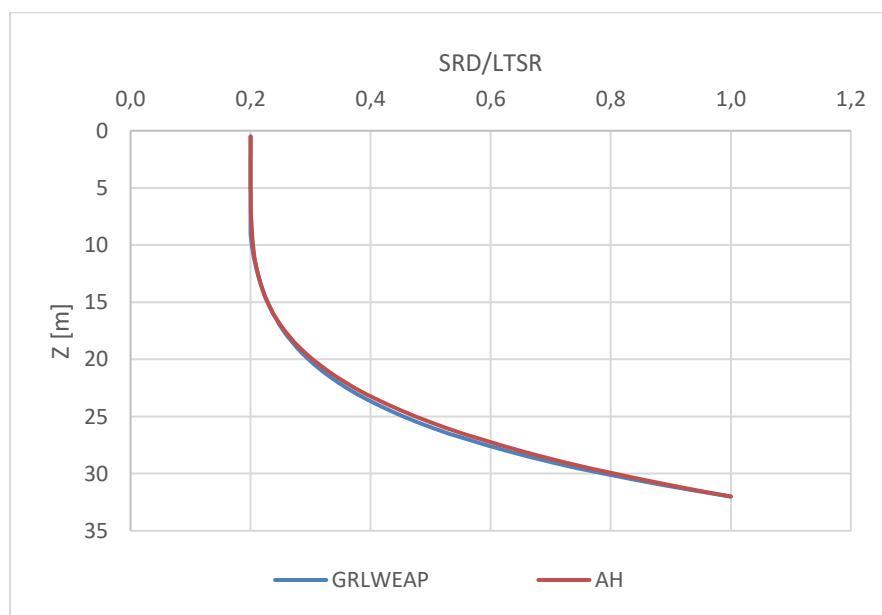


Figure 6.15 – Friction fatigue factor

It is possible to see that the reduction in friction commences immediately after the tip, this concerns the fraction of limit length. Evidently this factor should be made equal to 0, since it was already seen that AH considers an immediate reduction in friction, starting from the pile tip.

It is important to refer the fact that the setup factors used in the offshore option are largely different from the ones used in standard analysis, that being said, the best setup factor for sands was found to be 5 (instead of the recommended 1.2 value). Therefore the gain loss factor in a homogeneous sand layer should be $1/5$.

Having understood the underlying mechanisms in the correlation process it later was applied in the analysed location, P01. Figure 6.16 represents the relation obtained. A good match was obtained meaning that GRLWEAP will now consider the evolution of friction in the same way that it was calculated in AH. In this location a silt layer was encountered and as such the degradation curve will not be the simple curve seen in the previously figure. A higher setup factor accompanied by a higher limit distance were used to match the resistance distribution in such layer resulting in the “discrepancy” seen at around 23m to 28m.

With the friction fatigue factor used in GRLWEAP now correctly calibrated with the AH design, it is possible to perform the analysis. However special care should be given in the consideration of the unit side friction. Alm and Hamre consider that Equation 3.10 to be the total internal and external shaft friction. It was recommended to apply the friction only on the outside or to reduce by 50% the total resistance and apply it to both internal and external friction. Mathematically, this is the same as considering the full friction is only acting on the outside. Therefore, the user must model this assumption on the program. This can be done by only considering the outer perimeter over the entire length of the pile. If for example there was a need to model both internal and external shaft friction, then the perimeter should be made equal to the sum of the internal and external perimeter

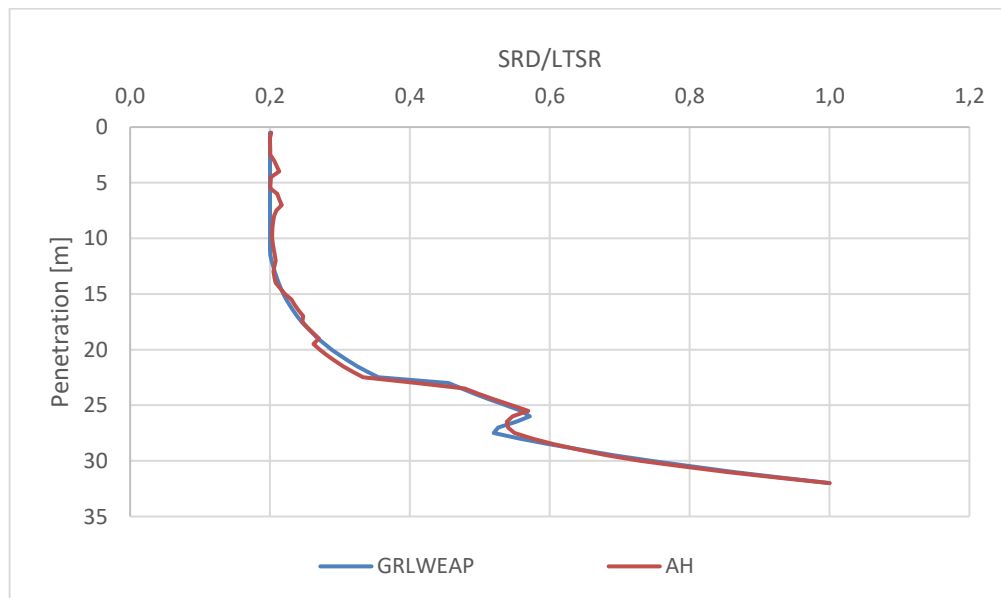


Figure 6.16 – Friction Fatigue Factor: Correlated

The consideration of internal and external friction is of great value, in order to consider plugged or core behaviour. It was already discussed that in large diameter piles a coring behaviour is to be expected, thus such conduct was assumed among all calculations and back analysis. Evidently the user must input the undegraded shaft resistance since the program will degraded it using the offshore option. Gain loss factor was made equal to the highest soil setup factor, $f_{fi} = 6$ in this case.

Figure 6.17 shows the final results obtained when comparing the AH analytical results with the GRLWEAP computed results.

By analysing the calculated resistance distribution, a good match can be seen between AH and GRLWEAP. The small differences between the software and the literature are probably due to the consideration of friction degradation not being exactly equal to the one proposed in AH, as well as the relation between Ll and k not being perfect. Also only a certain amount of different penetrations were analysed, to achieve better results a higher number of penetrations depths should be considered, despite this, the relation attained is more than adequate to proceed with the calculations and the back analysis.

As expected soil resistance increases as the pile penetrates deeper. It is also obvious that SRD is heavily dependent on the surrounding soil. With the sudden change of SRD, it is clear that the pile tip as reached a different layer.

It is also possible to compare the evolution of SRD with the CPT tip resistance. A certain proportionality between the two can be appreciated when comparing it to Figure 6.18. It is therefore important that CPT results should be made as accurate as possible, since it will evidently affect the final results.

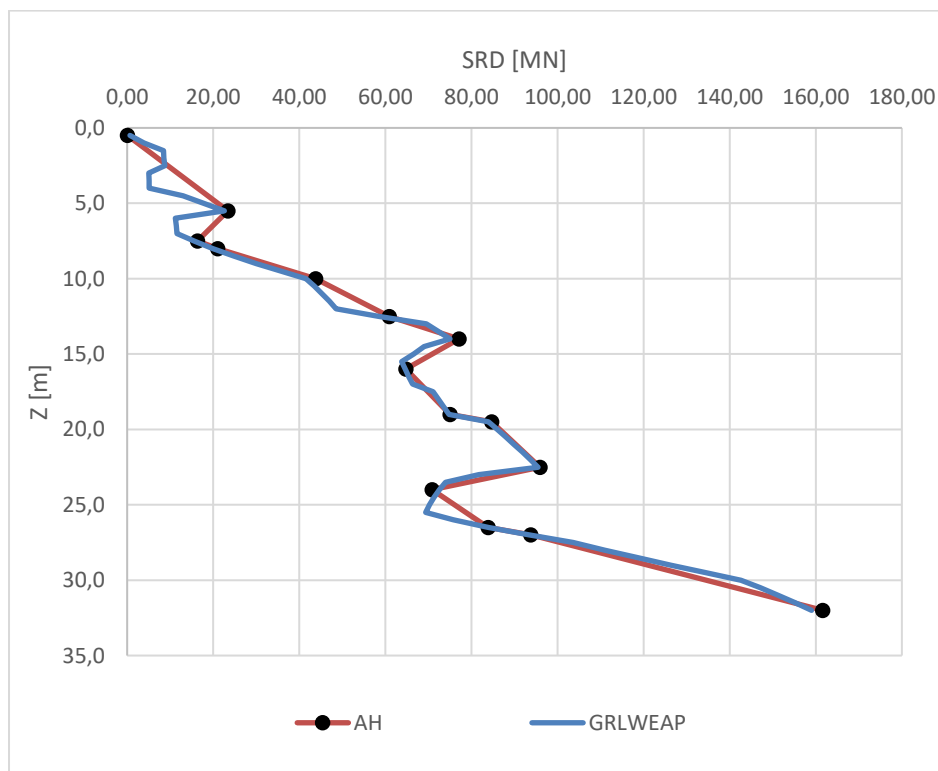


Figure 6.17. – SRD Evolution: AH

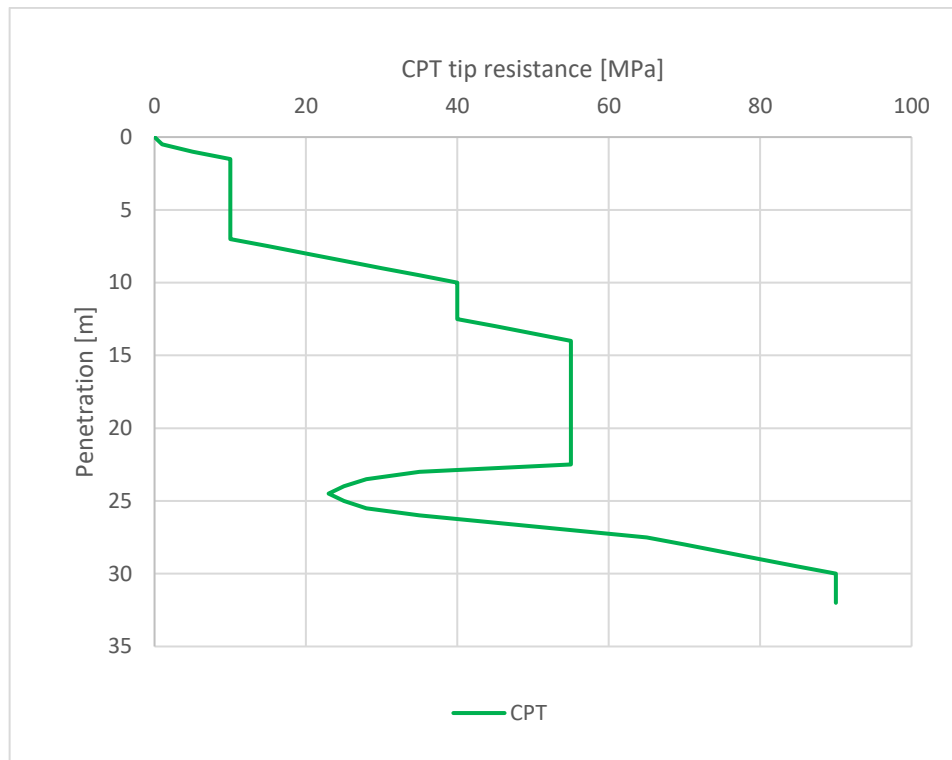


Figure 6.18. – CPT tip resistance; P01

6.7. IMPERIAL COLLEGE (ICP)

As it is known the resistance given by ICP is not the actual reduced resistance at the time of driving and as such, in order to perform the best driveability analysis possible it was decided to investigate the resistance at the end of driving. In GRLWEAP said resistance is reduced by the gain loss factors. Hence by calculating the maximum undegraded unit skin resistance, then inputting it in GRLWEAP and using a standard friction reduction based on setup and gain loss factors, the SRD can be calculated. As seen in Figure 5.7 a small silt layer was encountered, and to better represent said layer a setup factor of 1.5 was used, instead of the standard 1.2 value used in sands, since the factor, since the end of driving resistance (SRD) has to be multiplied to achieve the final long term capacity (LTSR) should be bigger, i.e. usually in silts and in clays, the time to achieve the LTSR is bigger than in sands. In order to actually reduce the resistance the gain loss factors analysed were:

