

Numerical modelling of liquefaction in offshore sites

Beatriz Maria Correia de Magalhães de Oliveira Osório

**Dissertação submetida para satisfação parcial dos requisitos do grau de
Mestre em Engenharia Civil — Especialização em Geotecnia**

Orientador: Professor Doutor José Eduardo Tavares Quintanilha de Menezes

Coorientadora: Professora Doutora Cristiana Maria da Fonseca Ferreira

Coorientador: Doutor Giovani Li Destri Nicosia

Outubro de 2020

Mestrado Integrado em Engenharia Civil 2019/2020

Departamento de Engenharia Civil

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ miec@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 - 2019/2020 - Departamento de Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal, 2020.

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.

To my dear family and amazing friends.

*“Surround yourself with people who challenge you, teach you,
and push you to be your best self.”*

Bill Gates

ACKNOWLEDGEMENTS

First of all, I would like to thank, very deeply, my co-supervisor Professor Cristiana Ferreira for her incredible support, not only in the theoretical and knowledge-wise of the work but also for always being on my side, giving me confidence and encouragement In the difficult times. Thank you for always being available to answer my question and give a kind word.

To my adviser Professor José Quintanilha for the facilities granted for the development of this report.

To my co-supervisor at COWI, Geotechnical and Earthquake Engineer Giovanni Nicosia, for all the patience answering my many questions about this new branch of work that I had prior not a lot of knowledge about. Thank you for helping me to better understand earthquake engineer.

A special thank you to my colleagues and friends José Calejo and Ivan Almeida. José, for showing me the beautiful city of Copenhagen, whelping me with all the logistics that moving to another country and working in a company environment imply. To Ivan, thank you for accompanying me in this adventure, for all the support and contagious optimism that you gave me through out these months. I am grateful to have your friendship and to have had the opportunity to experience this challenge with you by my side.

To my family, for giving me the opportunity to experience new cultures and new challenges. For even far away manage to support and always be present when I most needed. For supporting and allowing me to continue this adventure far from you, even when the world's situation was critical, and no one knew what was going to happen next. Thank you for being my rock

And finally, thank you to my friends. For all the encouragement and love you give me every day, for even miles apart you guys managed to be present in the important moments and in the everyday routine. Thank you for your friendship.

Thank you all.

Um grande obrigado a todos.

ABSTRACT

Over the last years, new forms of producing energy in a more sustainable way have been subject of study. One of the most common ways of extracting energy with a low environmental impact is by converting wind power into electricity.

With the development of this area, a new market of offshore wind turbines has emerged, especially in Europe and Asia. In turn, some European companies give use to their many resources and knowledge to develop and install foundations for wind turbines in offshore sites. These sites overseas are the choice of election for the implementation of such structures because they are places where the environmental and social impacts are low, and the prices are more competitive. However, this choice also brings disadvantages, because in these offshore locations, there are a high number of extraordinary natural events, such as violent sea storms, cyclones, and earthquakes, which leads to a great challenge in the design of these structures.

Reliable predictions of soil behaviour are essential to ensure the good performance of these structures and this is the main concern of geotechnical engineers. The combo between loose soils with strong earthquakes leads to a high risk of liquefaction of those soils, that can originate hazards to the foundation structure and the wind turbine.

There is still a lot to learn about soil liquefaction but predicting the soil response to the phenomena has proven to be increasingly essential to the design of the foundations. For that, several methods have been proposed based on in situ tests, cyclic laboratory tests, and numerical analysis.

This project seeks to analyze methods already documented in the literature, to understand which could be the best suited to the offshore locations. By simulating simple shear laboratory tests, using Plaxis 2D software, in order to obtain compliant results with those reported in the literature.

For this purpose, the present study focused on two profiles. One simple idealized three-layered profile, with only two different types of soil, and a second one derived from a real offshore site soil profile, with several layers and different soils. To perform this analysis, the geotechnical software PLAXIS2D was used, in conjunction with advanced scripts in Python language.

KEYWORDS: LIQUEFACTION, CSR-N CURVES, CRR-N CURVES.

RESUMO

Ao longo dos últimos anos, novos métodos de produzir de energia de forma sustentável tem sido grande alvo de estudo. Uma das formas mais comuns de extrair energia com baixo impacto ambiental é convertendo a força do vento em energia elétrica.

Com o desenvolvimento desta área, um novo mercado de turbinas eólicas offshore tem emergido, especialmente na Europa e Ásia. Em resposta, algumas empresas europeias aplicam à sua vasta experiência para desenvolver e instalar fundações de turbinas eólicas em parques offshore. Estes locais em alto mar são a escolha de eleição para a implementação de tais estruturas, pois são locais onde os impactos ambientais e sociais são menores e os preços mais competitivos. Contudo, esta escolha também acarreta desvantagens, devido ao facto de serem locais onde há um elevado número de fenómenos naturais extraordinários como as tempestades marítimas violentas, ciclones e sismos, o que leva a um grande desafio no projeto das fundações destas estruturas.

Previsões confiáveis do comportamento do solo são essenciais para garantir o bom funcionamento destas estruturas e, esta é a principal preocupação dos engenheiros geotécnicos. A combinação de solos soltos com fortes sismos leva a um elevado risco de ocorrência de liquefação, que pode originar graves consequências para as fundações e até mesmo para a turbina.

Ainda há muito para aprender sobre a liquefação dos solos, mas prever a resposta do solo a este fenómeno tem-se provado cada vez mais essencial para o design destas fundações. Para isso, diversos métodos têm sido propostos com base em ensaios in situ, testes cíclicos laboratoriais e análises numéricas.

Este projeto procurou analisar métodos que já documentados na literatura e perceber qual o que melhor se adaptava a locais offshore. Simulando ensaios laboratoriais de corte simples, com recurso ao software PLAXIS2D, de modo a obter resultados compatíveis com aquele dado pela teoria.

Para este propósito, o estudo centra-se em dois perfis. Um perfil simples, idealizado com apenas dois tipos de solo e três camadas, e um segundo perfil que deriva de um local offshore real, com diversas camadas e diferentes solos. Para a realização desta análise foi utilizado o software PLAXIS2D em conjunto com a linguagem Python.

PALAVRAS CHAVE: LIQUEFAÇÃO, CURVAS CSR-N, CURVAS CRR-N.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	V
ABSTRACT	VII
RESUMO	IX
TABLE OF CONTENTS	XI
LIST OF FIGURES	XV
LIST OF TABLES.....	XXI
LIST OF ABBREVIATIONS AND SYMBOLS	XXIII
1 INTRODUCTION	1
1.1 CONTEXT AND OBJECTIVES	1
1.2 PERSONAL MOTIVATION.....	2
1.3 THESIS LAYOUT	3
2 LITERATURE REVIEW.....	5
2.1 OFFSHORE WIND FARMS	5
2.2 LIQUEFACTION.....	10
2.2.1 GENERAL CONSIDERATIONS	10
2.2.2 LIQUEFACTION SUSCEPTIBILITY FOR SANDS	12
2.2.3 PARAMETERS AFFECTING SOIL RESPONSE TO UNDRAINED CYCLIC LOADING	18
2.2.4 EFFECT OF FINES.....	23
2.3 EMPIRICAL CORRELATIONS TO ASSESS LIQUEFACTION POTENTIAL.....	25
2.3.1 SPT-BASED LIQUEFACTION ASSESSMENT.....	27
2.3.2 CPT	31
2.4 CONSTITUTIVE MODELLING OF LIQUEFIABLE SOILS.....	38
2.4.1 INTRODUCTION.....	38
2.4.2 HARDENING SOIL MODEL.....	39
2.4.3 PM4SAND SOIL MODEL.....	42
3 SENSITIVITY STUDIES ON DIFFERENT LIQUEFACTION APPROACHES.....	55
3.1 INTRODUCTION	55
3.2 CRR _N FROM SPT	55

3.2.1	CLEAN SANDS	55
3.2.2	SANDS AND SILTY SANDS	57
3.3	CRR_N FROM CPT	60
3.3.1	CLEAN SANDS	60
3.3.2	SANDS AND SILTY SANDS	64
3.4	COMPARISONS	65
3.5	SILTS AND CLAYS.....	71
3.6	CALIBRATION OF THE MODEL DAMPING	ERROR! BOOKMARK NOT DEFINED.
3.7	CONCLUDING REMARKS.....	72
4	NUMERICAL SIMULATION.....	75
4.1	INTRODUCTION	75
4.2	SIMULATION OF CSR-N RESPONSE WITH THE PM4SAND MODEL	79
4.2.1	ORIGINAL PARAMETERS.....	79
4.2.2	INFLUENCE OF DIFFERENT RELATIVE DENSITIES DR.....	83
4.3	PRESENTATION OF THE TIME HISTORIES.....	85
4.4	SOIL TESTS SIMULATION	86
4.4.1	PROFILE 1: IDEALIZED PROFILE	86
4.4.2	PROFILE 2: REALISTIC PROFILE OF AN OFFSHORE SITE.....	88
4.4.3	PARAMETRIC ASSESSMENT	89
4.4.4	FITTING CURVES	91
5	SITE RESPONSE SIMULATIONS.....	97
5.1	INTRODUCTION	97
5.2	1D WAVE PROPAGATION ANALYSIS WITH PM4SAND MODEL	97
5.2.1	PROFILE 1: IDEALIZED PROFILE	97
5.2.2	PROFILE 2: REALISTIC PROFILE OF AN OFFSHORE SITE.....	99
5.2.3	PLAXIS	101
5.3	PYTHON EXPERIENCE	103
5.3.1	SCRIPTS.....	104
6	FINAL REMARKS.....	111
6.1	MAIN CONCLUSIONS.....	111
6.2	FUTURE DEVELOPMENTS.....	112
APPENDIX.....		113
A.....		114

TIME HISTORIES.....	114
REFERENCES	121

LIST OF FIGURES

Figure 2.1 Windmills.	6
Figure 2.2 Offshore wind farms (Getty, 2019).	7
Figure 2.3 Onshore wind farms(a); Offshore oil and gas facilities, (b).....	8
Figure 2.4 Common foundation types used in offshore wind turbine design: mono-pile (a), monopods or gravity-based foundations (b), jacket structure (c), tripod (d) and wind turbine anchors (e) (Kaynia, 2018).	9
Figure 2.5. Niigata Earthquake, 1964 (Ungtss, 1964).	10
Figure 2.6 The effect of contraction and dilation after shearing. And Deviatoric stress per axial strain (a), and per void ratio (b) (COWI, Liquefaction susceptibility, 2019).....	13
Figure 2.7 Typical triaxial test results of two samples of a sand, one of them dense and the other one loose, correlating the axial deformation to the void ratio. (adapted from Taylor, 1948; (Matos Fernandes, 2011)).	13
Figure 2.8 Use of the CVR line as a boundary between loose contractive states and dense dilative states (Kramer, 1996).....	14
Figure 2.9 Behaviour of initially loose and dense specimens under drained and undrained conditions for (a) arithmetic and (b) logarithmic effective confining pressure scales (Kramer, 1996).	14
Figure 2.10 Three-dimensional steady-state line showing projections on e - τ plane, e - σ' plane, and τ - σ' plane. (Kramer, 1996).	15
Figure 2.11 Definition of state parameter (Been and Jefferies, 1985).	16
Figure 2.12 Effective stress paths in the p' - q plane typically measured in undrained monotonic triaxial tests on loose, medium dense sand samples (Vaid et al., 1981).....	16
Figure 2.13 Response of five specimens isotropically consolidated to the same initial void ratio at different initial effective confining pressures. Flow liquefaction in specimens C, D, and E is initiated at the points marked with an X. The dotted line passing through these points is a line of constant principal effective stress ratio, KL (Kramer, 1996).....	17
Figure 2.14 Orientation of the flow liquefaction surface in stress path space.	17
Figure 2.15 Schematic illustration of typical curves $q(\epsilon_1)$ (Vaid and Sivathayalan, 2000) and $q(p)$ in cyclic triaxial tests on: a) very loose, b) loose and c) medium dense or dense sand (Wood, 2012).	18
Figure 2.16 Influence of void ratio on the number of cycles to a) initial liquefaction and b) a strain amplitude $\epsilon_1^{ampl} = 20\%$, undrained cyclic triaxial tests of Lee and Seed (1967). (Wood, 2012). ..	18
Figure 2.17 Increase of the liquefaction resistance with increasing density: tests of a) Mori et al. (1978) and b) Tatsuoka et al. (1986).	19

Figure 2.18 The use of an amplitude-pressure ratio $q_{amp}/2p_0'$ on the ordinate purifies the data from the influence of the initial effective mean pressure p_0' (Lee and Seed, 1967).....	20
Figure 2.19 Correction factor K_σ for a consideration of the decrease of the cyclic stress ratio $*q_{amp}/(2p_0')$ causing liquefaction with increasing pressure p_0' (Ishihara, 1995).....	20
Figure 2.20 Influence of a static initial shear stress on the liquefaction resistance depends on a) the chosen failure criterion (Vaid and Finn, 1979) and b) on soil density (Rollins and Seed, 1990).	21
Figure 2.21 Influence of a static initial stress on the liquefaction resistance depends on initial mean effective stress p_0' (Vaid and Chern, 1985).	22
Figure 2.22 Influence of the specimen preparation method on the liquefaction resistance, data of Mulilis et al. (1975).	22
Figure 2.23 Comparison of the liquefaction resistance of undisturbed samples obtained by ground freezing from a natural sand deposit and from a freshly deposited (artificial) sand layer (Yoshimi et. Al., 1989).	23
Figure 2.24 Critical state line of sand-silt mixtures in terms of void ratio, e (Papadopoulou & Tika, 2008).	24
Figure 2.25 Graphical representation of the proposed liquefaction susceptibility criteria: a) isotropically consolidated CTX testing data from this study (Bray & Sancio, 2006).	25
Figure 2.26 Representative relationship between CSR and Number of cycles to Cause Liquefaction (Reproduced from Seed and Idriss 1982) (Youd & Idriss, 2001)	27
Figure 2.27 Equivalent clean sand adjustments for SPT-based liquefaction triggering procedures.	30
Figure 2.28 Variation In the MSF relationship with qc_{1Ncs} and N_{160cs} for cohesionless soils.	30
Figure 2.29 Variation In the MSF relationship with qc_{1Ncs} and N_{160cs} for cohesionless soils.	31
Figure 2.30 Terminology for cone penetration (Mayne, 2018).....	32
Figure 2.31 Curve recommended for calculation of CRR from CPT data along with empirical liquefaction data from compiled case histories (reproduced from Robertson and Wride, 1998) (Youd & Idriss, 2001)	33
Figure 2.32 Variation in the MSF relationship with qc_{1Ncs} and N_{160cs} for cohesionless soils (Boulanger & Idriss, CPT - Based Liquefaction Triggering Procedure, 2016).....	35
Figure 2.33 Variation in the MSF relationship with qc_{1Ncs} and N_{160cs} for cohesionless soils.	35
Figure 2.34 Ratio of N_{clay} to N_{sand} for NEHRP class D soil sites, M_w of 7 to 8, and $r_e = 0.65$ (Boulanger & Idriss, 2007).....	37
Figure 2.35 MSF relationships for clay and sand (Boulanger & Idriss, 2007).	38

Figure 2.36 Results from the Hardin-Drnevich relationship compared to test data by Santos & Correia (2001),.....	40
Figure 2.37 Secant and tangent shear modulus reduction curve.	41
Figure 2.38 Stiffness parameters E_{50} , E_{ur} , and $E_0 = 2G_0(1 + \nu_{ur})$ of the Hardening Soil model with small-strain stiffness in a triaxial test.	41
Figure 2.39 Stiffness parameters in cyclic shear test.	42
Figure 2.40 Definition of the relative state parameter index (Tolosa, 2018).	44
Figure 2.41 Yield, critical, dilatancy and bounding lines in q - p space, on the left, and yield, dilatancy and bounding surfaces in the r_{yy} - r_{xy} stress-ratio plane on the right, (Tolosa, 2018).	45
Figure 2.42 Dilatancy, D , calculation based on the stress state with respect to M_dR , M_d and M_b surfaces during half-cycle of loading, from contraction to dilation (Boulanger & Ziotopoulos, 2015).	53
Figure 3.1 CRR-N curves of clean Sands based on Y&I (2001) methodology for SPT test results of $N_{160} = 15, 20$ and 25	56
Figure 3.2 CRR-N curves of Sands and Silty sands based on Y&I (2001) methodology for SPT test results of $N_{160} = 15$ and 20 (top and bottom respectively).	58
Figure 3.3 CRR-N curves of Sands and Silty sands based on B&I (2014) methodology for SPT test results of $N_{160} = 15$ and 20 (top and bottom respectively)	60
Figure 3.4 CRR-N curves of clean Sands based on Youd & Idriss (2001) methodology for SPT test results of $qc_{1Ncs} = 94.34$ and 153.09	62
Figure 3.5 CRR-N curves of clean sands based on Boulanger (2016) methodology for SPT test results of $qc_{1Ncs} = 91.85$ and 153.09	63
Figure 3.6 CRR-N curves of sands and silty sands based on Boulanger & Idriss (2014) methodology for CPT test results with fines content of 0% and 20%	65
Figure 3.7 Comparison between CRR-N curves of clean sands versus sands and silty sands based on Y&I (2001) methodology for SPT of $N_{160} = 15$ and 20	66
Figure 3.8 Comparison between CRR-N curves of Clean Sands Vs Sands and Silty Sands based on B&I(2016&2014) methodology for CPT with fines content of 0% and 20%	67
Figure 3.9 Comparison between Y&I(2001) Vs B&I(2014) methodologies for CRR-N curves of sands and silty Sands for SPT test results.	68
Figure 3.10 Comparison between Y&I (2001) Vs B&I(2016) methodologies for CRR-N curves of clean Sands for CPT test results.	68
Figure 3.11 Comparison Y&I (2001) methodology of clean sands between SPT versus CPT test results.	69

Figure 3.12 Comparison B&I (2014) methodology for sands and silty sands between SPT versus CPT test results.....	70
Figure 3.13 Plot of MSF curves by different approaches.	71
Figure 3.14 CRR-N curve for Silts and Clays.	72
Figure 3.15 Comparison between the results obtained with PLAXIS and the theoretical formula.	Error! Bookmark not defined.
Figure 3.16 B&I (2014) target curve with $Dr=54,5\%$ and $FC=10,1\%$	73
Figure 4.1 Layout of the PLAXIS2D Soil Test facility: cyclic direct simple shear (CDSS) tests.....	76
Figure 4.2 Flowchart of the process for deriving the target CSR-N curves using Plaxis with different hp_0 by comparison with the theoretical curve from Boulanger & Idriss (2014).	77
Figure 4.3 CSR-N relationship for case A and B material sets (modified from (Boulanger & Ziotopolou, PM4SAND (VERSION 3.1): A Sand plasticity model for earthquake engineering application, 2017)), (Vilhar, Brinkgreve, & Zampich, 2018).	81
Figure 4.4 Plots $\gamma_{xy}-\tau_{xy}$, $\gamma_{xy}-p_w$ and $\sigma_{yy}-\tau_{xy}$	82
Figure 4.5 Comparison between the results of the exercise using PLAXIS PM4Sand model and the expected results.	83
Figure 4.6 Results of the simulation with $Dr = 41,5\%$, (a), and $Dr = 55\%$, (b).	84
Figure 4.7 Plots of acceleration, velocity and displacement of the Imperial Valley time history.	86
Figure 4.8 Target curves for the sand layers of Profile 2: silty sand 1, $Dr=50.4\%$ (a); silty sand 2, $Dr=78.1\%$ (b); silty sand 3, $Dr=60.99\%$ and silty sand 4, $Dr=60.8\%$ (c).	89
Figure 4.9 CRR-N curves for different apparent relative density Dr and hp_0 (Toloza, 2018).....	90
Figure 4.10 CRR-N curves for different values of bounding surface parameter nb (Toloza, 2018).	91
Figure 4.11 CRR-N curves obtained with PLAXIS for $Dr_0 = 55\%$ and different hp_0 in comparison with the theoretical curve from B&I 2014.....	92
Figure 4.12 CRR-N curves obtained with PLAXIS with $Dr_0 = 55\%$ and different hp_0 and nb compared with theoretical curve from Boulanger 2014.....	92
Figure 4.13. CRR-N curves obtain with Plaxis Soil Test simulation, with $Dr_0 = 55\%$ $hp_0 = 0.4$ and $nb = 1$ compared with theoretical curve from Boulanger 2014.	93
Figure 4.14. CRR-N curves obtain with PLAXIS Soil Test simulation for Profile 2: silty sand 1, $Dr=50.4\%$ (a); silty sand 3 and 4, $Dr=60.99\%$ and $Dr=60.8\%$ (b).....	94
Figure 5.1 Profile 1 – idealized profile.	98

Figure 5.2 Profile 2 from a real offshore site: stratigraphy details (a), Plaxis representation (b).	100
Figure 5.3 Flowchart of how script <i>Generate</i> works.	105
Figure 5.4 Flowchart of how the <i>Post-processing</i> script works.....	106
Figure 5.5. Envelope results, taken with the python script, of time histories Izmit, on the top, and Landers Coolwater, on the bottom. From the left to the right: Excess pore pressure [kPa], Deviatoric Strain and, Horizontal acceleration [m/s ²].	107
Figure 5.6. Comparison between the envelop of the processed results taken adopting the Python scripts provided, (a) and (c), and the results taken directly from the output of Plaxis, (b) and (d). For the each time history: Izmit, (a) and (b), and Landers Coolwater, (c) and (d).	108
Figure 5.7 CSS versus CRR: a) Izmit; b) Landers Coolwater	109
Figure 5.8 Effective vertical stress of time history: a) Izmit; b) Landers Coolwater.	110

LIST OF TABLES

Table 2.1 Relationships for α and β are coefficients.....	29
Table 2.2 Normalized CPT Soil type index intervals (Robertson, 1990&2010)	36
Table 2.3 Number of cycles conversion from Sands to Clays.	37
Table 2.4 Stiffness parameters.	39
Table 2.5 Small strain stiffness parameters.	39
Table 3.1 Equivalent number of uniform stress cycles (Kramer, 1996).....	55
Table 3.2 Soil parameters for CRR-N curves for SPT Youd & Idriss (2001) approach.....	56
Table 3.3 Soil calculations for CRR-N curves for SPT Youd & Idriss approach.....	57
Table 3.4 Soil calculations for CRR-N curves for SPT Boulanger & Idriss (2014) approach.	59
Table 3.5 Suggested q_c/P_{aN60} ratios (Robertson, 2010).....	61
Table 3.6 Soil parameters and calculations for CRR-N curves for CPT Youd & Idriss approach.....	61
Table 3.7 Soil parameters and calculations for CRR-N curves for CPT Boulanger (2016) approach.....	63
Table 3.8 Soil parameters and calculations for CRR-N curves for Silty Sands: CPT B&I (2014) approach.	64
Table 3.9 Soil parameters and calculations for CRR-N curves for Silts and Clays: B&I (2007).....	72
Table 3.10 Linear Elastic layer properties.	Error! Bookmark not defined.
Table 3.11 Bedrock layer properties.	Error! Bookmark not defined.
Table 4.1 The calibrated parameters.	80
Table 4.2 The parameters with default values.	80
Table 4.3 The values of the contraction rate parameter h_{p0} for two cases A and B.	80
Table 4.4 Test conditions.....	81
Table 4.5 CSR-N values at single amplitude shear strain $\gamma = 3\%$ for Case A and B.	82
Table 4.6 Results of Exercise 1 with $D_r = 41,5\%$ on the top and $D_r = 55\%$ on the bottom.	85
Table 4.7 Time histories.	85
Table 4.8 PM4Sand primary parameters of sand.	87
Table 4.9 PM4Sand secondary parameters of sand.....	87
Table 4.10 Sand parameters provided by COWI for the offshore profile.	88
Table 4.11 Parameters for the sands of Profile 2.	95
Table 5.1 HSsmall parameters for the clay layers.....	98

Table 5.2 Bedrock Linear Elastic parameters.	99
Table 5.3 Clay parameters provided by COWI for the offshore profile.....	100
Table 5.4 Input parameters of the soil column for the Python Script.	104

LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations

CPT(U)	Cone penetration test (Piezocone)
CRR	Cyclic Resistance Ratio
CSL	Critical State Line
CSR	Cyclic Stress Ratio
CVR	Critical Void Ratio
D	Drained
FC	Fines Content
FLS	Flow Liquefaction Surface
MSF	Magnitude Scaling Factor
OCR	Over consolidation ratio
OWT	Offshore Wind Turbine
OWF	Offshore Wind Farm
QSS	Quasi Steady State
SPT	Standard penetration test
SS	Steady state
UD	Undrained

Symbols

a	Acceleration
C_{2D}	Correction for two-dimensional versus one-dimensional cyclic loading

C_D	Corrective coefficient for SPT test result to take into consideration the diameter of the borehole
C_E	Corrective coefficient for SPT test result to take into consideration the energy
C_N	Corrective coefficient for SPT test result to take into consideration the vertical effective stress
C_R	Corrective coefficient for SPT test result to take into consideration the length of the sample tube.
C_{rev}	Coefficient to avoid over-stiffness
D	Dilatancy, damping ratio
D_R	Apparent relative density
$D_{R,cs}$	Relative density at the critical state line
D_{50}	Mean grain size
e e_{min} e_{max}	Void ratio, minimum and maximum void ratio
e_c	Critical Void Ratio
E	Young's modulus
E_R	Energy ratio
E_{50}^{ref}	Plastic straining due to deviatoric loading / Secant stiffness in standard drained triaxial test
E_{oed}^{ref}	Plastic straining due to primary compression / Tangent stiffness for primary oedometer loading
E_{ur}^{ref}	Elastic Unloading/reloading stiffness at engineering strains ($\varepsilon \approx 10^{-3}$ to 10^{-2})
F	Robertson parameter
f	Frequency
F_s	Factor of safety
g	Acceleration of gravity
G	Shear modulus
G_0	Shear modulus coefficient
G_t	Tangent shear modulus
h	Thickness

h_{po}	Contractive Rate Parameter
H	Height
I_c	Soil behaviour type index
K_a, K_c	Correction factor
$K_\sigma(p'_0)$	Pressure dependent correction factor
K_p	Plastic modulus
L	Loading index
LL	Liquid limit
m	Power for stress-level dependency of stiffness
M	Magnitude
M^b	Bounding surface
M^d	Dilatancy surfaces
n	Generic exponent; Porosity, Deviatoric unit normal to the yield surface
n^b	Bounding surface parameter
n^d	Dilatancy surface parameter
N_{60}	SPT test results corrected for an energy of 60%
$(N_1)_{60}$	N_{60} corrected for an effective vertical stress of 1 atmosphere
N_{SPT}	Number of blows in the SPT test
p'	Mean effective stress
p'_0	Initial effective mean pressure/ effective consolidation pressure
p_A	Atmospheric pressure (reference pressure)
q_c	Tip resistance of CPT
q_{c1N}	q_c for effective total stress of 1 atmosphere
q_{c1Ncs}	Clean-sand tip resistance of CPT
Q	Robertson parameter, Bolton parameter, Critical state parameter
q^{ampl}	Stress amplitude
r	Deviatoric stress ratio tensor
r_d	Shear stress reduction coefficient
R	Bolton parameter, Critical state parameter
R^2	Correlation coefficient
S	Parameter

u	Pore water pressure
Δu	Increment of pore water pressure
V	Volume
V_P	Compression wave velocity
V_S	Shear wave velocity
W	Weight
w, w_c	Water content
z	Depth
α	Static stress ratio / the position of the yield surface in the deviatoric stress ratio space, Rayleigh parameter, coefficient
α^b	stress ratio that define the bounding surface
α^d	stress ratio that define the dilatancy surface
α_{in}	Initial back-stress ratio
α_{in}^{app}	Apparent back-stress ratio
α_{in}^{true}	True back-stress ratio tensor
β	Rayleigh parameter, coefficient
Δ	Increment, difference
ε_a	Axial strain
ε_s	Shear strain
ξ_R	Relative state parameter index
φ_{cv}	Critical state effective friction angle at a constant volume
φ_{pk}	Peak friction angle of the shearing resistance
$\gamma_{0.7}$	Shear strain level, at which the secant shear modulus, G_s , is reduced to about 70%
γ^{amp}	Shear strain amplitude
σ_{0v}, σ_{0h}	Initial vertical and horizontal total stresses
$\sigma'_{0v}, \sigma'_{0h}$	Initial vertical and horizontal effective stresses
σ_v, σ_h	Vertical and horizontal total stresses
σ'_v, σ'_h	Vertical and horizontal effective stresses
$\sigma'_x, \sigma'_y, \sigma'_z$	Effective stresses in the X, Y and Z directions
σ_{vc}	Vertical effective consolidation stress

ν	Poisson's ratio
τ	Shear stress
τ_0	Shear stress required for the static equilibrium of a soil mass
τ^{av}	Static shear stress
τ_{cyc}	Cyclic stress ratio
τ_{liq}	Shear strength of the soil in its liquefied state
τ_s	Horizontal static shear stress
ψ	Dilatancy angle

1 INTRODUCTION

1.1 CONTEXT AND OBJECTIVES

With the expansion of offshore wind farms in earthquake prone regions of the world, design of offshore wind farms has been confronted with new challenges. Soil liquefaction and the reduction of lateral support is one of the most important especially for foundations, such as monopiles, relying on the soil lateral support.

The development of constitutive models able to replicate the stress strain path in stress controlled cyclic tests where soils liquefaction and cyclic softening is included and their implementation in commercially available and user-friendly finite elements and finite difference packages, has created the conditions for practicing engineers to develop numerical models including soil liquefaction.

Challenges remain, including, but not limited to the following. Understanding all relevant aspects influencing soil liquefaction and cyclic softening even of a small volume of soil is complex and still subject of research. Characterizing the soil profile of each and every wind turbine in an offshore wind farm is therefore conceptually challenging and would in practice also require a vast amount of cyclic laboratory tests. In this work this challenge is addressed by describing how reference CRR/N for coarse grained and fine-grained soils can be developed for a given soil profile including potentially liquefiable layers.

Computationally challenges are the large number of curve fitting calibration required, building a large number of numerical finite element/finite difference models, running a large number of time histories and post processing poses also a challenge. In this work, this challenge is addressed by using PLAXIS in conjunction with Python scripts.

1.2 PERSONAL MOTIVATION

Regarding my personal motivation for this thesis, many factors had an important role in the decision on what I would do of it. The most important one was the fact that I wanted to experience, in this stage of my learning process, a company environment, where I could see how the everyday life of an engineer is.

I also always wanted to have a learning and living experience outside my home country, because I think that it is extremely important to observe and learn how different people from different cultures have different perspectives and methods to solve one problem.

After some searching, I learned that COWI A/S had history of accepting student to develop their master's thesis and I was lucky to be accepted to do this project.

The work that will be presented in this report is part of the final project of the postgraduate academic master's degree at the Faculty of Engineering of University of Porto (FEUP) in the field of specialization of Geotechnical Engineering.

1.3 THESIS LAYOUT

This document is organized into 6 chapters, where in the first it is possible to find the contextualization and objectives, and the personal motivation for the development of this thesis.

Chapter 2 is the state of art of this dissertation where firstly there is a short summary about offshore wind industry, followed by some considerations about liquefaction and what influences it. A scrutinized presentation of the empirical correlations most commonly used to access liquefaction and, finally, the presentation of the constitutive models of liquefiable soils.

In the first part of Chapter 3 there were made sensitivity studies on different liquefactions approached to understand which would make more sense to use in the present work. The results of these studies were compared, and it was concluded what curve, provided by the literature would be more interesting to used. In the end of the chapter the calibration of the model was presented.

In Chapter 4 numerical simulations were made to obtain the CSR-N curve, that corresponded to the one given by the literature in the previous chapter, using the model PM4Sand and the software Plaxis. This process was made both for the idealized profile and the realistic one of an offshore site, presented in the beginning of the current chapter.

In Chapter 5 soil profiles that will be subjected to analysis were presented, the Plaxis process was explained, and the python scripts were detailed. Also, site response simulations and results were presented.

Chapter 6 was reserved for conclusions and future developments.

2 LITERATURE REVIEW

2.1 OFFSHORE WIND FARMS

Throughout the history, in all the multiplicity of forms of human societies that humankind has created, the development of techniques that directly apply to problem solving has been sought, in a constant struggle, but that have allowed humankind to evolve.

Over time, it has, naturally, translated in progress of science. Initially connected to the survival of individuals and its communities, the matters to solve became more and more diverse what imply an increase on the complexity of the problems to solve, as the solutions that, in each era, humankind, with art and ingenuity, has managed to build. Humans are, therefore, builders that use architecture and engineering as their main resource.

Early, humans discovered the power of mother nature, such as rivers, oceans, wind, gravity, etc., and that they could use then for its own advantage. The wind power has been used by humans for centuries. Two of the most known and ancient ways that the humankind has used wind power are: by harness it in their favour to navigate through all the seas, and the other is to supply their cities with food and water.

First information about civilizations that used windmills goes back to the Persians in A.D. 500-900 and the Chinese in A.D. 1200 that used them for pumping water and grind seeds.

The windmill is a structure that converts wind power into rotational energy by means of vans or blades that are pushed by the wind. Their times were throughout the mediaeval and early modern eras where they were essential to the development and prosperity of modern civilization. Figure 2.1 presents an example of a windmill.



Figure 2.1 Windmills.

But this mediaeval technology was eventually replaced by the steamed engines that were fuelled by combustible materials such as coal, wood or oil. The industrial revolution was the beginning of a new era of consumerism, and with that there was the need generate more and more energy.

For much of the twentieth century, there was no interest in using renewable energies; however, with the beginning of the oil crisis in the 70s that led to a drastic increase of the oil price. Governments from all around the world started to fund research programs on how to use wind for generation of electricity.

In 1991, the first offshore wind farm was constructed at Vindeby, Denmark, consisting of eleven, 450 kW wind turbines located up to 3 km offshore (Kaynia, 2018). Figure 2.2 presents an example of an offshore wind farm. Throughout the 90s small number of offshore wind turbines (OWT) were placed close to shore, until 2002, when the Horns Rev was constructed at about 20 km off the western coast of Denmark. After that, the OWT's have kept increasing in size over the years.

With the 2000s, the environmental concerns grew bigger and, lowering the CO₂ emissions became a global agenda. The potential of wind energy to help slowing down climate change was big, but it was difficult to use these renewable energies for transportation and heating. The European Union declared that until 2020, 20% of all energy should originate from renewable sources and 32% by 2030, and more energy-related policies continue to be developed every year.



Figure 2.2 Offshore wind farms (Getty, 2019).

The onshore wind turbines were the first form of harvesting wind power for energy. However, on land, space is limited and can be more expensive and, since wind farms need greater spaces of implementation closer to big cities, going on sea seems an interesting alternative depending on the cost. Since the surface of the sea is smoother than on land and the wind is less turbulent, a greater and more reliable power production can be achieved there. On the sea, it is also possible to have lower elevations and larger turbines.

Offshore wind farms are born from the fusion between the industries of onshore wind and offshore oil and gas, presented in Figure 2.3. The development of these two industries alone and the knowledge that they gather, allowed the offshore wind industry to develop and prosper. In Figure 2.3 is presented each one of these industries.

a)



b)



Figure 2.3 Onshore wind farms(a); Offshore oil and gas facilities, (b).

For the offshore wind, and comparing with the onshore, taking a look at the loads that need to be considered, besides the designed to withstand the sea wind loads that are already greater than on land, there are the loads due to the waves and currents, that will become more severe with the increase of water depth and as we move further offshore.

Also, the piece of the support structure that links the waterline with the soil below will greatly increase the lever arm of the wind-induced moments, leading to a considerable rise in the maximum loads that have to be transferred to the soil by the foundation.

Moreover, it is important to consider in the design the support structures for maintenance, that in the offshore case will become increasingly necessary. This was an issue that the oil and gas offshore industry had already to deal with. But there are, again, some differences between these two industries. For instance, the wind loads are generally not determinant for oil platforms, but they are for offshore wind turbines. Specifically, the dynamic response of the support structures to the wind loading should also be considered along with the static, something that does not occur in oil and gas platforms.

Looking into an oil and gas facility, we are talking about one structure alone with its corresponding foundation. On the other hand, offshore wind farms consist of large groups of wind turbines, around 100 per farm, where each one of them has its own foundation, requiring 100 foundations in total. As it is easily understood, it would be impossible to design a specific foundation of each location separately, and so it becomes necessary to cluster the turbines into groups, each one characterized by a single foundation design.

As offshore wind turbines have a variety of options for their foundation depending on some requirements, in Figure 2.4 are represented the most common types of foundation used in the design of these kinds of structures. The most common is mono-pile (Figure 2.4 (a)), they can be installed in up to 40 meters water depth depending on the soil type; Monopods or gravity-based foundations (Figure 2.4 (b)), jacket structures, tripods (and tripile) (Figure 2.4 (c), and (d) respectively) are usually used for deeper waters because of the lower weight of the supporting structures; and, the floating wind turbine anchored to the seabed, used for much deeper depths since its structure is more economic (Figure 2.4 (e)).

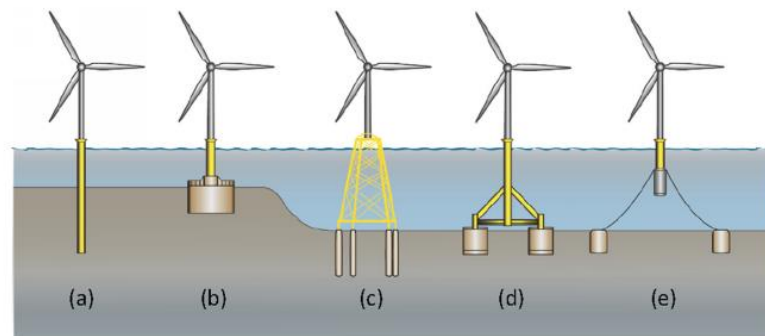


Figure 2.4 Common foundation types used in offshore wind turbine design: mono-pile (a), monopods or gravity-based foundations (b), jacket structure (c), tripod (d) and wind turbine anchors (e) (Kaynia, 2018).

Monopiles are by far the most common foundation type for these structures, it is simply a cylindrical steel tube driven into the seabed, using a hydraulic impact hammer or vibratory device. It relies mainly on the lateral resistance of the soil to transfer the loads to the seabed. As opposed to the mono-pile foundation, which relies on the restoring moment provided by the self-weight of the foundation and the turbine, to face the overturning moments. They are usually constructed on land and then transported to the offshore location. And due to the crane lifting capacities, these foundations are usually designed to be hollow, so they are light for transport and then filled with ballast material in situ. Although this last type of foundations is less effective at sea than on land, they remain the second most popular choice offshore.

The jacket is the third most common foundation type, and it is a lighter alternative against the two exposed before. The wide separation of the legs allows higher lever arms that resist the loadings of the overturning moments. Although there is a disadvantage in using this type of foundation, it requires expensive fabrication and welding of many geometrically complex joints.

Finally, we have tripods and tripiles, the concept behind these foundations is that they are structurally more efficient than monopiles in resisting the overturning moments. But are they not

much used, only around 5.2% of the cases, because their installation and fabrication are significantly more expensive (COWI, 2020). Another important thing to study when designing these structures is the soil that they are going to be founded in. While the initial development of offshore wind turbines has been concentrated in northern Europe where seismicity is low, in the last decade or so, the development of OWT has been in the agenda of several earthquake-prone regions and countries posing new challenges to the design. One of the most relevant challenges in the seismic design of OWT is indeed soil liquefaction, which can lead to the loss of soil bearing capacity to depths up to 20 meters. Although this is a matter to be alert for, it is important to underline that seismic loads, even with liquefaction, are usually not designed driving for monopiles and jackets, but instead storm loads are (COWI, 2020).

2.2 LIQUEFACTION

2.2.1 GENERAL CONSIDERATIONS

Liquefaction is one of the most important, interesting, and controversial topics in geotechnical earthquake engineering. Its devastating effects triggered the attention of geotechnical engineers in 1964, at the time of two earthquakes in Alaska and Japan (Kramer, 1996). Figure 2.5 shows an example of liquefaction after Niigata Earthquake in 1964. The term was first used by Magami and Kubo (1953), and it is a term used to describe soil deformations, which lead the soil to behave as a liquid caused by different types of disturbance in saturated cohesionless soils under undrained conditions. The recovery of the soil may take a while depending on the soil permeability and residual stiffness.



Figure 2.5. Niigata Earthquake, 1964 (Ungtss, 1964).

This phenomenon is most often observed in saturated loose sands, although sands are usually known to be in drained conditions, the fact that it occurs very rapidly allows that the conditions could be considered undrained. So, when a saturated soil, under undrained conditions, is loaded rapidly, either by monotonic, transient or repeated disturbance, excess pore pressure is generated leading to an increase that can equalize the total pressure, with the decrease of effective stress that

could turn into zero (Equation (2.1)), leading to losses in the soil strength and the soil behaves like a liquid.

$$\sigma'_v = \sigma_v - (u + \Delta u) \quad (2.1)$$

Liquefaction phenomena can be divided into two main groups: flow liquefaction and cyclic liquefaction. Both are very important, and the evaluation of both liquefaction hazards should be considered.

2.2.1.1 Flow liquefaction

Flow liquefaction is a type of liquefaction that can occur in static or dynamic conditions and it has a lower frequency occurrence rate, but its effects are usually far more severe. Failures caused by this type of liquefaction are often characterized by large and rapid movements. Examples of static loads can be the construction of new buildings or tailing dams, while for dynamic loads pile driving or earthquakes are good examples.

Kramer reported that it occurs when the shear stress required for the static equilibrium of a soil mass (static shear stress), τ_0 , is greater than the shear strength of the soil in its liquefied state, τ_{liq} , (Kramer, 1996). This means that the static equilibrium is destroyed by the load applied to a soil with low residual strength (Eq. (2.2)).

$$\tau_0 > \tau_{liq} \quad (2.2)$$

Once triggered, the large deformations are driven by the shear stress required for static equilibrium, τ_0 .