- $G/L=1$
- $G/L=0.66667$

The first gain loss factor will give the resistance with full set up, meaning no reduction in resistance. The second gain loss factor analysed should give the fully reduced resistance, since the maximum setup factor is being considered. However this is not the case, simply because the “reduced resistance” in the ICP is obtained after 10 days of driving, and as it is known, this is not the totally reduced resistance (the totally reduced resistance is achieved immediately after driving). Therefore, there is a need to further degrade this resistance within the GRLWEAP, as such the relation established in section 6.9 was used. Thus, in accordance with the trendline from Chow et al. (1998) by reducing the obtained shaft friction with the $G/L=0.66667$ by 50% the fully reduced resistance at the end of driving is obtained.

It is important to mention the fact that ICP experiments have revealed the inner shaft component to be concentrated near the pile tip, thus these components have been considered to contribute to the base and external resistance, meaning that shaft friction will only occur on the outside, thus it is only necessary to consider the external perimeter in the following calculations. The final results were plotted and can be viewed in Figure 6.19

As with the AH simulation, the results achieved when modelling the ICP proposal in GRLWEAP appear to give a good match, meaning that the assumptions made when inputting the main factors in GRLWEAP are most likely correct. The small differences that can be seen are also related to considerations on the soils actual setup factor and how friction is reduced in the program in contrast with ICP. Finally, it is possible to see the same proportionality between Jardine (2005) methodology and the CPT tip resistance.

The plotted results refer to the ICP modified version where the removal of setup was considered. A comparison of the final SRD results between AH and ICP will be provided in Table 6.3 and in Figure 6.21 the final SRD results are presented. However, here the AH best estimate (upper bound) will be considered, meaning that now SRD is to be considered with an 25% increase, this was found to give better results in the back calculation of the blow count since it accounts for soil variability, according to Alm and Hamre (2001). As it can be seen both theories predict SRD in a very similar manner with ICP estimating the soil resistance in a slighter higher fashion.

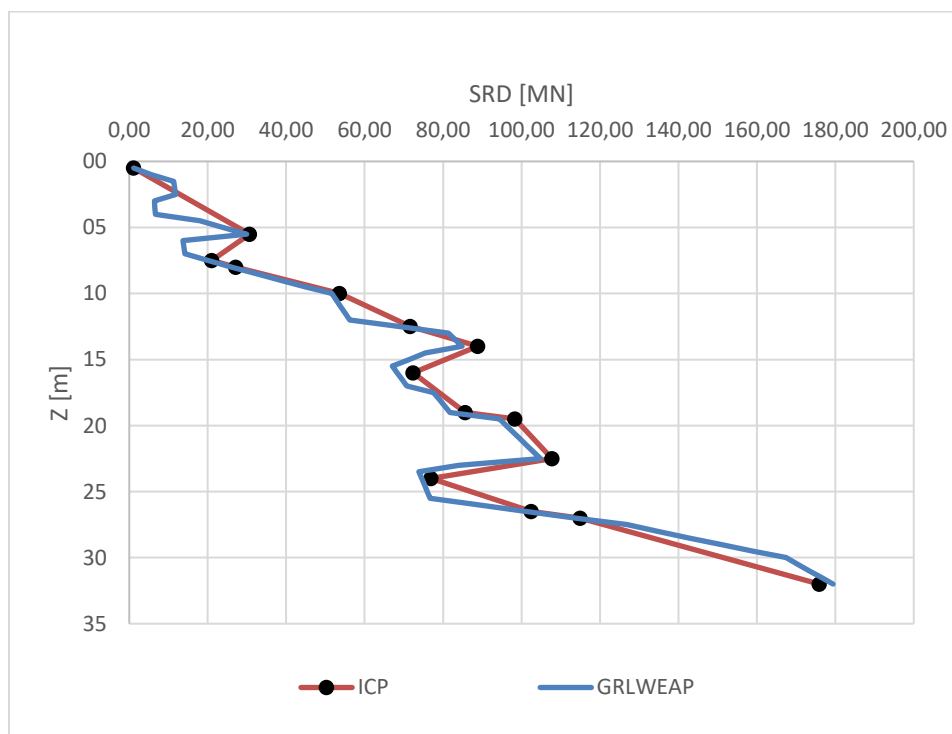


Figure 6.19– SRD Evolution: ICP

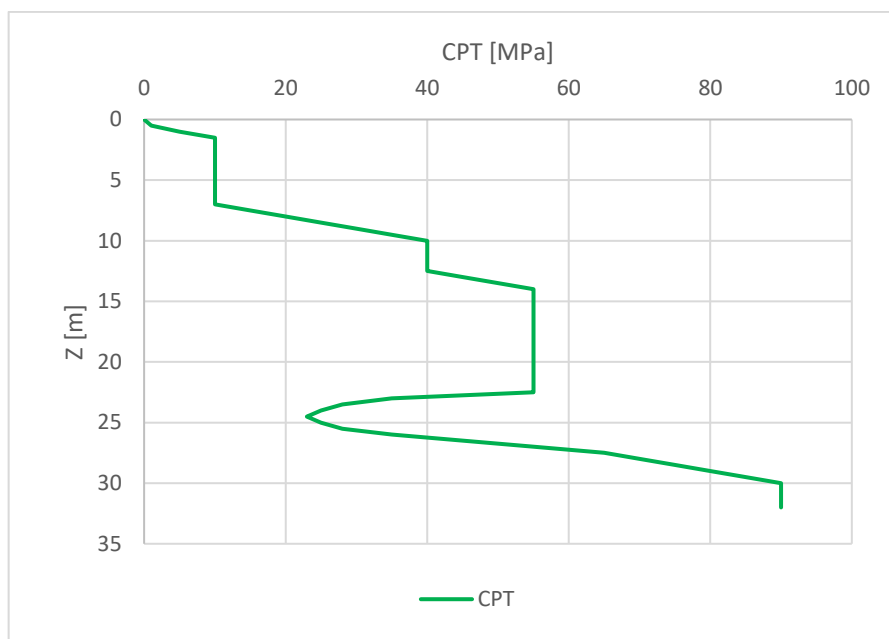


Figure 6.20. – CPT tip resistance; P01

Table 6.3 – SRD comparison between AH upper bound and ICP no setup

AH		ICP	
Depth (m)	SRD (MN)	Depth (m)	SRD (MN)
0.5	0.07	0.5	1.14
5.5	23.42	5.5	30.66
7.5	16.34	7.5	21.00
8.0	21.04	8.0	27.18
10.0	43.78	10.0	53.60
12.5	60.86	12.5	71.65
14.0	77.11	14.0	88.84
16.0	64.78	16.0	72.34
19.0	75.02	19.0	85.63
19.5	84.68	19.5	98.34
22.5	95.90	22.5	107.82
24.0	70.85	24.0	76.92
26.5	83.84	26.5	102.48
27.0	93.73	27.0	114.97
32.0	161.61	32.0	175.91

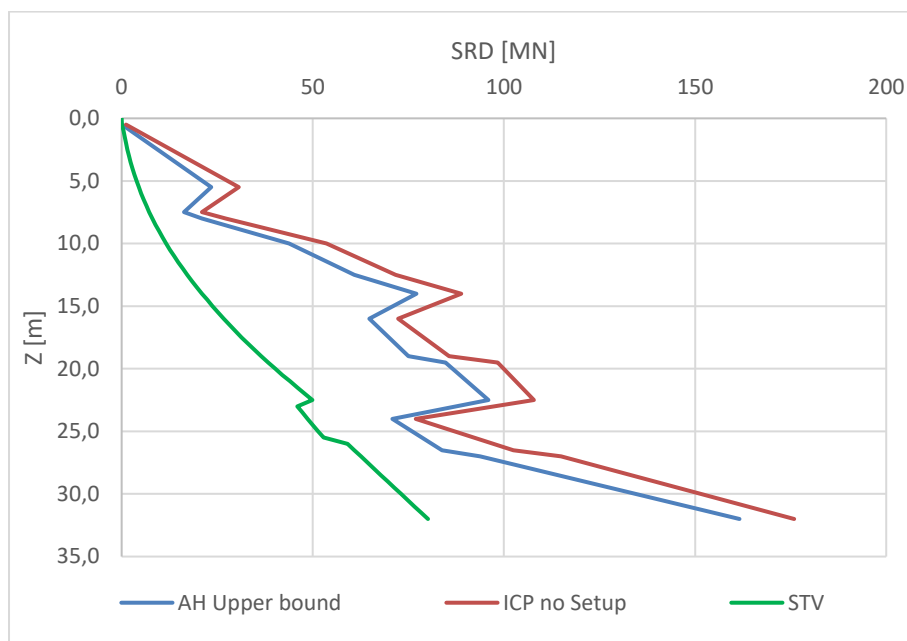


Figure 6.21– Final SRD comparison; P01

6.8. STEVENS (STV)

It was already described that Stevens design approach does not make any consideration on friction fatigue. As such the only care that has to be taken when simulating the process in GRLWEAP is to consider the correct input of shaft and base resistance. Since in this model both internal and external friction is considered, then it is required to enter both the internal and external perimeter. Note however, that no reduction in friction is to be considered, hence the gain loss factor as to be set equal to 1, meaning that no friction reduction was utilized by the program. The results obtained are exposed in Figure 6.22

It is shown an almost perfect correlation between STV and GRLWEAP, this should not come as a surprise, since no consideration of friction exists the evolution of resistance will always be the same for all penetration depths and since both GRLWEAP and the Stevens (1982) model calculate the total side friction and base resistance in the same way using the unit side and base resistance the match seen is to be expected. Therefore, with no consideration regarding the reduction in friction, the main cause of discrepancy is now gone, so a flawless match was to be expected. Note however that this model is not CPT related, instead it depends on the interface friction angle, bearing capacity factor and naturally the effective overburden stresses.

To conclude, it is possible to say that all methods give very different predictions for the soil resistance, particularly AH and ICP. The main reason for this discrepancy was concluded to be the consideration of the setup caused by 10 days waiting time in the ICP methodology. After removing the setup effects in the ICP and comparing it to AH it was seen that both theories are quite similar, especially when dealing with shaft resistance. This is due to the fact that both consider the Mohr Coulomb failure criteria to control the shaft failure. The main difference between the two is related to the calculation of the base resistance. Lastly, it is important to calibrate GRLWEAP in order to estimate the SRD in accordance with the proposed methods. As seen in Figures 6.17; 6.19 and 6.22 based on the good match seen, it is possible to say the program now takes in consideration all of the underlying aspects

of the design proposals. As such the next step of the driveability analysis can now commence, the back analysis.