2.2.1.2 Cyclic mobility

Cyclic mobility involves a broader range of soil and site conditions than flow liquefaction, and its effects can reach from insignificant to highly damaging. This range of soils can include fine-grained soil, as described in Bray and Sancio (2006), where the liquefaction susceptibility criterion was given by a ratio between water content, w_c , per liquid limit, LL, introduced by Wang in 1979, where if the ratio w_c/LL is high (e.g. >1) the soil is a prime candidate for liquefaction, especially if the soil is on low plasticity (Bray & Sancio, 2006). Thus, the damage that the liquefaction of these soils, in loose and saturated state, can cause is much smaller than for example loose clean sands would (Bray & Sancio, 2006). Boulanger (2007) studied the softening of clays and Silts and stated that the consequences of cyclic softening in claylike fine-grained soils can range from severe to inconsequential depending on soil sensitivity, the specific site conditions and earthquake ground motions. Normally or lightly overconsolidated (OCR), sensitive clays and silts can have relatively low cyclic strengths and can lose significant strength for large earthquakes. While higher OCR have much greater cyclic strengths and lower sensitivities, such as even strong shaking might not necessarily present a major problem (Boulanger & Idriss, 2007).

Cyclic mobility occurs when the static shear stress is lower than the shear strength of the liquefied soil. The deformations produced by cyclic mobility failures develop incrementally during earthquake shaking (Kramer, 1996).

2.2.2 LIQUEFACTION SUSCEPTIBILITY FOR SANDS

Liquefaction of a soil depends on a large range of conditions and since its hazards can be very significant, it is necessary to make a good evaluation of its susceptibility. Sometimes it can occur in the same location more than once, if the soil condition remains constant, so previous sites where there has been liquefaction before are important to be determined certain conditions of liquefaction. Also, for a given earthquake magnitude, the distance of the site to the seismic source is also a very important aspect (e.g. 2010 and 2011 Christchurch earthquakes in New Zealand). Historically, liquefaction effects have been observed within a certain distance of the seismic source.

As it is usually said among peers, water can be a one of the biggest enemies of a Geotechnical engineers so, geologically, it is important to know where the water level is, since liquefaction occurs in saturated (or nearly fully saturated) soils, and its susceptibility decreases with the increase of groundwater depth. Also, hazards related to liquefaction often fluctuate with the fluctuation of the ground water level.

As described before, for liquefaction to occur, it is necessary that excess pore pressure is developed, and for that, it is crucial to evaluate the way how variations in the volume of the soils develop. For high volume changes, excess pore pressure is generated and then so liquefaction.

So, different soils behave differently when it comes to volume change. For example, if there is a loose soil there is a certain void ratio (ratio between volume of the voids -filled by water and air- and volume of solids), e , within the particles of that soil; when sheared the particles will rearrange themselves, expelling the air between them and densify leading to a smaller void ratio. On the other hand, if there is a soil that is denser than the first one, its initial void ratio will be smaller than the initial void ratio of the first soil. When sheared, the particles of the denser soil that were already well arranged, will first contract and then rapidly move and be in a new position where the void ratio is higher than before. Therefore, loose sands tend to contract, and denser sands tend to dilate, as clearly represented in Figure 2.4 below.

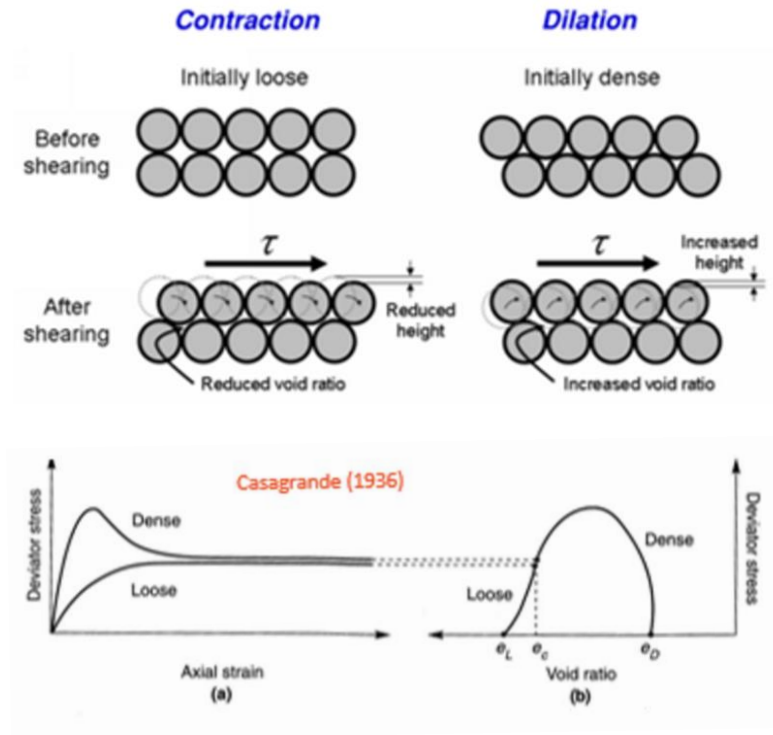


Figure 2.6 The effect of contraction and dilation after shearing. And Deviatoric stress per axial strain (a), and per void ratio (b) (COWI, 2019 after Kramer 1996).

At large strains, all soil specimens, irrespective of their initial density, will tend to the same soil density and continue to shear with constant shearing resistance, as it can be easily perceptible in Figure 2.7. The void ratio corresponding to this constant density was termed as critical void ratio (CVR or e_c) obtained by drained strain controlled triaxial tests (Casagrande, 1936).

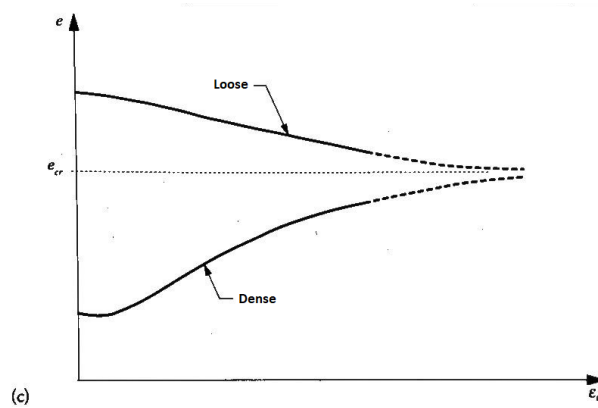


Figure 2.7 Typical triaxial test results of two samples of the same sand, one of them dense and the other one loose, correlating the axial deformation to the void ratio. (adapted from Taylor, 1948; (Matos Fernandes, 2011)).

Casagrande (1936) found that the CVR, or e_c , was uniquely related to the effective confining pressure, and it was after experimentally confirmed that positive excess pore pressure would be produced in loose specimens, due to contractive behaviour; and negative in dense specimens due

to dilative behaviour. Then it was possible to use the CVR line as a boundary between loose contractive states and dense dilative states (Figure 2.8) (Kramer, 1996).

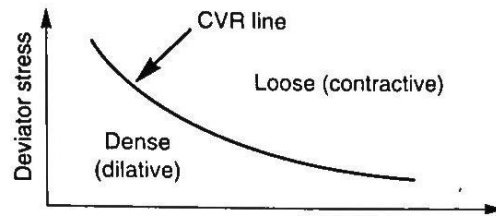


Figure 2.8 Use of the CVR line as a boundary between loose contractive states and dense dilative states (Kramer, 1996).

The CVR line, also called, Critical State Line (CSL) describes the state towards which any soil specimen would migrate at large strains, whether by volume change in drained conditions (D), changes in effective stresses in undrained conditions confining pressure (UD), or a combination (partially drained), as shown in Figure 2.9. It is also interesting to understand that the CVR (or CSL) can also be represented with a straight line when representing the effective confining pressure in a logarithmical scale (Figure 2.9 (b)), which can be useful.

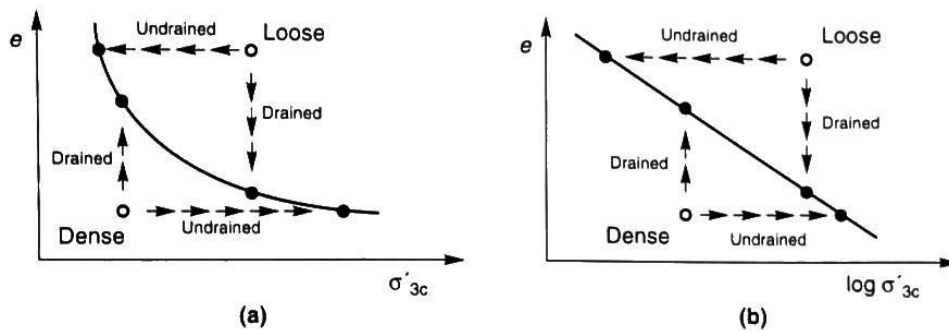


Figure 2.9 Behaviour of initially loose and dense specimens under drained and undrained conditions for (a) arithmetic and (b) logarithmic effective confining pressure scales (Kramer, 1996).

The steady state line (SSL) is the locus of points describing the relationship between void ratio, effective confining stress (and deviatoric stress in three-dimensional space) in the steady state of deformation. It can be viewed as a three-dimensional curve in $e - \sigma' - \tau$ or $e - p' - q$, as illustrated in Figure 2.10. A similar plot can be developed using the stress path parameters q and p' instead of τ and σ' .

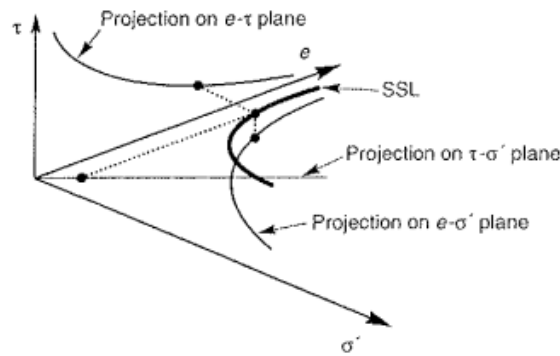


Figure 2.10 Three-dimensional steady-state line showing projections on e - τ plane, e - σ' plane, and τ - σ' plane. (Kramer, 1996).

The SSL can be used to identify if a certain soil, at a given confining pressure, sheared in stress-controlled (given deviatoric stress) and undrained loading (constant void ratio) is susceptible to flow liquefaction (if its state plots above the SSL) or not. If not, it means that liquefaction will not occur, but large deformations could occur, due to cyclic mobility.

Density affects the behaviour of the soils and so, it is important to create a framework, of properties and soil states, to explain why a soil with a particular density behaves in a particular way. By linking this theory with a particular condition of the soil critical state – the critical void ratio - Casagrande (1936) named critical state the end state when the soil is sheared continuously, With the knowledge of the end state it is easy to create models that are accurate with how the soil will react in reality. Castro (1969) showed that the knowledge of the end point, that was the critical state during rapid shearing, allowed the solution for most of the problems that include liquefaction, and so, the critical state after rapid shearing was termed the steady state.

Mathematically, there is no difference between the definitions of steady and critical states, and they are usually taken to be the same (Jefferies & Been, 2016).

The critical state is another name given to the steady state, at which the soil continues to deform at a constant shear stress and void ratio, and it has been more and more used as a fundamental state to characterize the strength and deformation properties of sands in limit equilibrium.

The critical state line (CSL), expressed in the e - p' plane in Figure 2.11, helps describe the true state of a sand by the location of its current state stress and volume relative to the CSL. Whenever it is above of the CSL, the sand has the tendency to contract, whereas when its state is below the sand has the tendency to dilate. A state parameter, ψ , has been determined to quantify the void ratio difference between the current state of a soil and the critical state at the same mean effective stress, p'_{cs} . This can be easily comprehended by observing Figure 2.11 (Papadopoulou & Tika, 2008).

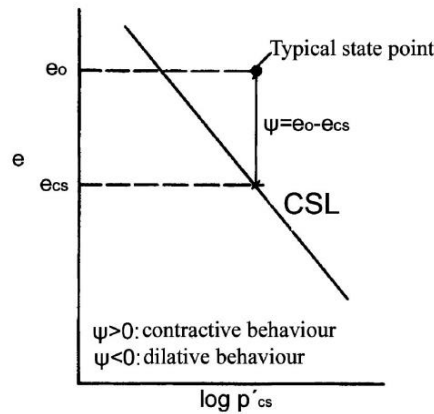


Figure 2.11 Definition of state parameter (Been and Jefferies, 1985).

Vaid et al (1981) analysed the response of a series of triaxial specimens initially consolidated to the same void ratio at different effective confining stress. Figure 2.12 shows the effective stress path for the same soil: a loose sand (curve A), a medium dense sand (curve B) and a dense sand (curve C). For a certain initial pressure, the shape of the effective stress paths mainly depends on density.

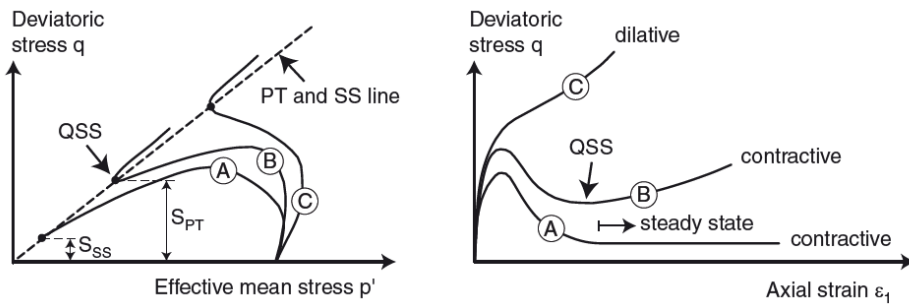


Figure 2.12 Effective stress paths in the p' - q plane typically measured in undrained monotonic triaxial tests on loose, medium dense sand samples (Vaid et al., 1981).

The curve A is showing a contractive behaviour, the deviatoric stress, q , increases until a maximum and then decreases until it reaches a steady state (SS). Then the effective stress does not change anymore although the shearing is continued. For the medium dense sand (curve B), the deviatoric stress first increases until a local maximum and then decreases up to a local minimum so called “quasi-steady state” (QSS). Up to this point the behaviour is contractive, meaning that p' has decreased. If the shearing continues the behaviour of the material changes to dilative, to a so called “phase of transformation” PT, and a subsequent increase of p' and q occurs. This last phase the effective stress path follows, in the p' - q -plane, an almost linear curve. In the case of curve C, its behaviour is similar to curve B, but the deviatoric stress is continuously increasing, the QSS is not observed and if the stress paths of B and C are connected they lay on the same linear curve through the origin (PT line) (Wood, 2012).

Kramer (1996) discussed the response of a series of triaxial specimens initially consolidated to the same void ratio at different effective confining pressures and sheared in undrained tests. Since all specimens have the same void ratio and are sheared in undrained tests (see Figure 2.12 above),

all specimens will reach the same effective stress conditions at the steady state, but through different paths.

Considering Figure 2.13, initial states A and B are located below the SSL which means that they will have a dilative behaviour upon shearing. On the contrary, states C, D and E have contractive behaviour. Each reaches an undrained strength peak after which the strain rapidly moves towards the steady state. Since each peak is marked with x's, these points can be used to define the flow liquefaction surface (FLS). The FLS marks the boundary between stable and unstable states in undrained conditions, describing the conditions at which the flow liquefaction will initiate (Figure 2.14).

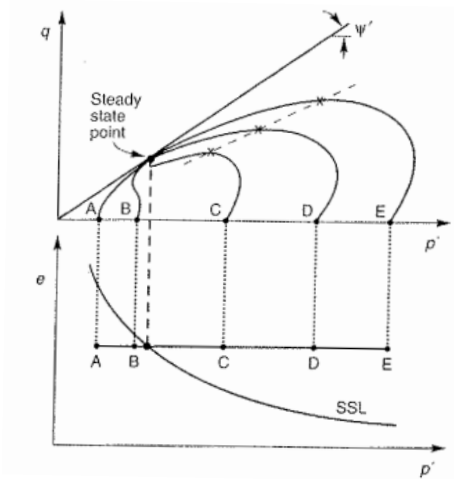


Figure 2.13 Response of five specimens isotropically consolidated to the same initial void ratio at different initial effective confining pressures. Flow liquefaction in specimens C, D, and E is initiated at the points marked with an X. The dotted line passing through these points is a line of constant principal effective stress ratio, K_L (Kramer, 1996).

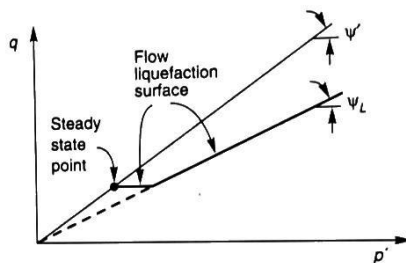


Figure 9.16 Orientation of the flow liquefaction surface in stress path space.

Figure 2.14 Orientation of the flow liquefaction surface in stress path space.

With respect to an undrained cyclic loading, Vaid and Chern (1985) distinguished three different cases depending on soil density. Figure 2.15 below shows three sands with different densities. In a) is presented as specimen looser than critical, that after a few cycles a “steady state” (or critical state), SS (or CS), is reached; a type of failure that is denoted as “flow liquefaction”. For b), a

sand with the critical void ratio, the stress path, after some cycles, the “quasi-steady state”, QSS, will be reached and during it large deformations are generated. This sand will show a “limited liquefaction”. For a sand denser than critical, as in c), the test ends at a cyclic mobility phase (Wood, 2012).

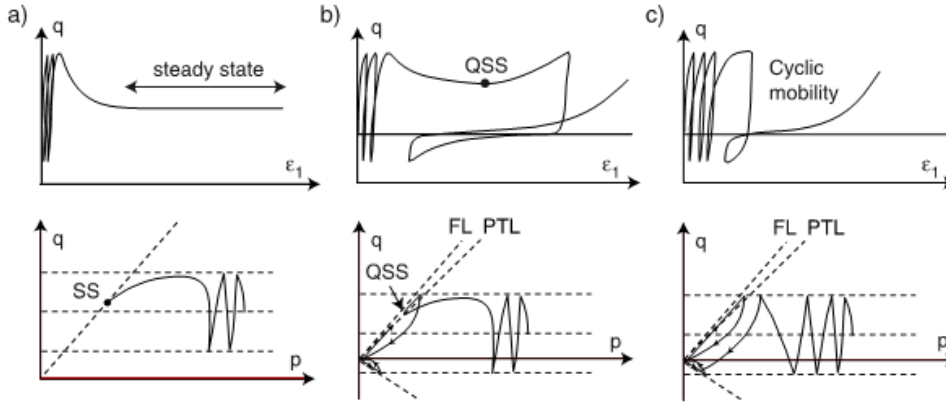


Figure 2.15 Schematic illustration of typical curves $q(\varepsilon_1)$ (Vaid and Sivathayalan, 2000) and $q(p)$ in cyclic triaxial tests on: a) very loose, b) loose and c) medium dense or dense sand (Wood, 2012).

The results shown above (Figure 2.15) refer mainly to clean sands and give a framework and overview of the main factors affecting liquefaction susceptibility in clean sands. To give a more complete picture of the current state of knowledge, more recent studies including the effects of plastic and non-plastic fines will be described in the following section.

2.2.3 PARAMETERS AFFECTING SOIL RESPONSE TO UNDRAINED CYCLIC LOADING

There are a number of parameters that highly influence the response of the soil under undrained conditions. Regarding the accumulation of pore water pressure, density is one of the most important. Seed and Lee (1966/1967) have demonstrated that for a certain stress amplitude, a much larger number of cycles is necessary to cause liquefaction in a dense sand compared to a loose one (Figure 2.16).

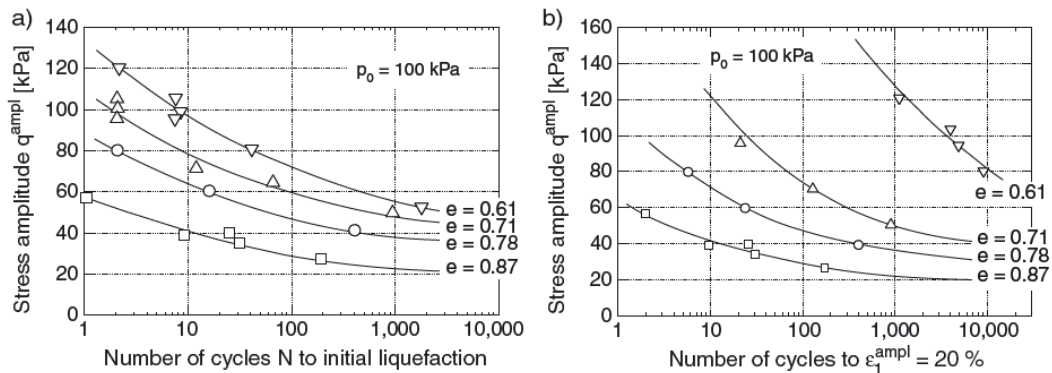


Figure 2.16 - Influence of void ratio on the number of cycles to a) initial liquefaction and b) a strain amplitude $\varepsilon_1^{ampl} = 20\%$, undrained cyclic triaxial tests of Lee and Seed (1967) (Wood, 2012)

Figure 2.16 shows the influence of void ratio, e , and for a certain stress amplitude, q^{ampl} , a much larger number of cycles is necessary to cause liquefaction in a denser sand than compared to a looser one. This means that, by increasing soil density, the CSR(N) curves are shifted upwards and to the right (Wood, 2012).

Mori et al. (1978) carried out some tests to show the increase of the liquefaction resistance with increasing density, which the results are presented in Figure 2.17 a). Similarly, Tatsuoka et al. (1986b) found a relationship between the liquefaction resistance and the relative density. As it is possible to observe in Figure 2.17 b), for larger relative densities the cyclic stress ratio causing liquefaction increases over linear with D_r (Wood, 2012).

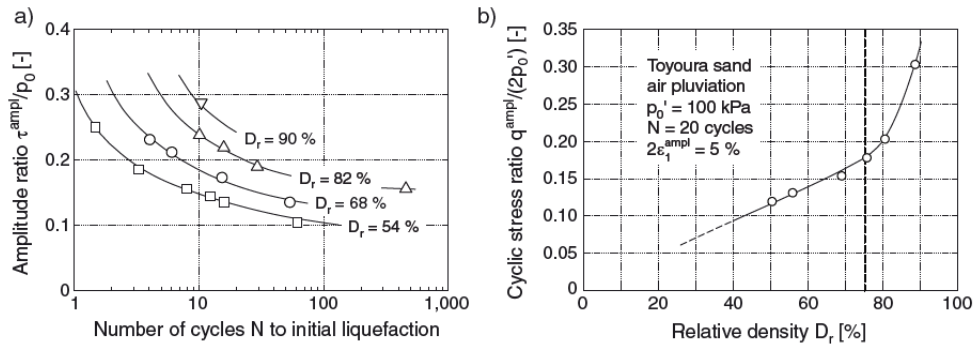


Figure 2.17 Increase of the liquefaction resistance with increasing density: tests of a) Mori et al. (1978) and b) Tatsuoka et al. (1986).

The influence of the initial effective mean pressure, p_0' , can be almost eliminated if the data is plotted, as in the lower row of Figure 2.18, by the amplitude pressure ratio, $q^{ampl}/2p_0'$, instead of the stress amplitude, q^{ampl} , as in the upper row of Figure 61, this was demonstrated by Seed and Lee (1967), (Wood, 2012).

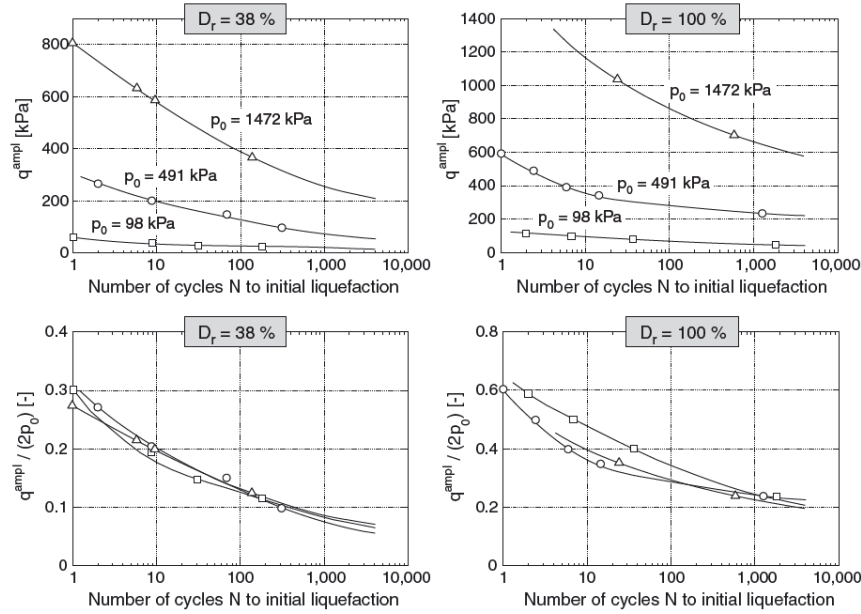


Figure 2.18 The use of an amplitude-pressure ratio $q^{amp}/2p'_0$ on the ordinate purifies the data from the influence of the initial effective mean pressure p'_0 (Lee and Seed, 1967).

A slight decrease in the curves $CSR(N)$ with increasing pressure may be conclude from the $D_r=100\%$ diagram in the Figure 2.18 above. This decrease has been observed by many other authors and, from several data previously presented, Ishihara (1995) proposed a pressure dependent correction factor, $K_\sigma(p'_0)$, with which the liquefaction resistance measured for $p'_0 = 100\text{ kPa}$ can be reduce for larger pressures (Wood, 2012).

Ishihara (1995) stated that K_σ strongly depends on the type of soil and Vaid and Thomas (1995) demonstrated that K_σ also depends on density. For loose specimens, $K_\sigma \approx 1$ was observed almost independent of p'_0 (Wood, 2012). After collecting data from several authors (Seed and Harder (1990), Kokusho et al. (1983), Frydman et al. (1980), and Vaid and Thomas (1994)), Ishihara (1995) proposed a pressure-dependent correction factor $K_\sigma(p'_0)$ (represented in Figure 2.19 by the solid line with which liquefaction resistance measured for a pressure $p'_0 = 100\text{ kPa}$ can be reduced for larger pressures.

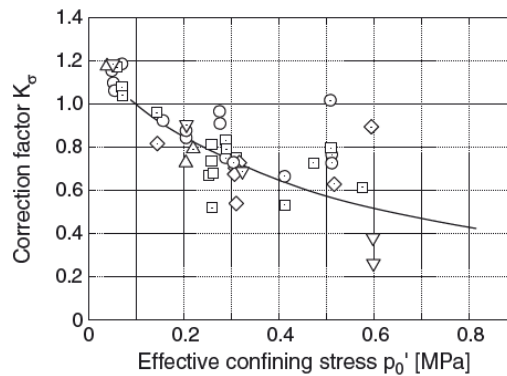


Figure 2.19 Correction factor K_σ for a consideration of the decrease of the cyclic stress ratio $*q^{amp}/(2p'_0)$ causing liquefaction with increasing pressure p'_0 (Ishihara, 1995).

Vaid and Finn (1979) showed, with simple shear tests, that the effect of a static shear stress depends on the failure criterion. Figure 2.20 a) presents the cyclic stress ratio (CSR) that causes, per 10 cycles, a shear strain amplitude $\gamma^{amp} = 2, 5 \text{ or } 10\%$ as a function of the static stress ratio $\alpha = \tau^{av}/\sigma_1$. It is easily observed that with increasing α , for $\gamma^{amp} = 2\%$, a lower CSR is necessary; but for $\gamma^{amp} = 5 \text{ and } 10\%$ larger amplitudes are needed (Wood, 2012).

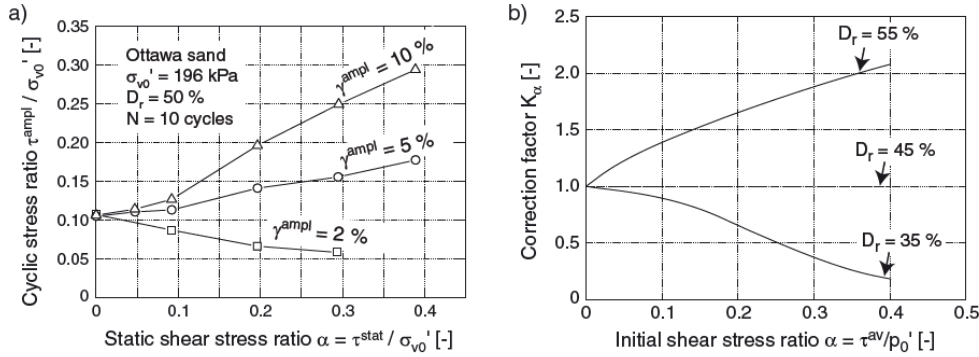


Figure 2.20 Influence of a static initial shear stress on the liquefaction resistance depends on a) the chosen failure criterion (Vaid and Finn, 1979) and b) on soil density (Rollins and Seed, 1990).

The dependency of density on the effect of static shear stress was revealed by Rollins and Seed (1990) by performing undrained cyclic triaxial tests.

In Figure 2.20 b), the correction factor K_α is defined as the CSR causing failure ($2\varepsilon_1^{amp} = 5\%$) for a certain static shear stress τ^{av} divided by the CSR causing failure at $\tau^{av} = 0$. For loose soil ($D_r = 35\%$), the liquefaction resistance decreases with increasing static stress ratio $\alpha = \tau^{av}/p_0'$. For $D_r = 45\%$, the τ^{av} influence is practically zero, and for $D_r = 55\%$ the liquefaction resistance increases with α . Some authors defend that the correction factor K_α presented in Figure 2.20 b) can be considered conservative, especially for loose sands (Wood, 2012).

Some tests on dense sand were performed by Vaid and Chern (1985) that moreover showed the influence of a static initial shear stress, on the liquefaction resistance of dense sands, depending also on the effective consolidation pressure p_0' .

Based on the data in Figure 2.20 Vaid and Sivathayalan (2000) concluded that the influence of p_0' (correction factor K_σ) and the influence of τ^{av} (correction factor K_α) cannot be treated independently of each other. A reduction in the liquefaction resistance by the product $K_\alpha * K_\sigma$ would drastically underestimate the liquefaction resistance, in particular for loose sands. For a specific sand, Vaid and Sivathayalan (2000) recommended to study the combined influence of p_0' and τ^{av} in laboratory tests (Wood, 2012).

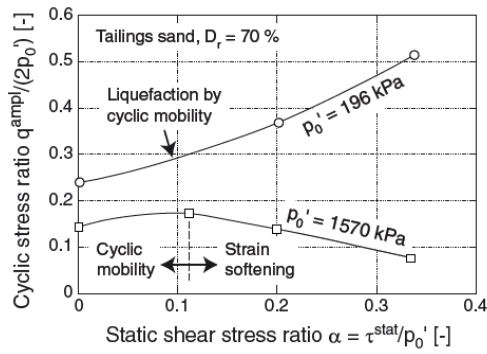


Figure 2.21 Influence of a static initial stress on the liquefaction resistance depends on initial mean effective stress p_0' (Vaid and Chern, 1985).

The specimen preparation procedure can have a significant influence on liquefaction resistance. In the field, soil particles can be disposed in many ways, and its orientation and arrangement, as well as particle contacts, all these issues can influence soil behaviour. In the laboratory, it is possible to recreate different initial soil fabric by implementing different specimen preparation methods, and all of this can have a clear influence on the liquefaction resistance of sand specimen. The example given by Mulilis et al. (1975) in Figure 2.22 clearly evidences the influence of specimen preparation.

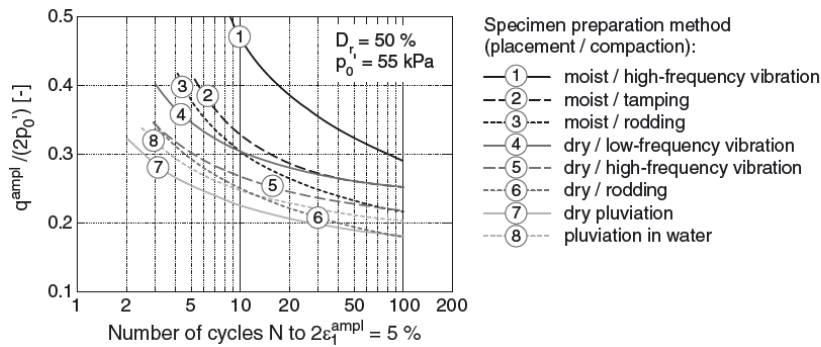


Figure 2.22 Influence of the specimen preparation method on the liquefaction resistance for the same soil, data of Mulilis et al. (1975).

In the work shown in Figure 2.22, all of the specimens were prepared with the same relative density of $D_r=50\%$ and $p_0'=55$ kPa, but the different preparation methods lead to different results on the liquefaction resistance. The specimens that were prepared in moist conditions have higher liquefaction resistance than others that have been prepared in dry conditions. The ones that were compacted, by vibration, tamping or rodding, have presented also higher liquefaction conditions than other preparation methods. The figure also shows that water pluviation can have a slightly higher resistance than air pluviation (Wood, 2012).

Many authors (e.g. Mulilis et al., 1975, 1977; Peck, 1979; Tokimatsu and Hosaka, 1986; Hatanaka et al., 1988; Seki et al., 1992; Porcino et al., 2004)) reported that undisturbed and laboratory reconstituted samples of the same material do not have the same liquefaction resistance, and normally the undisturbed ones are significantly larger. This can be due to the natural fabric and soil structure, as well as aging or cyclic preloading effects; therefore, testing reconstituted samples can lead to an underestimation of the liquefaction resistance of an in-situ soil deposit. High quality undisturbed samples can be obtained by a method of ground freezing technique (Yoshimi et al., 1984, 1989) or by advanced sampling techniques, such as gel-push or even Dames & Moore sampling (Molina-Gomez et al, 2020), and given the loose conditions of the natural soil, these samples need to be very carefully handled (Viana da Fonseca et al, 2019)).

Yoshimi et al. (1984, 1989) used this ground freezing technique to obtain undisturbed samples from a natural sand deposit in Niigata and produced some tests, comparing that sample with an artificial sand deposit prepared by water pluviation. As can be observed in Figure 2.23, some cyclic triaxial tests were made and revealed that the natural sand deposit (the undisturbed sample) had a much higher liquefaction resistance than the artificial one, even when the relative density is almost the same. This experiment can indicate that this ground freezing technique can preserve the in-situ fabric of the grain skeleton (Wood, 2012).

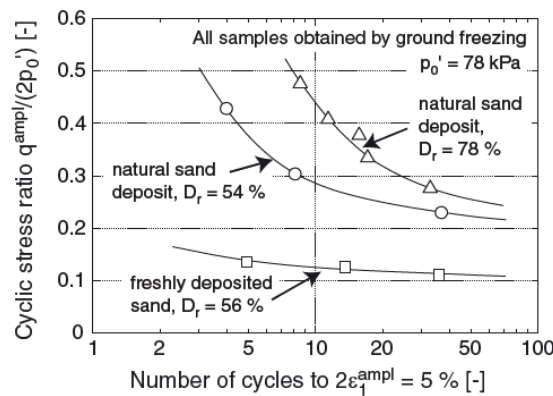


Figure 2.23 Comparison of the liquefaction resistance of undisturbed samples obtained by ground freezing from a natural sand deposit and from a freshly deposited (artificial) sand layer (Yoshimi et. Al., 1989).

2.2.4 EFFECT OF FINES

Drained and undrained monotonic tests produce unique critical state lines, and the location of the CSLs is different for each soil depending on its fines content. Figure 2.24 shows different CSLs of sand-silt mixtures for non-plastic fines, and it is possible to see that, for lower stresses, the critical state lines are slightly inclined and nearly parallel, and for increasing mean effective stress level, the CSLs steepen and then converge for stresses over 1000 kPa (Papadopoulou & Tika, 2008).

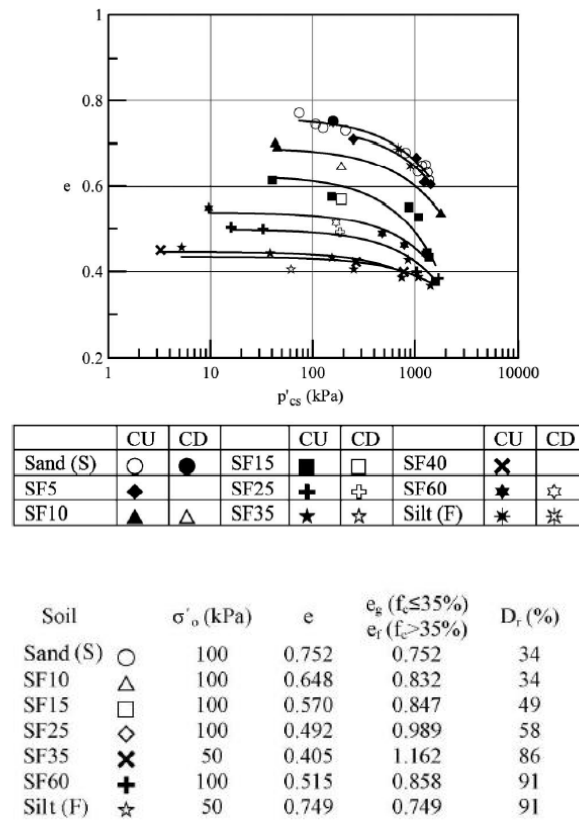


Figure 2.24 Critical state line of sand-silt mixtures in terms of void ratio, e (Papadopoulou & Tika, 2008).

By observing the figure above, it is also possible to realise that the critical state lines move downwards with the increasing of fines content, f_c , up to a threshold value, f_{cth} , of 35%. This happens because the non-plastic fines that fill the voids in between the particles are not really able to sustain, the sand gets “softer” and therefore more contractive. For a f_{cth} over 35%, the voids are less filled and the non-plastic fines start contributing to the stiffness of the soil, leading the CSLs to move upwards (Papadopoulou & Tika, 2008).

In the case of soils with a significant plastic fines content, Figure 2.25 below shows how fine-grained soils manifest in terms of plasticity index (PI) per water content to liquid limit ratios (w_c/LL). The way that the results are dispersed shows that young, shallow, non-plastic silts and clayey silts of low plasticity ($PI < 12$) at high w_c/LL (> 0.85) can liquefy under significant cyclic loading. The clayey silts and silty clays of moderate plasticity, with PI in between 12 and 18 ($12 < PI < 18$), at a $w_c/LL > 0.8$ can undergo liquefaction when shaken intensively for a significant number of loading cycles. Additionally, there may be cases where more sensitive soils with $PI > 18$ can suffer severe strength loss as a result of earthquake-induced straining (Bray & Sancio, 2006).

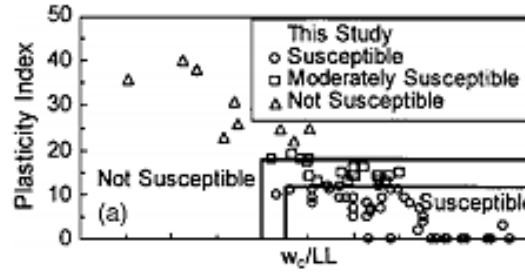


Figure 2.25 Graphic representation of the proposed liquefaction susceptibility criteria: a) isotropically consolidated CTX testing data from the study (Bray & Sancio, 2006).

Grain size distribution and plasticity index influence the susceptibility of liquefaction. The plasticity index of fines, I_p , or clay content, C_c , has been identified as a significant parameter to evaluate the liquefaction potential of silty sands. The more well-graded soils are, less susceptible to liquefaction they are; because, in undrained conditions, if the voids are filled with fine grains than there is lower volume change and there so lower excess pore pressure. Regarding plasticity index, non-plastic particles are more affected by liquefaction than plastic ones, i.e. clays don't usually suffer from liquefaction and coarse silts do.

2.3 EMPIRICAL CORRELATIONS TO ASSESS LIQUEFACTION POTENTIAL

2.3.1 INTRODUCTION

To evaluate the liquefaction resistance of soils two variables are required: the seismic demand on a soil layer, expressed in terms of CSR and the capacity of the soil to resist liquefaction, expressed in terms of CRR.

There are different approaches to assess the evaluation of liquefaction, the most used is the "simplified method" firstly proposed by Seed & Idriss, where the Factor of Safety against triggering of liquefaction can be determined, for sand-like soils, as the ration of the soil Cyclic Resistance Ratio (CRR) to the earthquake induced Cyclic Stress Ratio (CSR), with both, CRR and CSR, regarding the designed earthquake magnitude (Eq. (2.3)).

$$FS = \frac{CRR_M}{CSR_M} \quad (2.3)$$

The CSR is estimated by the following equations (Seed & Idriss, 1971):

$$CSR = 0.65 * \frac{a_{max}}{g} * \left(\frac{\sigma_{v0}}{\sigma'_{v0}} \right) * r_d \quad (2.4)$$

Where a_{max}/g is the peak ground acceleration at ground surface as fraction of gravity, σ_{v0} and σ'_{v0} are the total and effective initial vertical stresses at depth z , and r_d is the shear stress reduction coefficient that occurs for the dynamic response of the soil profile. The factor 0.65 was introduced to reduce the CSR from the peak value of the earthquake-induced stress, which occurs multiple times during the earthquake, to a more representative value that occurs several times during strong shaking. The value of r_d is obtained based on the following, proposed by Idriss (1999) based on numerical simulations (Idriss & Boulanger, 2014).

$$r_d = \exp [\alpha(z) + \beta(z) * M] \quad (2.5)$$

$$\alpha(z) = -1.012 - 1.126 * \sin \left(\frac{z}{11.73} + 5.133 \right) \quad (2.6)$$

$$\beta(z) = 0.106 + 0.118 * \sin \left(\frac{z}{11.28} + 5.412 \right) \quad (2.7)$$

To determine the maximum acceleration, a_{max} , Eurocode 8 approach was considered, that established the following steps:

- 1) Determine the medium value of the shear wave velocity.

$$v_{s30} = \frac{30}{\sum_{i=1}^n \frac{h_i}{v_i}} \quad (2.8)$$

Where h_i and v_i represent the thickness and the shear wave velocity (for distortions $\leq 10^{-5}$) of the i -th layer in a total of n existing in the upper 30 meters.

- 2) Select Ground type in table 3.1 of Eurocode 8.
- 3) Select the value of S in table 3.2 of Eurocode 8.
- 4) Calculate a_{max} by the following formula:

$$a_{max} = a_g * S \quad (2.9)$$

Where a_g is the peak ground acceleration, which should be selected considering the location of the site.

The CRR curves are normally defined for a given Magnitude of 7.5, to adjust to other magnitudes Seed and Idriss (1971) introduced a corrector factor termed Magnitude Scaling Factor (MSF), which is defined as:

$$CRR_M = MSF * CRR_{7.5} \quad (2.10)$$

For fine-grained soil, Boulanger (2007) presents the following equation:

$$CRR_{7.5} = 0.18 * OCR^{0.8} * K_\alpha \quad (2.11)$$

Where OCR is the over consolidation ratio; and K_α is a correction factor, introduced by Seed (1983), that represents the effects of an initial static shear stress ratio (α) on the liquefaction resistance of sands, and is calculated by the following equations:

$$K_\alpha = \frac{CRR_\alpha}{CRR_{\alpha=0}} \quad (2.12)$$

$$\alpha = \frac{\tau_s}{\sigma'_{vc}} \quad (2.13)$$

Being τ_s the horizontal static shear stress.

The Magnitude Scaling factor (MSF) defined by Seed and Idriss is plotted in Figure 2.26,

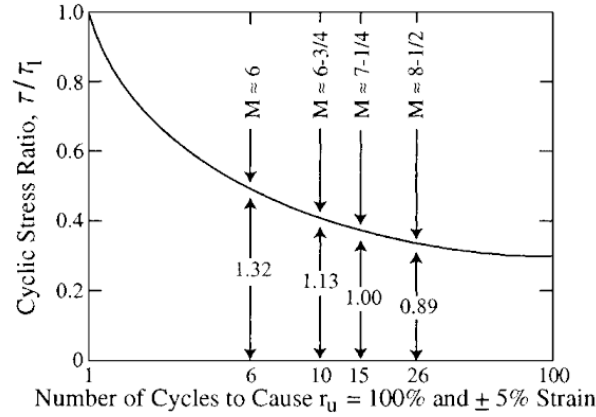


Figure 2.26 Representative relationship between CSR and Number of cycles to Cause Liquefaction
(Reproduced from Seed and Idriss 1982) (Youd & Idriss, 2001)

The revised MSFs are defined by the following equation:

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad (2.14)$$

In Boulanger 2014 there is another proposal for MSF:

$$MSF = 6.9 * e^{\frac{M}{4}} - 0.058 \leq 1.8 \quad (2.15)$$

2.3.2 SPT BASED LIQUEFACTION ASSESSMENT

2.3.2.1 Test description

The Standard penetration test (SPT) is by far one of the most used in situ tests around the world. This happens because it is simple and cheap, and because it allows to obtain borehole samples with reasonable quality. It basically consists in counting the number of blows given by a 63.5 kg hammer, falling from 76cm high, driving through a borehole a normalised sample tube in two phases. The first phase of 15cm is not taken into account because it corresponds to the superficial layer and where the soil is most disturbed, and another phase of 30cm. The number of blows from this 2nd phase provides the N number considered as the result of the test. But there are some corrections that need to be made in this test result.

Because from the fall of the hammer, not all the energy from it is, evidently, transmitted to the sample tube, and so it is important to consider the effective energy. Therefore, it is considered an energy ratio, E_R , given by Equation (2.16), where E_P is the weight of the hammer multiplied by

the height of the fall, and with that it was possible to establish standard value of 60 %, N_{60} , given by Equation (2.17).

$$E_R = \frac{E}{E_P} * 100 \quad (2.16)$$

$$N_{60} = C_E * N \text{ where } C_E = \frac{E_R}{60} \quad (2.17)$$

There is another correction that has to do with the effect of the effective stress at the depth of the test, for sandy soils, the overburden correction factor, C_N . Therefore, it was established a normalized result, N_1 , obtained for a vertical effective stress of rest of 1 atmosphere (100kPa) (Seed & Idriss, 1971). The correction is given by the Equation below:

$$(N_1)_{60} = C_N * N_{60} \text{ where } C_N = \left(\frac{p_a}{\sigma'_{v0}} \right)^{0.5} \quad (2.18)$$

And finally, two correction coefficients that consider the length, C_R , and the diameter, C_D , of the of the sample tube that have standardized intervals.

In short, to correct the raw result that comes from the SPT test, N , the following equation should be considered:

$$(N_1)_{60} = C_E * C_R * C_D * C_N * N \quad (2.19)$$

2.3.2.2 CRR curves for Clean Sands based on SPT test results

a) Youd & Idriss (2001)

In Youd and Idriss (2001) report, it is presented an equation to determine CRR criterion, for a magnitude of 7.5, for sands and silty sands based on the Standard Penetration Tests (SPT) results, as the followed:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 * (N_1)_{60} + 45]^2} - \frac{1}{200} \quad (2.20)$$

This equation is valid for $(N_1)_{60} < 30$, for higher results clean granular soils are too dense to liquefy and are classes as non-liquefiable (Youd & Idriss, 2001).

It is also possible to estimate the relative density (Idriss and Boulanger 2008):

$$D_r = \sqrt{\frac{(N_1)_{60}}{46}} \quad (2.21)$$

The Magnitude Scaling factor is the one previously presented in equation (2.14)

2.3.2.3 CRR curves for Sands and Silty Sands based on SPT test results

a) Youd & Idriss (2001)

To account for fines content, a relationship between SPT results, $(N_1)_{60}$, and SPT results for clean Sands, $(N_1)_{60cs}$, was developed by I. M. Idriss with the assistance of R. B. Seed. The point was to create a correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$:

$$(N_1)_{60cs} = \alpha + \beta * (N_1)_{60} \quad (2.22)$$

where α and β are coefficients determined by the following relationships presented in Table 2.1,

Table 2.1 Relationships for α and β are coefficients.

Fine Content (%)	$\leq 5\%$	$5\% < FC < 35\%$	$\leq 35\%$
α	0	$e^{1,76 - \left(\frac{190}{FC^2}\right)}$	5
β	1	$0,99 + \frac{FC^{1,5}}{1000}$	1,2

To calculate CRR values the Equation (2.20) for clean sands is also used for silty sands but replacing $(N_1)_{60}$ for $(N_1)_{60cs}$.

The Magnitude Scaling factor is the one previously presented in equation (2.14).

b) Boulanger & Idriss (2014)

Boulanger & Idriss (2014) also presented an approach to define CRR-N curves for SPT test of Sands and Silty sands.

$$CRR_{7,5} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \quad (2.23)$$

Where $(N_1)_{60cs}$, is the penetration resistance of SPT test. N_{60} , normalized and adjust to an equivalent clean sand (i.e. with a $FC \leq 5\%$) and it can be determined as:

To incorporate the fines content, the following expression and it corresponding curve (Figure 2.27) were used.

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \quad (2.24)$$

$$\Delta(N_1)_{60} = \exp\left(1,63 + \frac{9,7}{FC + 0.01} - \left(\frac{15,7}{FC + 0.01}\right)^2\right) \quad (2.25)$$

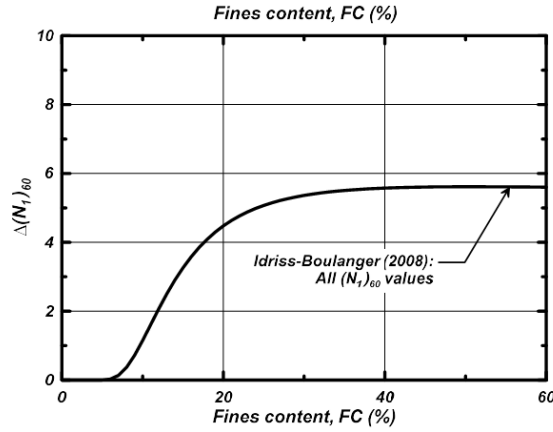


Figure 2.27 Equivalent clean sand adjustments for SPT-based liquefaction triggering procedures.

Then, the expression considered to calculate $CRR_{7.5}$ was:

The Magnitude Scaling Factor here is calculated based on Boulanger & Idriss (2014) where they propose a MSF that varies with a relationship between q_{c1Ncs} and $(N_1)_{60cs}$. This variation is represented in Figure 2.28.

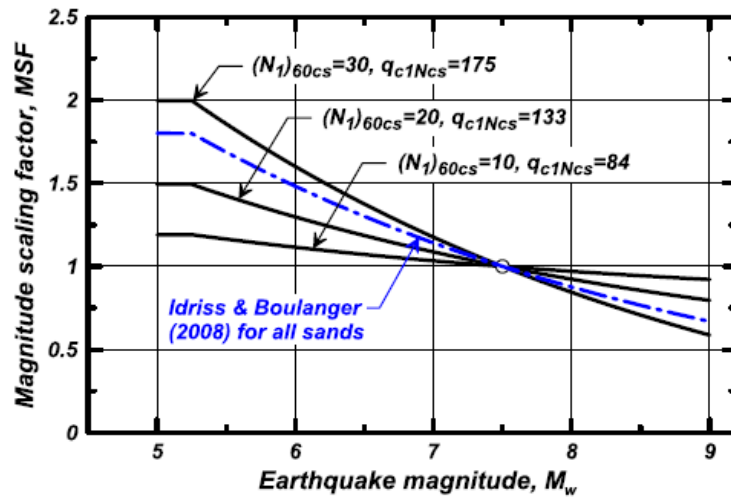


Figure 2.28 Variation In the MSF relationship with q_{c1Ncs} and $(N_1)_{60cs}$ for cohesionless soils.

Which can be expressed by the following equations:

$$MSF = 1 + (MSF_{max} - 1)[8.64 * \exp\left(-\frac{M}{4}\right) - 1.325] \quad (2.26)$$

$$MSF_{max} = 1.09 + \left(\frac{(N_1)_{60cs}}{31.5} \right)^2 \leq 2.2 \quad (2.27)$$

From this, it was possible to determine any variation. So, taking the same values has used for Yould & Idriss (2001) where the $q_{c1Ncs} \sim 75.9$, a new curve was determined (Figure 2.29).

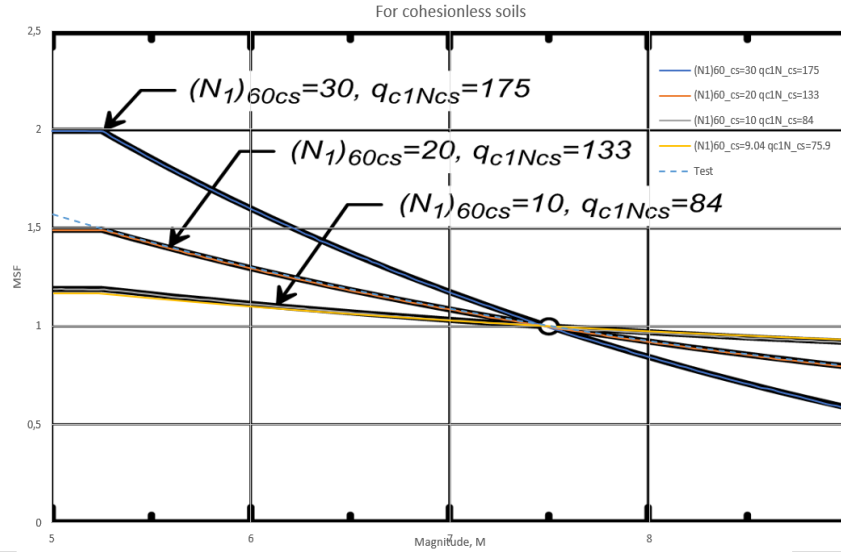


Figure 2.29 Variation In the MSF relationship with q_{c1Ncs} and $(N_1)_{60cs}$ for cohesionless soils.

2.3.3 CPT BASED LIQUEFACTION ASSESSMENT

2.3.3.1 Test description

The Cone Penetration Test (CPT) is another of the most currently used in situ tests. In comparison to the SPT, this test has the advantage of being statically pushed, highly automatized, and so its results are independent of the operator however, it does not allow to take soil samples. This test gives important information about the soil behavior throughout all its stratigraphy since it is continuous.

The test is performed through a hydraulic system that, at a continuous rate, pushes into the ground a cone on the end of a series of rods and constant measurements are made of the resistance to penetration of the tip of the cone, q_c , the lateral resistance to penetration of the surface sleeve, f_s , and in CPT(U) tests it is also possible to measure the pore water pressure, u . A schematic example is given, in Figure 2.30, of the penetrometer used for this test and the indication of the measurements that it takes.

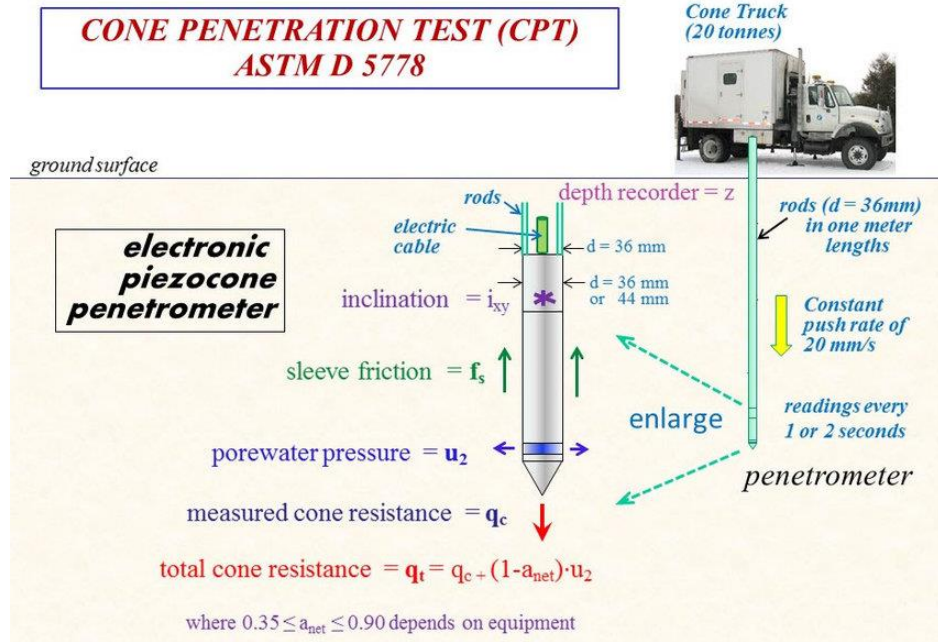


Figure 2.30 Terminology for cone penetration (Mayne, 2018).

The CPT is the most common in situ test made in offshore wind farms. Since it gives the most interesting results due to detailing, it is possible to obtain a continuous stratigraphy of the soil.

2.3.3.2 CRR curves for Clean Sands based on CPT test results

a) Youd & Idriss (2001)

From Youd & Idriss (2001), also a correlation for clean sand for CPT can be found. The normalization of the tip resistance is required and can be yielded by the following transformation, which will originate a dimensionless cone penetration resistance, q_{c1N} .

$$q_{c1N} = C_Q * \left(\frac{q_c}{P_a} \right) \quad (2.28)$$

Where,

$$C_Q = \left(\frac{P_a}{\sigma'_{v0}} \right)^n \quad (2.29)$$

C_Q is the normalizing factor for the cone penetration resistance; P_a is the atmospheric pressure and it is equal to 100 kPa = 1 atm; n is an exponent that varies from 0.5 to 1 depending on the grain characteristics of the soil type (Olsen 1997), however, for this matter will not be discussed and the value $n=0.75$ will be considered.

Now, to relate the q_{c1N} with CRR the following curve can be plotted (Figure 2.31):

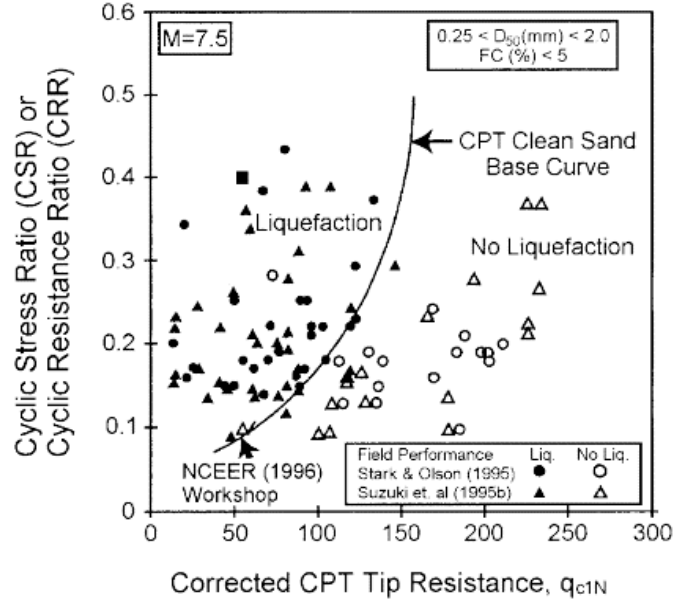


Figure 2.31 Curve recommended for calculation of CRR from CPT data along with empirical liquefaction data from compiled case histories (reproduced from Robertson and Wride, 1998) (Youd & Idriss, 2001)

The curve represented in Figure 2.31 was approximated by the following equation:

$$\text{If } (q_{c1N})_{cs} < 50 \text{ then } CRR_{7.5} = 0.833 * \left(\frac{(q_{c1N})_{cs}}{1000} \right) + 0.05 \quad (2.30)$$

$$\text{If } 50 \leq (q_{c1N})_{cs} < 160 \text{ then } CRR_{7.5} = 93 * \left(\frac{(q_{c1N})_{cs}}{1000} \right)^3 + 0.08 \quad (2.31)$$

Where $(q_{c1N})_{cs}$ is the clean-sand cone penetration resistance normalized to approximately 100 KPa (1atm). The normalized penetration resistance for silty sands, q_{c1N} , is corrected to an equivalent clean sand value by:

$$(q_{c1N})_{cs} = K_c * q_{c1N} \quad (2.32)$$

Where K_c is the correction factor for grain characteristics which depends on the soil behavior type index, I_c , and was defined by Robertson and Wride in 1998 by the following equation:

$$\text{For } I_c \leq 1.64 ; K_c = 1 \quad (2.33)$$

$$\text{For } I_c > 1.64 ; K_c = -0.403 * I_c^4 + 5.581 * I_c^3 - 21.63 * I_c^2 + 33.75 * I_c - 17.88 \quad (2.34)$$

In Youd (2001), it is possible to find how to determine soil behavior type index through the following equations (Robertson, 2010):

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \quad (2.35)$$

where,

$$Q = \left[\frac{q_c - \sigma_{v0}}{P_a} \right] * \left[\left(\frac{P_a}{\sigma'_{v0}} \right)^n \right] \quad (2.36)$$

and,

$$F = \left[\frac{f_s}{q_c - \sigma_{v0}} \right] * 100\% \quad (2.37)$$

The n exponent can be found in Robertson 2008 and it is given by:

$$n = 0.381 * I_c + 0.05 * \left(\frac{\sigma'_{v0}}{P_a} \right) - 0.15 \text{ where } n \leq 1 \quad (2.38)$$

Relative density can be estimated based on $(q_{c1N})_{cs}$ (Idriss and Boulanger, 2008) in (Basu, 2019):

$$D_r = 0.465 * \left(\frac{(q_{c1N})_{cs}}{0.9} \right)^{0.264} - 1.063 \quad (2.39)$$

The Magnitude Scaling factor is the one previously presented in equation (2.14).

b) Boulanger & Idriss (2016)

There is also an approach presented in Boulanger & Idriss (2016) where it is possible to find the following revised deterministic correlation:

$$CRR_{7.5-\sigma'_v=1atm} = \exp \left(\frac{(q_{c1N})_{cs}}{113} + \left(\frac{(q_{c1N})_{cs}}{1000} \right)^2 - \left(\frac{(q_{c1N})_{cs}}{140} \right)^3 + \left(\frac{(q_{c1N})_{cs}}{137} \right)^4 - 2.8 \right) \quad (2.40)$$

The rest of the parameter and calculation necessary are performed similarly as in the previous chapter.

The Magnitude Scaling Factor here is calculated based on Boulanger & Idriss (2016) where they propose a MSF that variates with a relationship between q_{c1Ncs} and $(N_1)_{60cs}$. This variation is represented in Figure 2.32.

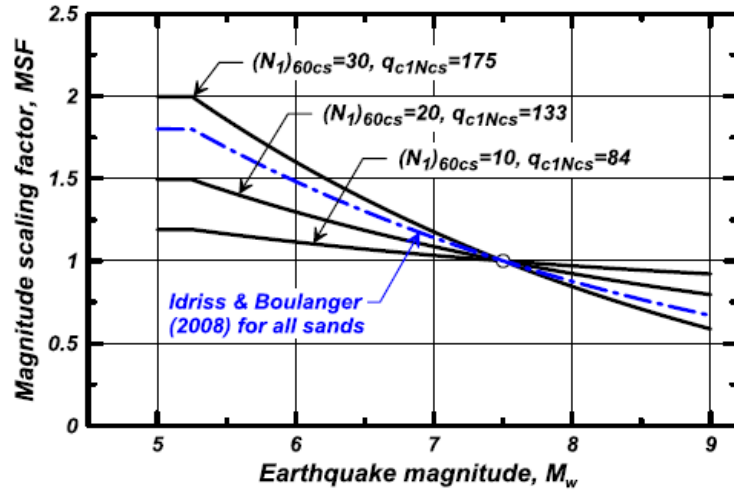


Figure 2.32 Variation in the MSF relationship with q_{c1Ncs} and $(N_1)_{60cs}$ for cohesionless soils (Boulanger & Idriss, CPT - Based Liquefaction Triggering Procedure, 2016).

Based on the regression in the figure, MSF can be expressed by the following equations:

$$MSF = 1 + (MSF_{max} - 1)[8.64 * \exp\left(-\frac{M}{4}\right) - 1.325] \quad (2.41)$$

$$MSF_{max} = 1.09 + \left(\frac{q_{c1Ncs}}{180}\right)^3 \leq 2.2 \quad (2.42)$$

From this, it was possible to determine any variation. So, taking the same values has used for Youd & Idriss (2001) where the $q_{c1Ncs} \sim 75.9$, a new curve was determined (Figure 2.33).

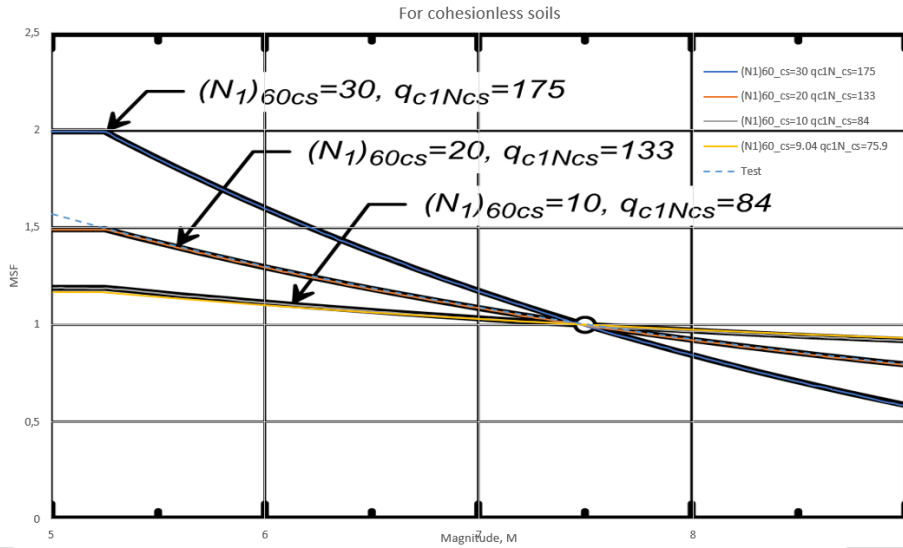


Figure 2.33 Variation in the MSF relationship with q_{c1Ncs} and $(N_1)_{60cs}$ for cohesionless soils.

However, this method is not only for clean sands since it changes with fines content, but not directly. The following Table 2.2 shows how the soil behavior type changes by intervals of soil behavior type index.

Table 2.2 Normalized CPT Soil type index intervals (Robertson, 1990&2010)

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

* Heavily overconsolidated or cemented

So, analyzing Table 2.2 where the $I_c = 1.64$, it is clear that the soil type is clean Sands, but if for instance the friction ratio (f_s) or other parameters the I_c would be higher and it would not be clean sands anymore.

2.3.3.3 CRR curves for Sands and Silty Sands based on CPT test results

For sands and silty sands, the calculations considered are the same, according to Boulanger & Idriss (2014) methodology, presented in the previous chapter 2.3.1.2, and the MSF_{max} is determined the same way as in Equation (2.42).

$$q_{c1N_{cs}} = q_{c1N} + \Delta q_{c1N} \quad (2.43)$$

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6} \right) * \exp \left[1.63 - \frac{9.7}{FC + 2} - \left(\frac{15.7}{FC + 2} \right)^2 \right] \quad (2.44)$$

2.3.3.4 CRR curves for Silts and Clays based on CPT test results

In Boulanger & Idriss (2007) it is possible to find a relation between the number of cycles for sands and the number of cycles for clays for a given magnitude of 7 to 8, that relationship is graphically represented in Figure 2.34.

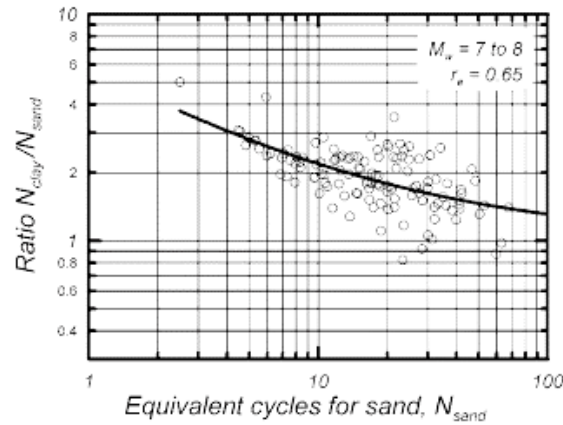


Figure 2.34 Ratio of N_{clay} to N_{sand} for NEHRP class D soil sites, M_w of 7 to 8, and $r_e = 0.65$ (Boulanger & Idriss, 2007).

In order to have the relationship this graphic was worked off and it was possible to obtain, by interpolation, the number of cycles for clay-like soils, N_{clay} at a given Magnitude as showed in Table 2.3.

Table 2.3 Number of cycles conversion from Sands to Clays.

M	Nsand	Nclay/Nsand	Nclay
5.25	2.53	3.71771362	9.40581547
6	6	2.61618756	15.6971253
6.75	10	2.18798981	21.8798981
7.5	15	1.93674971	29.0512456
8.5	26	1.67240969	43.482652

The Magnitude Scaling Factor for clays is not the same as for sand, it is given by (Boulanger & Idriss, 2007) and can be determined from the following equation:

$$MSF = 1.12 * e^{-\frac{M_w}{4}} + 0.828 \quad (2.45)$$

Figure 2.35 shows the MSF relationships for clay and sand, and it is possible to observe that the MSF for clays is much flatter than for sands (i.e. less dependency on number of cycles and hence, M), which reflects the differences in the slope of the cyclic strength versus number of cycles curves obtained for these soil types.

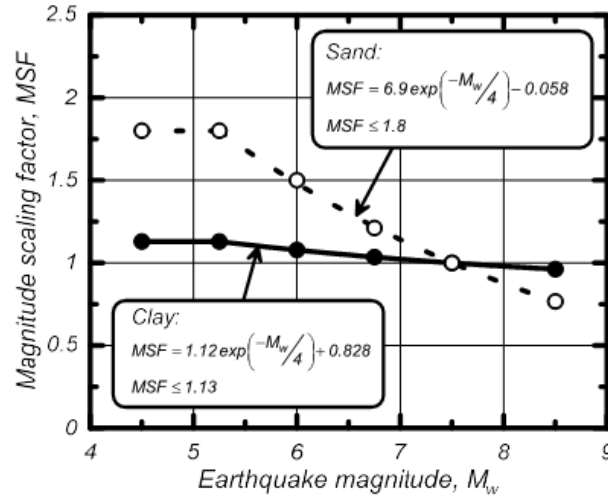


Figure 2.35 MSF relationships for clay and sand (Boulanger & Idriss, 2007).

The CRR of clay-like fine-grained soil, for a given magnitude earthquake, can be estimated by:

$$CRR_M = C_{2D} * \left(\frac{\tau_{cyc}}{s_u} \right)_{M=7.5} * \frac{s_u}{\sigma'_{vc}} * MSF * K_\alpha \quad (2.46)$$

Where,

- C_{2D} is a correction for two-dimensional versus one-dimensional cyclic loading;
- $\left(\frac{\tau_{cyc}}{s_u} \right)_{M=7.5}$ is the cyclic stress ratio to s_u for the number of equivalent uniform cycles of an $M=7.5$;
- $\frac{s_u}{\sigma'_{vc}}$ is the undrained shear strength ratio;
- σ'_{vc} is the vertical effective consolidation stress;
- K_α is the correction factor to represent the effects of an initial static shear stress ratio (α) on the liquefaction resistance of sands. In this case $\alpha = 0^\circ$ and so $K_\alpha=1$.

2.4 CONSTITUTIVE MODELLING OF LIQUEFIABLE SOILS

2.4.1 INTRODUCTION

There has been an important care with the mechanical behaviour of the soil and a numerous number of constitutive soil models have been developed through the years in order to best assess the behaviour of the soil. Being liquefaction an already complex event, it is very important to understand which soils are susceptible to it and how they behave during and after that event, therefore constitutive models have been developed for different soil types to better replicate the behaviour.

For this project, two constitutive soil models were chosen to recreate the behaviour of the soil: the HSsmall (Hardening Soil model with small-strain stiffness) for clays and the PM4Sand for sands. The choice of the models was based on the available models on the numerical modelling software PLAXIS2D, that will be used in this project, and their adequacy for correctly describing the behaviour of these soils, especially under earthquake loading.

2.4.2 HARDENING SOIL MODEL

HSsmall, a short term for Hardening Soil Model with small-strain stiffness, is an advanced model that simulates the behavior of the soil. It is an elastoplastic type of hyperbolic model and it is similar to the Hardening Soil model (HS). Moreover, this model incorporates the strain-dependent stiffness moduli, simulating the different reactions of soils from small strains (below 10^{-5}) to large strains (above 10^{-3}).

HSsmall model has the same parameters as the HS model and an additional two stiffness parameters as input. Being the input parameters from HS presented in Table 2.4 and the two parameters for small strain stiffness presented on Table 2.5.

Table 2.4 Stiffness parameters.

Parameter	Symbol	Unit
Plastic straining due to deviatoric loading / Secant stiffness in standard drained triaxial test	E_{50}^{ref}	N/m^2
Plastic straining due to primary compression / Tangent stiffness for primary oedometer loading	E_{oed}^{ref}	KN/m^2
Elastic Unloading/reloading stiffness at engineering strains ($\varepsilon \simeq 10^{-3}$ to 10^{-2})	E_{ur}^{ref}	KN/m^2
Power for stress-level dependency of stiffness	m	-

Table 2.5 Small strain stiffness parameters.

Parameter	Symbol	Unit
Shear strain level, at which the secant shear modulus, G_s , is reduced to about 70% of G_0 , $G_s = 0,722G_0$	$\gamma_{0,7}$	-
Reference shear modulus at very small strains ($\varepsilon \leq 10^{-6}$)	G_0^{ref}	KN/m^2

2.4.2.1 Describing small-strain stiffness with a simple hyperbolic law

The most frequently used model in soil dynamics is the Hardenin-Drnevich (1972) relationship

$$\frac{G_s}{G_0} = \frac{1}{1 + \left| \frac{\gamma}{\gamma_r} \right|} \quad (2.47)$$

$$\gamma_r = \frac{\tau_{max}}{G_0} \quad (2.48)$$

Santos & Correia (2001) suggested using $\gamma_r = \gamma_{0.7}$ at which the secant shear modulus G_s is reduced to 70% of its initial value, and so Equation (2.47) can be rewritten as:

$$\frac{G_s}{G_0} = \frac{1}{1 + a \left| \frac{\gamma}{\gamma_{0.7}} \right|} \text{ with } a = 0.385 \quad (2.49)$$

Using this value of a and $\gamma = \gamma_{0.7}$ then $G_s/G_0 = 0.722$ so the formulation is, after all, 72.2%. Figure 2.36 shows this modified relationship.

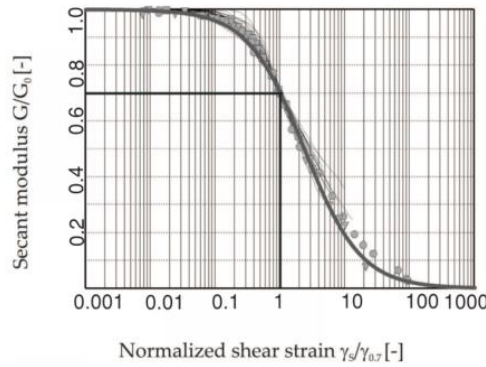


Figure 2.36 Results from the Hardin-Drnevich relationship compared to test data by Santos & Correia (2001),

2.4.2.2 Applying the Hardin-Drnevich relationship HS model

The tangent shear modulus is given by:

$$G_t = \frac{G_0}{\left(1 + 0.385 \frac{\gamma}{\gamma_{0.7}} \right)^2} \quad (2.50)$$

The stiffness reduction curve reaches far into the plastic material domain. In the models HS and HSsmall, stiffness degradation due to plastic straining is simulated with strain hardening. In the HSsmall model, the small-stiffness reduction curve is therefore bound by certain lower limit, determined by conventional laboratory tests:

- The lower cut-off of the tangent shear modulus G_t is introduced at the unloading reloading stiffness G_{ur} :

$$G_t \geq G_{ur} \text{ where } G_{ur} = \frac{E_{ur}}{2(1 + \nu_{ur})} \text{ and } G_t = \frac{E_t}{2(1 + \nu_{ur})} \quad (2.51)$$

- The cut-off shear strain $\gamma_{cut-off}$:

$$\gamma_{cut-off} = \frac{1}{0.385} \left(\sqrt{\frac{G_0}{G_{ur}}} - 1 \right) \gamma_{0.7} \quad (2.52)$$

With the HSsmall model, the quasi-elastic tangent shear modulus is calculated by integrating the secant stiffness modulus reduction curve over the actual shear strain increment. An example is presented in Figure 2.37.

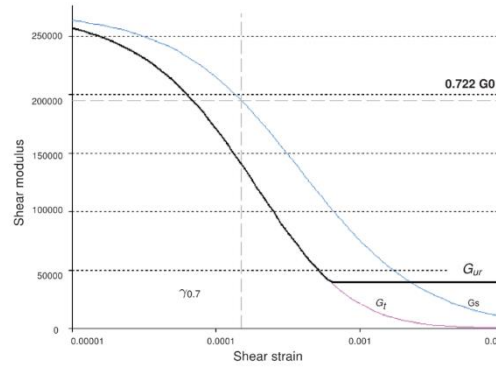


Figure 2.37 Secant and tangent shear modulus reduction curve.

2.4.2.3 Model parameters

The parameters required for HSsmall model are the same as for the HS model, as previously exposed, and the input stiffness parameters are presented in Table 2.4 and Table 2.5.

Figure 2.38 illustrates the model's stiffness parameters in a drained triaxial test. And Figure 2.39.

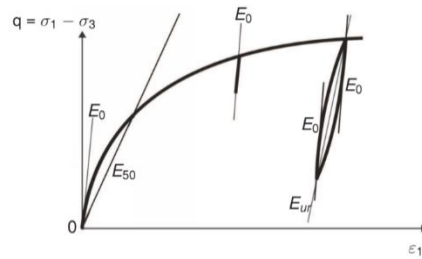


Figure 2.38 Stiffness parameters E_{50} , E_{ur} , and $E_0 = 2G_0(1 + \nu_{ur})$ of the Hardening Soil model with small-strain stiffness in a triaxial test.

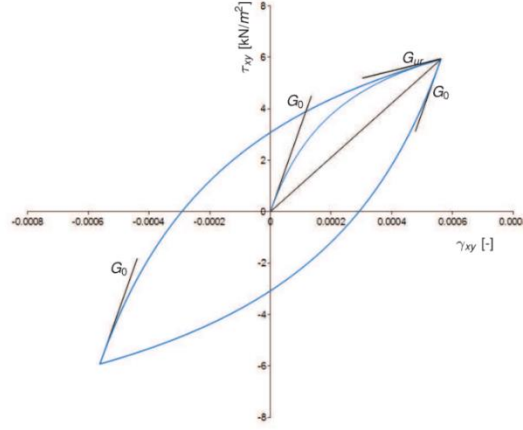


Figure 2.39 Stiffness parameters in cyclic shear test.

$$E_{50}^{ref} = 1.25 * E_{oed}^{ref} \quad (2.53)$$

In the HSsmall model, the stress dependency of the shear modulus G_0 is taken into account with the power law:

$$G_0 = G_0^{ref} \left(\frac{c * \cos \varphi - \sigma'_3 * \sin \varphi}{c * \cos \varphi - p^{ref} * \sin \varphi} \right)^m \quad (2.54)$$

which resembles the ones used for the other stiffness parameters.

Knowledge of a material's initial void ratio can be very helpful in deriving its small-strain shear stiffness G_0 , and there are many correlations proposed in the literature. The one given by Hardin & Black (1969) is a good estimation for different soils:

$$G_0^{ref} = 33 * \frac{(2.97 - e)^2}{1 + e} \text{ (MPa) with } p^{ref} = 100 \text{ (KPa)} \quad (2.55)$$

In the absence of test data, correlations for the threshold shear strain $\gamma_{0,7}$ are also available. For example, applying in Eq 7.2 and 7.3 the Mohr-Coulomb failure criteria (Vilhar, Brinkgreve, & Zampich, 2018):

$$\gamma_{0,7} \simeq \frac{1}{9G_0} (2c'(1 + \cos(2\varphi')) - \sigma'_1(1 + K_0) \sin(2\varphi')) \quad (2.56)$$

where K_0 is the earth pressure coefficient at rest and σ'_1 is the effective vertical stress (negative pressure).

2.4.3 PM4SAND SOIL MODEL

2.4.3.1 Introduction

The PM4Sand soil model is a sand plasticity model for earthquake engineering. It follows the basic framework of the stress-ratio controlled, critical-state compatible, bounding-surface plasticity model for sands that has been elaborated by (Dafalias & Manzari, 2004). In the version used in this research, a fabric-dilatancy tensor was added to take into consideration the effects of fabric change during loading. The model was developed based on effective stresses, considering the generation of excess pore pressure due to cyclic loading under undrained conditions. The model was formulated in two dimensions.

This model considers elastic and plastic strain increments which are composed by volumetric and deviatoric terms. The elastic strain increments are restricted by two parameters of the soil material: the shear modulus, G , and the bulk modulus, K ; and are generated according to acting stress levels. The plastic strain increments are restricted by dilatancy, D , and the distance between the stress levels in comparison to the position of the yield surface, through the normal tensor, n ; and are produced by the loading index L (Toloza, 2018).

2.4.3.2 Critical state soil mechanics framework

This model uses the relative state parameter index, ξ_R , which is defined as:

$$\xi_R = D_{R,cs} - D_R \quad (2.57)$$

Where D_R is the current apparent relative density and $D_{R,cs}$ is the relative density at the critical state line at the current mean effective stress, p , and it is defined as follows:

$$D_{R,cs} = \frac{R}{Q - \ln\left(100 * \frac{p}{p_A}\right)} \quad (2.58)$$

Being p_A the atmospheric pressure while Q and R are Bolton's parameter (Bolton, 1986)).

Figure 2.40 shows an example of the critical state line at $D_R - p'/p_A$ plane with the parameters Q and R , respectively, of 10 and 1.5.

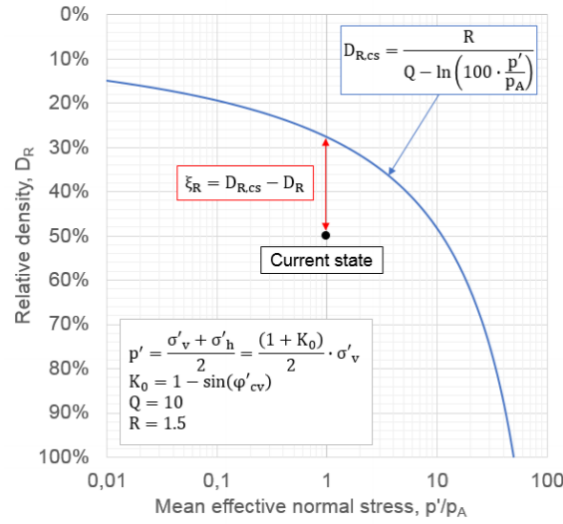


Figure 2.40 Definition of the relative state parameter index (Toloza, 2018).

2.4.3.3 Bounding, Dilatancy, Critical and Yield Surfaces

The model uses bounding, dilatancy, and critical surfaces according to Dafalias and Manzari (2004). The bounding and dilatancy surfaces are defined with the following stress ratios, which depend on the relative state index, ξ_R :

$$M^b = M * \exp(-n^b * \xi_R) \quad (2.59)$$

$$M^d = M * \exp(n^d * \xi_R) \quad (2.60)$$

$$M = 2 * \sin(\varphi_{cv}) \quad (2.61)$$

Where n^b and n^d are de model parameter defining the computation of M^b and M^d concerning M. φ_{cv} is the critical state, at a constant volume, effective friction angle, and it is also a model parameter.

During the shearing of the model, the bounding and dilatancy surfaces, M^b and M^d , tend to approach the critical surface, M, and at the same time the relative state parameter index approaches the critical state line, i.e., ξ_R tends to zero. (Vilhar, Brinkgreve, & Zampich, 2018).

The dilatancy surface defines the location where the modification from contractive to dilative behaviour occurs (Toloza, 2018). The yield surface is formulated as a small cone in the stress space with the following expression:

$$f = \sqrt{(s - p\alpha):(s - p\alpha)} - \sqrt{\frac{1}{2}} pm = 0 \quad (2.62)$$

where, the back-stress ratio tensor α denotes the position of the yield surface in the deviatoric stress ratio space, and m is the radius of the cone, hence the size of the yield surface with the predefined value of 0.01.