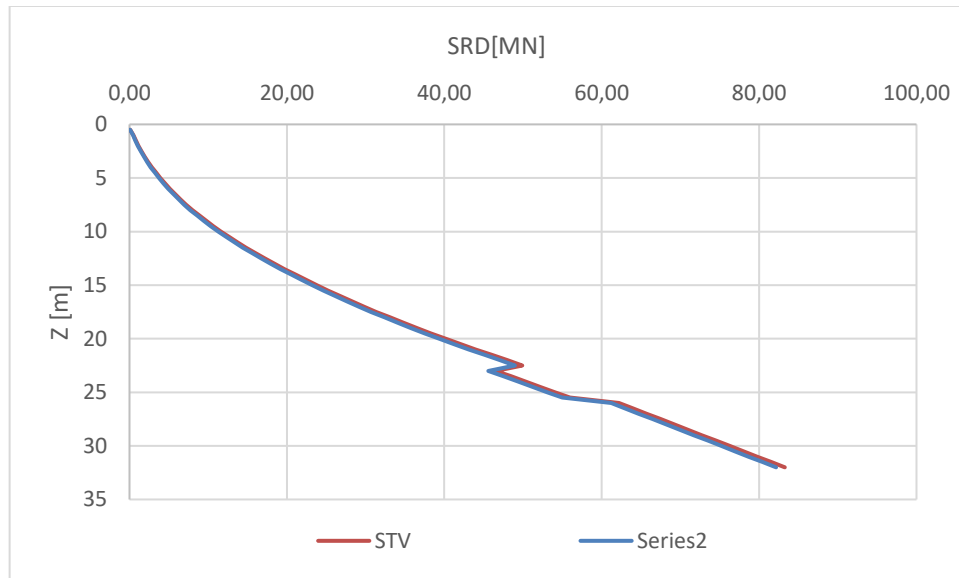


Figure 6.22 – SRD Evolution: STV

6.9. FINAL RESULTS FOR LOCATION P02 AND P03

As stated in the beginning of the section, only the results for P01 were analysed, however it is still important to cover the results achieved for the remaining locations. As such the final results were plotted in Figure 6.23 and 6.24. The plot was achieved in the same manner already described in the present section. It can be seen in all figures that STV predicts low resistance values. Meaning that in a axial analysis it is the most conservative method.

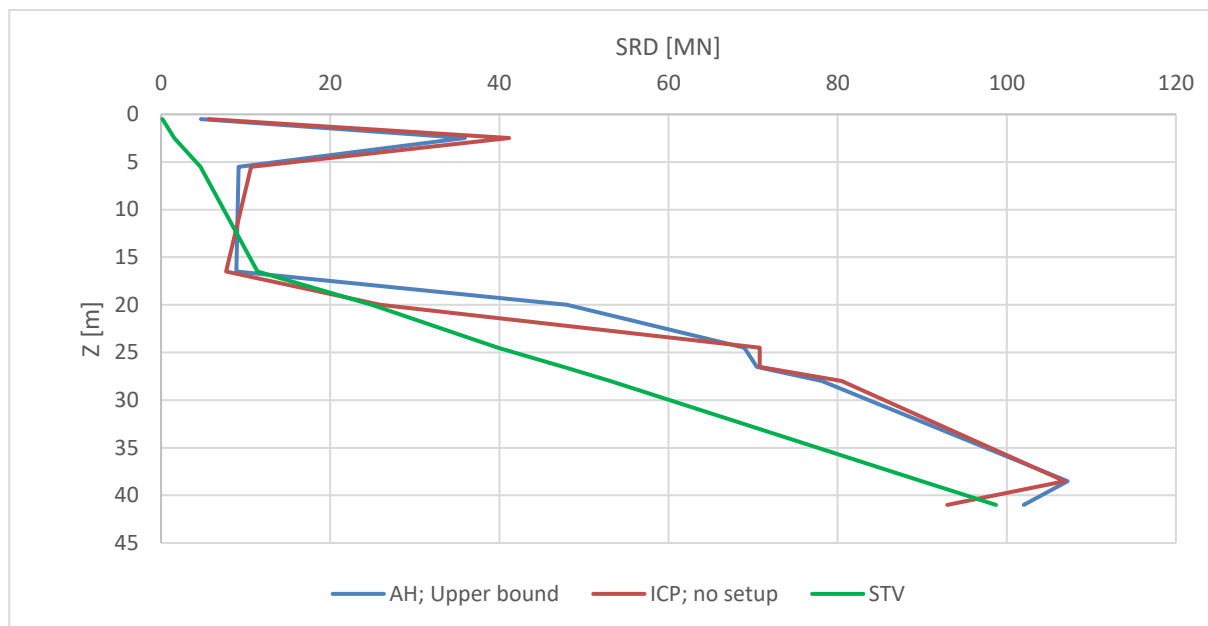


Figure 6.23 – Final SRD results; P02

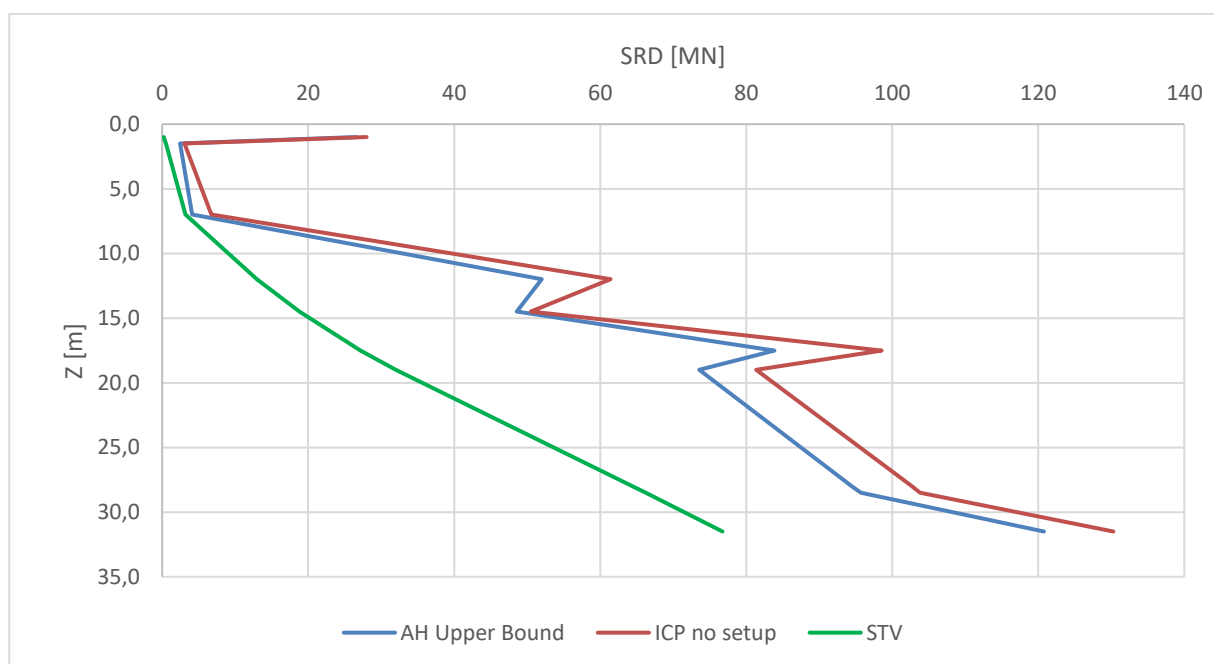


Figure 6.24 – Final SRD results; P03

7

BACK ANALYSIS

7.1. INTRO

In order to reach a safe and cost effective installation there is a need to perform an adequate pile driveability analysis. Driveability analyses provides assessments on the selection of an appropriate hammer as well as determination of pile accessories. It also involves a review of soil profile to reach the determined penetration or capacity stipulated in the design stage. What defines a cost effective installation is the number of blows required to actually drive the pile to the desired depth. In this context, the present chapter aims at providing a solid estimation of such blow count using different approaches for estimating the soil resistance to driving and back calculating the blow count using pile driving records from the North Sea.

The ultimate goal of this paper is to provide more insight on pile driveability, more specifically large offshore piles in the North Sea as well, as if the methods currently used to evaluate soil resistance on smaller diameter piles are still viable. Soil resistance (SRD) was estimated using three design approaches: Alm and Hamre (1998) and Alm and Hamre (2001); Imperial College Pilling (2005) and Stevens (1982). All of the methods are described in section 3 and a summarized calculation table is provided in chapter 6

7.2. DRIVING CONDITIONS

The presented piles were all driven in the North Sea with the soil conditions presented in chapter 5, pile geometry and other specifications are detailed in the same section. The pile installation was made possible with use of an IHC Hydrohammer with a rated energy of about 2000 kJ (IHC S-2000), ram weight of 990.81kN and with the equivalent rated stroke of 2m. Table 7.1 summarizes hammer properties as well as some pile details.

7.3. METHODOLOGY OF THE BACK ANALYSIS

7.3.1. SOIL MODEL

Several different ways to back calculate either soil resistance, hammer blows or even the energy transmitted from hammer to pile are available. However, as previously stated, the aim of this chapter is to estimate blow count, from the information provided in the driving log, namely hammer energy or the hammer stroke height. Since the pile experiences both static and dynamic resistance the presented method relies on the wave equation analysis program GRLWEAP based in Smith (1960). Dynamic soil parameters, such as soil damping and static stiffness parameters, such as soil quake, are essential to correctly assess the soil model and thus the pile driveability itself. Quake and damping values must be determined for skin and toe, the recommended value for both skin and toe quake in open ended piles is 2.5 mm, this value is based on the fact that such pile will not develop a plug, which seems reasonable since the detailed piles present a large diameter, as such no plug is to be expected. It should be noted that in the following calculations said toe quake value was used, however Webster et al. 2008 as well as Rausche et al. 2009 performed several dynamic tests and signal matching analysis with the

use of CAPWAP, and reached the conclusion that those values might not be appropriate, leading to the proposal of toe quake values proportional to the piles outer diameter, which in turn results in higher quake values. Nonetheless in order to follow the recommendations of the analysed SRD theories, such as Alm and Hamre (2001), it was decided that the quake value of 2.5 mm should be maintained. As a result, the blow count predictions will tend to slightly decrease as opposed to the blow prediction based on higher quake values, thus the achieved results might not be on the conservative side (in the driveability point of view). Regarding the soil damping, the recommended GRLWEAP values were used, since these values have been generally indicated from CAPWAP analysis performed on offshore dynamic tests. Note that both quake and damping values might vary based on the actual soil conditions and behaviour, but since it is extremely difficult to correctly ascertain these values it is recommended to use the proposed values. The proposed skin Damping values used in the calculations are presented in Table 7.2. For all situations, the quakes and toe damping values were made equal to: 2.5 mm and 0.5 s/m.