In the general formulation of the model, the bounding and dilatancy surfaces are defined in terms of the image back stress ratios α^b and α^d as:

$$\alpha^b = \sqrt{\frac{1}{2}} [M^b - m] * n \quad (2.63)$$

$$\alpha^d = \sqrt{\frac{1}{2}} [M^d - m] * n \quad (2.64)$$

where n is the deviatoric unit normal to the yield surface defined as:

$$n = \frac{\mathbf{r} - \alpha}{\sqrt{\frac{1}{2}} m} \quad (2.65)$$

and $\mathbf{r}$ is the deviatoric stress ratio tensor defined as:

$$\mathbf{r} = \frac{\mathbf{s}}{p} \quad (2.66)$$

Figure 2.41 presents a scheme of the yield, dilatancy, bounding, and critical surfaces, tensor $\mathbf{n}$ and the image back-stress ratios in $r_{yy} - r_{xy}$ plane (Vilhar, Brinkgreve, & Zampich, 2018).

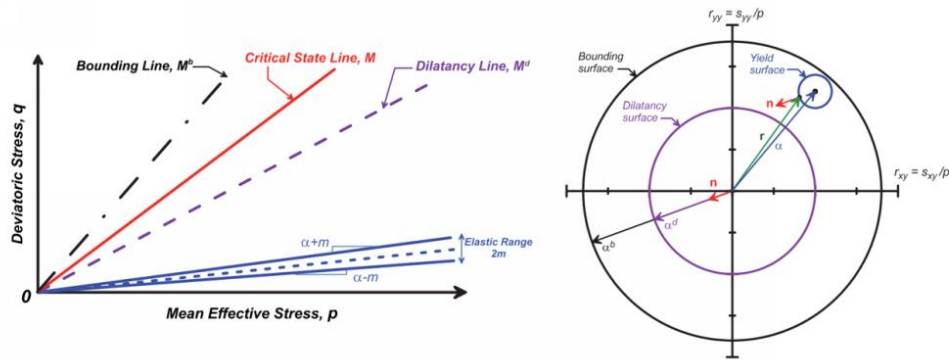


Figure 2.41 Yield, critical, dilatancy and bounding lines in q - p space, on the left, and yield, dilatancy and bounding surfaces in the $r_{yy} - r_{xy}$ stress-ratio plane on the right, (Toloza, 2018).

The distance between the yield surface axis α and the image back-stress ratio α^b defines the plastic modulus K_p , while the distance between α and α^d defines the amount of dilatancy or contraction.

2.4.3.4 Model parameters

For the soil model PM4Sand to be implemented in PLAXIS, 13 input parameters are necessary, that the user can manually manipulate. From those 13 parameters, 4 of them are considered the primary input parameters, which define the model behaviour, the remaining 9 are the secondary parameters and are considered as default values. Subsequently, all of the 13 parameters will be presented, and it will be specified how they can be determined from lab tests and the existing proposed correlations (Vilhar, Brinkgreve, & Zampich, 2018).

2.4.3.5 Primary input parameters:

1. Apparent Relative Density, D_R .

The relative density, D_R , has an important influence on all phases of the model formulation, this parameter controls the dilatancy and stress-strain relation response characteristics.

This variable can be estimated based on CPT or SPT test results, by the relations given by Idriss and Boulanger (2008):

$$D_R = \sqrt{\frac{(N_1)_{60}}{C_d}} \quad (2.67)$$

for SPT results and where C_d is the adopted value of 46 in the development of liquefaction triggering correlations, this value is a simplified approach that does not account for fines. If fines are present, a higher value can be expected. For CPT results, the following expression was developed by the same authors:

$$D_R = 0.465 * \left(\frac{q_{c1N}}{C_{dq}} \right)^{0.264} - 1.063 \quad (2.68)$$

where $C_{dq} = 0.9$ was adopted.

When laboratory tests are possible and available, the relative density can be determined taking into consideration the maximum (e_{max}) and minimum (e_{min}) void ratios as follows:

$$D_R = \frac{e_{max} - e}{e_{max} - e_{min}} \quad (2.69)$$

Also, D_R is best considered an "apparent relative density" rather than a strict measure of relative density from conventional laboratory tests. This parameter can be modified in some situations by

adjusting, relative to the above relationships, the input D_R up or down for calibration purposes (Vilhar, Brinkgreve, & Zampich, 2018).

2. Shear Modulus Coefficient, G_0 .

The shear modulus coefficient, G_0 , corresponds to the shear modulus at small strains, from which the elastic shear modulus G at a given mean effective stress can be computed as :

$$G = G_0 * p_A * \sqrt{\frac{p}{p_A}} \quad (2.70)$$

G_0 should be calibrated to fit estimated or measured V_s values, according to the following expression:

$$G_0 = \rho * (V_s)^2 \quad (2.71)$$

or fitted to values of estimated V_s by correlations with SPT results, such as:

$$V_s = 85 * [(N_1)_{60} + 2.5]^{0.25} \quad (2.72)$$

The G_0 parameter can also be calculated as proposed by Boulanger and Ziotopoulou (2017) by a correlation with SPT results in the form of:

$$G_0 = 167 * \sqrt{(N_1)_{60} + 2.5} \quad (2.73)$$

or as a correlation with relative density,

$$G_0 = 167 * \sqrt{46 * D_R^2 + 2.5} \quad (2.74)$$

(Vilhar, Brinkgreve, & Zampich, 2018).

3. Contractive Rate Parameter, h_{po}

The contractive rate parameter, h_{po} permits the adjustment of plastic volumetric strains during contraction, and hence the model is being calibrated to obtain a certain cyclic resistance ratio (CRR) accordingly to the penetration resistances, from in-situ soil results, or cyclic laboratory tests. This can be performed by using the soil facility 'cyclic DSS test' in PLAXIS 2D to produce uniform cyclic simple shear tests to calibrate the parameter. Also, the contractive rate parameter should be the last calibrated after the other parameter values have been assigned (Vilhar, Brinkgreve, & Zampich, 2018) and (Toloza, 2018).

4. Atmospheric pressure, p_A

The default value to the atmospheric pressure is 101.3 kPa, to be consistent with the original formulation (Vilhar, Brinkgreve, & Zampich, 2018).

2.4.3.6 Secondary parameters

5. Maximum and minimum void ratios, e_{max} and e_{min}

The maximum and minimum void ratios (e_{max} and e_{min}) influence the computation of the relative state index, ξ_R , and they affect the relationship between volume changes and the relative state index. Boulanger and Ziotopoulou (2017) recommend that e_{max} and e_{min} values are 0.8 and 0.5, respectively (Vilhar, Brinkgreve, & Zampich, 2018).

6. Bounding surface parameter, n^b

The bounding surface parameter, n^b , default value is 0.5 and it controls the relative position of the bounding surface to the critical state surface. It also controls dilatancy and thus also the peak effective friction angles. For looser than critical states ($\xi_R > 0$), M^b is computed using $n^b/4$.

7. Dilatancy surface parameter, n^d

The dilatancy surface parameter, n^d , default value is 0.1 and it controls the stress ratio at which the contraction transitions to dilation and also when dilation transitions into contraction, these transformations are often referred to as the phase transformation. The default value of 0.1 for n^d , produces a phase transformation angle slightly smaller than φ_{cv} , which is consistent with the experimental data (Boulanger and Ziotopoulou, 2017). As with the bounding surface, for looser than critical states ($\xi_R > 0$), M^d is computed using $4n^b$ (Vilhar, Brinkgreve, & Zampich, 2018).

8. Critical state friction angle, φ_{cv}

This parameter defines the position of the critical state surface, i.e. the value of M stress ratio from (2.61). The default value for the critical state friction angle is 33° (Vilhar, Brinkgreve, & Zampich, 2018).

9. Poisson's ratio, ν

The Poisson ratio default value is 0.3

10. Critical state line parameters, Q and R

The parameters Q and R define the critical state line. And its default values, recommended by Bolton (1986) are 10 and 1, respectively. However, Boulanger and Ziotopoulou (2017) used a slightly higher R, with the value of 1.5, in order to lower the critical state line to have a better approximation of the typical results for direct shear loading (Vilhar, Brinkgreve, & Zampich, 2018).

11. Post shake switch, **PostShake**

This parameter is used to activate the reduction of elastic stiffness to simulate the post-shaking reconsolidation. Its default value is 0.

12. Other parameters

The remaining model parameters were taken equal to the default values from the index properties according to Boulanger & Ziotopoulou (2017).

2.4.3.7 Elastic Part of the model

The elastic volumetric and deviatoric terms are defined as:

$$d\varepsilon_v^{el} = \frac{dp}{K} \quad (2.75)$$

$$de^{el} = \frac{ds}{2G} \quad (2.76)$$

Where G is the elastic shear modulus and K the elastic bulk modulus. And the elastic shear modulus depends on the effective stress, stress ratio and fabric according to the expression:

$$G = G_0 * p_A \sqrt{\frac{p}{p_A}} C_{SR} \left(\frac{1 + \frac{z_{cum}}{z_{max}}}{1 + \frac{z_{cum}}{z_{max}} * C_{GD}} \right) \quad (2.77)$$

where C_{SR} represents the stress ratio effects and is defined by:

$$C_{SR} = 1 - C_{SR,0} \left(\frac{M}{M^b} \right)^{m_{RS}} \quad (2.78)$$

The two equations put together are as follows:

$$G = G_0 * p_A \sqrt{\frac{p}{p_A}} \left(1 - C_{SR,0} \left(\frac{M}{M^b} \right)^{m_{RS}} \right) * \left(\frac{1 + \frac{z_{cum}}{z_{max}}}{1 + \frac{z_{cum}}{z_{max}} * C_{GD}} \right) \quad (2.79)$$

G_0 is the shear modulus coefficient, and it considers the small shear modulus; $C_{SR,0}$ and m_{SR} impose the stress ratio effects according to Yu & Richard Jr. (1984) and they take the values of 0.5 and 4 respectively. This keeps the effect of stress ratio on elastic modulus to be small at small stress ratios but lets the effect increase to a 60% reduction when the stress ratio is on the bounding surface (Boulanger and Ziotopoulou, 2017). z_{cum} represents the cumulative value of absolute changes of the fabric tensor z :

$$dz_{cum} = |dz| \quad (2.80)$$

And z_{max} is the parameter computed at the time of the model initialization according to the initial relative state index ξ_{R0} as:

$$z_{max} = 0.7 \exp(-6.1\xi_{R0}) \leq 20 \quad (2.81)$$

C_{GD} is the factor that controls the shear modulus degradation at large values of z_{cum} , and it is set internally to a value of 2.0.

The elastic bulk modulus, K , is related to the shear modulus G by the following equation:

$$K = \frac{2(1 + \nu)}{3(1 - 2\nu)} G \quad (2.82)$$

being ν the Poisson ratio at a recommended constant value of 0.3.

2.4.3.8 Plastic Part of the model

2.4.3.9 Plastic strains loading index and stress increment

The increment of plastic deviatoric and volumetric strains are calculated according to the following expressions:

$$d\varepsilon^{pl} = \langle L \rangle D \quad (2.83)$$

$$de_v^{pl} = \langle L \rangle n \quad (2.84)$$

Where $\langle _ \rangle$ are the MacCauley brackets set negative values to zero; L is the loading index, D is the dilatancy, and $\mathbf{n}$ is the deviatoric unit normal to the yield surface. The loading index, L , can be calculated as follows:

$$L = \frac{2G\mathbf{n} : d\mathbf{e} - \mathbf{n} : \mathbf{r} K d\varepsilon_v}{K_p + 2G - K D \mathbf{n} : \mathbf{r}} \quad (2.85)$$

The stress incremented is calculated as:

$$d\sigma = 2G d\mathbf{e} + K d\varepsilon_v \mathbf{I} - \langle L \rangle (2G \mathbf{n} + K D \mathbf{I}) \quad (2.86)$$

2.4.3.10 Hardening-Softening Rule and Plastic Modulus

The evolution of the back-stress ratio α corresponding to the axis of the yield surface is using according to Dafalias & Manzari (2014) as:

$$d\alpha = \langle L \rangle \frac{2}{3} h (\alpha^b - \alpha) \quad (2.87)$$

where h is the hardening coefficient and is related to the plastic modulus K_p through the expression:

$$h = \frac{3}{2} * \frac{K_p}{p(\alpha^b - \alpha) : \mathbf{n}} \quad (2.88)$$

The plastic modulus is defined as:

$$K_p = Gh_0 * \frac{\sqrt{(\alpha^b - \alpha):n}}{[\exp\left(\left((\alpha - \alpha_{in}^{app}):n\right) - 1\right) + C_{\gamma l}] + C_{k\alpha}} C_{rev} \quad (2.89)$$

$$* \frac{C_{k\alpha}}{1 + C_{Kp} \left(\frac{Z_{peak}}{Z_{max}}\right) \langle (\alpha^b - \alpha):n \rangle \sqrt{1 - C_{zpk2}}}$$

where,

$$C_{rev} = \frac{(\alpha - \alpha_{in}^{app}):n}{(\alpha - \alpha_{in}^{true}):n} \quad \text{for} \quad (\alpha - \alpha_{in}^{app})n \leq 0 \quad (2.90)$$

Otherwise $C_{rev} = 1$

h_0 is the parameter adjusting the ratio between plastic and elastic moduli, and it is internally set according to the initial relative density, D_{R0} , by the expression:

$$h_0 = \frac{(0.25 + D_{R0})}{2} \geq 0.3 \quad (2.91)$$

The constant $C_{\gamma l}$ is defined to avoid division by zero, and it is internally set as $h_0/200$. The initial back-stress ratio α_{in} is chosen between an apparent back-stress ratio α_{in}^{app} and a true back-stress ratio tensor α_{in}^{true} through the implementation of C_{rev} coefficient to avoid over-stiffness.

2.4.3.11 Plastic volumetric strains - Contraction

The plastic volumetric contraction occurs when $(\alpha^d - \alpha):n > 0$, so dilatancy > 0 and can be calculated as:

$$D = A_{dc} [(\alpha - \alpha_{in}):n + C_{in}]^2 * \frac{(\alpha^d - \alpha):n}{(\alpha^d - \alpha):n + C_D} \quad (2.92)$$

An upper limit is defined to prevent numerical issues:

$$D \leq 1.5 * A_{d0} \frac{(\alpha^d - \alpha):n}{(\alpha^d - \alpha):n + C_D} \quad (2.93)$$

where,

$$A_{dc} = \frac{A_{d0}(1 + \langle z:n \rangle)}{h_p C_{dz}} \quad (2.94)$$

The dilatancy, D , is proportional to the constant A_{d0} and the distance of the back-stress ratio α to de dilatancy back-stress ratio α^d . The constant A_{d0} is related to the dilatancy relationship proposed by Bolton (1986), who showed the difference between peak and constant volume friction angles could be approximated according to:

$$\varphi_{pk} - \varphi_{cv} = -0.8\psi \quad (2.95)$$

where φ_{pk} is the peak friction angle of the shearing resistance; φ_{cv} is the angle of shearing resistance at constant volume and ψ is the dilatancy angle.

$$A_{d0} = \frac{1}{0.4} * \frac{\arcsin\left(\frac{M^b}{2}\right) - \arcsin\left(\frac{M}{2}\right)}{M^b - M^d} \quad (2.96)$$

And h_p function controls the dilatancy and depends on the contraction parameter h_{p0} and the current relative state parameter index, ξ_R , as presented in the following equation:

$$h_p = h_{p0} \exp(-0.7 + 7(0.5 - \xi_R)^2) \quad \text{for} \quad \xi_R \leq 0.5 \quad (2.97)$$

$$h_p = h_{p0} \exp(-0.7) \quad \text{for} \quad \xi_R > 0.5$$

The contraction rate parameter h_{p0} allows to calibrate the model for a specific cyclic stress ratio and number of cycles, the term C_{in} depends on the fabric and it is used to enhance the contractions rate at the start of an unloading cycles. The C_D constant is internally set as 0.16. The C_{dz} term improves modeling of cyclic strength of denser sands.

2.4.3.12 Plastic volumetric strains - Dilation

The plastic volumetric dilation occurs when $(\alpha^d - \alpha):n < 0$, so dilatancy < 0 and can be calculated as:

$$D = A_{d0}[(\alpha_d - \alpha):n] \quad (2.98)$$

A rotated dilatancy surface with slope M^{dR} has been added which involves with the history of the fabric tensor z to facilitate earlier dilation at low stress ratios under certain loading paths (Boulanger & Ziotopoulou, 2015). The rotation surface is defined as:

$$M^{dR} = \frac{M^d}{C_{rot1}} \quad (2.99)$$

$$\alpha^{dR} = \frac{1}{\sqrt{2}} * (M^{dR} - m)n \quad (2.100)$$

From this, D is computed by two expressions. One corresponds to the rotated dilatancy surface while the other corresponds to the non-rotated dilatancy surface.

$$D_{rot} = A_d * \frac{\langle z:n \rangle}{\sqrt{2} * z_{max}} * (\alpha_d - \alpha): \frac{n}{C_{DR}} \quad (2.101)$$

$$D_{non-rot} = A_d(-(\alpha_d - \alpha):n) \quad (2.102)$$

where A_d is defined by:

$$A_d = \frac{A_{d0}(C_{zin2})}{\frac{z_{cum}^2}{z_{max}} * \left(1 - \frac{\langle -z:n \rangle}{\sqrt{2}z_{peak}}\right)^3 (C_\varepsilon)^2 (C_{pzp})(C_{pmin})(C_{zin1}) + 1} \quad (2.103)$$

Equation (2.103) presents some terms that must follow certain roles. The first term, A_{d0} , facilitates the progressive growth of strains under symmetric loading; (C_{zin2}) facilitates strain hardening when the plastic shear strain reaches the prior peak value; the term C_e is a calibration constant that modifies the rate of plastic shear strain accumulation; the fourth term, C_{pzp} , causes the effects of fabric on dilation to be diminished whenever the current value of p is near to p_{zp} enabling the model to provide reasonable predictions of responses to a large number of loading cycles; C_{pmin} provides a minimum amount of shear resistance for a soil after it has temporarily reached an excess pore pressure ratio of 100%; C_{zin1} facilitates strain-hardening when stress reversals are not causing fabric changes. And lastly, C_{zin2} causes the dilatancy to be decreased by up to a factor of 3 under conditions of large strains and full stress reversals, which improves the prediction of cyclic strain accumulation during undrained cyclic loading (Boulanger and Ziotopoulou, 2015).

Factor C_{DR} depends on the initial relative density of the soil. The dilatancy, D , is defined as follows:

$$\text{if } D_{non-rot} < D_{rot} \quad \text{then} \quad D = D_{non-rot} \quad (2.104)$$

$$\text{else} \quad D = D_{non-rot} + (D_{rot} - D_{non-rot}) * \frac{\langle M^b - M^{cur} \rangle}{\langle M^b - M^{cur} \rangle + 0.01} \quad (2.105)$$

Figure 2.42 shows the dilatancy calculated based on the stress state with respect to M^{dR} , M^d and M^b surfaces during half-cycle of loading, from contraction to dilation.

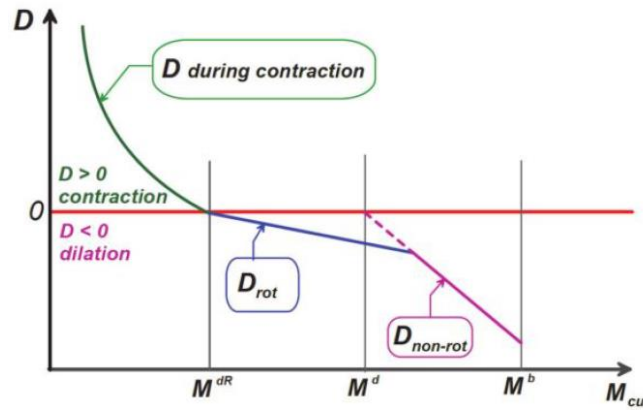


Figure 2.42 Dilatancy, D , calculation based on the stress state with respect to M^{dR} , M^d and M^b surfaces during half-cycle of loading, from contraction to dilation (Boulanger & Ziotopoulou, 2015).

The distance between α and α^d defines the amount of dilatancy or contractancy experienced by the soil.

3 SENSITIVITY STUDIES ON DIFFERENT EMPIRICAL LIQUEFACTION APPROACHES

3.1 INTRODUCTION

In this chapter, it will be exposed the work carried out with the empirical correlations for liquefaction potential assessment presented in the previous chapter. For a better understanding of this subject and in order to understand how and which different parameters and different methodologies affect CRR-N curves, an extensive exercise of plotting these curves, using the theory presented in the literature review, was executed in Excel.

The purpose was to study the influence of the different approaches on the plot of the CRR-N curves, previously presented in section 2.3, and from those conclude which would be the best to consider as the theoretical target curve for the present work.

It is now crucial to present, in Table 3.1, the relationship considered between the earthquake magnitude, M_w , and the number of cycles, N , which was taken from Kramer (1996):

Table 3.1 Equivalent number of uniform stress cycles (Kramer, 1996).

Earthquake Magnitude	Number of Significant Stress Cycles
$5\frac{1}{4}$	2-3
6	5
$6\frac{3}{4}$	10
$7\frac{1}{8}$	15
$8\frac{1}{2}$	26

For clean sands, the MSF considered was the first one presented in section 2.3.

3.2 CRR_N FROM SPT

3.2.1 CLEAN SANDS

The starting point parameters considered, and respective calculations were as those shown in Table 3.2 and the results are plotted in Figure 3.1.

Table 3.2 Soil parameters for CRR-N curves for SPT Youd & Idriss (2001) approach.

$(N_1)_{60} = 15$	ϕ'	30	p'	66,67	N	M	MSF	CRR_7.5	CRR_M
	z	12,5			2,5	5,25	2,49	0,160	0,399
	γ	18	K0	0,5	6	6	1,77		0,283
	γ_w	10			10	6,75	1,31		0,210
	$\sigma'v$	100	N1_60		11,67	7	1,19		0,191
	CN	0,8		15	15	7,5	1,00		0,160
	N60	18,4	Dr(%)		20,5	8	0,85		0,136
	N1_60	15,0		57	26	8,5	0,73		0,116
$(N_1)_{60} = 20$	ϕ'	30	p'	66,67	N	M	MSF	CRR_7.5	CRR_M
	z	12,5			2,5	5,25	2,49	0,215	0,537
	γ	18	K0	0,5	6	6	1,77		0,381
	γ_w	10			10	6,75	1,31		0,282
	$\sigma'v$	100	N1_60		11,67	7	1,19		0,257
	CN	0,8		20	15	7,5	1,00		0,215
	N60	24,5	Dr(%)		20,5	8	0,85		0,183
	N1_60	20,0		66	26	8,5	0,73		0,156
$(N_1)_{60} = 25$	ϕ'	30	p'	66,67	N	M	MSF	CRR_7.5	CRR_M
	z	12,5			2,50	5,25	2,49	0,292	0,727
	γ	18	K0	0,5	6,00	6	1,77		0,517
	γ_w	10			10,00	6,75	1,31		0,382
	$\sigma'v$	100	N160		11,67	7	1,19		0,348
	CN	0,8		25	15,00	7,5	1,00		0,292
	N60	30,6	Dr(%)		20,50	8	0,85		0,247
	N1_60	25,0		74	26,00	8,5	0,73		0,212

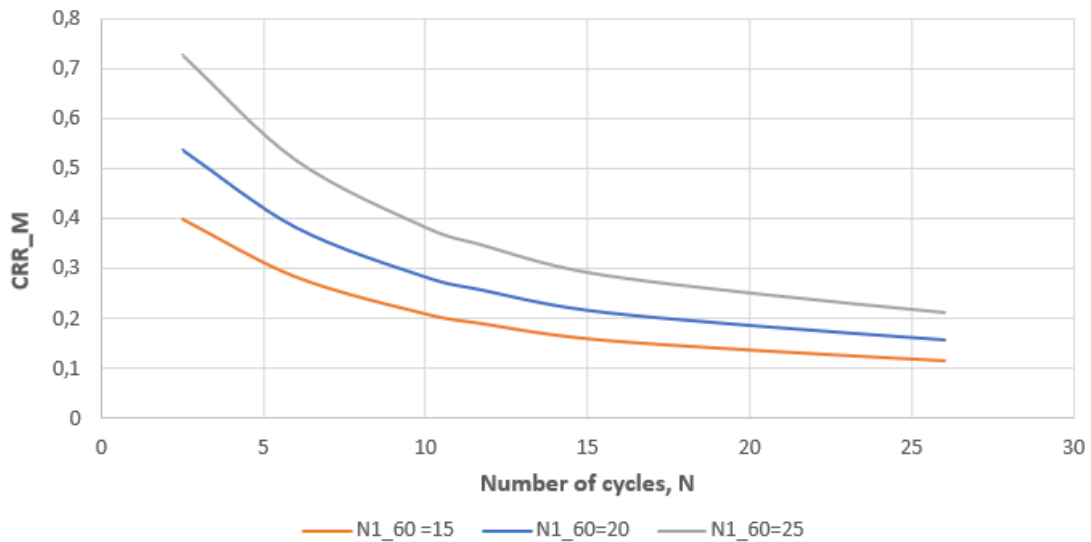


Figure 3.1 CRR-N curves of clean Sands based on Y&I (2001) methodology for SPT test results of $(N_1)_{60} = 15, 20$ and 25 .

From plotting the curves, it is visible that with the increase of $(N_1)_{60}$ the curve moves upwards and so to reach the same CRR value it is necessary a higher number of cycles.

3.2.2 SANDS AND SILTY SANDS

Youd & Idriss (2001) methodology, for SPT test results of Sands and Silty sands, calculations are presented in Table 3.3 and the respective plots in Figure 3.1.

Table 3.3 Soil calculations for CRR-N curves for SPT Youd & Idriss approach.

			N			M			MSF					
			6	10	15	6	10	15	6	10	15	6	10	15
			6	6,75	7,5	6	6,75	7,5	6	6,75	7,5	6	6,75	7,5
			1,770	1,309	1,193	1,770	1,309	1,193	1,770	1,309	1,193	1,770	1,309	1,193
			CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7
			0,283	0,210	0,191	0,283	0,210	0,191	0,283	0,210	0,191	0,283	0,210	0,191
			0,305	0,226	0,205	0,305	0,226	0,205	0,305	0,226	0,205	0,305	0,226	0,205
			0,377	0,279	0,254	0,377	0,279	0,254	0,377	0,279	0,254	0,377	0,279	0,254
			0,429	0,317	0,289	0,429	0,317	0,289	0,429	0,317	0,289	0,429	0,317	0,289
			0,455	0,336	0,306	0,455	0,336	0,306	0,455	0,336	0,306	0,455	0,336	0,306
			0,455	0,336	0,306	0,455	0,336	0,306	0,455	0,336	0,306	0,455	0,336	0,306
			CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7
			0,160	0,136	0,116	0,160	0,136	0,116	0,160	0,136	0,116	0,160	0,136	0,116
			0,172	0,146	0,125	0,172	0,146	0,125	0,172	0,146	0,125	0,172	0,146	0,125
			0,213	0,181	0,155	0,213	0,181	0,155	0,213	0,181	0,155	0,213	0,181	0,155
			0,242	0,205	0,176	0,242	0,205	0,176	0,242	0,205	0,176	0,242	0,205	0,176
			0,257	0,218	0,186	0,257	0,218	0,186	0,257	0,218	0,186	0,257	0,218	0,186
			0,257	0,218	0,186	0,257	0,218	0,186	0,257	0,218	0,186	0,257	0,218	0,186
			Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr
			57,104	59,333	65,618	57,104	59,333	65,618	57,104	59,333	65,618	57,104	59,333	65,618
			70,711	70,711	70,711	70,711	70,711	70,711	70,711	70,711	70,711	70,711	70,711	70,711

			N			M			MSF					
			6	10	15	6	10	15	6	10	15	6	10	15
			6	6,75	7,5	6	6,75	7,5	6	6,75	7,5	6	6,75	7,5
			1,770	1,309	1,193	1,770	1,309	1,193	1,770	1,309	1,193	1,770	1,309	1,193
			CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7
			0,381	0,282	0,257	0,381	0,282	0,257	0,381	0,282	0,257	0,381	0,282	0,257
			0,411	0,304	0,277	0,411	0,304	0,277	0,411	0,304	0,277	0,411	0,304	0,277
			0,524	0,387	0,353	0,524	0,387	0,353	0,524	0,387	0,353	0,524	0,387	0,353
			0,641	0,474	0,432	0,641	0,474	0,432	0,641	0,474	0,432	0,641	0,474	0,432
			0,726	0,537	0,489	0,726	0,537	0,489	0,726	0,537	0,489	0,726	0,537	0,489
			0,726	0,537	0,489	0,726	0,537	0,489	0,726	0,537	0,489	0,726	0,537	0,489
			CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7	CRR M=6	CRR M=6,75	CRR M=7
			0,215	0,183	0,156	0,215	0,183	0,156	0,215	0,183	0,156	0,215	0,183	0,156
			0,197	0,169	0,142	0,197	0,169	0,142	0,197	0,169	0,142	0,197	0,169	0,142
			0,251	0,215	0,186	0,251	0,215	0,186	0,251	0,215	0,186	0,251	0,215	0,186
			0,307	0,263	0,228	0,307	0,263	0,228	0,307	0,263	0,228	0,307	0,263	0,228
			0,348	0,298	0,257	0,348	0,298	0,257	0,348	0,298	0,257	0,348	0,298	0,257
			0,348	0,298	0,257	0,348	0,298	0,257	0,348	0,298	0,257	0,348	0,298	0,257
			Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr	Dr
			65,94	68,05	74,02	65,94	68,05	74,02	65,94	68,05	74,02	65,94	68,05	74,02
			77,73	79,40	79,40	77,73	79,40	79,40	77,73	79,40	79,40	77,73	79,40	79,40

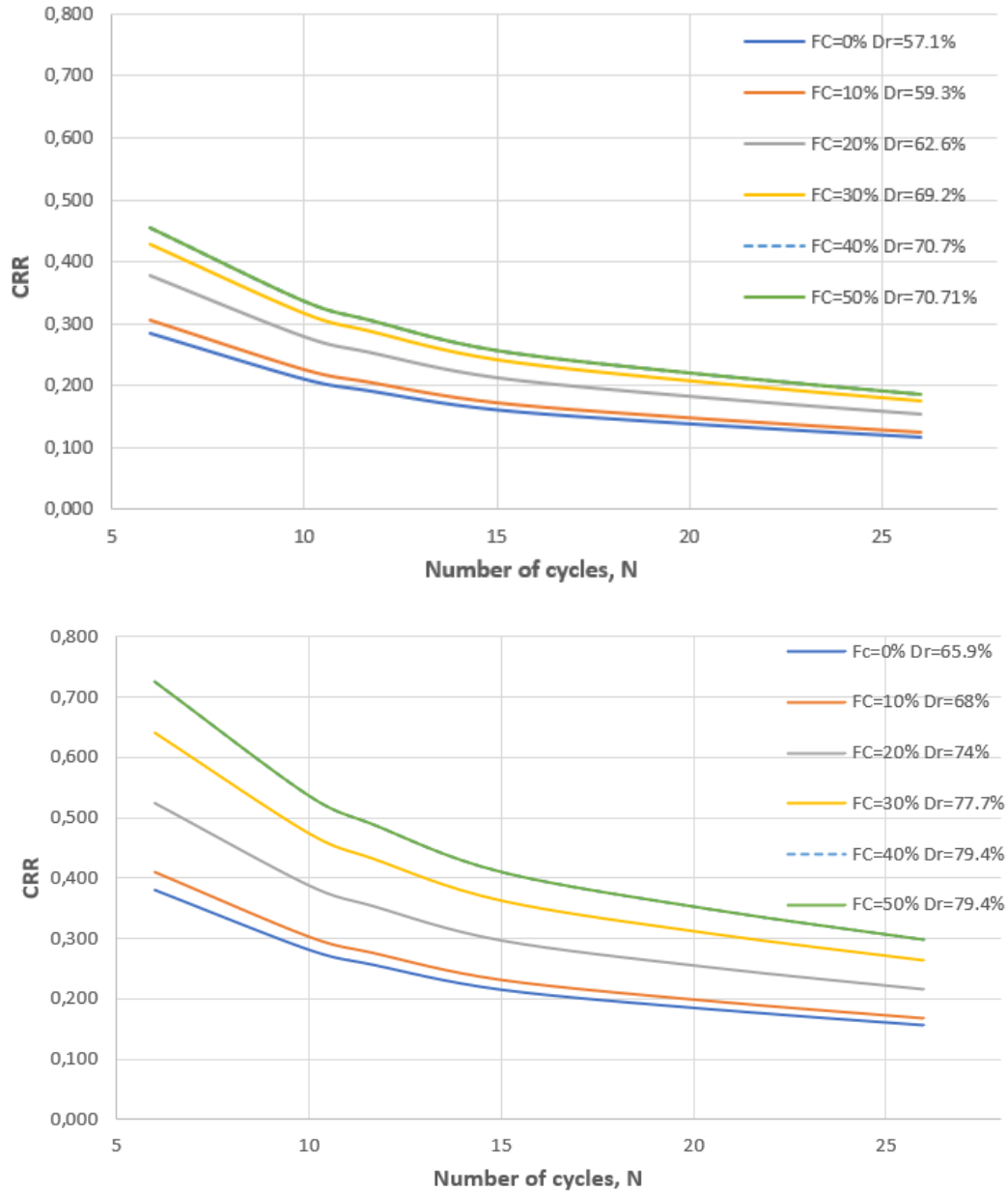


Figure 3.2 CRR-N curves of Sands and Silty sands based on Y&I (2001) methodology for SPT test results of $(N_1)_{60} = 15$ and 20 (top and bottom respectively).

From the plots it is possible to understand how, for a fixed value of $(N_1)_{60}$, the fines content and the relative density influence the curve. In both plots with the increase of fines content and the relative density, the curves move upwards. For fines content of 40% and 50% the curves overlap, this happens because, for fines content over 35%, the values of $(N_1)_{60_{cs}}$ are the same (see Table 2.1 and Equation(2.22)), but the relative density is different. As in clean sands, for a higher $(N_1)_{60}$ and comparing for the same FC, the curves also move upwards what is in agreement with the theory because the $CRR_{7,5}$ base equation is the same.

That means that for higher values of $(N_1)_{60}$, and with increasing fines content and relative density, a higher number of cycles is needed for the same CRR.

Boulanger & Idriss (2014) methodology for SPT test results of sands and silty sands are presented in Table 3.4 and the respective plots in Figure 3.3.

Table 3.4 Soil calculations for CRR-N curves for SPT Boulanger & Idriss (2014) approach.

N		6		10		11,66667		15		20,5		26	
M		6		6,75		7		7,5		8		8,5	
FC (%)	(N1)60_cs	CRR_M=6	CRR_M=6,75	CRR_M=7	CRR_7,5	CRR_M=8	CRR_M=8,5	Dr					
0	15	0,186	0,170	0,165	0,156	0,148	0,142	57,1					
10	16,149	0,201	0,182	0,176	0,166	0,157	0,149	59,3					
20	19,478	0,257	0,225	0,216	0,200	0,185	0,172	65,1					
30	20,363	0,275	0,240	0,229	0,210	0,194	0,179	66,5					
40	20,576	0,279	0,243	0,232	0,213	0,196	0,181	66,9					
50	20,615	0,280	0,244	0,233	0,214	0,196	0,181	66,9					

N		6,0		10,0		11,667		15,0		20,500		26,0	
M		6,0		6,75		7,0		7,5		8,0		8,5	
FC (%)	(N1)60_cs	CRR_M=6	CRR_M=6,75	CRR_M=7	CRR_7,5	CRR_M=8	CRR_M=8,5	Dr					
0	20,00	0,503	0,341	0,293	0,206	0,129	0,061	65,94					
10	21,15	0,547	0,368	0,316	0,221	0,137	0,062	67,81					
20	24,48	0,696	0,468	0,401	0,278	0,170	0,075	72,95					
30	25,36	0,744	0,501	0,429	0,299	0,184	0,082	74,25					
40	25,58	0,756	0,509	0,436	0,304	0,188	0,085	74,57					
50	25,61	0,758	0,511	0,438	0,305	0,188	0,085	74,62					

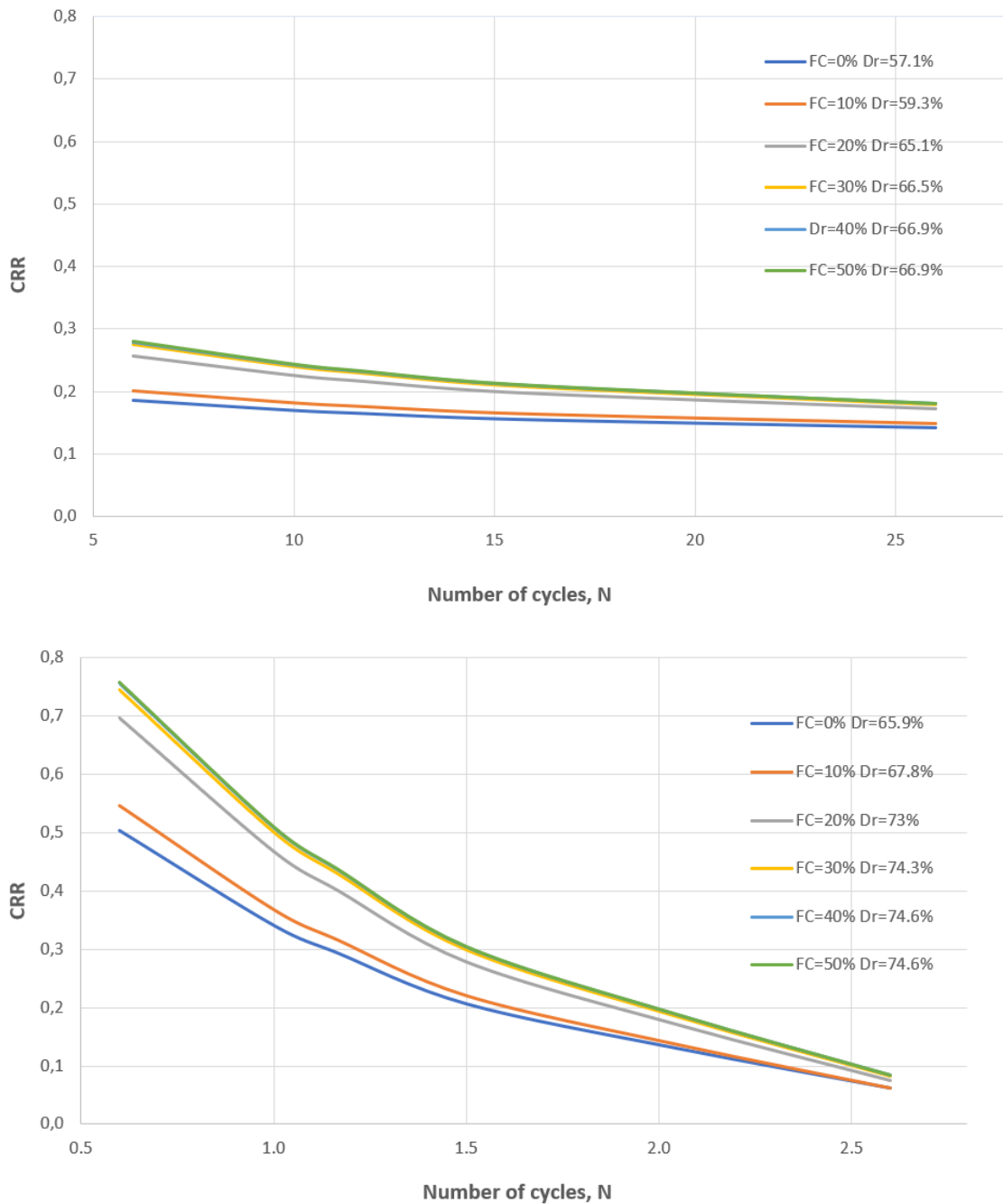


Figure 3.3 CRR-N curves of Sands and Silty sands based on B&I (2014) methodology for SPT test results of $(N_1)_{60} = 15$ and 20 (top and bottom respectively) .

3.3 CRR_N FROM CPT

3.3.1 CLEAN SANDS

In order to have comparable results, the soil must be similar to the ones considered in the previous chapter for SPT, the following chart from Robertson, 2010 with $\frac{q_c/P_a}{N_{60}}$ ratios, was considered (Table 3.5).

Table 3.5 Suggested $\frac{q_c/P_a}{N_{60}}$ ratios (Robertson, 2010).

Zone	Soil Behavior Type	$\frac{(q_c/P_a)}{N_{60}}$
1	Sensitive fine grained	2.0
2	Organic soils – clay	1.0
3	Clays: clay to silty clay	1.5
4	Silt mixtures: clayey silt & silty clay	2.0
5	Sand mixtures: silty sand to sandy silt	3.0
6	Sands: clean sands to silty sands	5.0
7	Dense sand to gravelly sand	6.0
8	Very stiff sand to clayey sand*	5.0
9	Very stiff fine-grained*	1.0

For $(N_1)_{60} = 15$ and 20 the corresponding value of N_{60} can be seen in Table 3.2 and then q_c is determined by the ratio presented in Table 3.5 above.

Youd & Idriss (2001) methodology for CPT test results of clean sands, considers the soil behavior type for clean sands corresponding to the ratio $\frac{q_c/P_a}{N_{60}}$ equal to 5, q_c was determined and the calculations needed to plot the CRR-N curves for two soil test results are presented in Table 3.6, as the plots in Figure 3.4.

Table 3.6 Soil parameters and calculations for CRR-N curves for CPT Youd & Idriss approach.