Table 7.1 – Pile and hammer details

	P01	P02	P03
Hammer	IHC S-2000	IHC S-2000	IHC S-2000
Rated Energy (kJ)	1999.23	1999.23	1999.23
Eq. Rated Stroke (m)	2.01	2.01	2.01
Efficiency	0.95	0.95	0.95
Pile Length (m)	62.80 m	65.10m	60.30m
Penetration (m)	31.90 m	40.75m	31.75m
Diameter (m)	5.94 m	5.94 m	5.94 m
Cap Type	Sleeve	Sleeve	Sleeve
Pile Cushion	Anvil	Anvil	Anvil
Pile Material	Steel	Steel	Steel

Table 7.2 – Soil Model

Soil Behaviour	Method	Skin Damping [s/m]
Clay	Alm and Hamre	0.25
	Imperial College	0.65
	Stevens	0.65
Sand	Alm and Hamre	0.25
	Imperial College	0.16
	Stevens	0.16

7.3.2. PILE AND HAMMER MODEL

Modelling of the hammer and pile will naturally be of extreme importance. The input of the pile model into GRLWEAP is normally a straightforward procedure, using the pile details (pile makeup, wall thickness, and length) provided by COWI A/S the pile segments were modelled in GRLWEAP. However due to the fact that the analysed piles were used in an actual offshore project in the North Sea, most of the details will not be presented.

Since the main focus of this paper is the prediction of blow counts and not pile stresses evaluation, no inclusion of stabbing guides was included, thus no consideration of the increase in compressive stresses in the pile were made. Though in actual driving analysis this is of extreme importance, excessive stresses on the pile section might lead to pile buckling, severely hindering the driving process.

Now regarding the energy applied by the hammer to the pile. Before the use of modern hydraulic hammers, steam hammers were usually employed for driving. These hammers did not have an option to vary the stroke and so the energy applied to the pile resulted on the weight of the hammer, and as such in order to drive a single pile most of the times it was required to change the hammer. For the initial stages of driving a light pile was normally used, whilst for the last phases a heavier hammer was in order to apply more energy. The piles analysed were all driven with a hydraulic hammer as a result there is no need to change the hammer meaning that the weight will be the same, since it is now possible to vary the stroke and the velocity of the ram.

To simulate the driving conditions that actually occurred, the hammer transferred energy needs to be correctly inserted in GRLWEAP. This can easily be made by imputing the hammer efficiency, i.e. the ratio of the actual energy generated or transferred through any point of the system (hammer, cushion, anvils, etc.) to an ideal energy definition. In other words, the IHC hammer can control the velocity and height that the ram impacts the pile, resulting in different energies in which the pile can be driven.

$$\eta = \frac{E/Blow}{E_r} \quad (7.1)$$

With the hammer efficiency, GRLWEAP can calculate the energy used to drive the pile at each depth. The analysis was performed with the full hammer stroke of 2m, meaning that it was the hammer velocity that was changed in order achieve the ideal energy criteria. It is also possible to back calculate the blow counts varying the hammer stroke instead of its energy. By adjusting the hammer stroke, the actual hammer energy recorded in the driving log at the time of installation is used to demonstrate how the predicted SRD suits soil conditions. Note however that an anvil was used in the driving, thus the energy that the hammer transfers to the pile will be reduced, said anvil was modelled in GRLWEAP and the energy that the pile now receives can be seen in Figure 6.6

7.4. CPT RECORDS AND SOIL CONDITIONS

In order to better compare the blow count prediction with the soil CPT borehole logs as detailed in section 5 were provided and the plotted tip resistance can be seen in Figure 5.8; 5.9 and 5.10. The predominant soil encountered in all three locations was sand, dense sand to very sand to be more specific, with the exception of small layers of silt and clay.

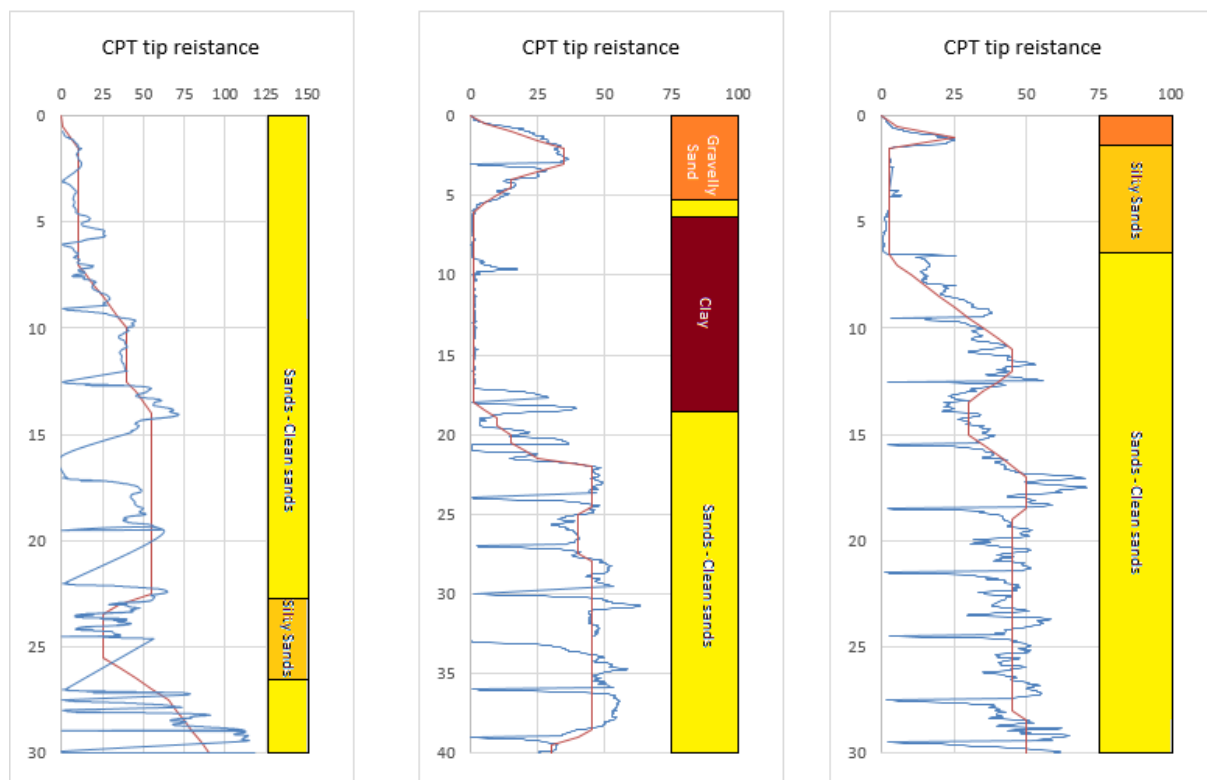


Figure 7.1 – CPT tip resistance and soil description for Location P01; P02; P03

7.5. BACK ANALYSIS

With the driving conditions now properly matched GRLWEAP can calculate the number of blows that the different methods predict. In all the following calculations, for AH as well as for STV the upper bound was used. I.e. in AH the resistance was increased by 25% whilst in STV both internal and external friction were considered with the following ratio, $\tau_{f,int}/\tau_{f,out} = 1$. Again it was assumed that all piles were driven in a coring scenario, due to the large diameter. Figures 7.1; 7.2 and 7.3 represent the measured blow counts as well as the methods predictions.

All of the recorded hammer energies and total number of blows required to drive the piles were plotted in Figures 7.5 and 7.6. Note that the hammer does not start the driveability with full efficiency, this is common practice in all offshore driveability works, as well as onshore. The first layers of soil are usually not that hard to drive into, using a lower energy will mean that the pile will not overstress and the hammer will work at a safe rate. Of course as the pile is being driven deeper there is a need to increase the efficiency, transferring more energy into the pile. Do keep in mind, that the plotted energy is the energy that the pile actually receives (ENTHRU), meaning that it already suffered losses when transferred from the hammer to the pile.

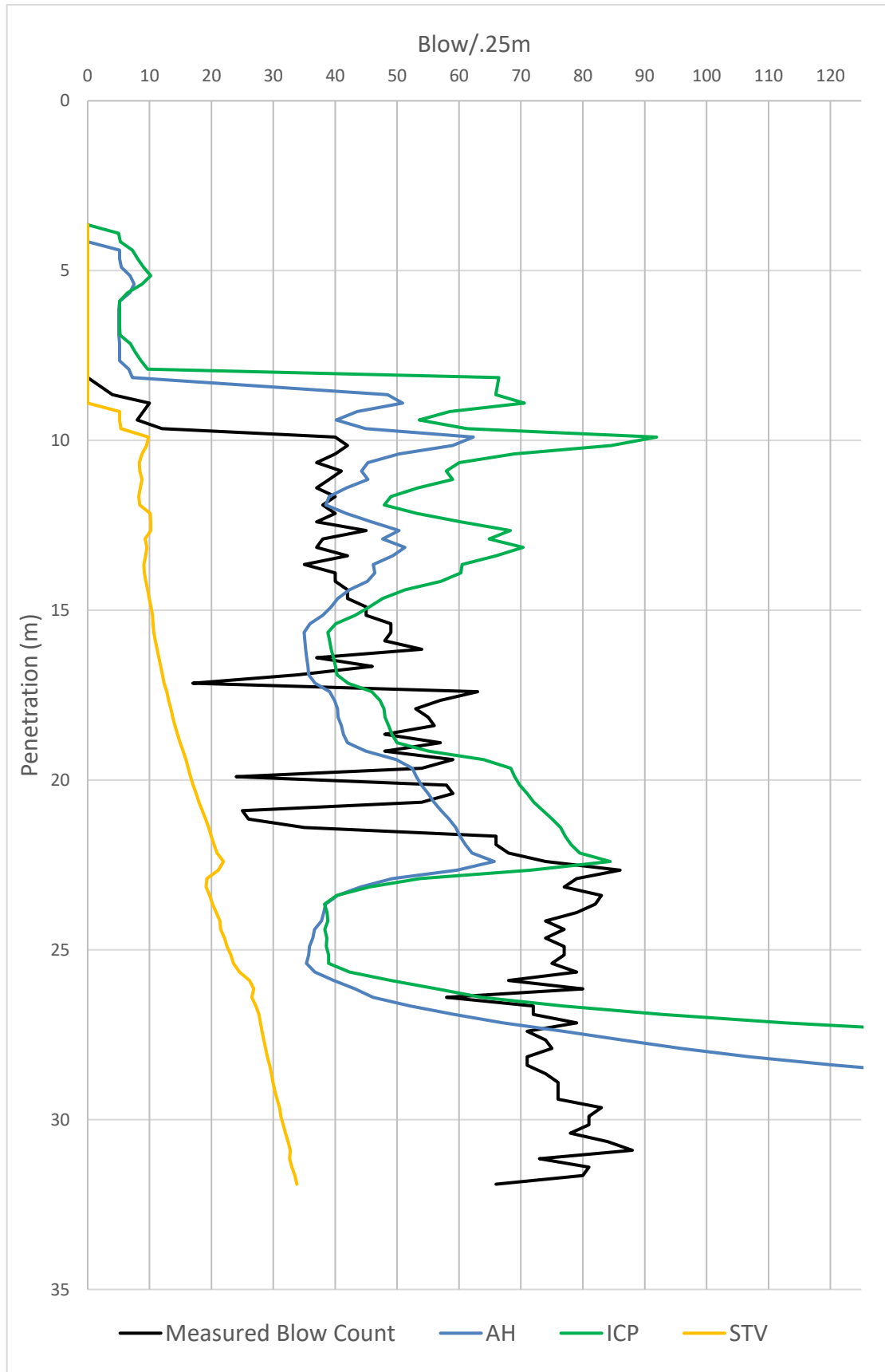


Figure 7.2- Driveability for Location P01

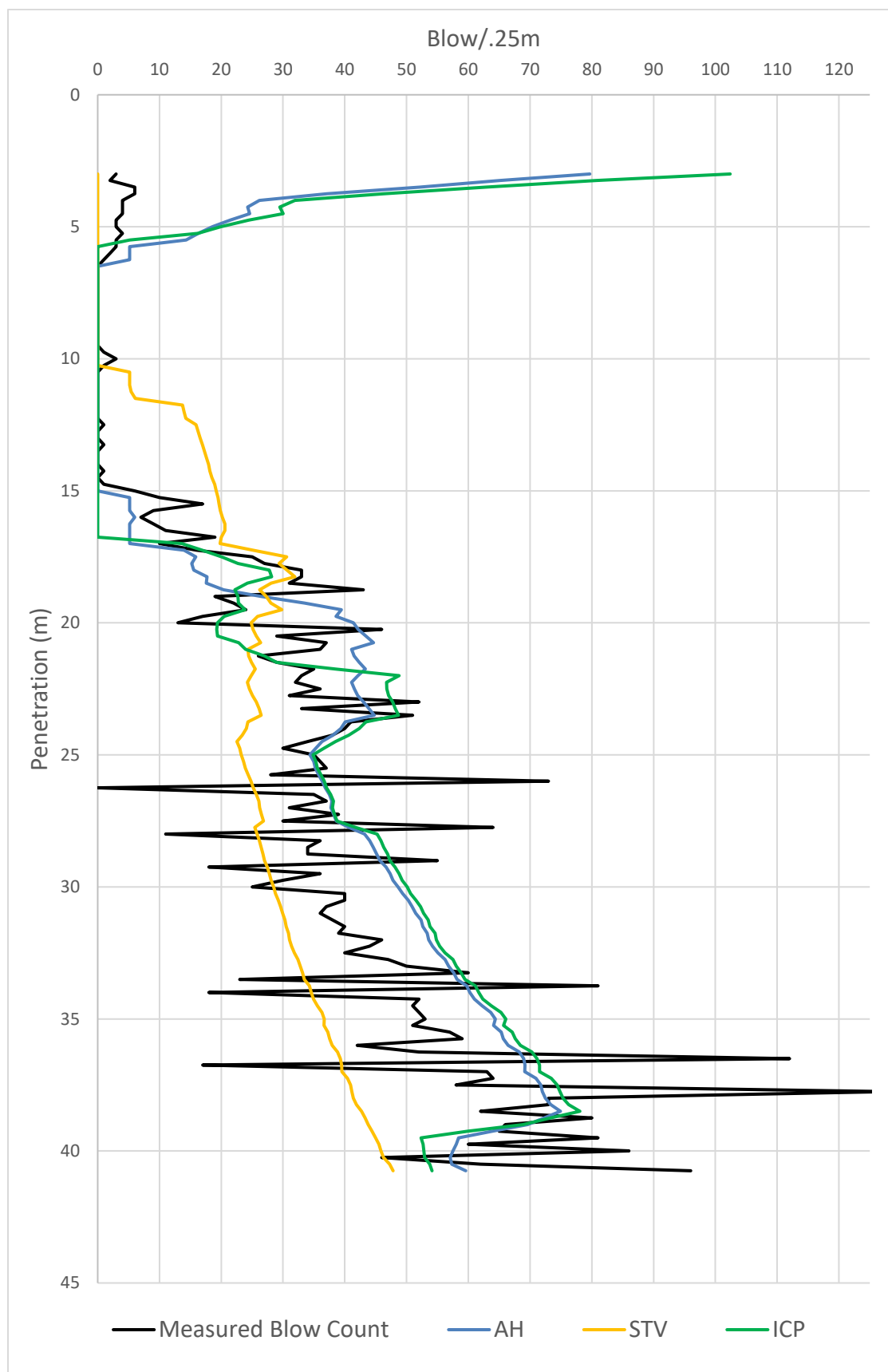


Figure 7.3 – Driveability for Location P02

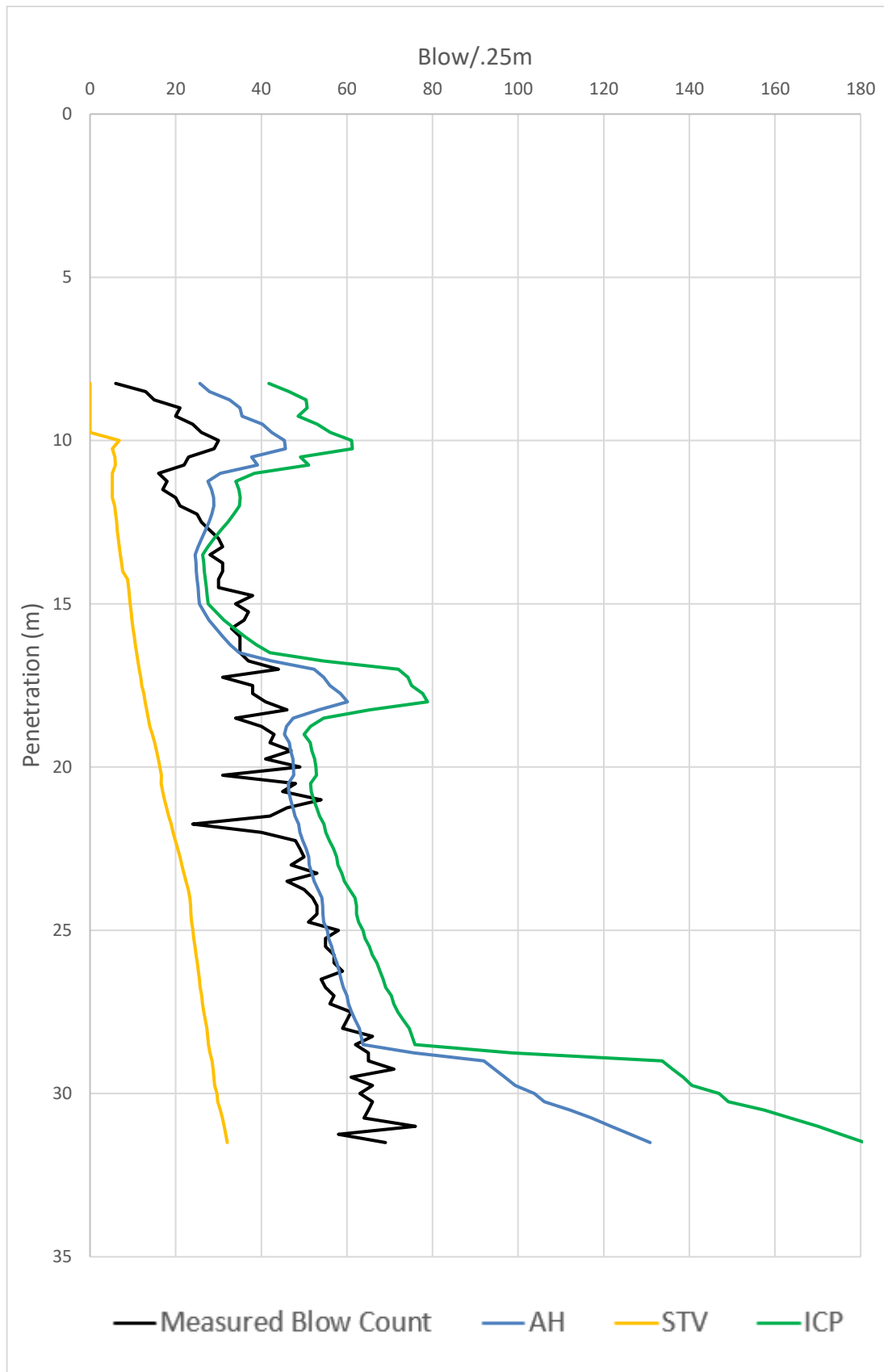


Figure 7.4 – Driveability for Location P03

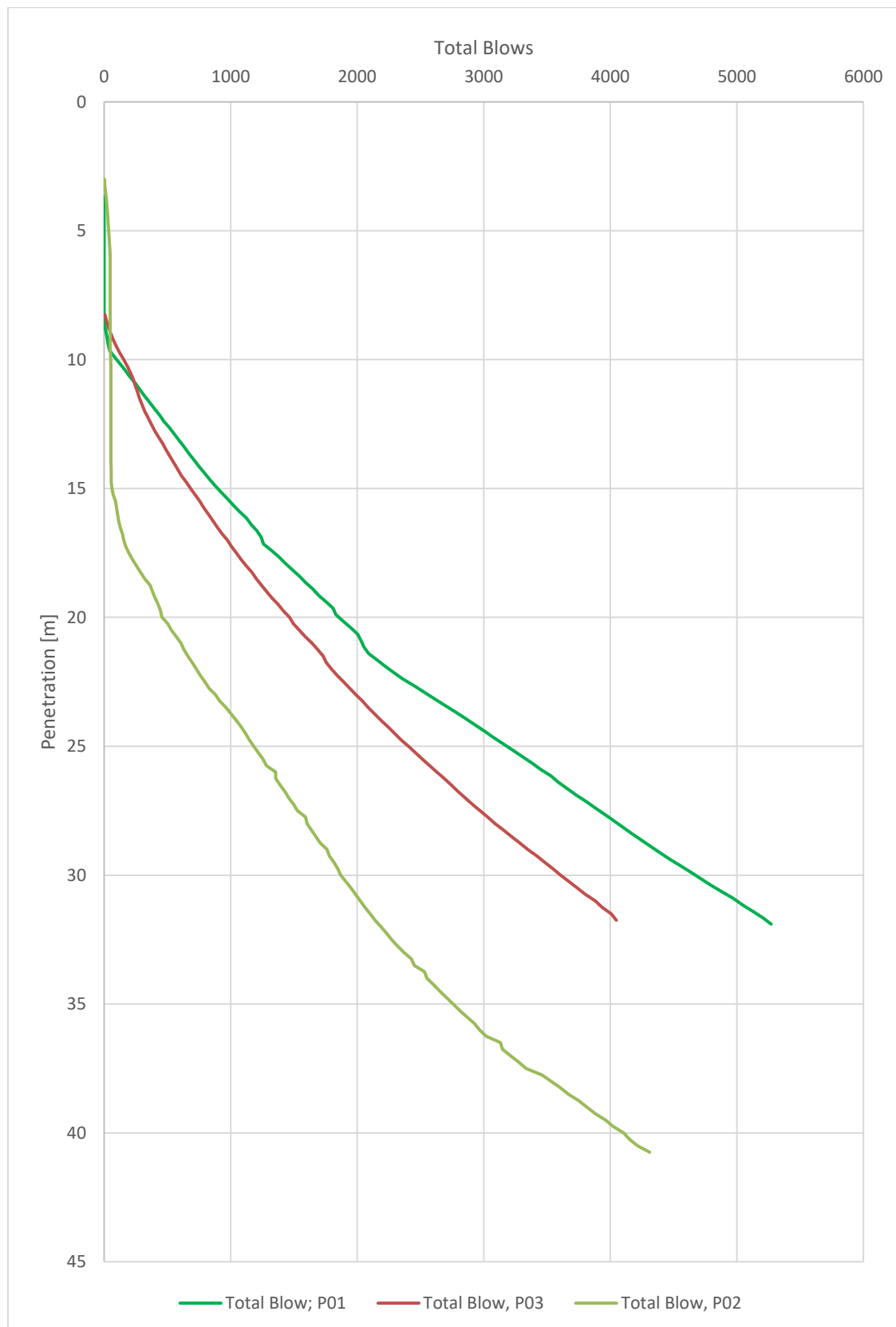


Figure 7.5 – Total blows

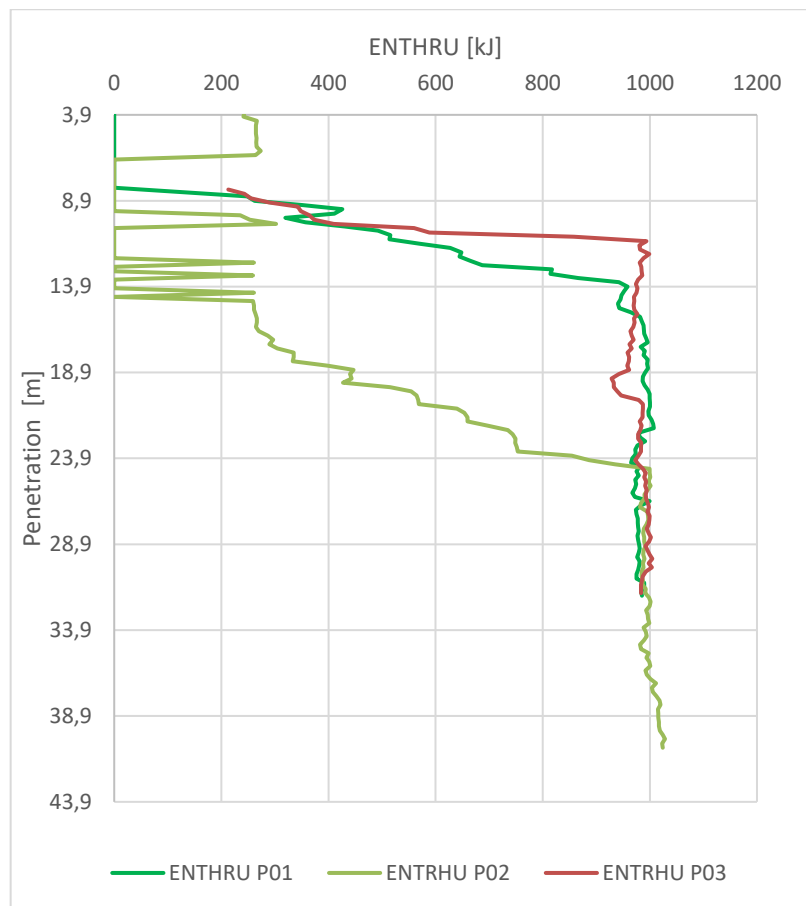


Figure 7.6 – Energy transmitted to the piles

Regarding pile, P03. The predominant soil type is sand (dense sand to very dense sand) with the exception of a small silty sand layer at around 2 m to 7m depth. However the driveability itself only begun at 8.25m so no data regarding energy or number of blows is provided until said depth is reached. It was assumed that the pile runs through the first 8.25m of soil. As the soil mainly consists of sand it is expected that the CPT based tests will provide a good estimate of the number of blows. Indeed both AH and ICP provide good estimates, however they foresee an unusual blow count at around 17m. One possible explanation could be that since these methods are based on CPT results, they will be very sensitive to sudden variations of the tip resistance. In this case we have an increase of the CPT tip resistance at that depth causing an increase in the SRD which in turn will then rise the number of blows predicted by these methods. Based on Figure 7.4 it is possible to see that both AH and ICP provide very similar results, with a slight over prediction on behalf of the ICP. This is to be expected since both designs are quite similar, and with the removal of the setup in the ICP method the major differences between the two can be summed up to on how they consider tip resistance and the reduction of friction. It is seen that STV underpredict the number of blows in the sand layer, this will be a recurring theme for this method in all three locations. Stevens's model is known to be on the conservative side, meaning that the pile should not count with much resistance, this is why two bounds should be considered, an upper bound for driveability analyses and a lower bound for loading. Since in a static loading point of view, if a pile is calculated considering a low static capacity it will lead to a safe design, however in a driveability point of view, said reduction in axial capacity is not in

accordance with reality and will lead to a non-conservative driveability analyses, i.e. low blow count. However even when using STV upper bound, an under estimation will occur. The two analysed CPT based methods, predicted refusal at around 29m meaning that the predicted soil resistance is most likely greater than the real encountered resistance. In this particular location AH gave the best prediction whilst STV gave the worst.

Figure 7.2 shows back analysis results for the second location analysed, P02. Yet again the site is mainly comprised of dense sands. However in this location a 12m layer of silty clay was encountered at 6m below the mudline. The soil in this layer was handled as if it behaved like a clay. As explained, the main focus of this work will be on sands, and as such the main conclusions will only refer to this type of soil. Nevertheless in this location there was a need to apply the described methods in clays. As it can be seen, both CPT methods provide precise predictions, especially AH and ICP. Once the sand layer has been reached, the two designs proposals will marginally over predict blows, but it can be said that in a driveability analyses these results appear to be rather reasonable. Also in the initial stage of the driveability an abnormal high blow count is predicted by the CPT methods. It is possible that in the CPT borehole a small layer of very dense gravely sand or maybe rock was encountered. Based in Figure 5.9 it is possible to see that at around 3m the tip resistance is around 35 MPa, this seems rather unusual. If such high tip resistance was encountered then the measured blow count in that depth would be greater, since the actual blow count is fairly low it is possible to assume that the pile was not actually driven through said layer. As a result the high blow prediction should be disregarded since it was not directly caused by a wrong interpretation of the methods. If the tip resistance were to be reduced to better represent the soil conditions were the pile is situated the blow prediction would certainly be in accordance with reality. It could be said that such increase could also be explained by the low confinement stresses, however the number of blows is just too great to be simply explained by the application of the methods in low confinement stresses. STV appears to give a good prediction in this location especially in the sand layer.

Lastly Figure 7.3 concerns the analysis developed for P01. In this location, a small 3m layer of sandy silt was encountered, and it was regarded as sand. Again the remaining soil mainly consists of clean sands. In this specific location all methods, to some extent failed to provide an accurate prediction on the number of blows required to drive the pile, demonstrating inconsistency in the methods even in the CPT based ones. STV design again underperformed, predicting a lower number of blows. The remaining methods over predict blows in the first meters of driving, one possible explanation for this over prediction could be the fact that these methods were developed, in sands, for high confinement stresses, meaning they will not give their best results in the first meters of driving. Further, AH and ICP predict a drastic reduction of blows in the silt layer, while in actuality an increase in blows was registered revealing a harder driving than foreseen. This underperformance reveals a flaw in the CPT based methods, seemingly when dealing with mixtures of soils, the models will not perform adequately. An alternative explanation for the increase in blows could be that around 21m depth the pile could began to plug, meaning that base resistance will increase resulting in the increase seen in Figure 7.3. Although even though the plug could explain the increase in blows, it is not likely to actually occur and since no measurements of the plug were made, the main effect that explains such inconsistency is the fact that the silt layer was actually much harder to drive than anticipated. As in location P03 both AH and ICP predict refusal.