$(N_1)_{60} = 15, (q_{c1N})_{cs} = 94.34$	qc	Cq	qc_1Ncs	CRR_7,5	M	MSF	CRR_M	N
	9185,59	1	94,34	0,158	5,25	2,491	0,394	2,5
	Pa				6	1,770	0,280	6
	100	qc_1N	Dr		6,75	1,309	0,207	10
	σ'_{v0}	91,85586535	53		7	1,193	0,189	11,67
	100				7,5	1,000	0,158	15
	n	Ic			8	0,847	0,134	20,5
	0,54	1,684			8,5	0,726	0,115	26
		Q						
	σ_{v0}	90,056						
	180	F						
	fs	0,011						
	1	n Robertson 2009						
		0,54						
		$(q_c / P_a) / N_{60}$						
		5						

$(N_1)_{60} = 25, (q_{c1N})_{cs} 153,09$	qc	Cq	qc_1Ncs	CRR_7,5	M	MSF	CRR_M	N	
	15309,31	1	153,09	0,414	5,25	2,491	1,031	2,5	
	Pa	qc_1N	Dr		6	1,770	0,732	6	
	100				6,75	1,309	0,542	10	
	$\sigma'v0$	153,0931	74		7	1,193	0,493	11,67	
	100	Ic	Q		7,5	1,000	0,414	15	
	n				8	0,847	0,351	20,5	
	0,51	1,6080	F		8,5	0,726	0,300	26	
	$\sigma v0$	151,2931							
	180	F							
	fs	0,0066							
	1	n Robertson 2009							
		0,51							
		(q _c / P _a) / N ₆₀							
		5							

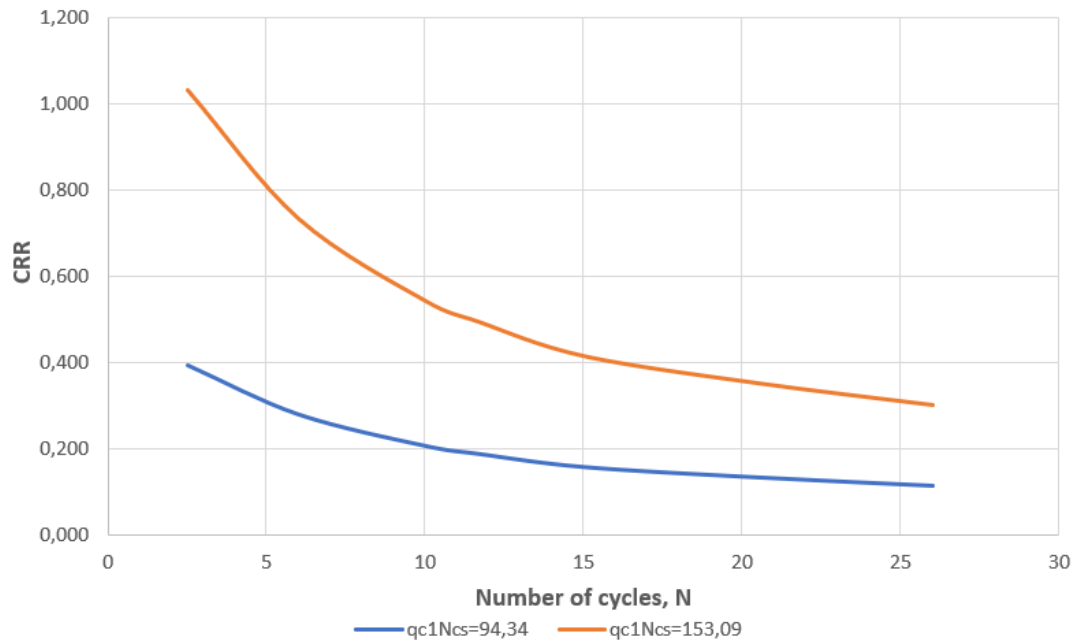


Figure 3.4 CRR-N curves of clean Sands based on Youd & Idriss (2001) methodology for SPT test results of $(q_{c1N})_{cs} = 94.34$ and 153.09 .

As the $(q_{c1N})_{cs}$ increases the curves move upwards and also, for smaller number of cycles, the inclination of the curve starts to rise.

Boulanger & Idriss (2016) approach for CPT test results of Clean Sands calculations (Table 3.7) and corresponding plots (Figure 3.5) are presented below.

Table 3.7 Soil parameters and calculations for CRR-N curves for CPT Boulanger (2016) approach.

$(N_1)_{60} = 15, (q_{c1N})_{cs} = 91,85$

qc	CN	qc_1Ncs	CRR_7,5	M	MSF	CRR_M	N
9185,59	1	91,85587	0,1276	5,25	1,223	0,156	2,5
Pa				6	1,134	0,145	6
100	qc_1N	Dr		6,75	1,061	0,135	10
$\sigma'v0$	91,856	51		7	1,039	0,133	11,7
100				7,5	1,000	0,128	15
m	Ic	FC		8	0,965	0,123	20,5
0,52	1,684	0,92845		8,5	0,935	0,119	26
	Q	$\Delta qc1N$					
$\sigma v0$	90,056	1,11E-12					
180	F						
fs	0,011	MSFmax					
1	n Robertson 2009	1,22					
	0,52						

$(N_1)_{60} = 25, (q_{c1N})_{cs} = 153,09$

qc	Cn	qc_1Ncs	CRR_7,5	M	MSF	CRR_M	N
15309,3	1	153,093	0,31035	5,25	1,7055	0,52932	2,5
Pa				6	1,4252	0,4423	6
100	qc_1N	Dr		6,75	1,1927	0,37016	10
$\sigma'v0$	153,0931089	74		7	1,1244	0,34897	11,7
100				7,5	1	0,31035	15
m	Ic	FC		8	0,8902	0,27627	20,5
0,40	1,608048329	0,44387		8,5	0,7933	0,2462	26
	Q	$\Delta qc1N$					
$\sigma v0$	151,2931089	2,6E-18					
180	F						
fs	0,006609686						
1	n Robertson 2009						
	0,40						

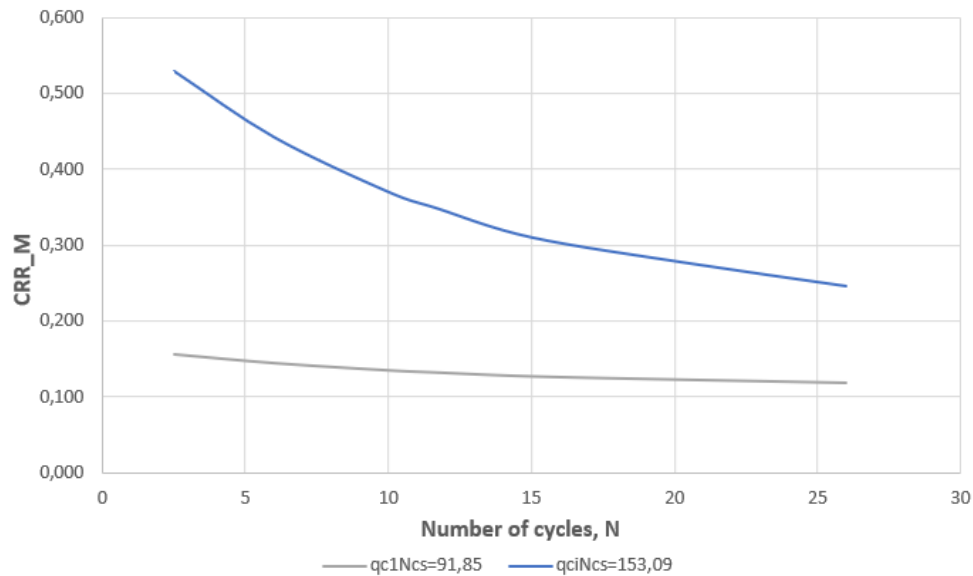


Figure 3.5 CRR-N curves of clean sands based on Boulanger (2016) methodology for SPT test results of $(q_{c1N})_{cs} = 91.85$ and 153.09 .

Here a similar situation that happened in the previous methodology occurs, which is the higher the $(q_{c1N})_{cs}$ the more inclined the curve moves upward, especially for smaller number of cycles.

3.3.2 SANDS AND SILTY SANDS

For Boulanger & Idriss (2014) methodology, considering CPT test results of sands and silty sands, the calculations are presented in Table 3.8, and the respective curves CRR-N plotted in Figure 3.6.

Table 3.8 Soil parameters and calculations for CRR-N curves for Silty Sands: CPT B&I (2014) approach.

qc	Cq	qc_1Ncs	CRR_7,5	M	MSF	CRR_M	N
12247,449	1	122,47	0,177	5,25	1,405	0,249	2,5
Pa				6	1,244	0,220	6
100	qc_1N	Dr		6,75	1,111	0,197	10
σ'v0	122,474	64		7	1,071	0,190	11,67
100				7,5	1,000	0,177	15
n	Ic	FC(%)		8	0,937	0,166	20,5
0,52	1,632	0,4		8,5	0,881	0,156	26
	Q						
σv0	122,374						
10	F						
fs	0,008						
1	n Robertson 2009						
	0,52						
	(qc / Pa) / N60						
	5						
	N	6,0	10,0	11,7	15,0	20,5	26,0
	M	6,0	6,8	7,0	7,5	8,0	8,5
	MSF	1,770	1,309	1,193	1,000	0,847	0,726
qc1N = 20	FC=0,4%	CRR_M=6	CRR_M=6,75	CRR_M=7	CRR_7,5	CRR_M=8	CRR_M=8,5
Δqc1N	qc1Ncs	qc1Ncs					
1,29E-19	122,474	122,474	0,313	0,232	0,211	0,177	0,150
							0,128

qc	Cq	qc_1Ncs	CRR_7,5	M	MSF	CRR_M	N
12247,45	1	142,74	0,247	5,25	1,6	0,393	2,5
Pa				6	1,4	0,335	6
100	qc_1N	Dr		6,75	1,2	0,287	10
σ'v0	122,474	71		7	1,1	0,273	11,67
100				7,5	1,0	0,247	15
n	Ic	FC(%)		8	0,9	0,224	20,5
0,52	1,874	19,7		8,5	0,8	0,204	26
	Q						
σv0	122,374						
10	F						
fs	0,003						
0,4	n Robertson 2009						
	0,61						
	(qc / Pa) / N60						
	5						
	N	6,0	10,0	11,7	15,0	20,5	26,0
	M	6,0	6,8	7,0	7,5	8,0	8,5
	MSF	1,770	1,309	1,193	1,000	0,847	0,726
qc1N = 20	FC=19,7%	CRR_M=6	CRR_M=6,75	CRR_M=7	CRR_7,5	CRR_M=8	CRR_M=8,5
Δqc1N	qc1Ncs	qc1Ncs					
39,332	161,806	188,580	1,861	1,377	1,254	1,052	0,891
							0,763

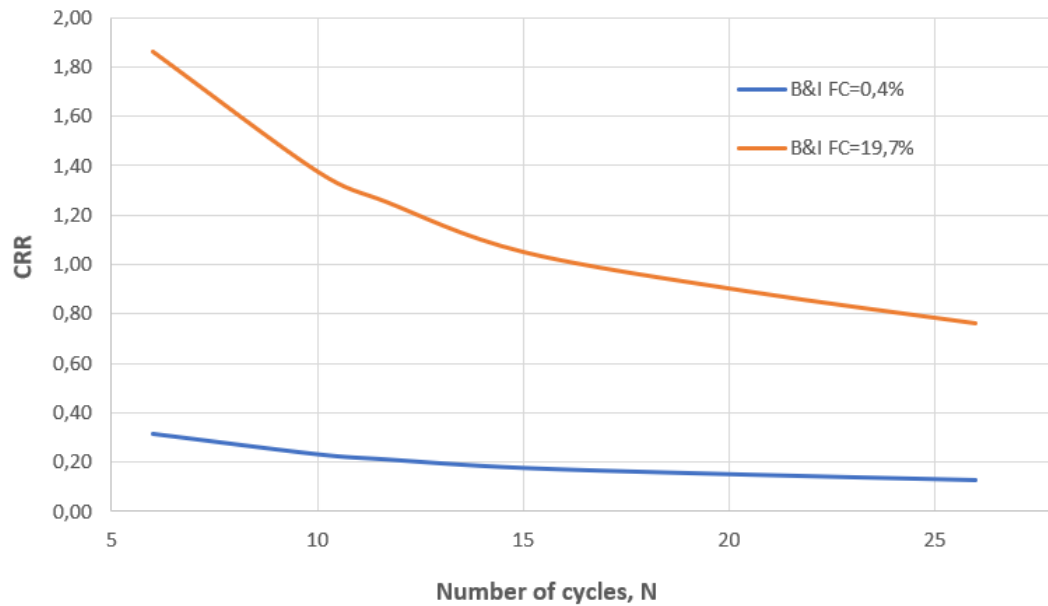


Figure 3.6 CRR-N curves of sands and silty sands based on Boulanger & Idriss (2014) methodology for CPT test results with fines content of 0% and 20%.

The curves continue to show the same behavior for this methodology, with shows that the different methodologies are in agreement with each other and the results will be similar for this type of soil and test.

3.4 COMPARISONS

3.4.1.1 Clean Sands *versus* Sand and Silty Sands

The following Figure 3.7 and Figure 3.8 compare two different soil types: clean sands and sands and silty sands. One for Y&I (2001) methodology for SPT test results and the other for B&I (2016) methodology for CPT test results, respectively.

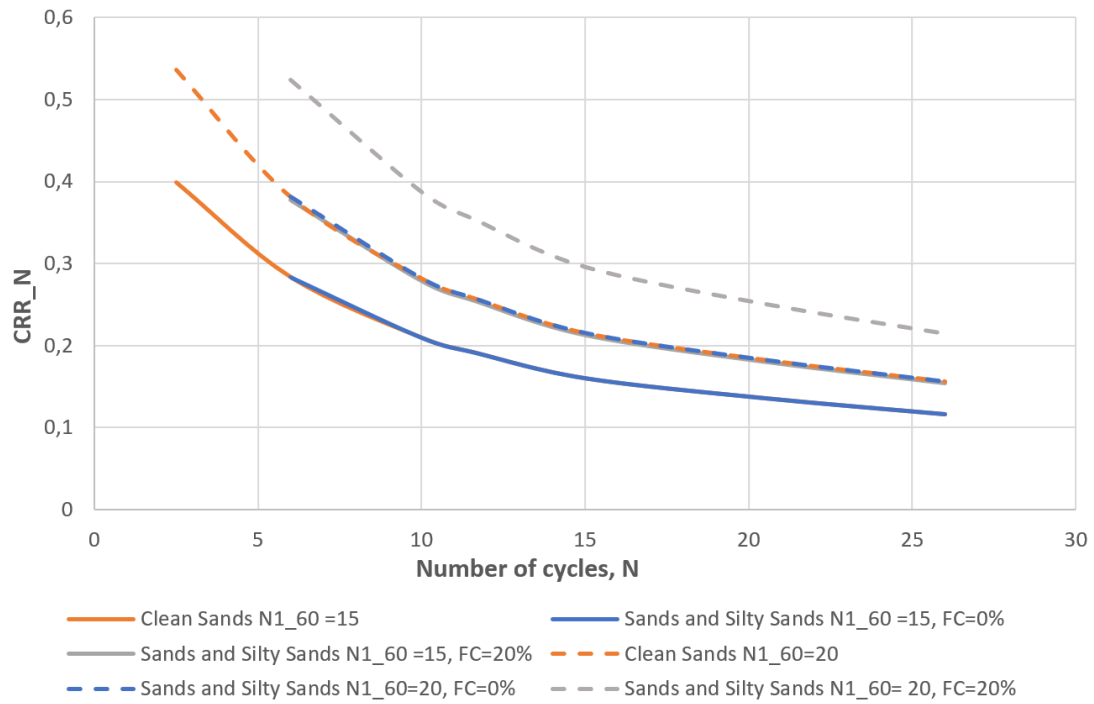


Figure 3.7 Comparison between CRR-N curves of clean sands versus sands and silty sands based on Y&I (2001) methodology for SPT of $(N_1)_{60} = 15$ and 20.

For the same methodology both types of soil have the same shape for CRR-N curves. The variations are due to the increase of fines content or higher $(N_1)_{60}$.

Curves for sands and silty sands of $FC=0\%$ (in blue) overlap the ones with the same $(N_1)_{60}$ (in orange), which makes sense, since the base equation is the same. What it is interesting to see is that the curve for sands and silty sands with $(N_1)_{60} = 15$ and $FC=20\%$ (in full gray) is overlapped by clean sands $(N_1)_{60} = 20$ (in dashed orange).

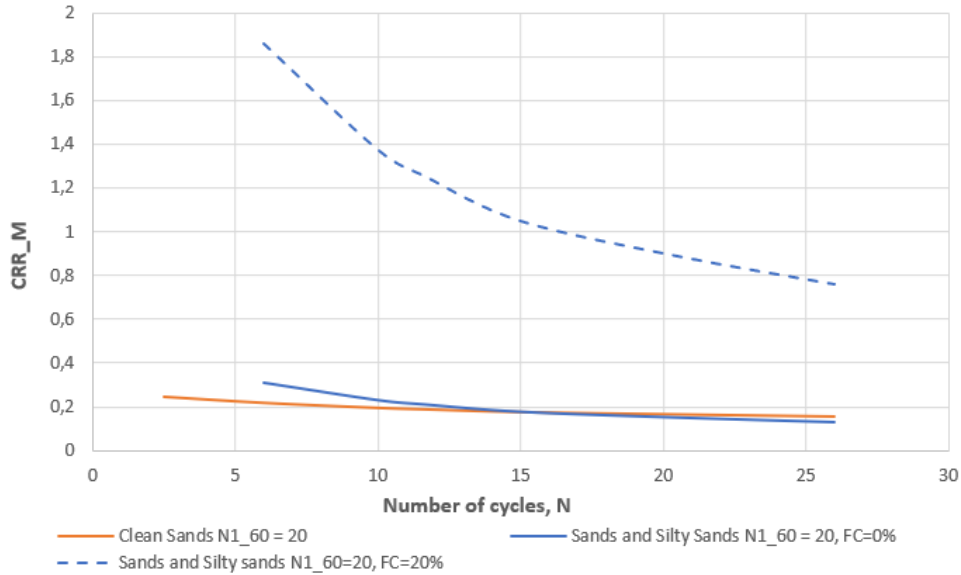


Figure 3.8 Comparison between CRR-N curves of Clean Sands Vs Sands and Silty Sands based on B&I(2016&2014) methodology for CPT with fines content of 0% and 20%.

When analyzing the B&I (2016&2014) approach, it is clear that by adding the fines content, the curves CRR-N change their shape. Changing the type of sand, for the same FC=0%, (full curves) the curve for clean sands is flatter than the sands and silty sands curve. And for a 20% of fines content, the curve for sands and silty sands (in dashed blue) rises the inclinations on the left side, which translates in high CRR for small number of cycles and large number of cycles for lower values of CRR.

3.4.1.2 Youd & Idriss (2001) versus Boulanger & Idriss (2014&2016)

Figure 3.9 and Figure 3.10 reflect the comparison between two different methodologies, Y&I (2001) and B&I (2014 & 2016). One for SPT test results of sands and silty sands and the other for CPT test results of clean sands, respectively.

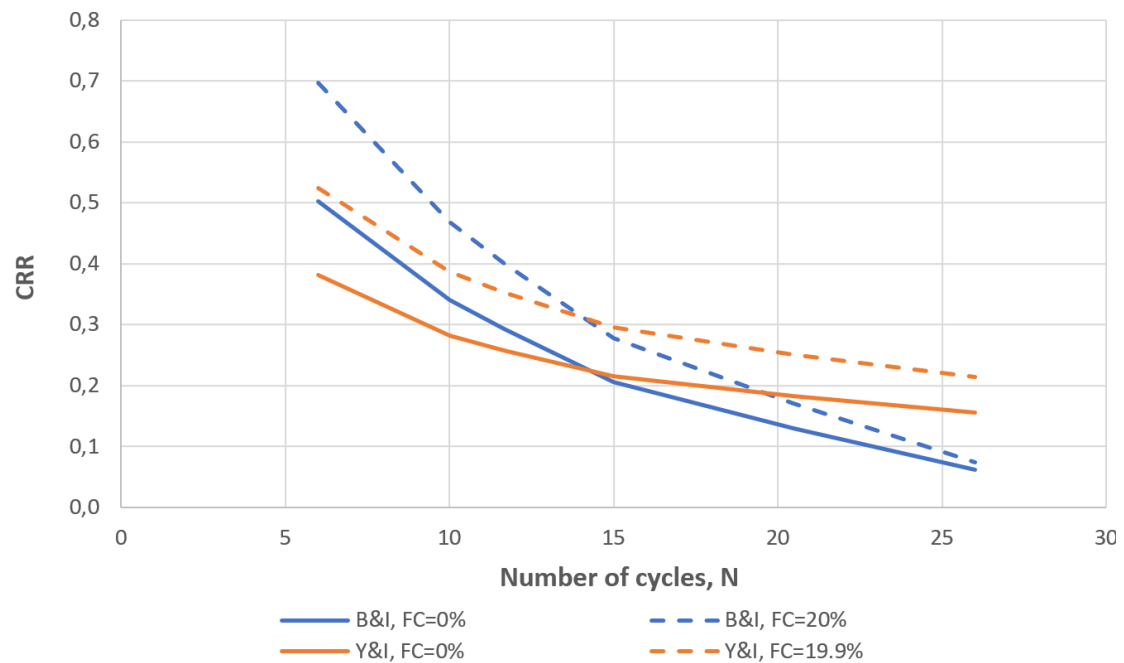


Figure 3.9 Comparison between Y&I(2001) Vs B&I(2014) methodologies for CRR-N curves of sands and silty Sands for SPT test results.

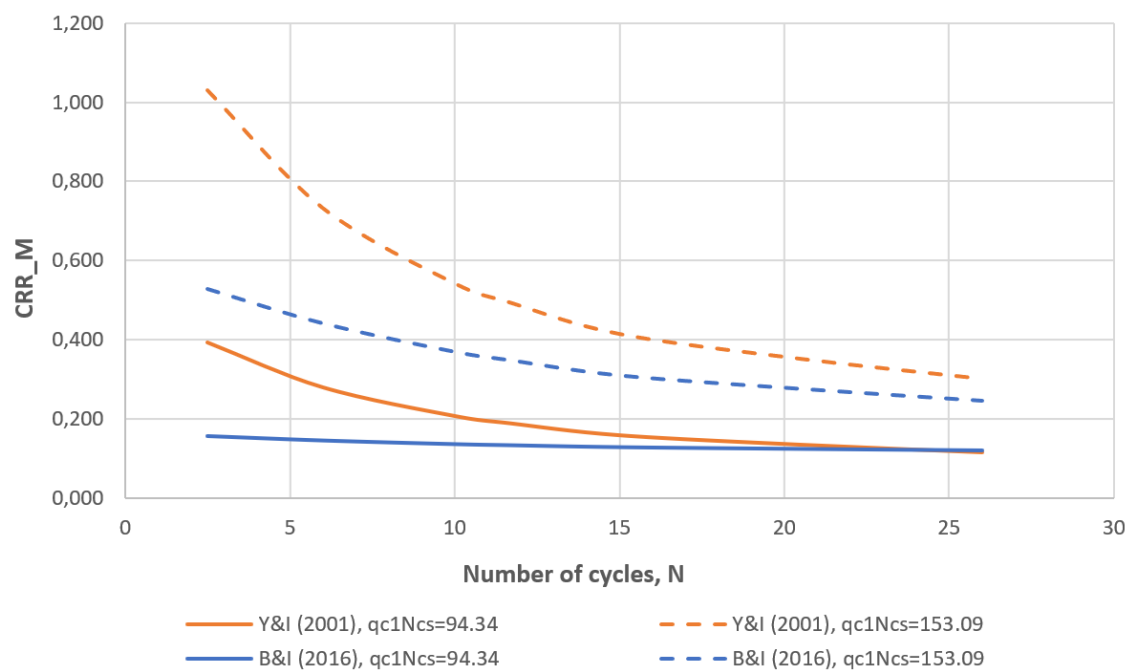


Figure 3.10 Comparison between Y&I (2001) Vs B&I(2016) methodologies for CRR-N curves of clean Sands for CPT test results.

Now comparing B&I (2014) and Y&I (2001) methods, when the number of cycles is small, the differences between the two methods are large and increase with the increase of $(qc_1)_{Ncs}$ value. But once the number of cycles, N , starts to decrease, the two curves stiffen, being the Y&I curve who has the most variety and is more optimistic, thus less conservative. For higher values of $(qc_1)_{Ncs}$, and consequently higher relative density, the curves from both methods move upwards.

3.4.1.3 SPT versus CPT

Below in Figure 3.11 and Figure 3.12 reflect the comparison between two different types of tests, SPT and CPT. One for clean sands with Y&I (2001) methodology and another one for sands and silty sands considering B&I (2014) methodology, respectively.

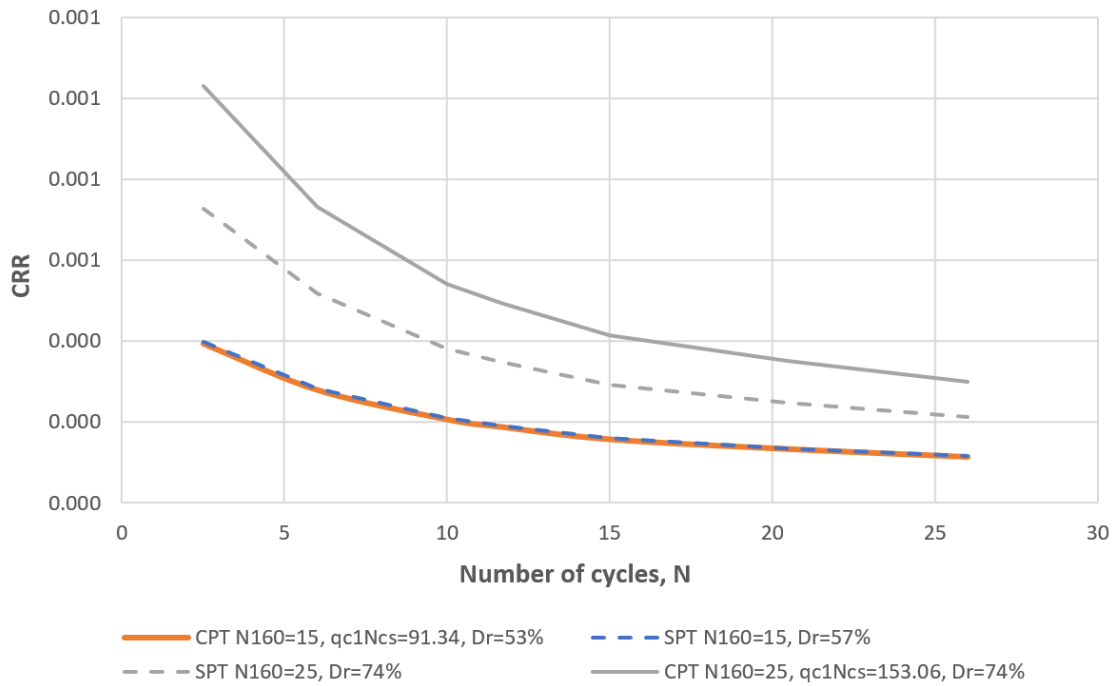


Figure 3.11 Comparison Y&I (2001) methodology of clean sands between SPT versus CPT test results.

This comparison shows that for lower values of $(qc_{1N})_{cs}$ the curves for the two tests are compliant since these are overlapping (full orange and dashed blue). When $(qc_{1N})_{cs}$ value increases, both curves move upwards, and the one from CPT moves more compared to the SPT one. This change can easily be understood when analyzing Equations (2.30) and (2.31).

The shapes of the curves are very similar among them all.

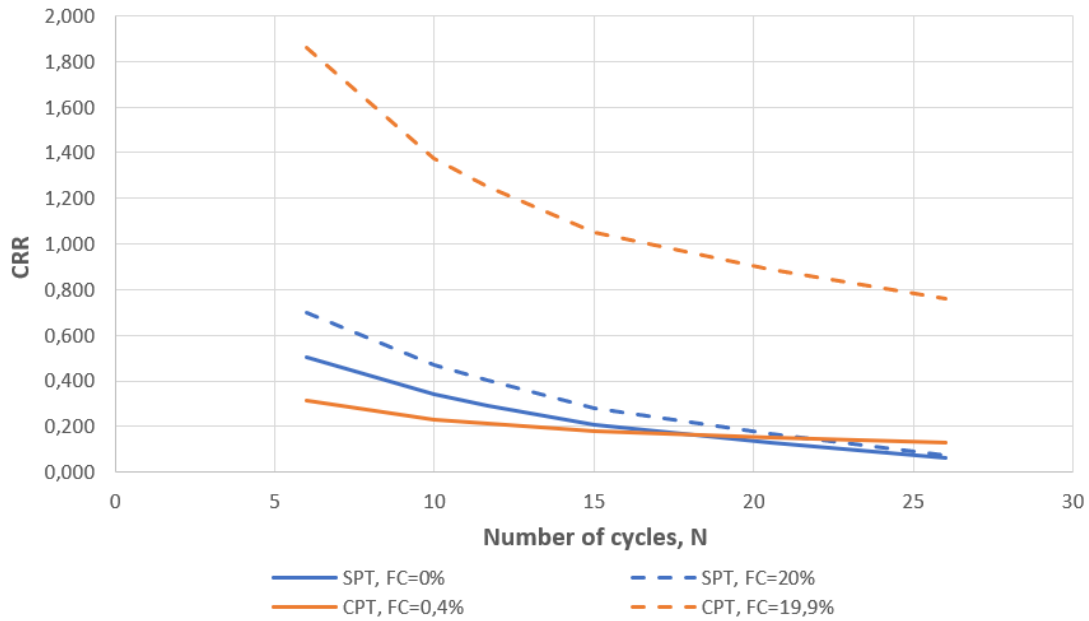


Figure 3.12 Comparison B&I (2014) methodology for sands and silty sands between SPT versus CPT test results.

In the previous plot, it is possible to see that, for the same difference in the fines content, the two tests have different reactions. From 0% to 20% of FC, the CPT test curve (in blue) experiences an enormous upward move in comparison with the one experienced by the SPT test curve.

3.4.1.4 Magnitude Scaling Factor

From the equations represented in Figure 2.28, it was possible to plot the curves and add two other curves that represent, for the same soil – a clean sand determined with the methodologies presented for CPT tests in Chapter 2.3.3 with a $(q_{c1})_{NCS} = 122.47$ – the different approaches proposed by Y&I (2001) and B&I (2016), represented in orange and blue, respectively.

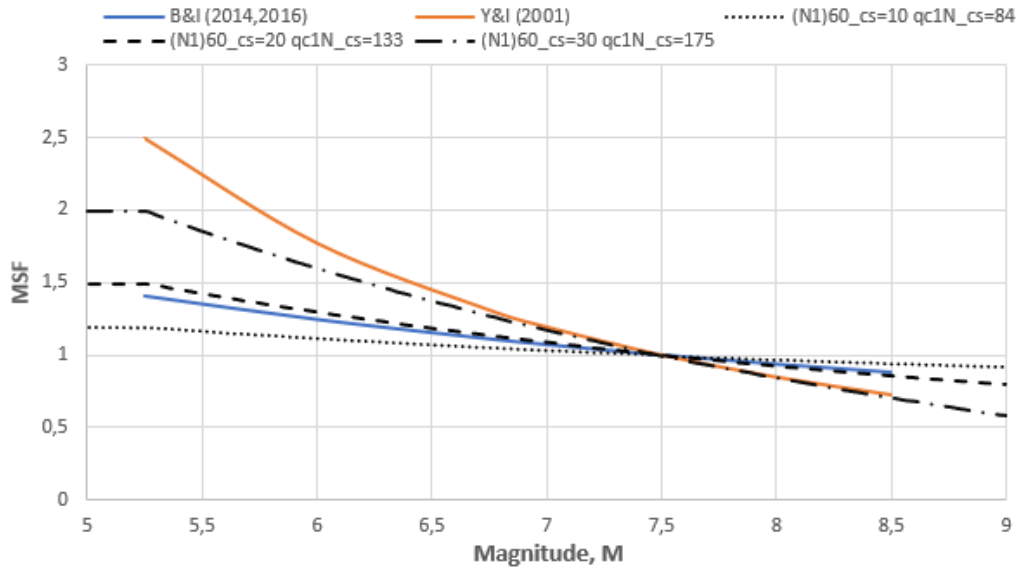


Figure 3.13 Plot of MSF curves by different approaches.

It is interesting to see how the curve of the MSF from B&I (2016) changes if the $(qc_1)_{Ncs}$ is altered. If it is altered to $(qc_1)_{Ncs}=175$, the blue curve will get much closer to the Y&I (2001). From this analysis it can be concluded that the MSF is more conservative for the B&I (2014, 2016) than the Y&I (2001).

3.5 SILTS AND CLAYS

A quick assessment of the approaches for silts and clays was also developed, based on the theory already presented in Chapter 2. The following Table 3.9 presents the parameters considered for the soil and the respective calculations, in order to find the CRR-N curve, which is represented in Figure 3.14.

Table 3.9 Soil parameters and calculations for CRR-N curves for Silts and Clays: B&I (2007).

ϕ'	32	S	0,22	τ_{cyc}/Su	0,83
z	10	OCR	1	C_2D	0,96
γ	18	m	0,8		
γ_w	10	Sub'vc	0,22	τ_s	0
σ'_v	80			α	0
				K_α	1

M	N _{clay}	MSF	CRR_M	τ_{cyc}/Cu
5,25	9,406	1,12944391	0,197987	0,99993434
6	15,697	1,07790578	0,18895257	0,95430592
6,75	21,880	1,03517917	0,18146277	0,91647862
7,5	29,051	0,99975756	0,1752535	0,8851187
8,5	43,483	0,96176492	0,16859354	0,85148255

CRR_7,5
0,176

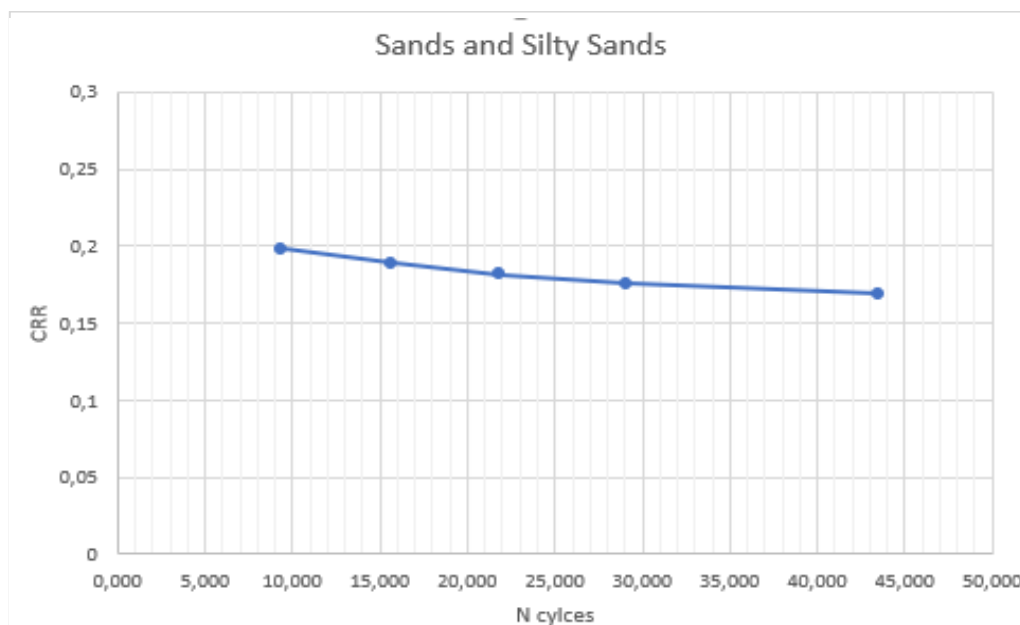


Figure 3.14 CRR-N curve for Silts and Clays.

3.6 CONCLUDING REMARKS

From this analysis, it was possible to conclude that the approach presented by Boulanger & Idriss (2014) is the one that presents the curve that better adjusts to the purpose of this work, since it gives more conservative results and also because it is the one that affects the most CPT results that are the most common used tests for offshore wind sites.

Therefore, the curve that will be considered in the present work as the target curve of the literature is the one given by Boulanger & Idriss (2014) approach for CPT test. Figure 3.15 presents this curve for a relative density corresponding to 54.5%~55% and a fines content of 10%.

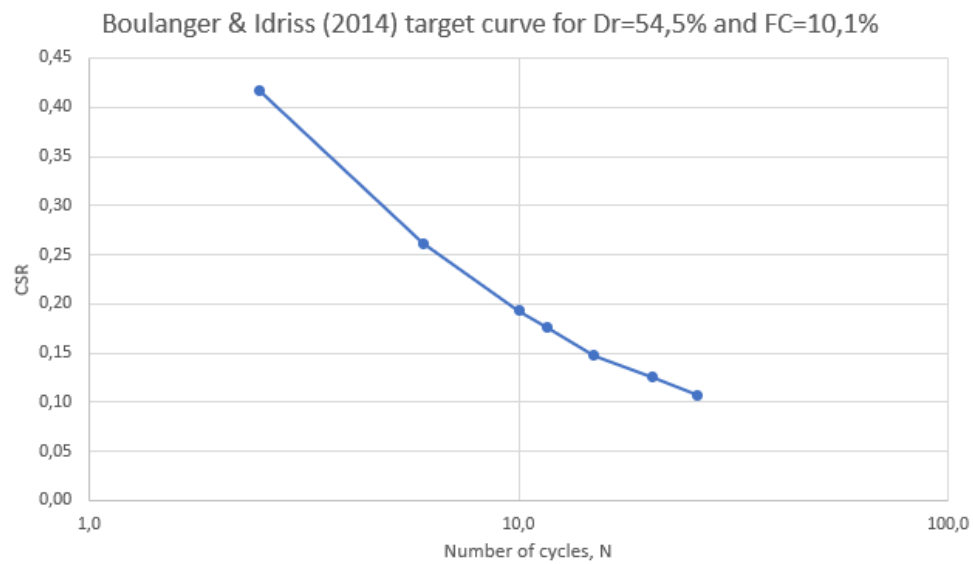


Figure 3.15 B&I (2014) target curve with $D_r=54,5\%$ and $FC=10,1\%$.

4 NUMERICAL SIMULATION

4.1 INTRODUCTION

In the present chapter, the PM4Sand soil model will be used to simulate CSR-N target curves. This soil model has been developed to capture liquefaction triggering of sands throughout the undrained cyclic response of the model by determining primary model parameters: relative density D_r , shear modulus coefficient G_0 , and the contraction rate parameter h_{p0} ; and, if necessary, it is possible to modify secondary parameters, such as the bounding surface parameter, n^b .

In order to understand how to do this simulation, the paper on the "PLAXIS The PM4Sand model 2018" by Vilhar, Brinkgreve, & Zampich (2018) was taken as support for this work. In the referred paper, there are detailed examples on how to use the tools necessary in the software Plaxis 2D and how to process the simulation.

The goal was to carry out an iterative process by changing some of the primary and secondary parameters of the sands, in order to obtain CSR-N curves that were as close as possible to the target curves given by the literature (which were discussed in the previous chapter). For this, it is important to understand how the alteration of these parameters influence the CSR-N curve, and that will also be exposed here.

For the iterative process of finding the best parameters to fit the B&I (2014) target curves, the *Soil test* facility of PLAXIS2D was used as a means to simulate a series of cyclic direct simple shear test (CDSS). The layout of this tool in PLAXIS is presented in Figure 4.1.

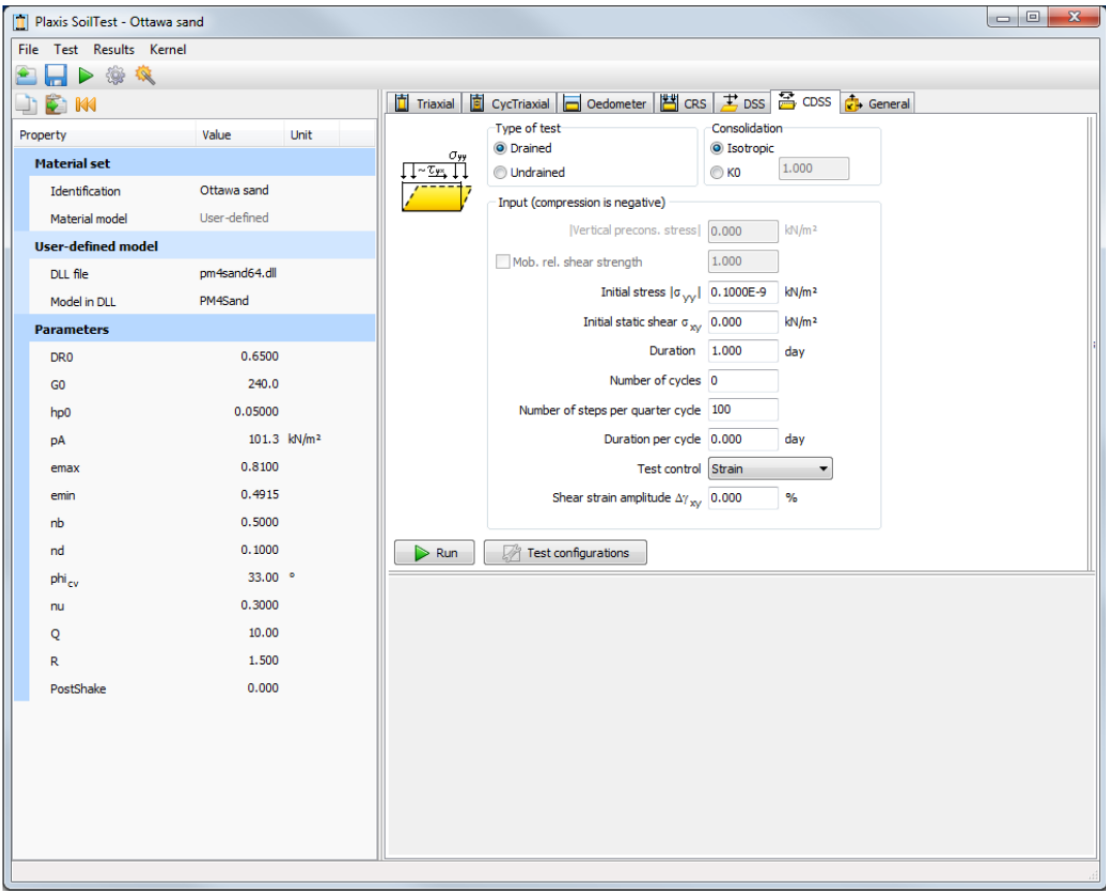


Figure 4.1 Layout of the PLAXIS2D Soil Test facility: cyclic direct simple shear (CDSS) tests.

To reach the intended curves, the iterative process described in the flowchart in Figure 4.2 was conducted for each point of each curve, until a curve matching the target curve (obtained from the analysis in chapter 3) was derived.

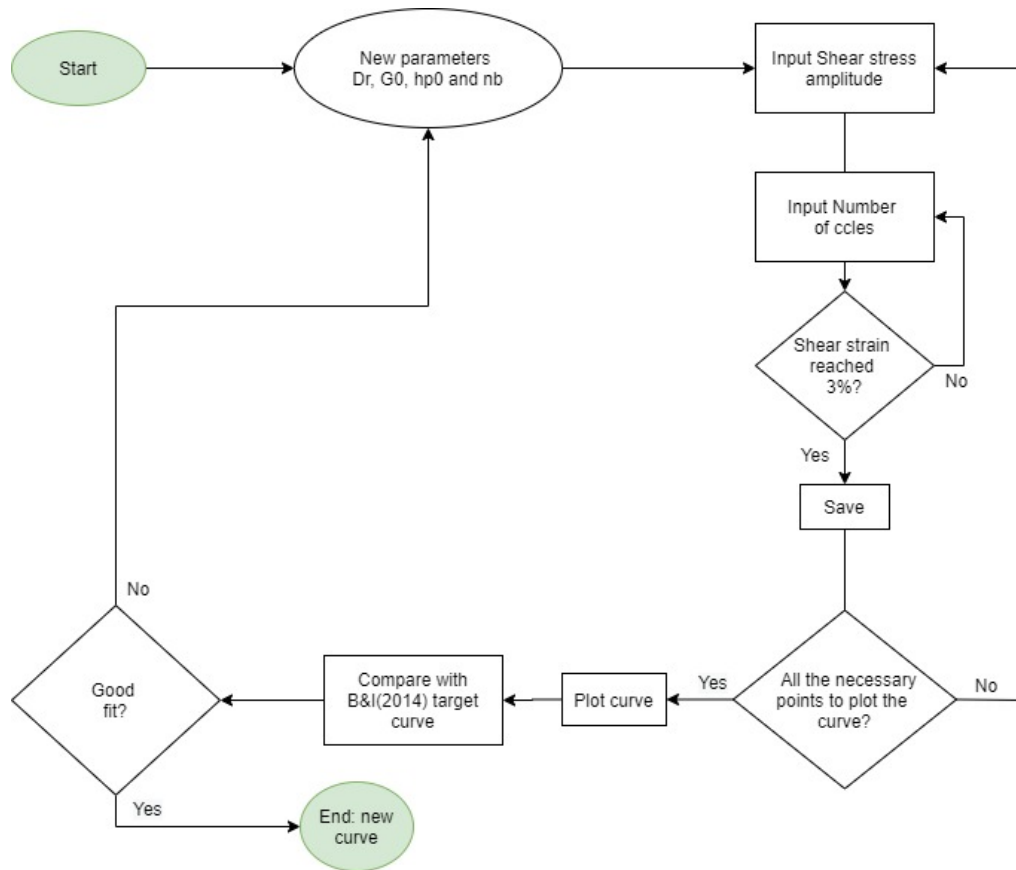


Figure 4.2 Flowchart of the process for deriving the target CSR-N curves using Plaxis with different hp_0 by comparison with the theoretical curve from Boulanger & Idriss (2014).

4.2 CALIBRATION OF THE MODEL DAMPING

Before conducting any dynamic analysis with a numerical model, it is necessary to perform a calibration. And this can be done with any chosen dynamic parameters, comparing the results of the response of the model with the solutions given by the theory.

Therefore, to secure that the model was working as intended, a calibration to control the system damping was performed, where a Linear Elastic soil column was considered, with 40 meters plus 1 meter of bedrock layer. The bedrock was considered as infinitely rigid and the soil properties for both layers can be seen in Table 4.1 and Table 4.2.

Table 4.1 Linear Elastic layer properties.

Parameter	Symbol	Value	Unit
Drainage type	-	Drained	-
Unsaturated unit weight	γ_{unsat}	14	KN/m^3
Saturated unit weight	γ_{sat}	14	KN/m^3
Young's modulus	E'	$138 * 10^3$	KN/m^3
Poisson's ratio	ν'	0,2	-
Shear velocity	V_s	200	m/s
Layer depth	H	40	m

Table 4.2 Bedrock layer properties.

Parameter	Symbol	Value	Unit
Drainage type	-	Drained	-
Unsaturated unit weight	γ_{unsat}	22	KN/m^3
Saturated unit weight	γ_{sat}	22	KN/m^3
Young's modulus	E'	$8 * 10^6$	KN/m^3
Poisson's ratio	ν'	0,2	-
Shear wave velocity	V_s	12190	m/s
Multiplier	q_x	27340	$kN/m/m$

Before the first run, the Rayleigh parameter were set for a damping of $\xi = 1\%$ and frequency targets of $f_1 = 0.5$ and $f_2 = 1$, which corresponded to $\alpha = 0.04189$ and $\beta = 2.122 * 10^{-3}$. Then it was possible to conclude from the Furrier Amplification plots that $f_{equk} = 7.971$ and from a perturbation/free vibration analysis $f_{soil} = 1.41$ and so $\frac{7.971}{1.41} = 5.63 \sim 5Hz$.

Hence, the new target for the Rayleigh parameters have the frequency targets of $f_1 = 1$ and $f_2 = 5$, maintaining the damping of $\xi = 1\%$.

With the information from a point situated in the top of the soil column and another one on the bottom of the bedrock layer, it was possible to plot, with the help of an Excel sheet, the ratio between the values on the top and the bottom of the bedrock, and compared with Equation used also by (Visone, Bilotta, & Magistris, 2011).

$$A(f) = \frac{1}{\sqrt{\cos^2\left(2\pi \frac{H}{V_s} f\right) + \left(2\pi \frac{HD}{V_s} f\right)^2}} \quad (4.1)$$

Where f is the frequency, $H = 40m$ is the height of the soil column and $V_s = 200m/s$

Figure 4.3 clearly shows the peaks of frequency and presents the comparison between the results obtained with the calibration exercise in PLAXIS using the python script and the theoretical formula.

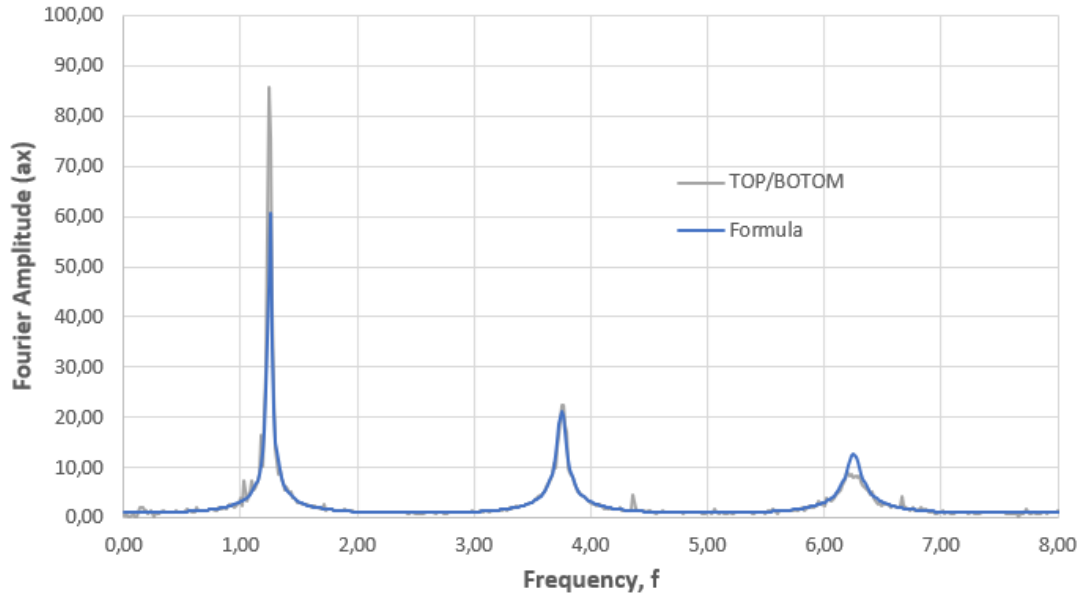


Figure 4.3 Comparison between the results obtained with PLAXIS and the theoretical formula.

Analyzing the plot, it is a good match, but it is still possible to see a difference between the two sets of results. This can be due to the damping considered, but for this project, this difference will be accepted. To make sure that this difference was not affecting the model outcome, another way of calibration was considered, which was by comparing the acceleration from the input and the one plotted from a point on the bottom of the bedrock, and they were found to be the same. And so, it was concluded that the model was working correctly.

4.3 SIMULATION OF CSR-N RESPONSE WITH THE PM4SAND MODEL

4.3.1 ORIGINAL PARAMETERS

The paper (Vilhar, Brinkgreve, & Zampich, 2018) "PLAXIS The PM4Sand model 2018" explains how the PM4Sand model works and presents two simple but very illustrative exercises. These exercises were used to better understand the model and as a basis for the work developed in this thesis.

The first exercise was a simulation of CSR-N response with the PM4Sand model, and the purpose was to reproduce two CSR-N curves, that were already published before, generated by the original

PM4Sand model (Boulanger & Ziotopoulous, 2015) using cyclic direct simple shear simulations in Plaxis *Soil Test* tool. The tests were stress-controlled, in undrained conditions and the consolidation set to $K_0 = 0.5$.

For this first exercise, the soil material chosen was Ottawa sand with a relative density of $Dr = 65\%$, and a summary of the selected parameters are presented in Table 4.3. and Table 4.4.

Table 4.3 The calibrated parameters.

Parameter	D_{r0} [-]	G_0 [-]	e_{max} [-]	e_{min} [-]
Value	0.65	240	0.81	0.4915

Table 4.4 The parameters with default values.

Parameter	p_A [KPa]	n^b [-]	n^d [-]	ϕ_{cv} [-]	ν [-]	Q [-]	R [-]	$PostShake$ [-]
Value	101.3	0.5	0.1	33	0.3	10	1.5	0

Table 4.5 The values of the contraction rate parameter h_{p0} for two cases A and B.

Parameter	Units	Case A	Case B
h_{p0}	[-]	0.05	0.2

The target CSR-N curves for both cases A and B are presented in Figure 4.4, in red and green respectively, and were obtained from cyclic stress-controlled DSS simulations using the original PM4Sand model (as reported by (Boulanger & Ziotopolou, PM4SAND (VERSION 3.1): A Sand plasticity model for earthquake engeneering aplication, 2017)). Additionally, the plots from simulations by the PM4Sand model implemented in Plaxis using the *Soil Test* tool are shown in blue.

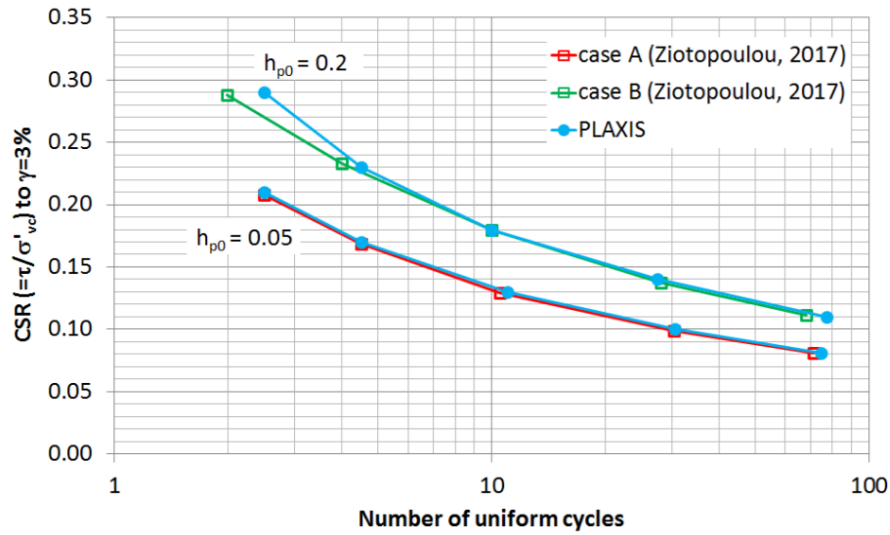


Figure 4.4 CSR-N relationship for case A and B material sets (modified from (Boulanger & Ziotopoulou, PM4SAND (VERSION 3.1): A Sand plasticity model for earthquake engineering application, 2017)), (Vilhar, Brinkgreve, & Zampich, 2018).

After the material and parameters are defined, the CDSS soil test should have the conditions presented in Table 4.6. The values represented as **n** and **x** in red, are the ones that will change in each simulation.

Table 4.6 Test conditions.

Parameter	Value
Type of test	Undrained
Consolidation/ K_0	0,5
Initial stress $ \sigma_{yy} $	$100 \text{ KN} / \text{m}^2$
Initial static shear σ_{xy}	$0 \text{ KN} / \text{m}^2$
Duration	Not relevant
Number of cycles	n
Number of steps per quarter cycles	100
Duration per cycle	Not relevant (0 days)
Test control	Stress
Shear stress amplitude $\Delta\sigma_{xy}$	x KN / m^2

There are a lot of plots that is possible to generate in the CDSS test, the most interesting ones for this exercise are: $\gamma_{xy} - \tau_{xy}$, $\gamma_{xy} - p_w$ and $\sigma_{yy} - \tau_{xy}$ as shown in Figure 4.5.

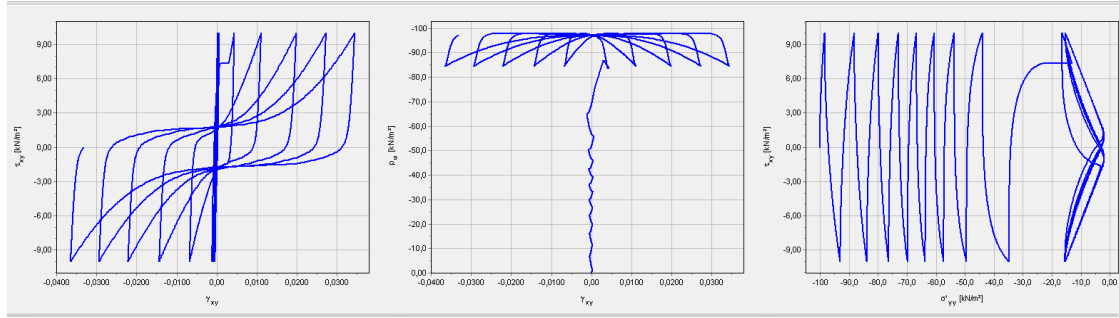


Figure 4.5 Plots $\gamma_{xy} - \tau_{xy}$, $\gamma_{xy} - p_w$ and $\sigma_{yy} - \tau_{xy}$.

For both cases A and B, the points in Table 4.7 were calculated. Each point corresponds to one simulation.

Table 4.7 CSR-N values at single amplitude shear strain $\gamma = 3\%$ for Case A and B.

Point	CSR [-]	Shear Stress amplitude $\Delta\sigma_{xy}$ [KPa]: Case A	Shear Stress amplitude $\Delta\sigma_{xy}$ [KPa]: Case B
1	0.21	21	29
2	0.17	17	23
3	0.13	13	18
4	0.1	10	14
5	0.081	8.1	11

Some of the results were slightly different than the expected, but that difference it is not significant and, for the purpose of the exercise, the obtained result is very satisfactory. Those results are plotted in Figure 4.6 and it is possible to observe that the results are compliant with the target curves, which means that PLAXIS and the *Soil Test* tool are working as expected.

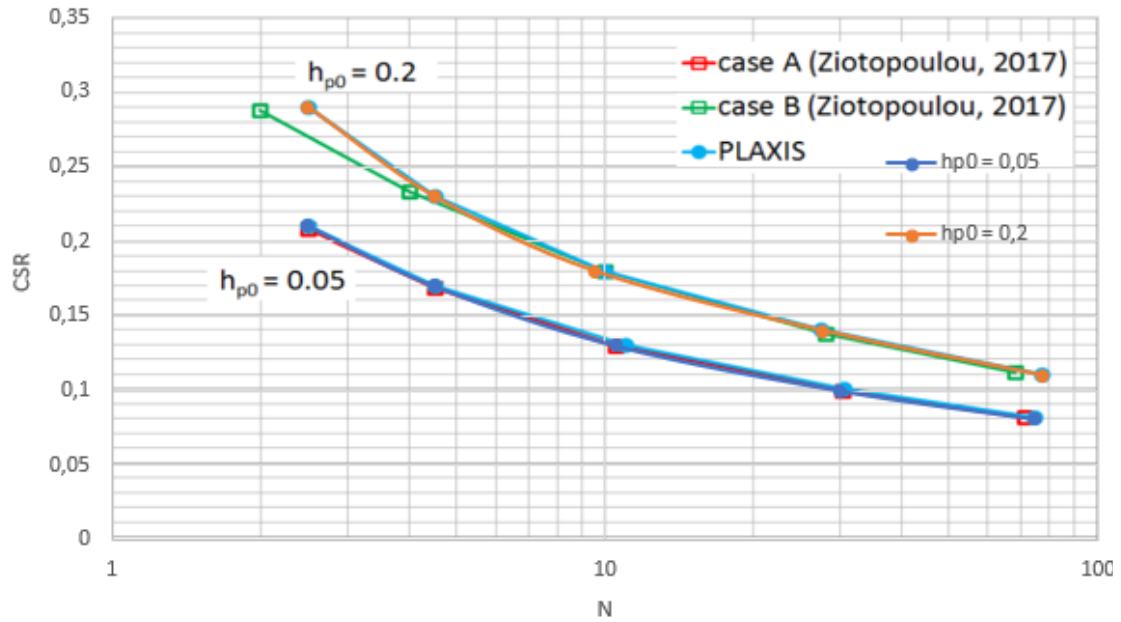


Figure 4.6 Comparison between the results of the exercise using PLAXIS PM4Sand model and the expected results.

The iterative process of finding the best parameters to fit the target curves (cases A and B, in this example) consists of the following steps, until a good match for the target curve is found:

- 1) Choose between the parameters D_{r0} , h_{p0} and n^b , previously determined based on experience of COWI, which are going to be iterated.
- 2) Input the load and the number of cycles in *Soil Test* tool, CDSS test ($\Delta\sigma_{xy}$ and N , respectively).
- 3) Run the test and check if the shear strain reaches 3%; if not, repeat the previous two steps.
- 4) When a shear strain of 3% is reached, and all the necessary points to plot the curve are gathered, it can be plotted; if not, all the process is repeated.
- 5) Finally, this new curve is compared with the target curve from B&I (2014). If it is not a good fit, all of the process should be repeated from the top. If the fit is good, the purpose of iteration has been accomplished, the iterative process is concluded and, therefore, the obtained curve can be used to define the behavior of the sand in question.

The flowchart present in Figure 4.2 in the beginning of this section shows this same process but in a more illustrative way.

4.3.2 INFLUENCE OF DIFFERENT RELATIVE DENSITIES D_r

The previous exercise was repeated with the same parameters but for different relative densities, namely of 41.5% and 55%, with the purpose of reaching a CSR-N curve close to the one from the theory from B&I (2014) for sands and silty sands with $p' = 100 \text{ kPa}$ and fines content of 10%, for the corresponding relative density.

To calculate the shear modulus coefficient, G_0 , Equation (4.2) was considered.

$$G_0 = 167 * \sqrt{46 * D_R^2 + 2,5} \quad (4.2)$$

The obtained results are presented in Figure 4.7 and Table 4.8:

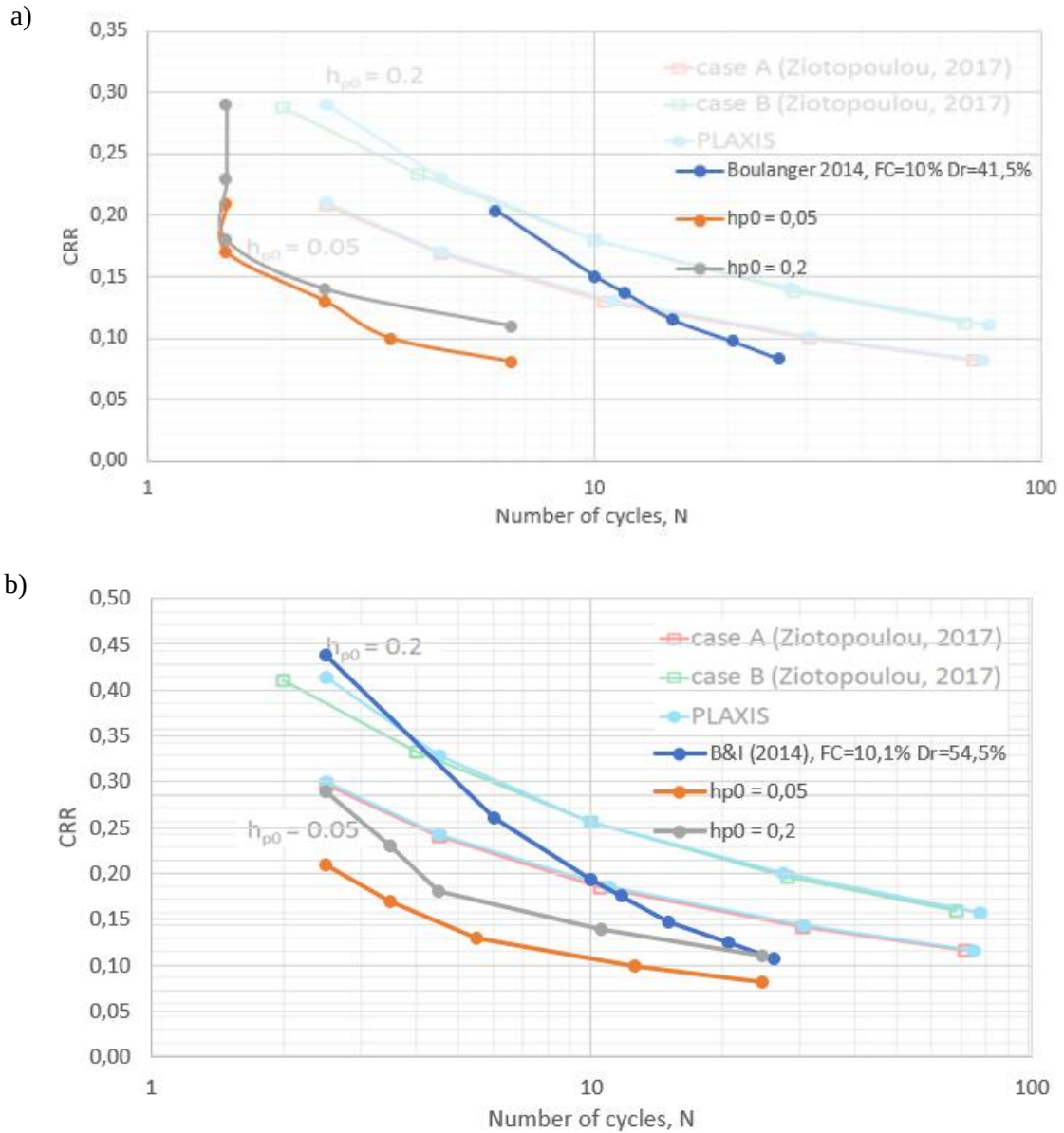


Figure 4.7 Results of the simulation with $D_r = 41,5\%$, (a), and $D_r = 55\%$, (b).

Table 4.8 presents the adopted values of each simulation from the plots above.

Table 4.8 Results of Exercise 1 with $D_r = 41.5\%$ on the top and $D_r = 55\%$ on the bottom.

Points	$D_r = 41.5\%$				$D_r = 55\%$			
	$h_{p0} = 0.05$		$h_{p0} = 0.2$		$h_{p0} = 0.05$		$h_{p0} = 0.2$	
	CSR	N	CSR	N	CSR	N	CSR	N
1	0.21	1.5	0.29	1.5	0.21	2.5	0.29	2.5
2	0.17	1.5	0.23	1.5	0.17	3.5	0.23	3.5
3	0.13	2.5	0.18	1.5	0.13	5.5	0.18	4.5
4	0.1	3.5	0.14	2.5	0.1	12.5	0.14	10.5
5	0.081	6.5	0.11	6.5	0.081	24.5	0.11	24.5

Comparing the results between the two relative densities, it is possible to conclude that, with a higher relative density, the results are closer to the ones presented in the in the previous chapters. But this is not enough to fit the theoretical curve and so, it is necessary to change other parameters to improve the match and better fit the curve.

4.4 PRESENTATION OF THE TIME HISTORIES

For this project, 7 time-histories were chosen to run the dynamic analyses, which are summarily presented in Table 4.9. In the Appendix, the acceleration and velocity plots of all these time histories are provided.

Table 4.9 Time histories.

Event	Country	Station	Magnitude	Distance (Km)	Comp.	PGA (g)
Imperial Valley 10/15/1992	USA	Cerro Prieto	6.5	26.7	X	0.17
Izmit 8/17/1999	Turkey	Goynuk-Delvet Hastanesi	7.6	73	X	0.14
Kern County 07/21/1952	USA	Taft Lincon School Tunnel	7.4	43.5	Y	0.18
Landers 06/28/1992	USA	Coolwater	7.3	23	Y	0.42
Landers 06/28/1992	USA	Desert Hot Spring	7.3	23	X	0.17
Loma Prieta 10/18/1989	USA	Saratoga-Aloha Ave	6.9	27.6	X	0.51
Tabas 09/16/1978	Iran	Tabas	7.4	52	Y	1.10

The main criterion by which these time histories were chosen was the magnitude, which was around 7.5. In addition, the distance is of about 30 km and PGA about 0.3g.

Because the peak ground acceleration (PGA) is usually smaller offshore, all the time histories were scaled to 0.2g being the Scaling Factor, $SF = 0.2/0.3$ for accelerations. As for the velocities, these were converted from cm/s to be in m/s .

For each time history, data corresponding to Acceleration (g), Velocity (cm/s) and Displacement (cm) were available. Figure 4.8 shows this data for the first time-history in Table 4.9, that is, for *Imperial Valley*.

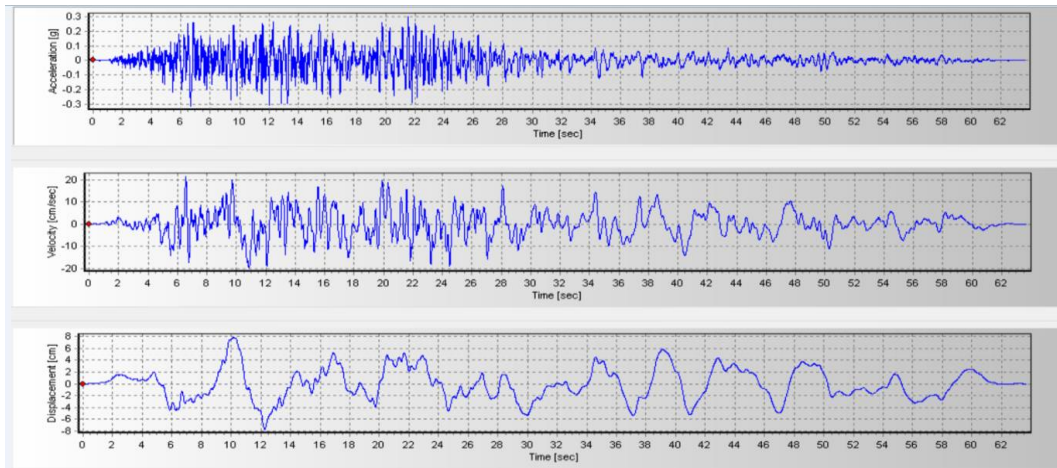


Figure 4.8 Plots of acceleration, velocity and displacement of the Imperial Valley time history.

4.5 SOIL TESTS SIMULATION

The second exercise of PLAXIS PM4Sand model consists of performing dynamic numerical analyses using PLAXIS to predict the onset of liquefaction on the sandy layer of the soil column modelled with the PM4Sand model.

In this work, two profiles will be analyzed: Profile 1, which is an idealistic three-layered profile and two soil types; and, Profile 2, which is more realistic, corresponding to an offshore site, with several layers and different soils, including sands with varying densities. The profiles will be presented in other sections further ahead, and in this section, the analysis will focus on the different sands that constitute each profile.

4.5.1 PROFILE 1: IDEALIZED PROFILE

This profile comprises only one sand layer, with the parameters regarding PM4Sand model presented in Table 4.10 and Table 4.11, in terms of primary and secondary parameters, respectively. These parameters were taken directly from the mentioned paper.

Table 4.10 PM4Sand primary parameters of sand.

Parameter	Symbol	Value	Unit
Drainage type	-	Undrained (A)	-
Unsaturated unit weight	γ_{unsat}	14	kN/m^3
Saturated unit weight	γ_{sat}	18	kN/m^3
Rayleigh damping coefficient	α	0.04189	-
Rayleigh damping coefficient	β	$2.122 * 10^{-3}$	-
Initial relative density (D_{R0})	D_{R0}	0,55	-
Shear modulus coefficient (G_0)	G_0	677	-
Contraction rate (h_{p0})	h_{p0}	0,4	-

Table 4.11 PM4Sand secondary parameters of sand.

Parameter	Symbol	Value	Unit
Atmospheric pressure	p_A	101.3	kPa
Maximum void ratio	e_{max}	0.8	-
Minimum void ratio	e_{min}	0.5	-
Bounding surface position according to ξ_R	n^b	0.5	-
Dilatancy surface position according to ξ_R	n^d	0.1	-
Critical state friction angle	φ_{cv}	33	°
Poisson's ratio	ν	0.3	-
Critical state line parameter	Q	10	-
Critical state line parameter	R	1.5	-
Post-shaking reconsolidation	$PostShake$	0	-
Earth pressure coefficient	K_0	0.5	-

The curve that will be considered as the target of this calibration, as explained in section 3.6, is the one from the literature by Boulanger & Idriss (2014) approach. For profile 1, this curve corresponds to the one in Figure 3.15, with a relative density D_r of 55%.

4.5.2 PROFILE 2: REALISTIC PROFILE OF AN OFFSHORE SITE

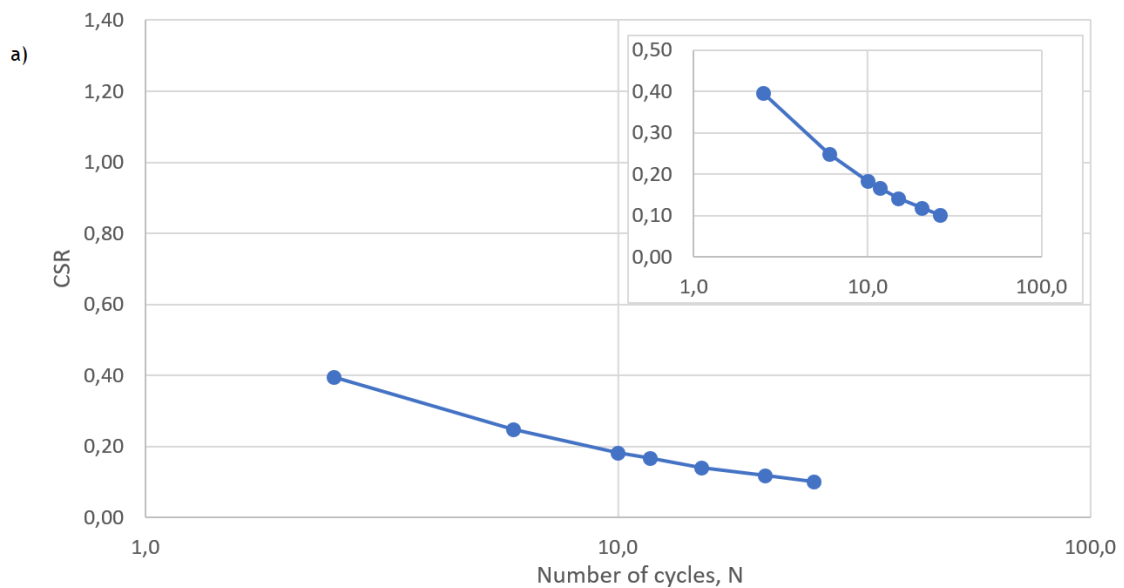
Profile 2 corresponds to an offshore site profile is a much more complex profile with more layers, it has a total of four sandy layers, and the parameters provided by COWI for those sands are presented in Table 4.12.

Table 4.12 Sand parameters provided by COWI for the offshore profile.

Sand layer	Dr [%]	G0 (PM4Sand)	γ [KN/m^3]	φ_{cv} [°]	ν	K_0
Silty sand 1	50.41	629.1	17.4	27	0.2	0.55
Silty sand 2	78.13	923.5	19.1	33	0.2	0.46
Silty sand 3	60.99	739.6	19.5	37	0.2	0.4
Silty sand 4	60.84	738	19.5	38	0.2	0.38

The parameters, needed for the PM4Sand model (Table 4.10 and Table 4.11), and that were not provided, were taken as the same as presented for Profile 1, with the exception of n_b and h_{p0} .

The target curves for the sands of Profile 2 were defined with the same theory as in Profile 1, that is, Boulanger & Idriss (2014) approach. By adjusting the parameters, it was possible to obtain, for each sand the target curves presented in Figure 4.9. Since sand 3 and sand 4 have a very close value of relative density, it was decided that the curve would be the same for both sands, since the difference was not significant.



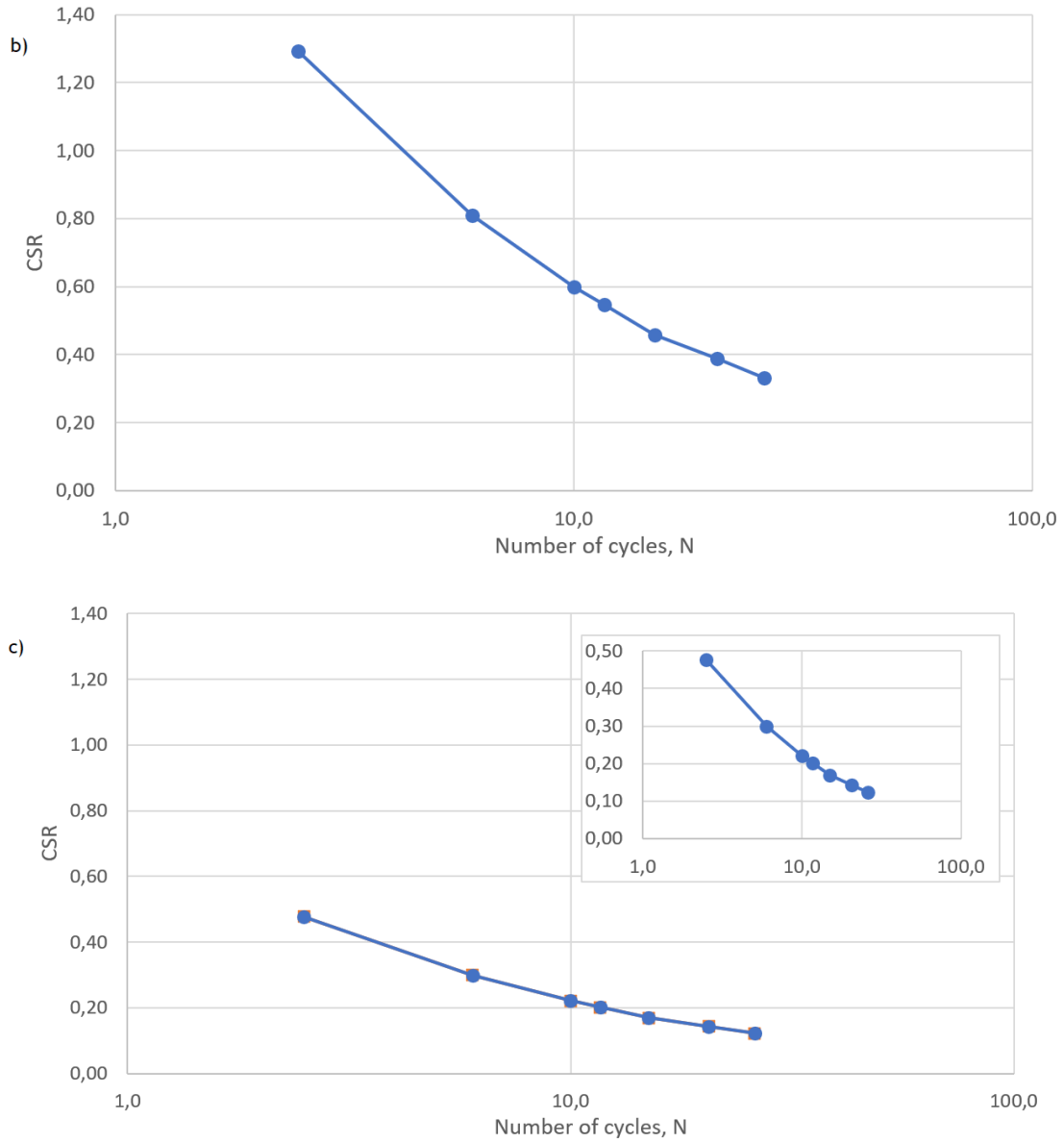


Figure 4.9 Target curves for the sand layers of Profile 2: silty sand 1, Dr=50.4% (a); silty sand 2, Dr=78.1% (b); silty sand 3, Dr=60.99% and silty sand 4, Dr=60.8% (c).

4.5.3 PARAMETRIC ASSESSMENT

Here is important to introduce the difference between the dimensional and dimensionless G_0 . The dimensionless G_0 is the shear modulus coefficient and it is given by Equation:

$$G_0^{adm} = 167 \sqrt{46 * D_r^2 + 2,5} \quad (4.3)$$

And this one is related to the model. As per the dimensional one, it is related to the material of the soil, and it can be determined through equation:

$$G_0^{dmn} = \frac{G}{p_A} \sqrt{\frac{p_A}{p}} \quad (4.4)$$

where G is the dimensionless G_0 of Equation (4.3).

To proceed with this iterative process, it was necessary to understand the effect of these parameters on the CSR-N curves. Toloza (2018) showed in his thesis that depending on which parameters are changed, the CSR-N curves move in certain directions.

From Figure 4.10 it is possible to see that by increasing the values of Dr or hp_0 , the curves tend to move to the right.

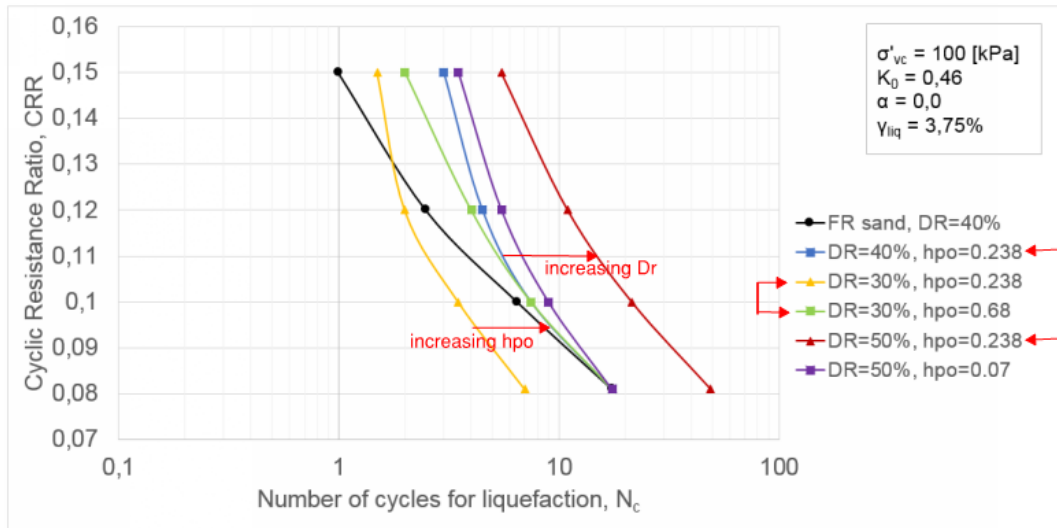


Figure 4.10 CRR-N curves for different apparent relative density Dr and hp_0 (Toloza, 2018).

For the bounding surface parameter, n^b - the secondary parameter that is used to define the initial bounding surface of the model and defines the rate of increment with which the bounding surface approaches the critical surface, as presented in Chapter 2.4.3 - a decrease in the value of this parameter makes the curves move to the left, in a greater scale for higher values of CRR, as shown in Figure 4.11.

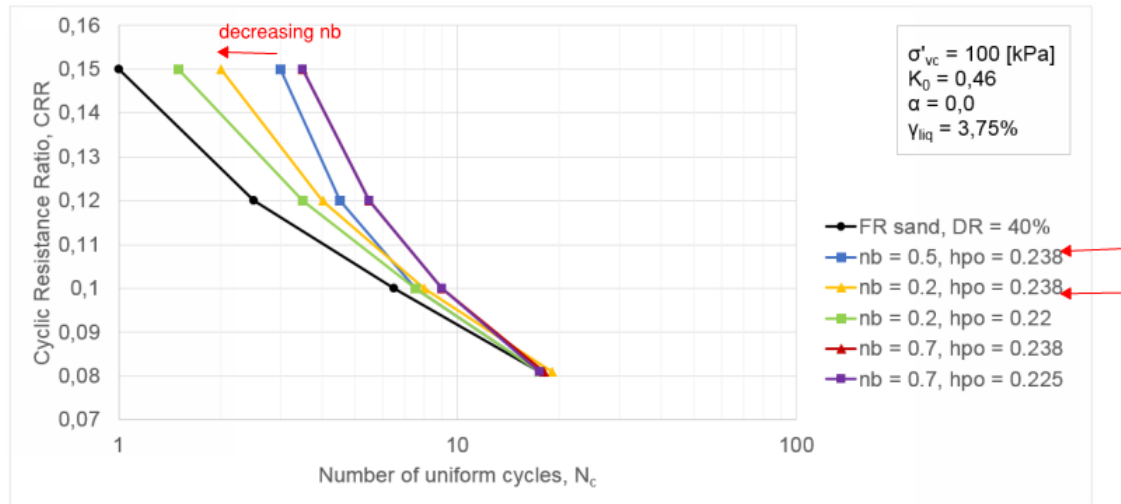


Figure 4.11 CRR-N curves for different values of bounding surface parameter n^b (Toloza, 2018).