Based on the three sites analysed it is difficult to say if the proposed methodologies are still a viable option to use in the large piles seen today in the offshore industry. However, some conclusions can be attained. These conclusions will be discussed in chapter 8

8

CONCLUSIONS

8.1. CONCLUSIONS

Firstly, it is clear that in a general way, CPT based methods provide better estimations when dealing with simple soils, such as clean sands. Yet, they will underachieve if dealing with soil mixtures, as seen in Figure 7.1. Thus it is of the outmost importance that the information regarding the layer represents the actual state of the soil. This recommendation is specially emphasized in Jardine et al (2005). More information was needed to attain the correct interface friction angle, undrained shear stress and the over consolidation ratio. These parameters have shown to have a great influence in accurately predicting the blow count. Instead of the proposed correlations in an actual offshore project it is heavy recommended to perform several testes to attain said parameters.

It is interesting to compare the final blow count prediction with the final SRD results. It can be clearly seen that the soil resistance influences the blow count tremendously.

Also quake and damping values used might not be representative of the actual soil conditions, correctly assessing this values is extremely difficult, the same could be said for parameters used in GRLWEAP, such as the limit distance. Regarding the damping and quakes, the manner in which the pile drives the soil and the way it responds to the driving is also complex, as a result the recommended values seen in the literature and in GRLWEAP were used. Having said that, a sensitivity analysis was performed where several shaft damping factors were analysed, the final results did not differ from the ones presented, it appears the recommended values can be used with some degree of confidence.

As it was stated all the piles were driven assuming they would not plug, meaning that the tip resistance would only be considered on the pile annulus, this consideration naturally gives lower blow counts. If a plugged scenario was admitted the number of predicted blows would naturally increase and the methods predictions (all assuming the coring scenario) would give low estimates. Since the blows predicted are already larger than the measured blows, it is possible to say that the piles were indeed driven in a coring manner. This conclusion might not be considered relevant for some, since it is well known that large diameter piles are usually driven in a coring manner. However recent driving analysis performed by Taylor et al (2015) in the Taranaki basin of New Zealand revealed that for one of the investigated pile location the 2m diameter pile was partially plugged towards the middle stages of driving. Although this was not ultimately concluded, since no measurements of plug height were taken, and the site conditions differ from the ones analysed here, it should serve as a reminder that unexpected plug could develop, even in large piles. As such it is always recommended to perform a driving analysis assuming both a coring and a plugged situation, or more conveniently assuming a degree of plug, especially in the depths were the pile is driven from a hard layer into a soft one.

Regarding Stevens proposal, based on the analysed locations, it is clear that it severely underestimates the blow count. It is believed that this proposal is too conservative when calculating the resistance to driving, said conservatism will not translate into the driveability analysis, even when using the upper bound which should be used in these sort of studies. Further, it is the author's opinion that the limits imposed on unit tip and skin resistance could possibly not be considered in a driveability analysis for large piles. It is also interesting to point that even though this model does not consider the effects of

friction degradation, which would lead to a harder driving, it still is the method that gives the lowest blow prediction. Based on the results this method should not be used in driveability analysis for large offshore monopiles.

Finally, based on only three locations it is not possible to ascertain if the analysed methods can be applied. However these results point towards a positive answer. Especially the proposals linked to CPT, even though some inconsistency is expected in applying them in soil mixtures, if a detailed geochemical knowledge of the site is presented it is estimated that these proposals will have a good match with the actual blow count measured on site. Also worth mentioning that the use of this methods should be used to provide a degree of confidence in the driving predictions, however actual driving experience from similar sites should be used and is encouraged as to increase the accuracy of the predictions.

In this work a driveability analysis was performed and calibrated. Three large diameter piles located in the North Sea were analysed. All piles were driven assuming they would not plug. On a first stage the soil resistance to driving was calculated, based on three different proposals. Since the methods were based on smaller diameter piles, the main point of the work was to ascertain if the methods could still be applied in the large diameter piles currently used in the offshore industry. The main conclusions regard granular soils.