Using lower values for the parameter n^b will generate less steep CRR-N curves.

4.5.4 FITTING CURVES

Proceeding the same way as exposed in chapter 4.3, where the iterative process of finding the best parameters to fit the B&I (2014) target curve was already carefully described, for each sand of the two profiles, the goal was to reach new CSR-N curves close to the target curves from the theory from B&I (2014) for sands and silty sands with $p' = 100kPa$, by changing the parameters D_{r0} , h_{p0} and n^b in order to find the best match.

The followed process consisted of fixing the initial relative density D_{r0} and start by changing the n^b with $h_{p0} = 0.2, 0.3$ and 0.4 , and then plotting the curves to see which one best fitted the target, and then repeat it with a different n^b , until a good match was found. Figure 4.12 shows a variety of curves obtained in the process.

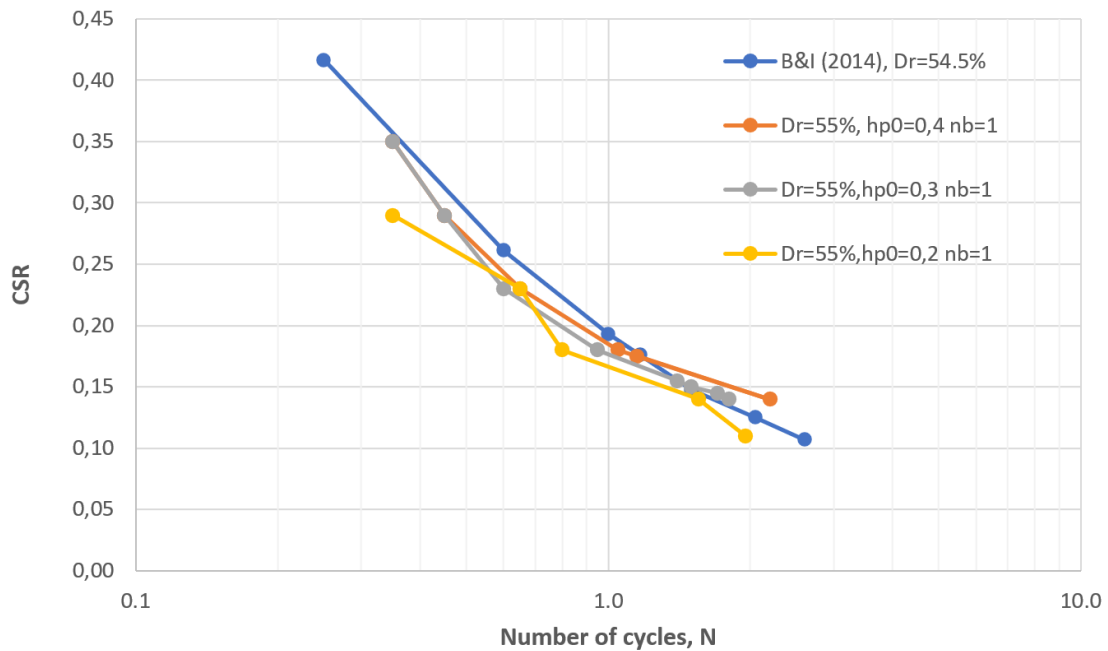


Figure 4.12 CRR-N curves obtained with PLAXIS for $D_{r0} = 55\%$ and different hp_0 in comparison with the theoretical curve from B&I 2014.

So, bearing in mind what was previously presented, the first step was to see what would happen with the increase of the bounding surface parameter, n^b . Figure 4.13 shows the results obtained for different values of this parameter. From the curves with orange tones, the movement of the curves caused by increasing values of n^b is clearly perceptible.

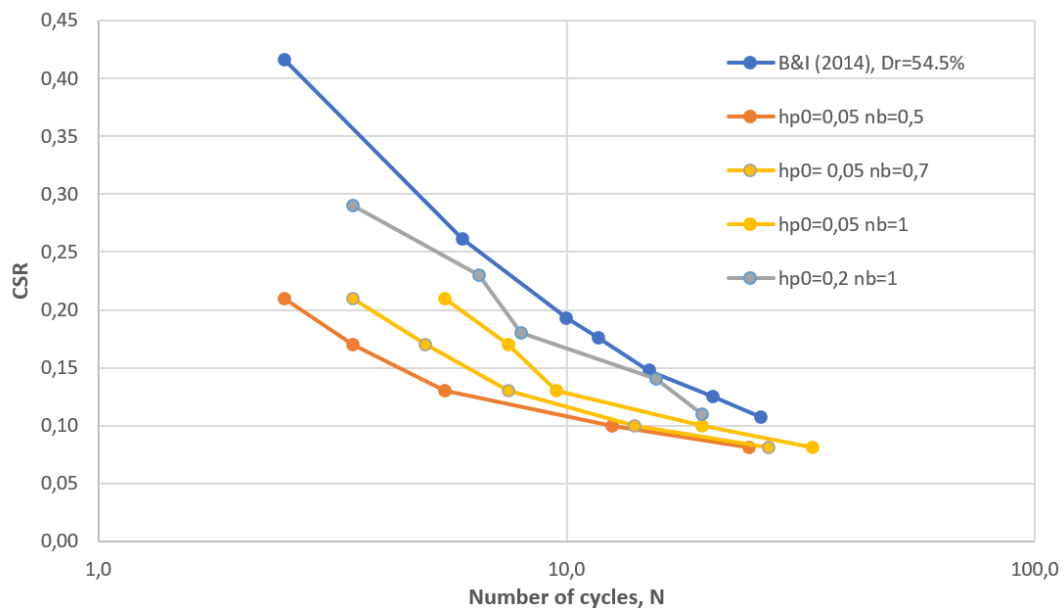


Figure 4.13 CRR-N curves obtained with PLAXIS with $D_{r0} = 55\%$ and different hp_0 and nb compared with theoretical curve from Boulanger 2014.

With this change in the bounding surface parameter, the curves resulting from the tests are getting closer to the target one, and therefore getting closer to the desired goal.

It is important to refer that there was the intention to use the optimization tool in *Soil Test* in PLAXIS, which theoretically would take two points from CRR-N curves with different values of h_{p0} and n^b , and find the optimal solutions between those values that would give the better fitting curve with the one given by Boulanger. Still, this was not possible because the tool was not working properly, and so this option was discarded.

Then, maintaining the bounding surface parameter as $n^b = 0.5$ it was tested the fitting changing the primary parameter h_{p0} .

4.5.4.1 Modelling the sand layer in Profile 1

The result of the previous processes is presented in Figure 4.14, which shows the best fitting achieved for the sand layer in Profile 1, for the target curve from the literature.

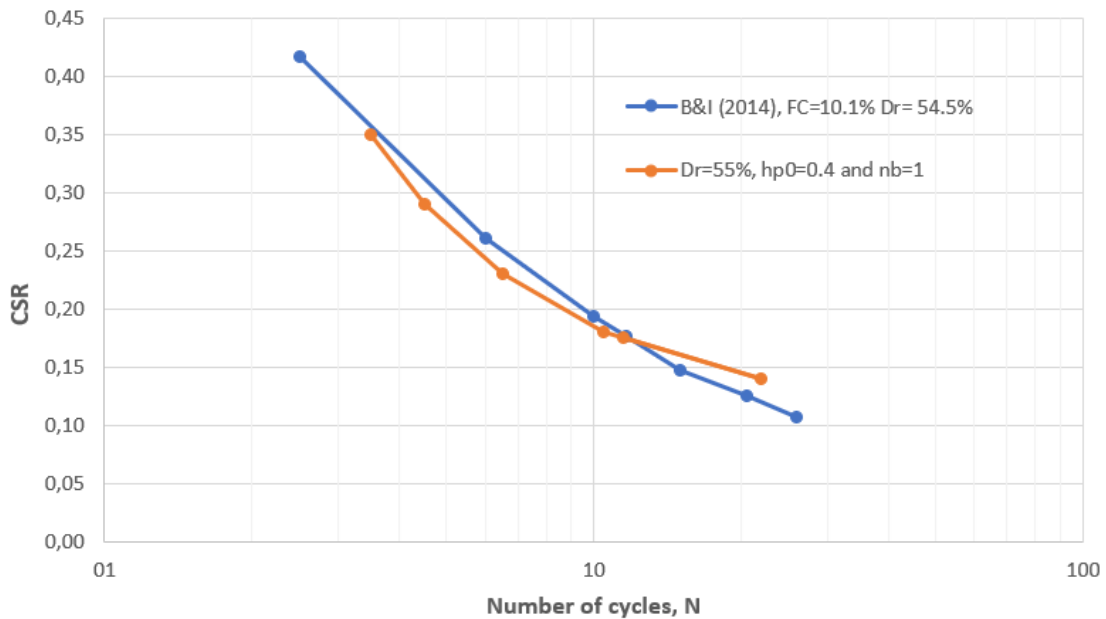


Figure 4.14. CRR-N curves obtain with Plaxis Soil Test simulation, with $Dr_0 = 55\%$ $h_{p0} = 0.4$ and $n^b = 1$ compared with theoretical curve from Boulanger 2014.

The curve represented in orange above and the parameters $Dr_0 = 55\%$, $h_{p0} = 0.4$ and $n^b = 1$, now define the sand of Profile 1. In the following sections, it will be used to assess liquefaction, it will define the limits of the resistance for this sand, corresponding to the CRR-N curve.

4.5.4.2 Modelling the different sand layers in Profile 2

The same process of finding the best parameters that fit curves from the literature (Figure 4.9) was repeated for each of the four sands of Profile 2. Because sands 3 and 4 have very similar relative densities, $Dr=61\%$ and $Dr=61\%$ respectively, it was concluded that the same parameters were a good match for both. As for sand 2, with a $Dr=78.13\%$, a variety of simulations was performed with many different parameters and the conclusion was that the sand would not liquefy. Therefore, for sand 2 it was concluded that it is better to use a model such as Linear Elastic rather than PM4Sand.

The fitting of the curves is presented below in Figure 4.15, and the parameters that will from now on define the sands of Profile 2 are in Table 4.13.

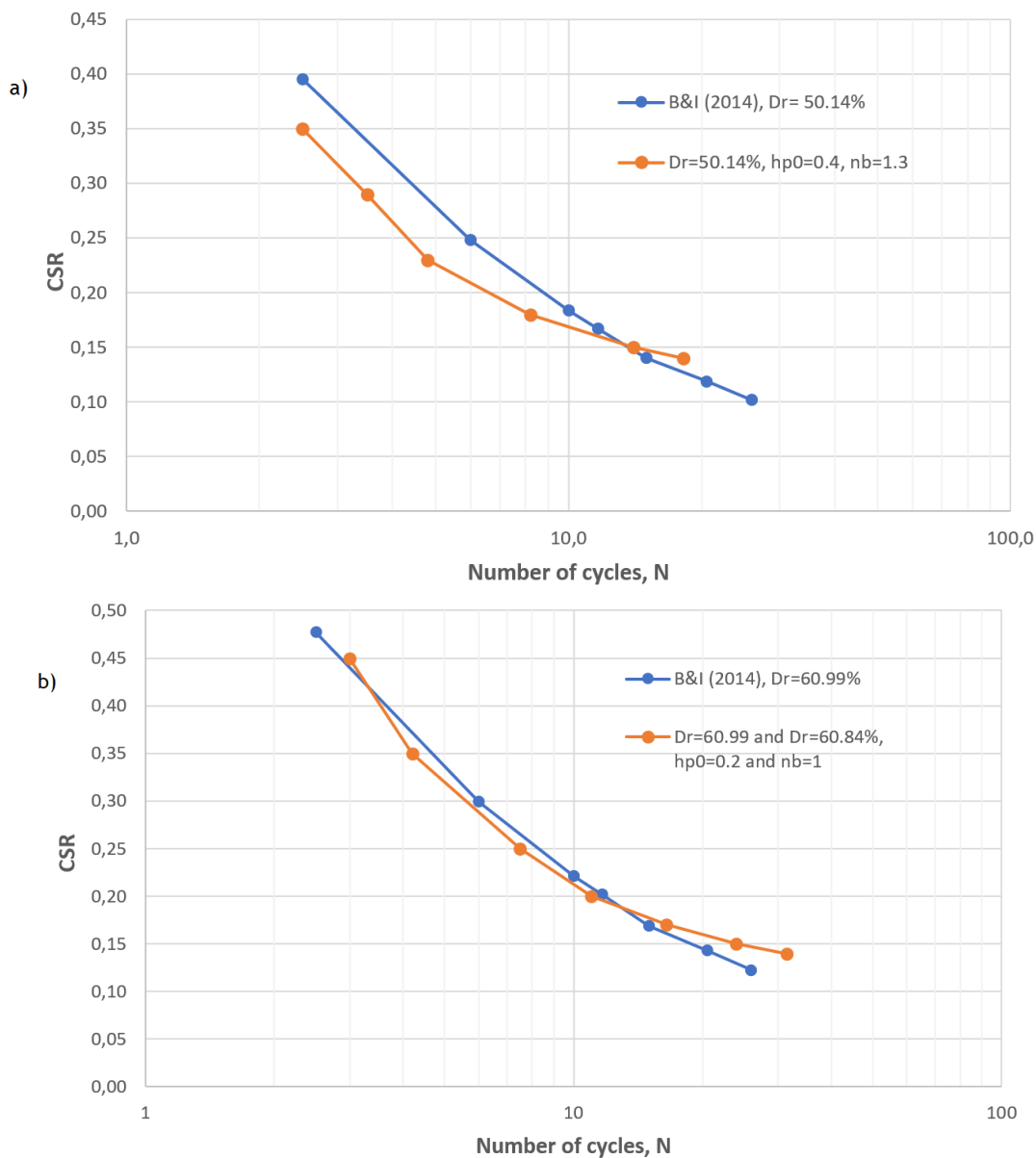


Figure 4.15. CRR-N curves obtain with PLAXIS Soil Test simulation for Profile 2: silty sand 1, $Dr=50.4\%$ (a); silty sand 3 and 4, $Dr=60.99\%$ and $Dr=60.8\%$ (b).

Table 4.13 Parameters for the sands of Profile 2.

Sand layer	Dr [%]	G0 (PM4Sand)	h_{p0}	n^b	Soil model
Silty sand 1	50.41	629.1	0.4	1.3	PM4Sand
Silty sand 2	78.13	923.5	-	-	Linear Elastic
Silty sand 3	60.99	739.6	0.2	1	PM4Sand
Silty sand 4	60.84	738	0.2	1	PM4Sand

In the following chapters, these parameters will be considered for assessing liquefaction and for defining the limits of the resistance for this sand, corresponding to the CRR-N curve.

5 SITE RESPONSE SIMULATIONS

5.1 INTRODUCTION

In this chapter, the main results of the soil profile simulations will be presented and discussed. As previously mentioned, this work is made using the software PLAXIS2D, and to generate the model and then to post-process the results, specific scripts were developed and used based on the programming language Python.

Two Python scripts were specifically developed and provided by COWI, as originally arranged for the purpose of this project. The first script was called *Generate*, and its purpose was to create the model of the soil column with all its properties and the time histories in PLAXIS and to calculate it. The second script, called *Post-Processing*, was designed to take the output of PLAXIS and transfer and organize the required results onto an Excel file.

After the generated model and the excel file with all the necessary data, an evaluation of liquefaction was to be made and the results and conclusions presented. But, as it will be exposed in the following sections, there were some complications along with this work, mostly because the initial scripts were unsuccessful and required several unexpected adjustments.

5.2 1D WAVE PROPAGATION ANALYSIS WITH PM4SAND MODEL

For this thesis, two soil profiles were defined and considered for the study of seismic site response: an idealized one (Profile 1) and a real one (Profile 2) derived from a typical offshore soil profile. These two profiles will be presented in detail in this section.

5.2.1 PROFILE 1: IDEALIZED PROFILE

The soil stratigraphy considered for Profile 1 is represented in Figure 5.1, and consists of a three-layered profile, composed of an overconsolidated clay deposit of medium compressibility from the ground surface to the bedrock at 40 m deep, interrupted for a sand layer, at the depth of 10m down to 20m, with a relative density $Dr = 55\%$.

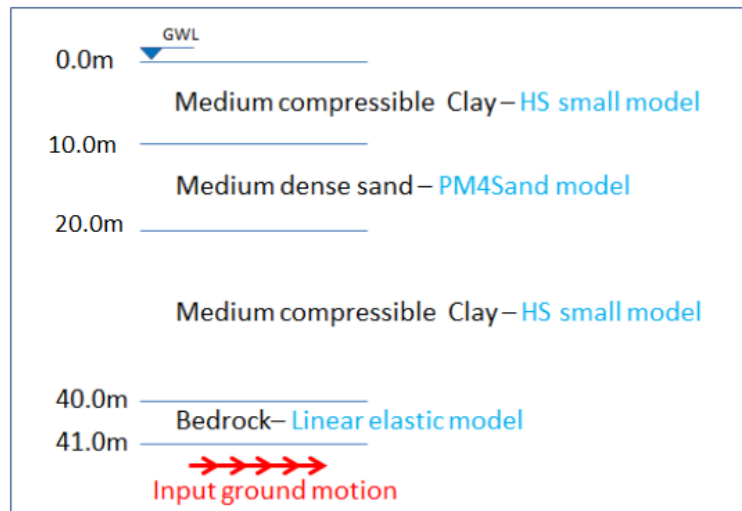


Figure 5.1 Profile 1 – idealized profile.

The clay layers were modelled considering the HSsmall model, and the sand layer was modelled using the PM4Sand model. The bedrock layer was modelled as a 1m layer of Linear Elastic material. In Table 5.1 and Table 5.2 are listed the material parameters for the HSsmall model and the bedrock, respectively. The parameters of the sand layers were already presented in Table 4.10 and Table 4.11 in the previous chapter.

Table 5.1 HSsmall parameters for the clay layers.

Parameter	Symbol	Value	Unit
Drainage type	-	Undrained (A)	-
Unsaturated unit weight	γ_{unsat}	19	KN/m^3
Saturated unit weight	γ_{sat}	21	KN/m^3
Rayleigh damping coefficient	α	0.04189	-
Rayleigh damping coefficient	β	$2.122 * 10^{-3}$	-
Secant stiffness in standard drained TX test	E_{50}^{ref}	9000	KN/m^2
Tangent stiffness for primary oedometer loading	E_{oed}^{ref}	9000	KN/m^2
Unloading-reloading stiffness	E_{ur}^{ref}	27000	KN/m^2
Power for stress-level dependency of stiffness	m	1	-
Cohesion	c'_{ref}	30	KN/m^2

Friction angle	φ'	26	°
Dilatancy angle	ψ	0	°
Shear strain at which $G_s = 0.722G_0$	$\gamma_{0,7}$	0.0007	-
Shear modulus at very small strains	G_0^{ref}	60000	KN/m^2
Poisson's ratio	ν	0.2	-
Reference stress	p_{ref}	100	KN/m^2
Normally consolidated earth pressure at rest	K_0^{nc}	0.5616	-
Cohesion increment	c'_{inc}	0	$KN/m^2 / m$
Reference coordinate	y_{ref}	0	m
Failure ratio	R_f	0.9	-
Tension cut-off	-	True	-
Tensile strenght	σ_t	0	KN/m^2
Over-consolidation ratio	OCR	2	-

Table 5.2 Bedrock Linear Elastic parameters.

Parameter	Symbol	Value	Unit
Drainage type	-	Drained	-
Unsaturated unit weight	γ_{unsat}	22	KN/m^3
Saturated unit weight	γ_{sat}	22	KN/m^3
Young's modulus	E'	$8'10^6$	KN/m^3
Poisson's ratio	ν'	0,2	-

5.2.2 PROFILE 2: REALISTIC PROFILE OF AN OFFSHORE SITE

Profile 2 was derived from a real offshore site soil profile and the parameters and stratigraphy details were provided by COWI. This profile has seven layers consisting of silty sands and clays, and in Figure 5.2 it is possible to see its schematic representation. The parameters of the sands were already presented and are summarized in section 4.5; the parameters for the clays are those exposed in Table 5.3.

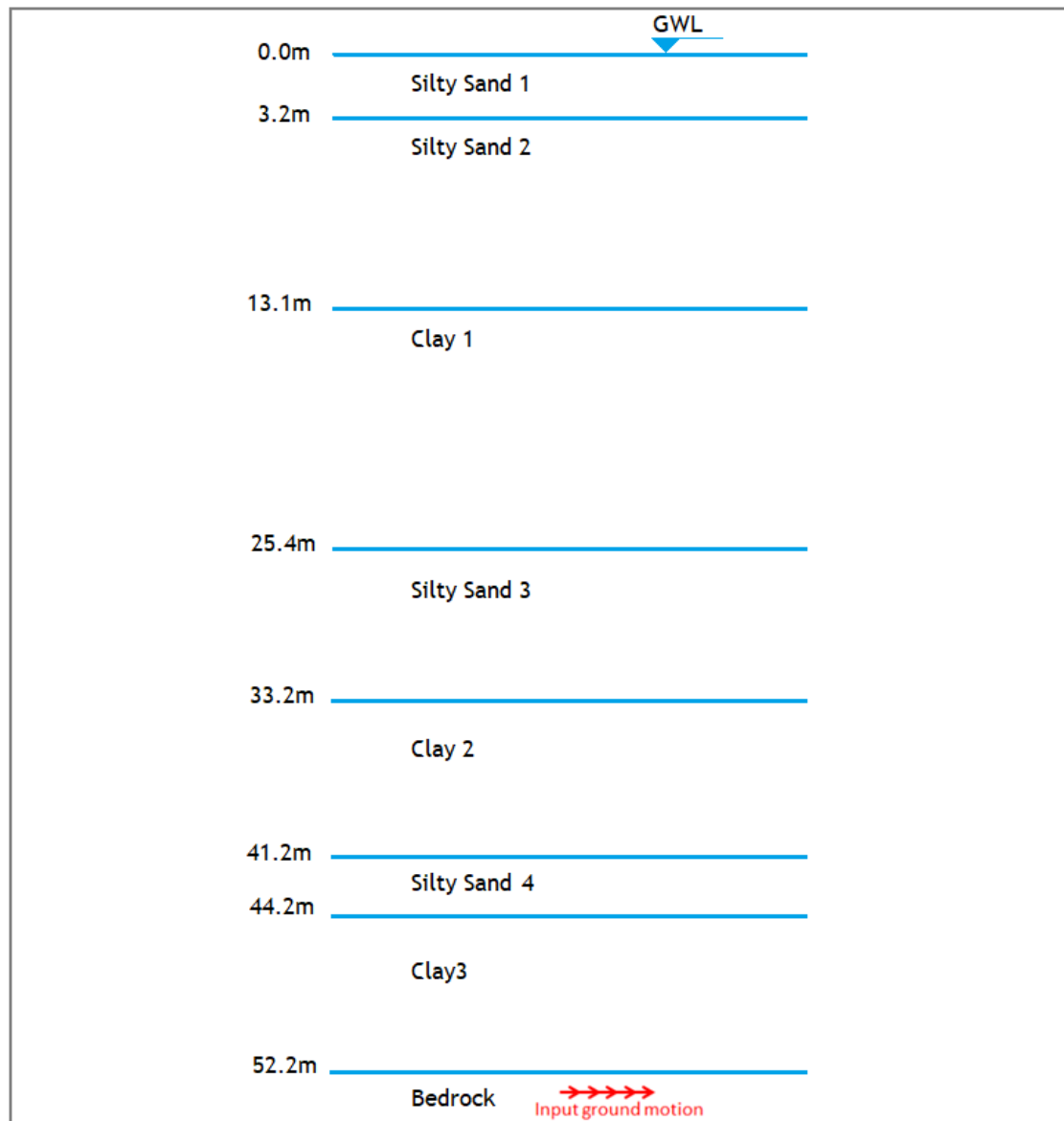


Figure 5.2 Profile 2 from a real offshore site: stratigraphy details (a), Plaxis representation (b).

Table 5.3 Clay parameters provided by COWI for the offshore profile.

Parameters	Clay 1	Clay 2	Clay 3
γ [KN/ m ³]	18.5	18.5	18.5
ϕ [°]	33	33	33
c' [KPa]	0.1	0.1	0.1
c_u [KPa]	70	100	240
V_s [m/s]	19.	235	330

$\gamma_{72\%}$	0.0002	0.000271	0.000305
E_{uref} [MPa]	49.512	86.189	173.174
E_{oed} [MPa]	13.203	22.984	46.180
E_{50} [MPa]	16.504	28.730	57.725
ν	0.2	0.2	0.2

For the bedrock, the same parameters of Profile 1 were considered, with the exception of the unit weight that was also provided and had the value of $\gamma=21 \text{ kN/m}^3$.

5.2.3 PLAXIS

The software PLAXIS2D was an essential tool in this project. After the simulations were completed, it was time to run the profiles in PLAXIS and obtain the results, to generate the model of the soil and, after the model was calculated, collect those results using two Python scripts, which will be addressed in detail in the following section.

These runs in PLAXIS consisted of defining a soil column, generated by the Python scrip, applying the intended loading action, that is, each selected time-history corresponding to an earthquake. For the definition of the earthquake ground motion, although the paper (Vilhar, Brinkgreve, & Zampich, 2018) suggests the creation of a line displacement, it was instead created a line load and, in the panel selection explorer, its definitions should be changed as (Laera & Brinkgreve, 2015) suggests in chapter 3.5. Because the line load is in the bedrock since the effect of the earthquake is transmitted to the soil by this layer, the bedrock properties need to be taken into account to determine the multiplier, which is calculated as follows.

$$E = 8 * 10^9 \text{ N/m}^2; \nu = 0,2; \gamma = 22 \text{ kN/m}^3 \quad (5.1)$$

$$Rh_0 = \gamma * \frac{1000}{9,8} = 22 * \frac{1000}{9,8} = 2242,61 \text{ kg/m}^3 \quad (5.2)$$

$$G_0 = \frac{E}{2(1+\nu)} = \frac{8 * 10^9}{2(1+0,2)} = 3,333 * 10^9 \text{ N/m}^2 \quad (5.3)$$

$$Vs_0 = \left(\frac{G_0}{Rh_0} \right)^{0,5} = \left(\frac{3,333 * 10^9}{2242,61} \right)^{0,5} = 1219,17 \text{ m/s} \quad (5.4)$$

$$\text{Multiplier} = \frac{Vs_0 * Rh_0}{1000} = \frac{1219,17 * 2242,61}{1000} = 2734,1 \text{ kN/m} \quad (5.5)$$

In the same chapter, Lerea & Brinkgræve (2015) also discuss how to define dynamic boundaries. For dynamic phases, in the staged construction mode and on the model explorer panel, the prescribed displacements and its dynamic component must be active. Then, in the same panel, the vertical boundaries are modeled with tied degrees of freedom that will allow modelling a reduced geometry of the problem. The compliant base option can be selected at the base (Y_{min}) and the viscous option should be chosen.

It is also important not to forget to add an interface on the bottom of the model to have a compliant base and make sure that the boundary conditions are properly applied.

Another important aspect to consider whenever using Plaxis or similar software is the mesh size. For these types of analysis, the mesh size should be smaller than 1/10 of the minimum wavelength, λ_{min} , which corresponds to the maximum transmitted frequency. The frequency content of the earthquake is usually up to 5 or 8 Hz; above this frequency, the energy of the earthquake is less significant.

Since the clay layer is the less stiff, for Profile 1 the calculations for the mesh were made as follows:

$$Rh_0 = \gamma * \frac{1000}{9.8} = 20 * \frac{1000}{9.8} = 2038.74 \text{ Kg/m}^3 \quad (5.6)$$

$$G_0 = 60000 \text{ N/m}^2 \quad (5.7)$$

$$Vs_0 = \left(\frac{G_0}{Rh_0} \right)^{0.5} = \left(\frac{60000 * 10^3}{2242.61} \right)^{0.5} = 171.6 \text{ m/s} \quad (5.8)$$

$$\lambda_{min} = \frac{Vs}{f} = \frac{171.6}{8} = 21.45 \text{ m} \quad (5.9)$$

$$\frac{1}{10} * \lambda_{min} = \frac{1}{10} * 21.45 = 2.1 \text{ m} \quad (5.10)$$

Therefore, the mesh size should be 2.1m.

The analysis was made in two points: one point at the ground surface of the soil column and another one at the base of the profile, that is, the bottom of the bedrock. And it is important to take into account the coarseness factor of 1 on the bottom of the model, so that the stress equation $Rh_0 * Vs$ is valid, because when adding the interface this factor can change.

For the Dynamic phase, there are some *numerical control parameters* that need special attention, and that can change from time history to time history. Here, the adopted parameters for the Imperial Valley time history will be presented.

- The *Dynamic time interval* should be equal to the duration of the earthquake in seconds (e.g. for Imperial Valley equal to 63.72s);
- The *Max steps* is equal to the duration of the earthquake per time steps of the earthquake (both can be seen in the time history files, on top of each column there is the number of lines/point of

the time-histories and the time of steps of the earthquake) (e.g. for Imperial Valley 63.72/0.01=6372).

- The *number of sub-steps* is set so that the fastest shear wave, which is the one on the bedrock, ($V_s=1219$ m/s) cannot travel for more than one element during one sub-step. Since one element has 2m, as previously presented the *number of sub-steps* should be calculated as:

$$\frac{\text{time steps of the earthquake} * V_s}{\text{number of substeps}} = \frac{0,01 * 1219}{\text{number of substeps}} = 2m \Leftrightarrow \quad (5.11)$$

$$\Leftrightarrow \text{number of substeps} = 7$$

All of the considerations mentioned above were in a first phase of the project done manually. With the development of the work, it was possible to implement all of them in the Python scripts in order to speed up the preparation of the model.

5.3 PYTHON EXPERIENCE

As previously mentioned, this work was based around two Python scripts that were designed and developed at COWI for the purpose of this project and provided to the student, ready to be used. It is important to refer that there was no previous experience working with Python, and with this language in PLAXIS2D or the dynamic tool of PLAXIS2D.

When the scripts were first provided to proceed with the development of the final part of the work, the task was simply for the student to run them and get the post-processed results, in order to proceed with the analysis required for liquefaction, a key part of this dissertation. Along with the usual difficulties and issues associated with understanding and learning how to use Python scripts and PLAXIS together, the scripts, that were developed as part of an internal development activity in COWI, had never been used before. Contrarily to what was expected, a number of adjustments and additional checks were required, which strongly affected the workflow and especially the outputs.

The original script suffered various modifications along the way, as some parts were removed and others added. This was only possible to do with the experiments that allowed us to understand which important information was needed that was not being extracted by the scripts.

One of the first difficulties encountered was the size of the output files, which were too big, and the computation time, making it impossible to manage between computers. For instance, a file with 2 time-histories would take around 16 hours to run and would be 37GB due to the fact that time histories from too many nodes had been stored. The main reason for this was later attributed to the selection of output nodes, adopted in the post-calculation mode.

Subsequently, several nodes and stress points were selected along the soil column in pre-calculation mode. After this change, the total computation time of *generating+calculating+postprocessing* took around 1-hour, generating an output file of a few megabytes, which was a significant improvement. This process took around three weeks.

Since the main purpose of using Python scripts for post-processing is to speed up the process, there are still some improvements that could be made to the *Post-processing* script to facilitate

the analysis, namely adding a couple of lines in the code to extract the effective vertical stress and the shear stresses of the outputs of PLAXIS. And this is just one example, with a little more time, many other useful improvements could be made.

5.3.1 SCRIPTS

To generate the model in PLAXIS and post-process the results, two Python scripts were created and provided by COWI. The first script was called *Generate*, and its purpose was to create the model with the time histories in PLAXIS and to calculate it. The second script, *Post-Processing*, would take the output of PLAXIS, and transfer and organize the results needed to an Excel file.

For these scripts to work, two text files were needed with relevant information, one with the input for the creation of the model of the soil and another one with the time histories. The *Input* file had the usual information required for the models HSsmall and PM4Sand, already presented in Chapter 2.4, and information about the soil column presented in Table 5.4

Table 5.4 Input parameters of the soil column for the Python Script.

Input	Symbol	Unit
Depth of the model	H	m
Width of the model	l	m
Sea level	H_w	m
Dynamic time interval = duration of the earthquake	-	s
Number of steps in each phase	-	-
Number of Sub-steps in each phase	-	-

5.3.1.1 Generate script

To use this script, it was first necessary to open Plaxis 2D software, on the tab “*Expert*” select “*Configure remote scripting server*” and introduce the password that was on the script code. The port of the Input of Plaxis should be set to 1000, in order to use the code. In the final version of this script, the Output of Plaxis also needed to be opened and the same operation should be performed, with the difference that the *Port* number should be 1001.

Then, still in the “*Expert*” tab, the under-tabs “*Python*” and the “*Interpreter*” should be selected and a blank window pops up, and in there, the script should be imported and then the script can be run. The usual was to comment out the line “*calculate*” of the *Generate* code, so it was possible to check manually in the Plaxis interface if everything was as intended and then pressed manually the button to “*calculate*” in the Plaxis Input interface.

Figure 5.3 presents a simplified flowchart on how to use the script *Generate*.

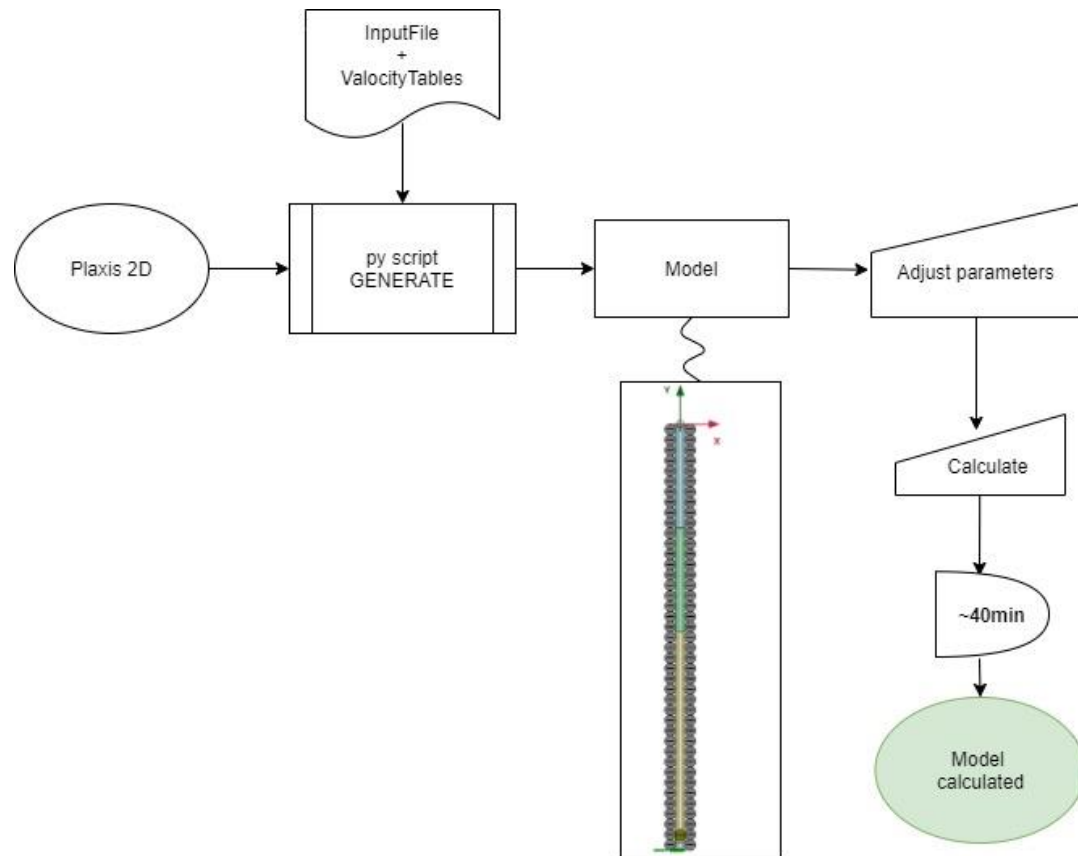


Figure 5.3 Flowchart of how script *Generate* works.

5.3.1.2 *Post-processing* script

After having the model generated and the calculations finalized, it was possible to run the Post-processing script. For this, the Plaxis Output had to be open and, as explained for the *Generate* script, the password of the code should be introduced and the *Port* set for 1001.

In Figure 5.4, a simplified flowchart on how to use the Post-processing script is presented. The script will then select a point, 1 meter apart along the soil model, and take the dynamic results for each point and then save all the information gathered in an Excel file.

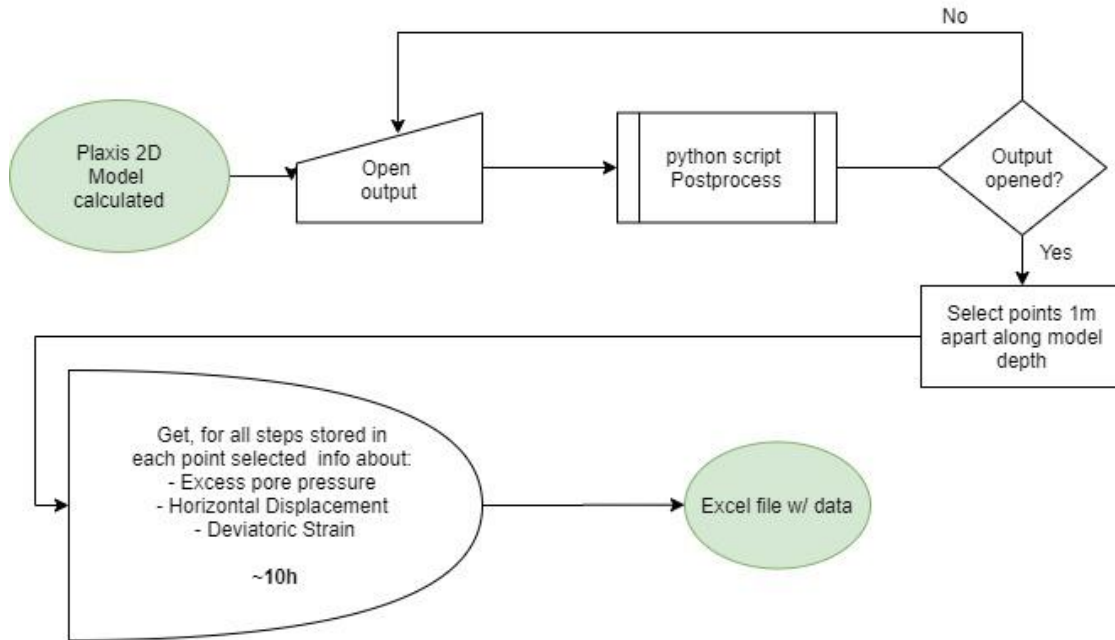


Figure 5.4 Flowchart of how the *Post-processing* script works.

The last version of the postprocessing script that was granted for the project, worked perfectly and was very quick to get the results from the Output of Plaxis, around 1-hour for two time histories (ten times faster than the first versions of the script). It was just necessary to make it run and then the Excel file with the data was ready to be analysed.

The main purpose of using Python for post-processing it is to speed up the process. Ideally, in this specific case, the following information is sought after in a few stages:

- 1) Profiles of maximum (envelope) shear strains: to detect if the value of 3% is exceeded at any depth and at any time during the time history
- 2) Profiles of maximum (envelope) excess pore pressure: to detect if the corresponding excess pore pressure ratio value gets close to 100% at any depth and at any time during the time history
- 3) Profiles of maximum (envelope) horizontal acceleration: from which it is possible to derive profiles of CSR that can be used to estimate the factor of safety against liquefaction together with profiles of CRR.

If the *post-processing* script is correct, the results obtained the results obtained in stages 1, 2 and 3 above and the conclusions obtained from the time histories of shear strains, vertical effective stress and horizontal accelerations extracted by the output program in PLAXIS should be consistent.

5.3.1.3 Post-processing results

After going through the process previously described the data from the script was collected in the excel file and then processed and organized. The results obtained from that are presented in the graphic in Figure 5.5.