Alm and Hamre (2001) and the Imperial College Pilling as seen in Jardine (2005) are both CPT dependent. Furthermore, both consider the degradation of friction, meaning that as the driving progresses the resistance encountered will decrease, being maximum at the pile tip where the pile does not suffer shearing and being minimum near the mudline where shearing due to consecutive hammer blows is encountered. The main difference between the two is the consideration of 10 days' worth of setup by the ICP methodology. So the presented original method does not provide the fully reduced resistance at driving, it provides instead the capacity mobilized after 10 days. On the other hand, the approach presented in Alm and Hamre (2001) does provide the fully reduced resistance at the time of driving. This difference explains the differences seen in the predicted resistance, and would nullify the use of the ICP proposal in a driveability analysis. As such it was decided to remove the consideration of setup using the Skov and Denver (1988) relation of capacity with time. By normalizing said relation with the ICP method and using a growth trendline it was possible to back calculate the resistance at the end of driving. The third analysed method was proposed by Stevens et al (1982). This theory does not consider the effects of friction fatigue nor does it relate to CPT. Thus the resistance predicted will not be reduced. Also worth mention the fact that STV is a largely conservative method, when estimating pile axial capacity, however it was noted that this conservative nature does not mean a conservative driveability analysis, quite the contrary. In a driveability stand point STV will often underpredict the blowcount.

Secondly all methods were implemented in GRLWEAP. Special care was given to the degradation of resistance in the program. As explained, for simulating the rate of degradation as seen in ICP a standard gain loss approach based on setup factors were used.

- $G/L = 1/1$ Full setup i.e. non reduced resistance, LTSR
- $G/L = 1/f_{s,max}$ Reduce Resistance

It is important to reinforce the idea that the diminish resistance is not fully reduced, it is the resistance seen after 10 days. As such in order to achieve the residual value, the proposed ratio $Q_{t=1day}/Q_{ICP;10\ days} = 0.5$ was used.

Regarding the implementation of Alm and Hamre (2001) rate of friction in GRLWEAP. A relation between the two was created and using an iterative process they were matched. Note that a true theoretical relation between the AH shape factor and the limit distance was not presented, since it would be a slow and difficult process, instead the iterative process was used, and by estimating the limit distance and the GRLWEAP shape factor a good match was established, i.e. the shape of the degradation curve in GRLWEAP was matched to the Alm and Hamre (2001) theory. The actual degradation is achieved with the gain loss factors.

Stevens (1982) provides the non-degraded resistance, as such in GRLWEAP the full resistance with a gain loss factor equal to the unity was analysed.

After calibrating GRLWEAP with the underlying aspects of the proposed methods, the back analysis process was initiated. The pile model was a straightforward process, in GRLWEAP the pile was modelled to behave to drive in a coring manner. Regarding the hammer and the energy used. By using driving logs of the driven piles, the energy used to drive the pile was simulated in the software. The losses that occurred from the transfer of energy from the hammer to the pile were accounted for. The hammer used was a hydraulic hammer, there was no need to model the hammer since it was available in the software library. Finally the soil model was made largely consistent with that used in Smith (1960) and consists primarily of static and dynamic parameters, i.e. quake and damping parameters. The final results were shown in section 7. There it is shown the match between the methods predictions and the actual measured blows required to drive the pile.

A conclusive answer regarding the application of the analysed proposals in the offshore industry is not possible to achieve since only three locations were analysed. Having said that, the results of the back analysis do point to a positive application of the CPT based methods, especially if dealing with clean soils. However, when facing with soil mixtures the same methods will underachieve. On the other hand, Stevens (1982) model clearly under predicts, resulting in a non-conservative driveability analysis. Hence it is the author's opinion that it should not be used in large diameter offshore driven piles.

Driveability analysis are of great value in the offshore industry, it allows for the correct selection of a hammer that should be capable of driving a given pile in a cost and time effective manner, an accurate driveability analysis, could lead to real driving scenarios that were otherwise considered impossible, such as the driving at Heather Field analysed in Heerema (1980). Essentially in any offshore project said analysis should be performed. However, using CPT base methods to predict the soil resistance can lead to some inconsistencies, especially when dealing with soil mixtures. Thus a model that can accurate predict the SRD in these soil mixtures would result in further accurate predictions.

8.2. FUTURE DEVELOPMENTS

It was seen that one of the main drawbacks of the CPT based methods was the inability to accurately assess the blowcount in more complex soil mixtures, such as silts or chalk like materials. When dealing with these soils it is of the utmost importance to attain a valid relation between the methods and the number of blows. This might involve small calibrations in the CPT proposals. Unfortunately, only one location containing a significant silt layer was encountered and as such it was impossible to attain any conclusions regarding the possible methods modification.

REFERENCES

- A. Viana da Fonseca;(2013) Ebook Abordagem Unificada ao tratamento de ensaios CPT CPTU SCPu para fundacoes, ..., 2013. ISBN: Em finalização (O livro está em edição)
- Achmus, M., Kuo, Y. S., & Abdel-Rahman, K. (2009). Behavior of monopile foundations under cyclic lateral load. *Computers and Geotechnics*, 36(5), 725-735.
- Alm, T., & Hamre, L. (1998, May). Soil Model for Driveability Predictions. In *Offshore Technology Conference*, Houston, Paper No OTC (Vol. 8835).
- Alm, T., & Hamre, L. (2001). Soil model for pile driveability based on CPT interpretations. In *International Conference On Soil Mechanics and Foundation Engineering*.
- Ardus, D. A., & Society for Underwater Technology. (1993). Offshore site investigation and foundation behaviour: Papers presented at a conference organized by the Society for Underwater Technology and held in London, UK, September 22-24, 1992. Dordrecht: Kluwer Academic.
- Brunning, P., Chow, F. C., Keep, A., Thomas, J., Yong, D., & Pennington, (2015) D. *Frontiers in Offshore Geotechnics III*.
- Brzezinski, L. S., & Baba, H. U. (2012, March). Offshore Open End Steel Tubular Piles-A Case History. In *GeoCongress 2012*.
- Das, B. M. (2015). *Principles of foundation engineering*. Cengage learning
- Energy, D. O. N. G. (2016) Comparison of pile driveability methods based on a case study from an offshore wind farm in North Sea.
- Fellenius, B. H. The response of a "plug" in an open-toe pipe pile. *Journal of the SEAGS & AGSSEA*, 46(2), 82-86.
- Gavin, K. G., & O'Kelly, B. C. (2007). Effect of friction fatigue on pile capacity in dense sand. *Journal of Geotechnical and Geoenvironmental Engineering*, 133(1), 63-71.
- Gavin, K. G., Igoe, D. J. P., & Kirwan, L. (2013). The effect of ageing on the axial capacity of piles in sand. *Proceedings of the Institution of Civil Engineers–Geotechnical Engineering*, 166(2), 122-130.
- Gavin, K., Jardine, R., Karlsrud, K., & Lehane, B. (2015). The effects of pile ageing on the shaft capacity of offshore piles in sand. *Frontiers in Offshore Geotechnics III*, 129.
- George, P., & Wood, D. (1976). *Offshore soil mechanics*. Cambridge University Engineering Department; Lloyd's Register of Shipping.
- Hannigan, P. J., Goble, G. G., & Likins, G. E. (2006). Design and construction of driven pile foundations-Reference manual Vol. II. Federal Highway Administration, Pub. No. FHWA NHI-05-043.
- Hannigan, P. J., Goble, G. G., Likins, G. E., & Rausche, F. (2006). Design and construction of driven pile foundation, vol I. US Department of Transportation, Federal Highway Administration, publication no. FHWA NHI-05-042.
- Heerema, E. P. (1978, January). Predicting pile driveability: Heather as an illustration of the "friction fatigue" theory. In *SPE European Petroleum Conference*. Society of Petroleum Engineers.

- Henke, S., & Grabe, J. (2008). Numerical investigation of soil plugging inside open-ended piles with respect to the installation method. *Acta Geotechnica*, 3(3), 215-223.
- Hussein, M. H., McGillivray, R. T., & Brown, D. A. (2012, March). Knowledge is bliss—A case for supplemental pile testing to ascertain fidelity. In *Full-Scale Testing and Foundation Design: Honoring Bengt H. Fellenius* (pp. 322-332). ASCE.
- Jardine, R. J., Standing, J. R., & Chow, F. C. (2006). Some observations of the effects of time on the capacity of piles driven in sand. *Géotechnique*, 56(4), 227-244.
- Jardine, R. J., Thilsted, C. L., Thomsen, N. V., & Mygindt, M. (2015). Axial capacity design practice for North European wind-turbine projects. In *Proceedings of 3rd international symposium on frontiers in Offshore Geotechnics* (pp. 581-586).
- Jardine, R., Chow, F., Overy, R., & Standing, J. (2005). ICP design methods for driven piles in sands and clays (p. 112). London: Thomas Telford.
- Kelly, R. B., and P. K. Wong. "CASE STUDY: ASSESSMENT OF THE SHAFT CAPACITY OF A DRIVEN PILE IN SAND USING CPT BASED METHODS."
- Kikuchi, Y., Otani, J., Kimura, M., & Morikawa, Y. (Eds.). (2007). *Advances in Deep Foundations: International Workshop on Recent Advances of Deep Foundations (IWDPF07) 1–2 February 2007, Port and Airport Research Institute, Yokosuka, Japan*. Taylor & Francis.
- Lehane, B. M., & Jardine, R. J. (1994). Displacement pile behaviour in glacial clay. *Canadian Geotechnical Journal*, 31(1), 79-90.
- Likins, G. E., Rausche, F., & Goble, G. G. (2000, September). High strain dynamic pile testing, equipment, and practice. In *Proceedings of the Sixth International Conference on the Application of Stress-wave Theory to Piles* (pp. 327-333).
- Likins, Garland E., Rausche, Frank, Thendean, Gabriel, and Svinkin, Mark (1996). "CAPWAP Correlation Studies," *STRESS WAVE '96 Conference*, Orlando, Florida
- Lunne, T., Robertson, P. K., & Powell, J. J. M. (1997). *Cone penetration testing. Geotechnical Practice*.
- Matos Fernandes, M. (2006). *Mecânica dos Solos – Conceitos e Princípios Fundamentais*. FEUP Edições, Porto.
- Matos Fernandes, M. (2011). *Mecânica dos Solos – Introdução à Engenharia Geotécnica*. FEUP Edições, Porto.
- Mhaiskar, S. Y., Khare, M., & Vaidya, R. (2010, December). High Strain Dynamic Pile Testing and Static Load Test—A Correlation Study. In *Proc. Indian Geotechnical Conference 2010*.
- Randolph, M. F., Dolwin, J., & Beck, R. (1994). Design of driven piles in sand. *Geotechnique*, 44(3), 427-48.
- Randolph, M., & Gourvenec, S. (2011). *Offshore geotechnical engineering*. CRC Press.
- Rausche, F. (2004). *Non-destructive evaluation of deep foundations*.
- Rausche, F. (2008). PDA Testing: 2008 State of the art. In *The Application of Stress-wave Theory to Piles: Science, Technology and Practice: Proceedings of the 8th International Conference on the Application of Stress-Wave Theory to Piles: Lisbon, Portugal, 8-10 September 2008* (p. 395). IOS Press.

- Rausche, F., & Klesney, A. (2007). Hammer Types. In *Efficiencies and Models in GRLWEAP-PDCA 11th Annual International Conference and Exposition*, Nashville, TN (pp. 97-118).
- Rausche, F., Nagy, M., Webster, S., & Liang, L. (2009, January). CAPWAP and Refined Wave Equation Analyses for Driveability Predictions and Capacity Assessment of Offshore Pile Installations. In *ASME 2009 28th International Conference on Ocean, Offshore and Arctic Engineering* (pp. 375-383). American Society of Mechanical Engineers.
- Rausche, F., Robinson, B., & Seidel, J. (2000, September). Combining static pile design and dynamic installation analysis in GRLWEAP. In *Proceedings of the 6h International Conference on the Application of Stress-Wave Theory to Piles*, Rotterdam.
- Rimoy, S., Silva, M., Jardine, R., Yang, Z. X., Zhu, B. T., & Tsuha, C. H. C. (2015). Field and model investigations into the influence of age on axial capacity of displacement piles in silica sands. *Géotechnique*, 65(7), 576-589.
- Robertson, P. K., & Cabal, K. L. (2015). *Guide to cone penetration testing for geotechnical engineering*. Gregg Drilling & Testing, Inc. 6th Edition July 2015.
- RP2A-WSD, A. P. I. (2000). Recommended practice for planning, designing and constructing fixed offshore platforms—working stress design—. In *Twenty-*.
- Saraiva, João Pedro (2015). ASSESSMENT OF THE SOIL RESISTANCE DURING DRIVING IN OFFSHORE TUBULAR PILES
- Schneider, J. A., & Harmon, I. A. (2010). Analyzing Drivability of Open Ended Piles in Very Dense Sands. *DFI Journal-The Journal of the Deep Foundations Institute*, 4(1), 32-44.
- Smith, E. A. (1960). L., 1960. Pile Driving Analysis by the Wave Equation. *Journal of the Soil Mechanics and Foundations Division*, 35-61.
- Stevens, R. S., Wiltsie, E. A., & Turton, T. H. (1982, January). Evaluating Drivability for Hard Clay, Very Dense Sand, and Rock. In *Offshore Technology Conference*. Offshore Technology Conference.
- Toolan, F. E., & Fox, D. A. (1977). GEOTECHNICAL PLANNING OF PILED FOUNDATIONS FOR OFFSHORE PLATFORMS. *Proceedings of the Institution of Civil Engineers*, 62(2), 221-244.
- Wang, J. (1992). Prediction of pile drivability from CPT and WEAP analysis (Doctoral dissertation, University of British Columbia).
- Webster, S., & Robinson, B. (2013, June). Driveability Analysis Techniques for Offshore Pile Installations. In *ASME 2013 32nd International Conference on Ocean, Offshore and Arctic Engineering* (pp. V006T10A036-V006T10A036). American Society of Mechanical Engineers.
- White, D. J., & Lehane, B. M. (2004). Friction fatigue on displacement piles in sand. *Géotechnique*, 54(10), 645-658.
- Works, V. I. (2004). *vulcanhammer. info*.
- Yang, J., Tham, L. G., Lee, P. K. K., Chan, S. T., & Yu, F. (2006). Behaviour of jacked and driven piles in sandy soil. *Géotechnique*.
- Zaaijer, M. B. (2006). Foundation modelling to assess dynamic behaviour of offshore wind turbines. *Applied Ocean Research*, 28(1), 45-57.

APPENDIX

APPENDIX I – CPT BOREHOLE REPORT FOR P01; P02 AND P03

APPENDIX I

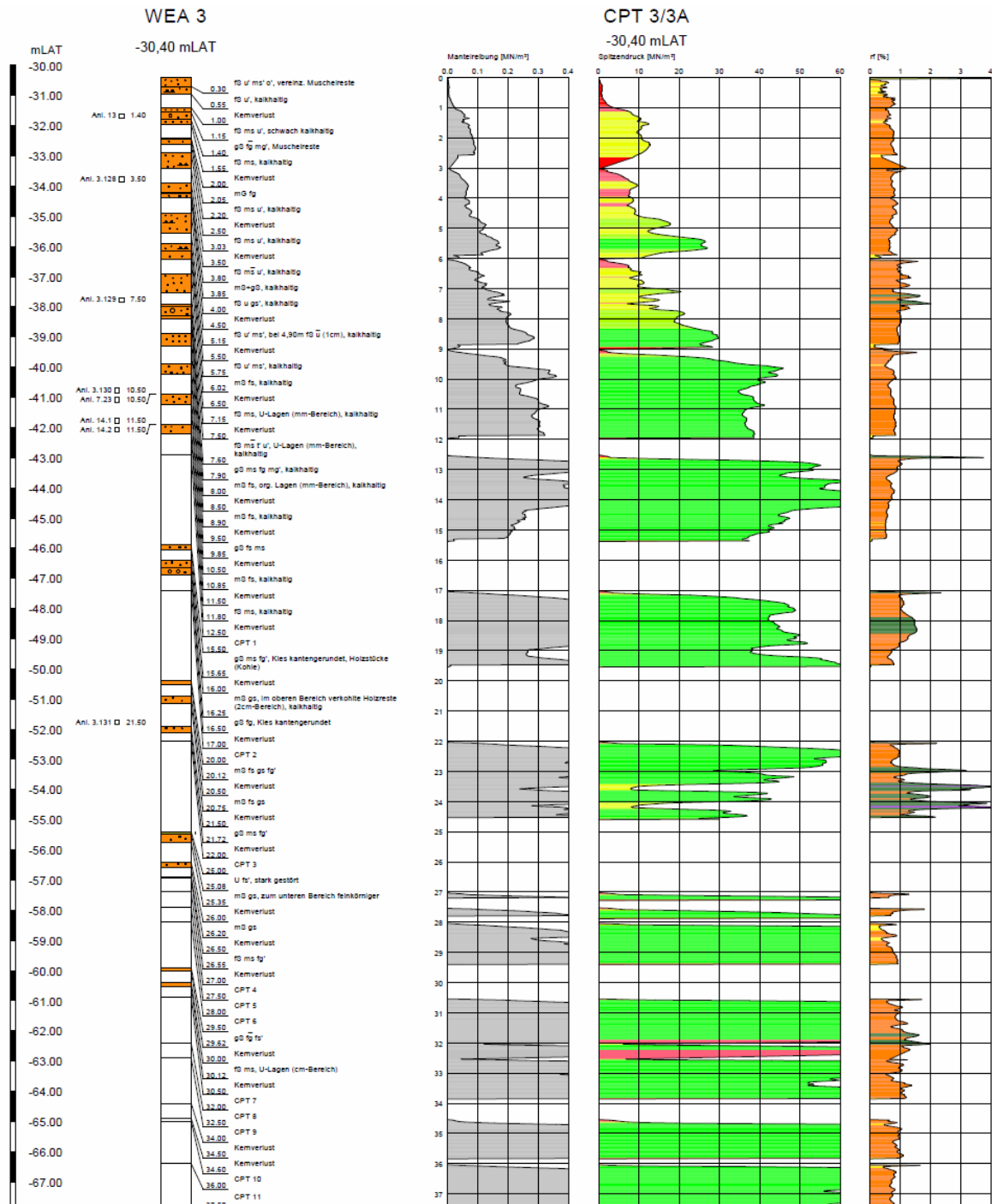
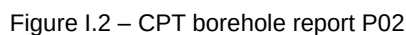


Figure I.1 – CPT borehole report P01



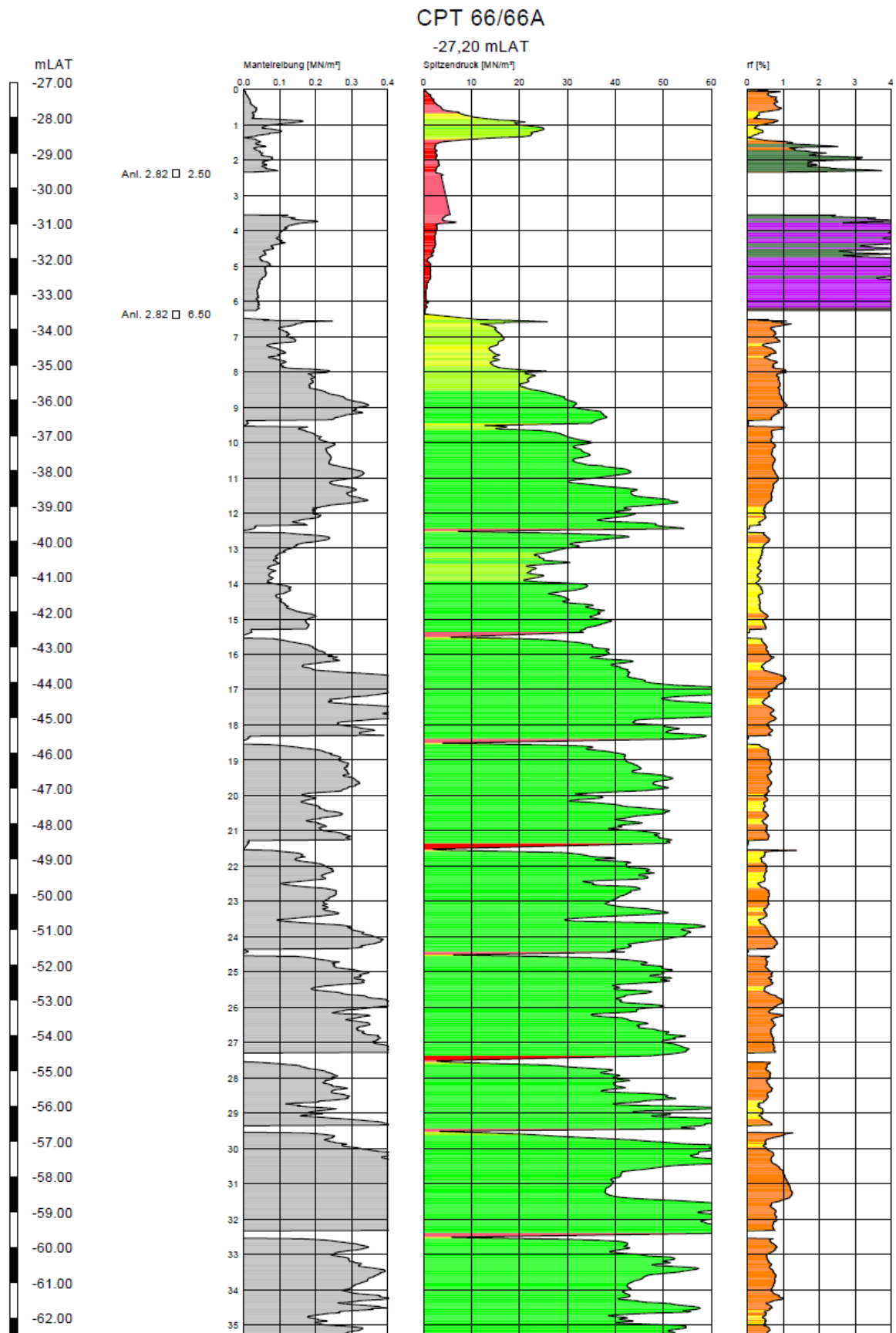


Figure I.3 – CPT borehole report P02e