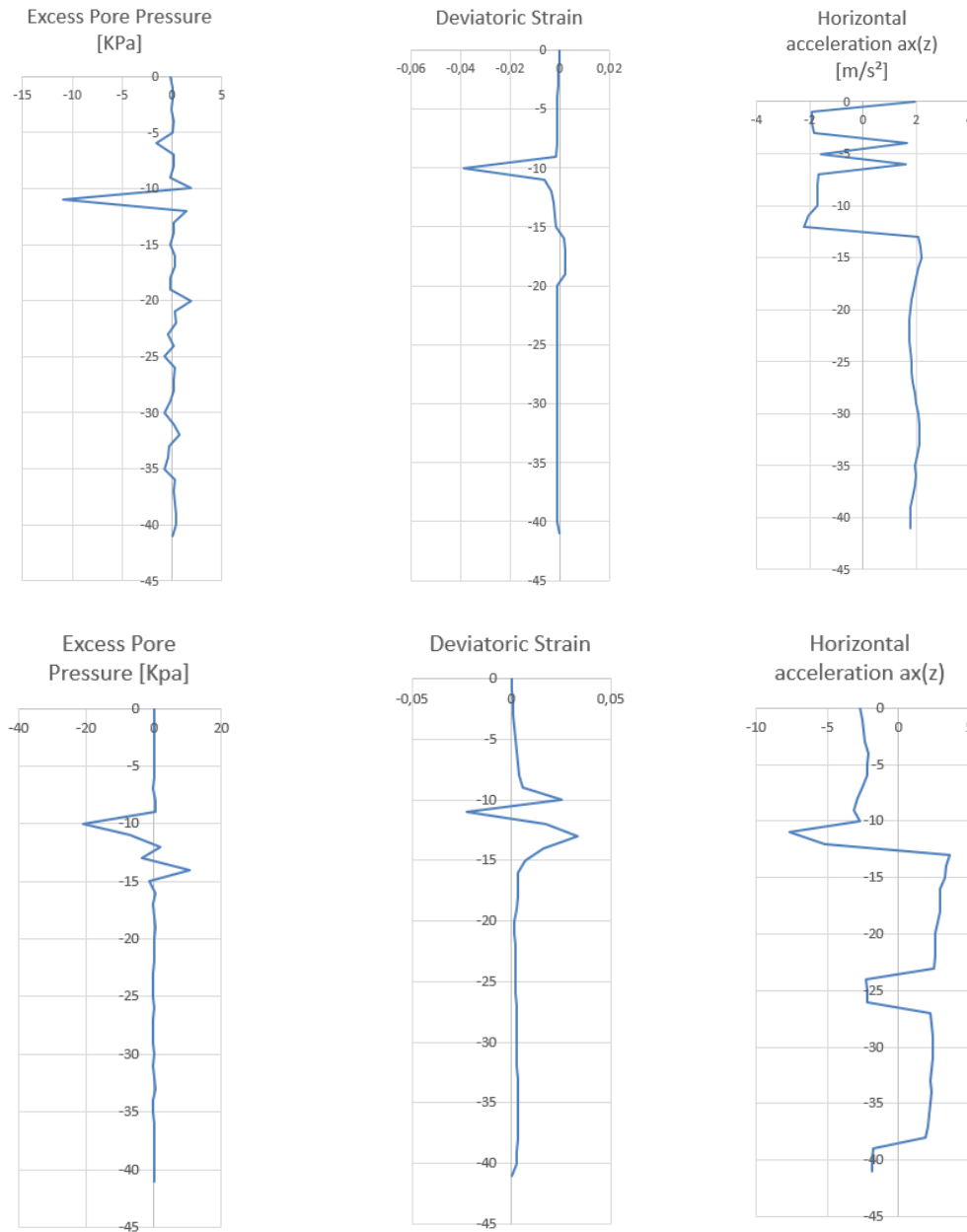


Figure 5.5. Envelope results, taken with the python script, of time histories Izmit, on the top, and Landers Coolwater, on the bottom. From the left to the right: Excess pore pressure [kPa], Deviatoric Strain and, Horizontal acceleration [m/s^2].

To better assess if the results taken from the script were correct, shear strain values, γ_{xy} , were collected manually from the Output of Plaxis. The comparison between them is presented in Figure 5.6, for both time histories. Once the results are sufficiently compatible, it is possible to conclude that the scripts are working as intended and can be used for this kind of evaluations.

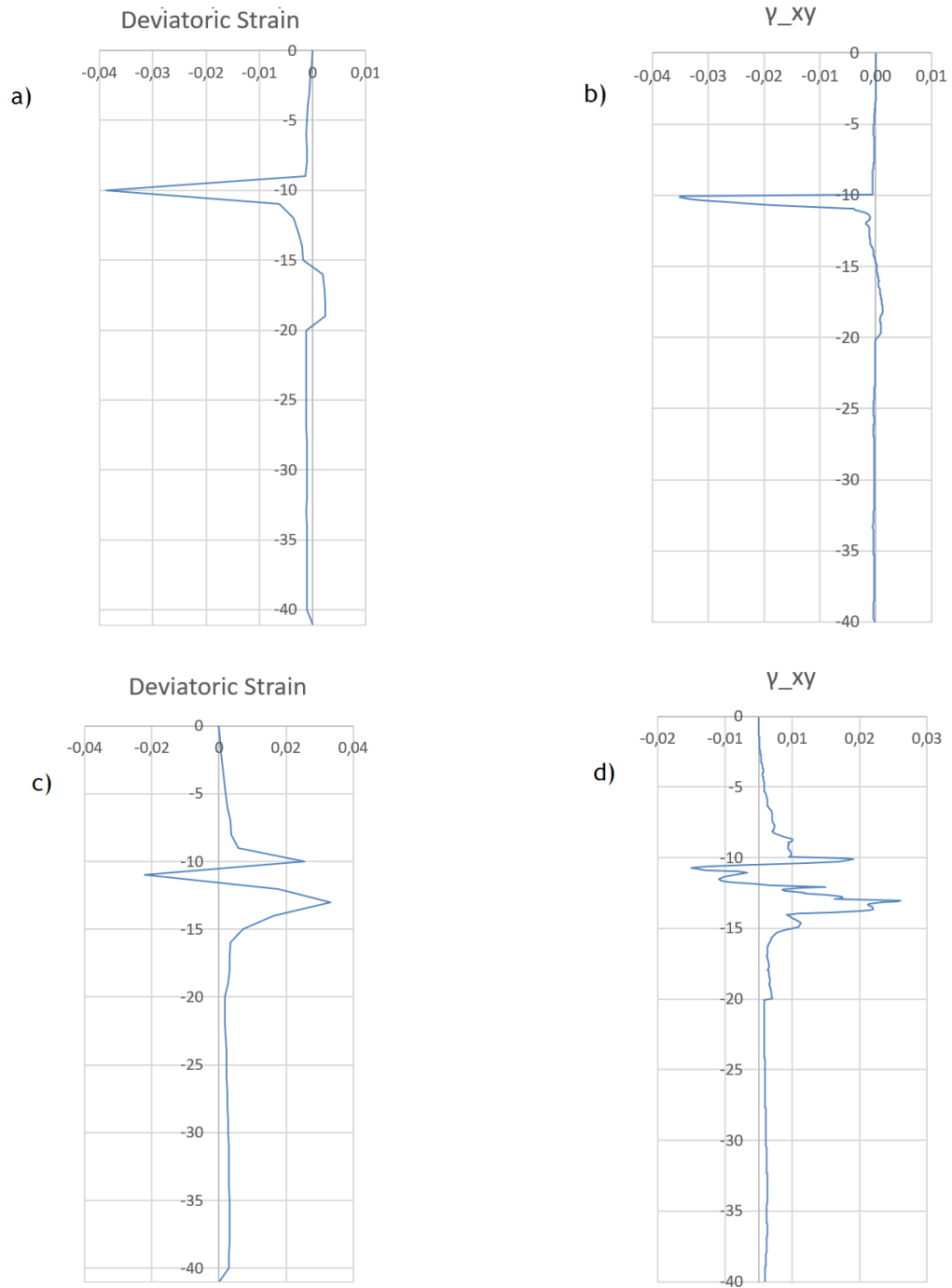


Figure 5.6. Comparison between the envelop of the processed results taken adopting the Python scripts provided, (a) and (c), and the results taken directly from the output of Plaxis, (b) and (d). For the each time history: Izmit, (a) and (b), and Landers Coolwater, (c) and (d).

Taking back the target curve already determined in Figure 4.14 this, as mentioned before, is the CRR curve to access liquefaction. Knowing that the two time-histories in study have an approximate magnitude of 7.5, from Table 2.3 the corresponding number of cycles is 15, and from the target curve it is possible to take the value that will represent the CRR curve for this specific case. And so, CRR has a constant of 0.165, and it is represented in green in Figure 5.7.

The CSR curves, for both time histories, were determined based on Eurocode 8 methodology, and they are in red in Figure 5.7.

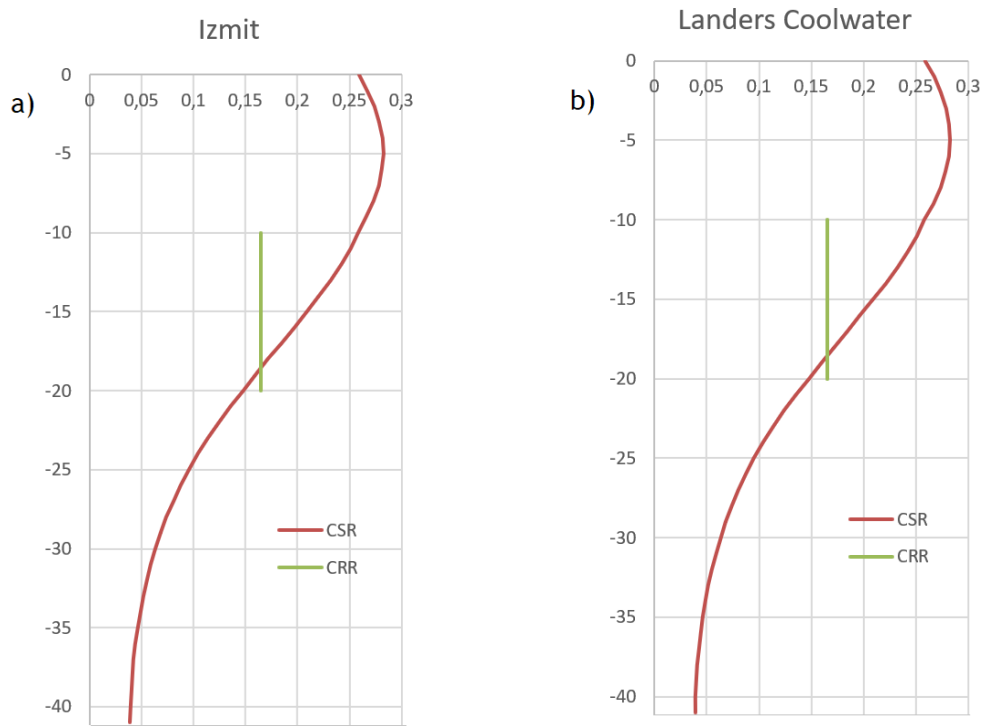


Figure 5.7 CSS versus CRR: a) Izmit; b) Landers Coolwater

It was decided to plot the CRR curve only in the zone correspondent to the sand layer (between 10 and 20 m deep) to have a better perception of the layer in terms of liquefaction potential. By analyzing these graphs, it seems that, for both time histories, the sand layer is practically liquefied for the Eurocode 8 approach.

To check whether this approach was doing a good assessment, the effective vertical stresses of both cases were plotted in Figure 5.8. Looking to these two plots, it can be observed that, for Izmit, liquefaction is only occurring in the first meter of the sand layer, and for Landers Coolwater time history, liquefaction is present for the first 5 meters of sand.

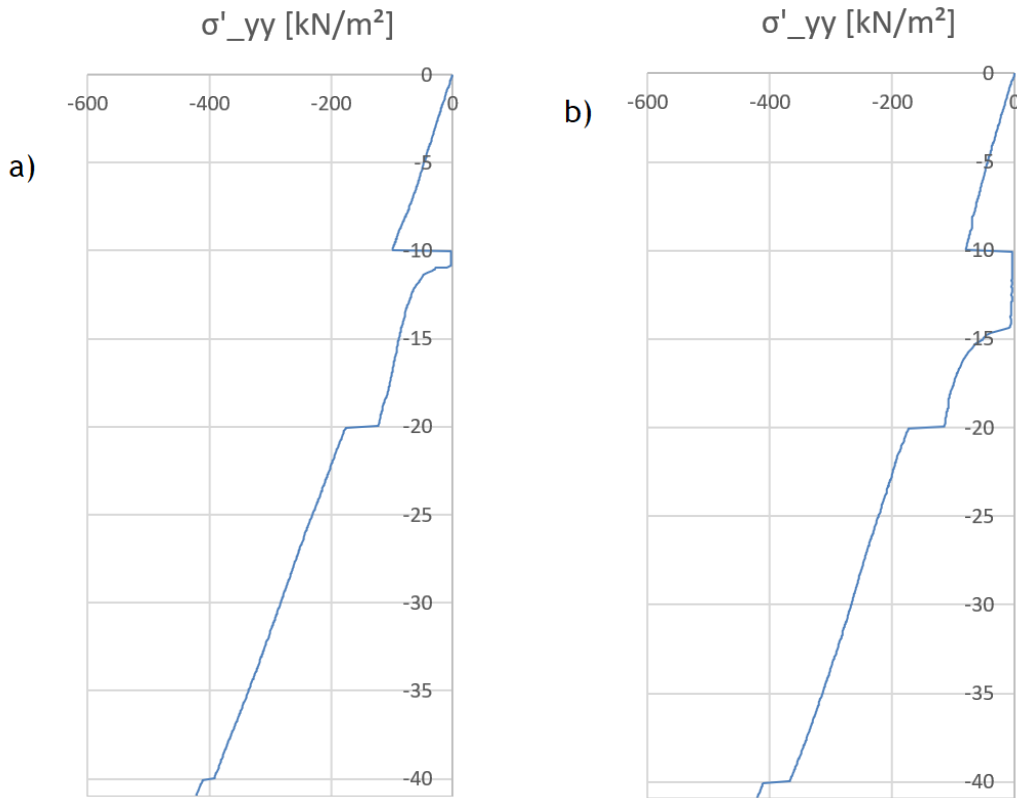


Figure 5.8 Effective vertical stress of time history: a) Izmit; b) Landers Coolwater.

From these two analyses it can be concluded that there is a large discrepancy between what the Eurocode 8 predicts and the reality.

As it is noticeable in the plots presented above, the CSR curves of both time histories are very similar. Keeping in mind that for liquefaction to occur, CSR must have a higher value than CRR, it is easily understood that liquefaction is occurring in the sand layer, that is between 10 and 20 meters deep.

The vertical lines in green, that represent CRR were determined based on the target curve presented in Figure 4.14. Knowing that the magnitudes for Izmit and Landers Coolwater time-histories are around 7.5, respectively, the corresponding value of CRR in the target curve were 0.165.

These examples evidence flaws in the computation code, that have been gradually solved during the time of dissertation, but not fully eliminated. The script is still under development and was not finalized in due time, but the results presented here show that, in the very near future, it will be possible to predict seismic response in an offshore site, considering liquefaction.

6 FINAL REMARKS

6.1 MAIN CONCLUSIONS

Reliable predictions of soil behaviour are essential to ensure the good performance of civil engineering structures and even more so, for the case of offshore wind farms. These mega-projects have been expanding worldwide and are now being implemented in locations where extraordinary natural events, such as violent sea storms, cyclones, and earthquakes, are recurrent, which lead to an even greater challenge in terms of the geotechnical design. In particular, the combination of loose soils with strong earthquakes leads to a high risk of liquefaction of those soils, that can originate hazards to the foundation structure and the wind turbine. This was the challenge proposed by the Danish engineering company, COWI, to be developed in this dissertation.

This project was divided in a number of tasks, involving new and complex subjects, which required plenty of studying, learning and computing. The first task consisted of understanding soil liquefaction, the different methodologies available in the literature to the assessment of this phenomenon, and the constitutive models capable of reproducing this soil behaviour. This is presented in Chapter 2, in the form of a literature review on these different topics.

In order to derive the most adequate and comprehensive approach to liquefaction susceptibility assessment to be used in the numerical studies, different methods were analysed and compared. Different types of soils were also considered in this second task, with emphasis on clean sands and on sands to silty sands. This is discussed and analysed in detail in the sensitivity studies in Chapter 3.

The third task involved the definition and selection of the most representative soil parameters for the adopted constitutive soil model, considering liquefaction. PM4Sand was the selected constitutive model, as it is a specific sand plasticity model for earthquake engineering, and PLAXIS2D, the designated geotechnical software for this work. The quantification of those soil parameters, according to the state of the liquefiable soil (expressed in terms of its relative density), required specific cyclic testing or, in the absence of experimental data, simulations of its response, in numerical element tests. The details and main results of the numerical simulations of cyclic direct simple shear tests on different sands are provided in Chapter 4.

With the calibrated model and the respective soil parameters, it was possible to pursue the main objective of this project: the simulation and prediction of the seismic site response for a series of real earthquake time histories. For this final task, the team at COWI had already been working on advanced scripts to generate and post-process the outputs of the dynamic analyses, since data management is particularly critical when dealing with these time-dependent phenomena. However, the implementation and successful completion of this task was compromised due to the need for successive adjustments to the scripts and operational limitations of computation time and file size. These drawbacks are explained in Chapter 5, evidencing how the problems were detected and solved, although not completely.

There is still a lot to learn about predicting the soil response to earthquakes and especially to liquefaction, but this dissertation aims to represent an important contribution to the process of dealing with liquefaction for the estimate and analysis of seismic site response.

6.2 FUTURE DEVELOPMENTS

At the end of this work, there were still a few developments that were supposed to be presented and that were not finalized due to a lack of time, and others that could be performed to better assess liquefaction and predict seismic response in an offshore site.

First of all, the number of time histories studied is considered scarce as opposed to the 7 that were firstly proposed to study. This occurred because when this evaluation started, the time of process of each time history was extremely long to be manageable, and so it was decided to just do the analysis for the two shortest time histories. When the process was improved, there was not enough time to redo the process with all of the time histories, and so a choice was made to continue with the same two time-histories. Therefore, it would be interesting to do extend these analyses, at least to the remaining 5 selected time histories, in a future development of this work.

Another thing that was proposed to do in the present work and was not possible due to the lack of time, was the same analysis but for a real profile of soil of an offshore site. The profile evaluated here is an idealistic one and to expand to a more realistic soil profile with a number of layers would be very interesting and relevant.

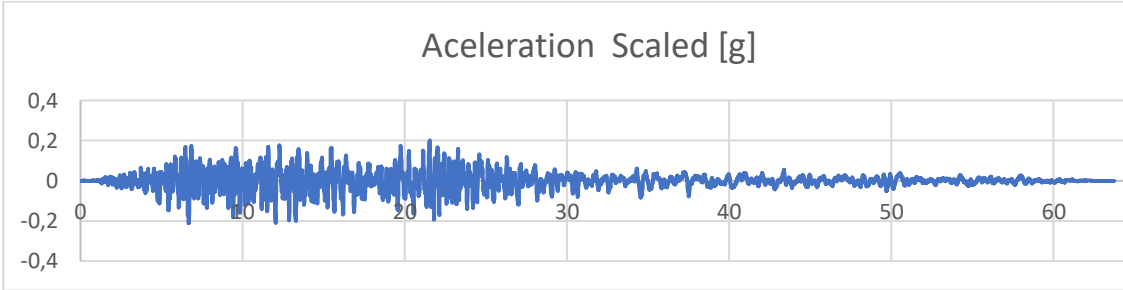
Regarding the Python script, a more objective set of tests should be performed in order to evaluate if they are working as intended. Also, more lines of code could be added with the intention of collecting additional basic data directly from the Output, namely the effective vertical stresses and the pore water pressures.

Appendix

A

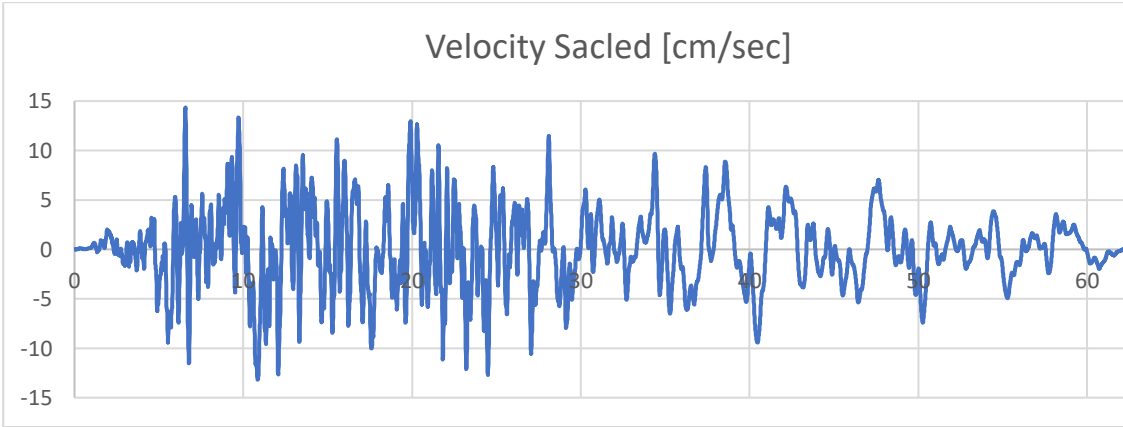
Time histories

Imperial Valley



Imperial Valley Acceleration

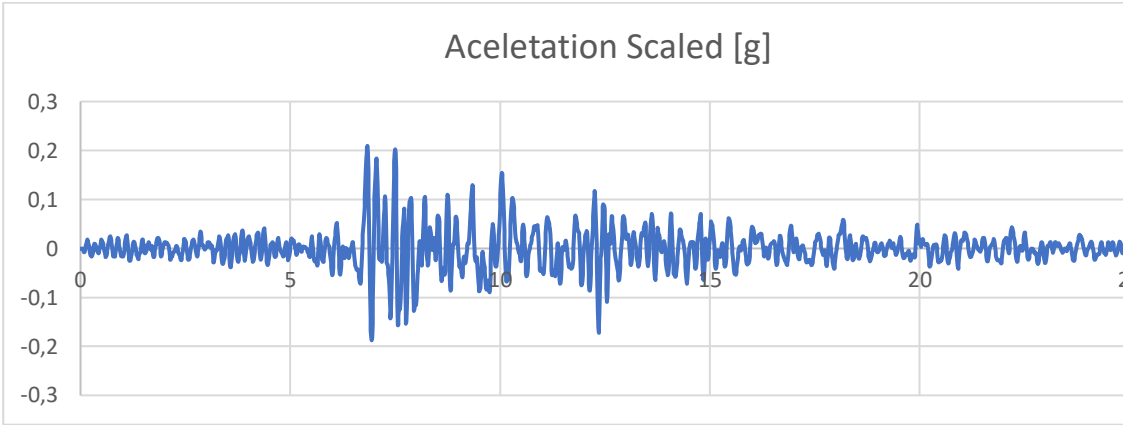
P



Imperial Valley Velocity

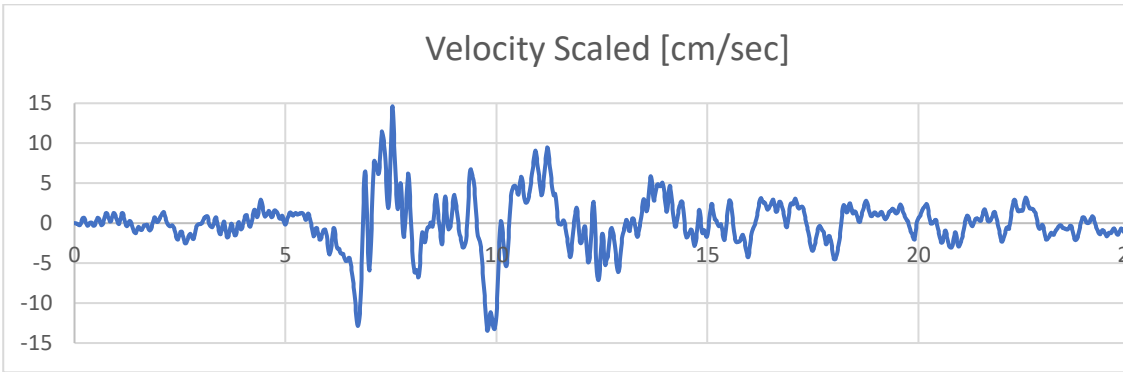
P

Izmit



Izmit Acceleration

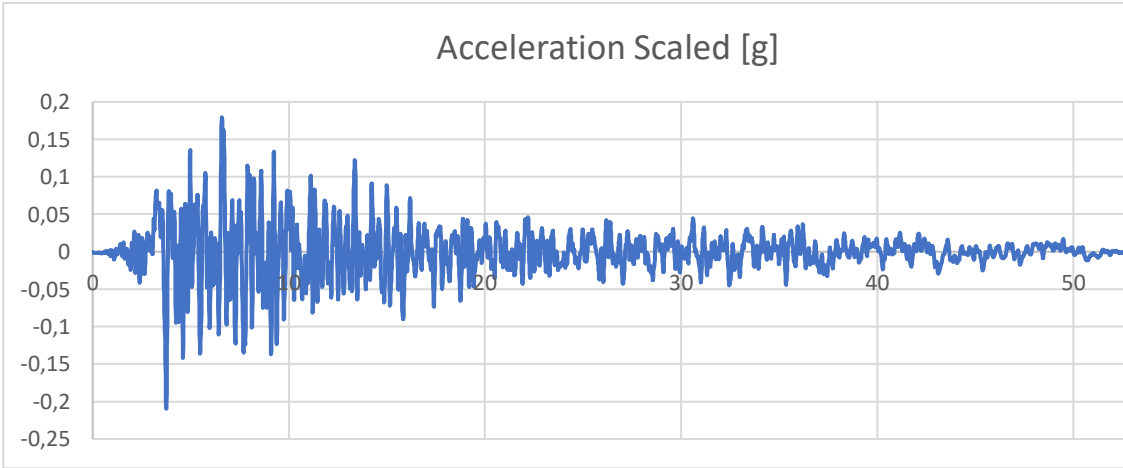
P



Izmit Velocity

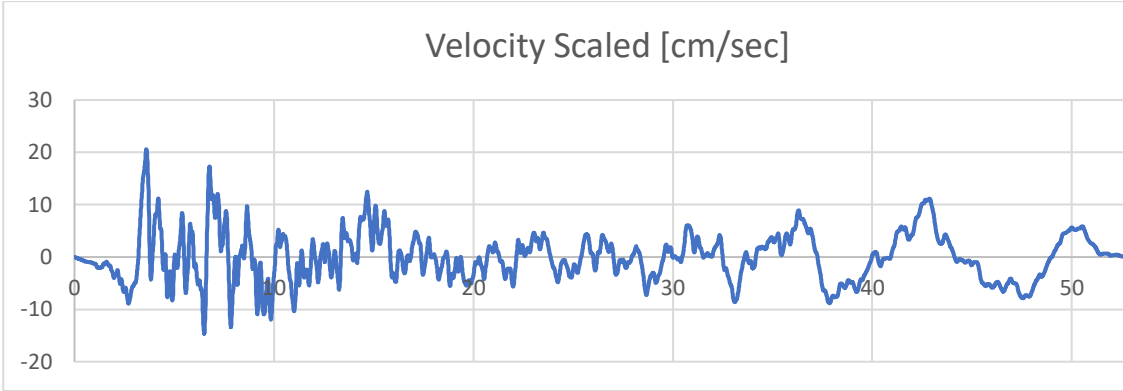
P

Kern County



Kern County Acceleration

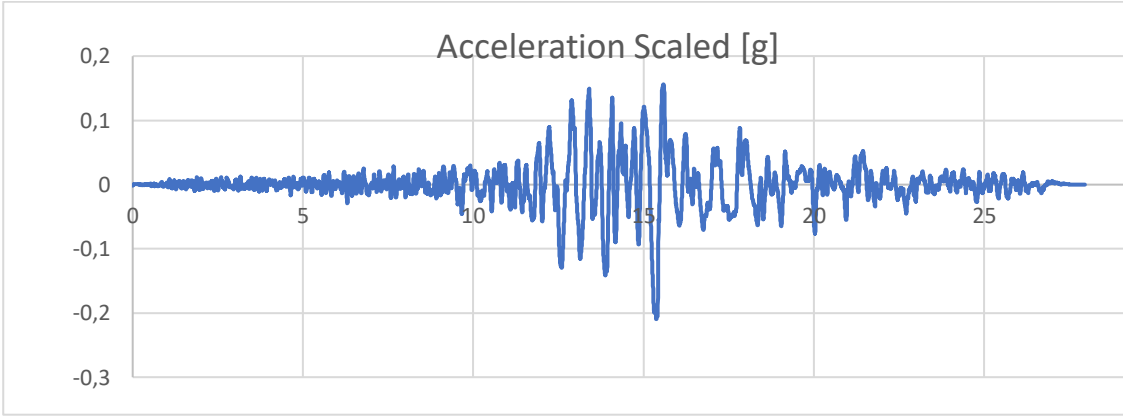
P



Kern County Velocity

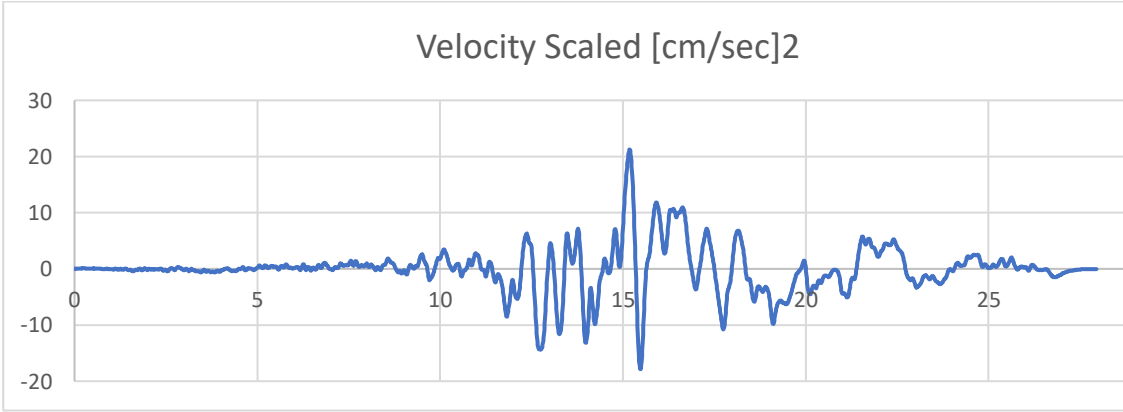
P

Landers Coolwater



Landers Coolwater Acceleration

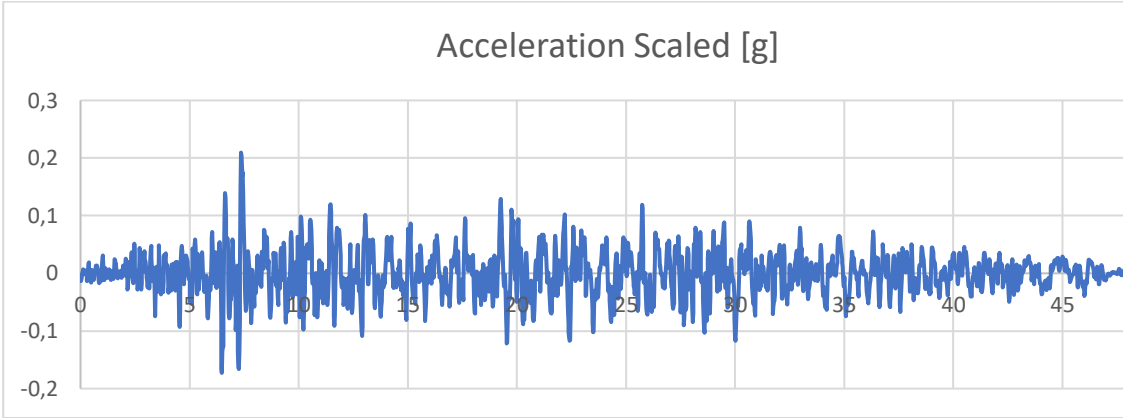
P



Landers Coolwater Velocity

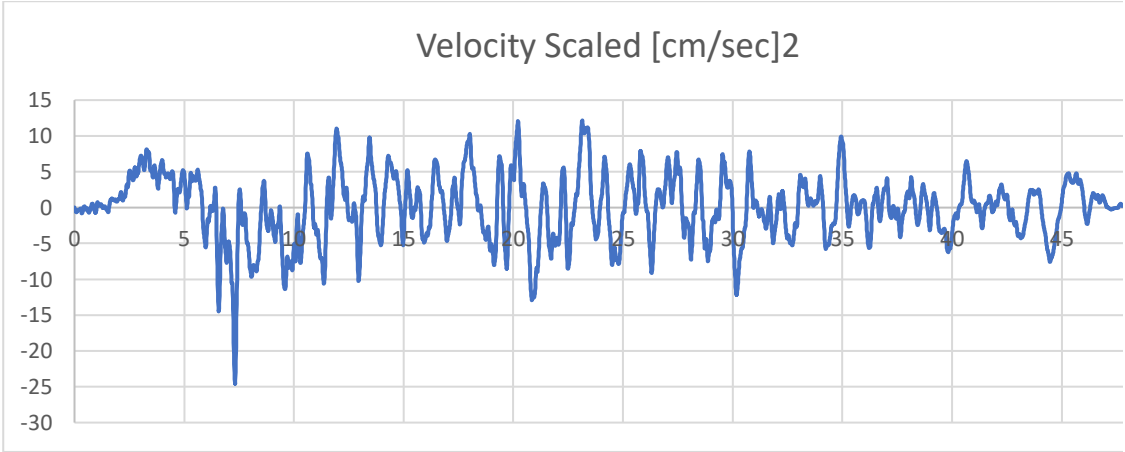
P

Landers DHS



Landers DHS Acceleration

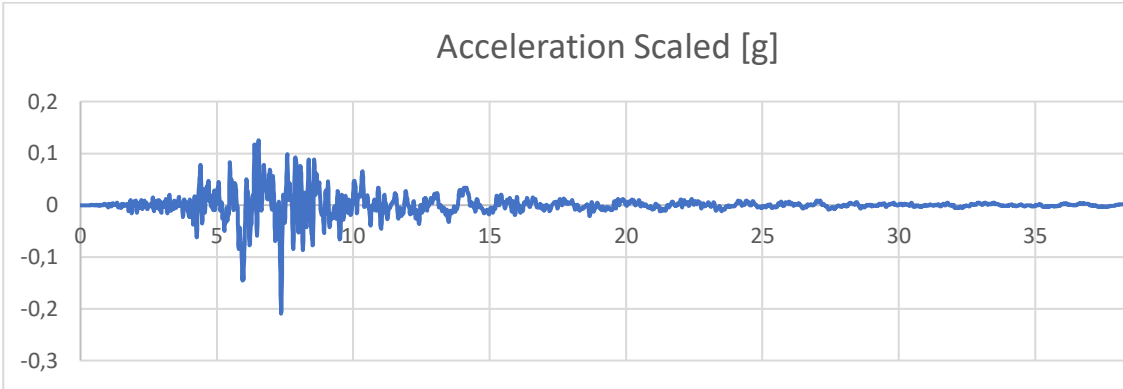
P



Landers DHS Velocity

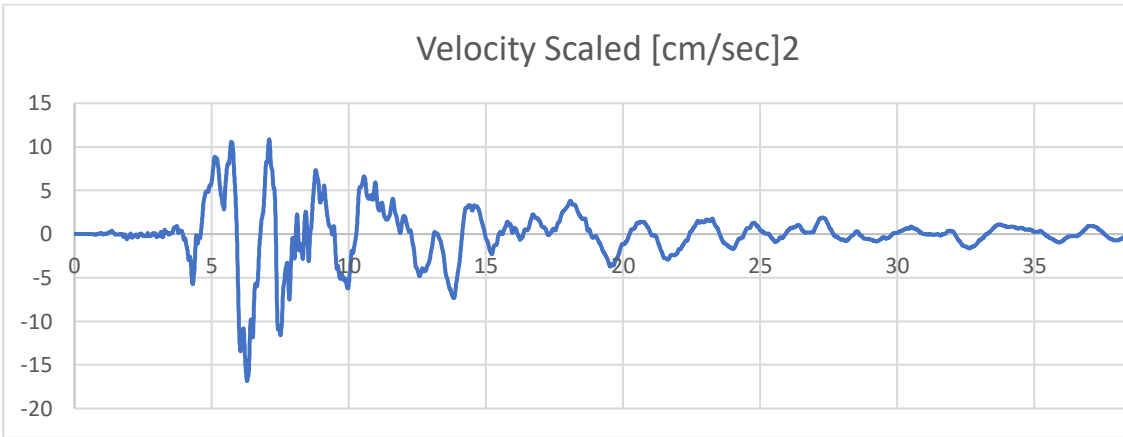
P

Loma Prieta



Loma Prieta Acceleration

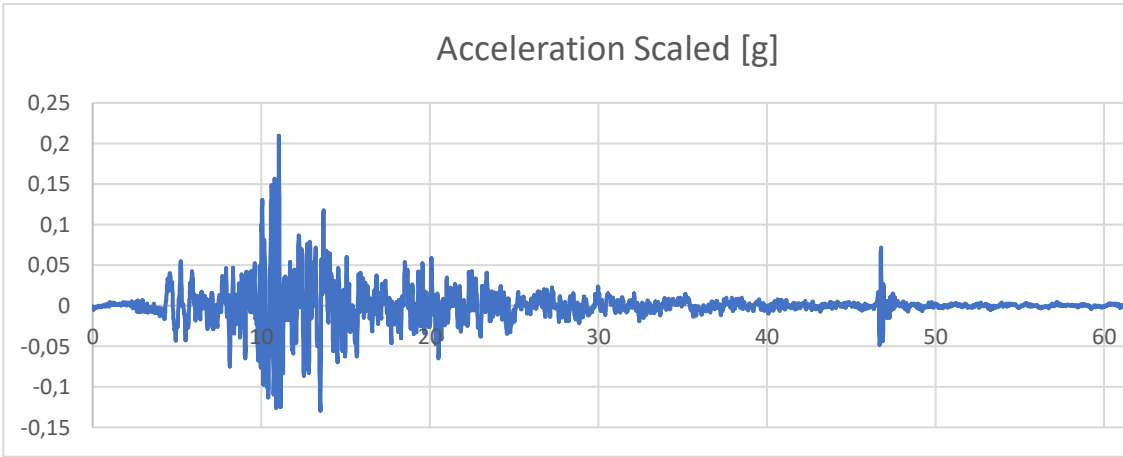
P



Loma Prieta Velocity

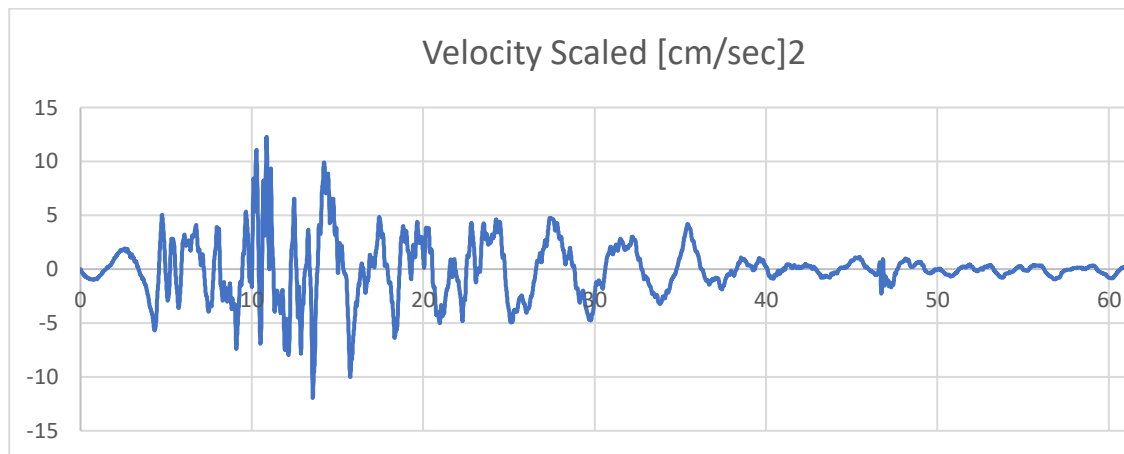
P

Tabas



Tabas Acceleration

P



Tabas Velocity

P

REFERENCES

- Bolton, M. (1986). Strength and dilatancy of sands. *Géotechnique*.
- Boulanger, & Ziotopoulous. (2015). *PM4Sand Version 3: A Sand Plasticity Model for Earthquake Engineering*.
- Boulanger, R. W., & Idriss, I. M. (2007). Evaluation of Cyclic Softening in Silts and Clays. *JOURNAL OF GEOTECHNICAL AND GEOENVIRONMENTAL ENGINEERING*, 13.
- Boulanger, R., & Idriss, I. (2016). CPT - Based Liquefaction Triggering Procedure. *J. Geotech. Geoenviron. Eng.*, 2016, 142(2): 04015065.
- Boulanger, R., & Ziotopolou, K. (2017). PM4SAND (VERSION 3.1): A Sand plasticity model for earthquake engeneering aplication. *Center for Geotechnical Modeling*.
- Bray, J. D., & Sancio, R. B. (September de 2006). Assessment of the Liquefaction Susceptibility of Fine-Grained Soils. *JOURNAL OF GEOTECHNICAL AND GEOENVIRONMENTAL ENGINEERING*.
- Casagrande, A. (1936). *Characteristics of cohesionless soils affecting the stability of slopes and earth fills*. Journal of the Boston Society of Civile Engineers, reprinted in Contributions to Soils Mehanics, Boston Society of Civil Engineers, Massachusse.
- COWI. (2019). Liquefaction susceptibility.
- COWI. (2020). *Introduction to suport structores - Internal presentation*.
- Dafalias, Y., & Manzari, M. (2004). *Simple Plasticity Sand Model Accounting for Fabric Changes Effects*.
- Getty, I. (25 de October de 2019). Obtido de Forbes (last accessed at June, 2020): <https://www.forbes.com/sites/davidrvetter/2019/10/25/offshore-wind-can-power-the-world-say-former-skeptics/#7ed0e094265f>
- Idriss, & Boulanger. (2014). *Guide to cone penetration test*.
- Jefferies, M., & Been, K. (2016). *Sil liquefaction: A critical state approach*. CRC press.
- Kaynia, A. M. (2018). Seismic considerations in design of offshore wind turbines. *Soil Dynamics and Earthquake Engineering*, 9.
- Kramer, S. L. (1996). *Geotechnical earthquake engineering*. Prentice Hall, Upper Saddle River, N.J: Prentice Hall, Upper Saddle River, N.J.
- Laera, A., & Brinkgreve, R. (2015). *Site response analysis and liquefaction evaluation*. PLAXIS.
- Matos Fernandes, M. (2011). *Soil Mecânicos - Introduction to Geothectical Engineering Vol II*. Porto: FEUP edições.
- Mayne, P. (2018). *ResearchGate*. Obtido de https://www.researchgate.net/figure/Setup-and-procedures-for-cone-penetration-testing-CPT_fig5_331021415
- Papadopolou, A., & Tika, T. (2008). The effect of fines on critical state and liquefaction resistance characteristic of non-plastic silty sands.
- Robertson, P. K. (2010). *Guide to Cone Penetration Testing*. Gregg.
- Seed, H., & Idriss, I. (1971). Simplified procedure for evaluating soil liquefaction potential. *J Geotech Eng Div ASCE* 97(9):1249–1273.

- Toloza, P. (2018). LiquefactionModelling using the PM4Sand Constitutive Model in PLAXIS 2D.
- Ungtss. (1964). *English Wikipedia (last accessed at June, 2020)*. Obtido de https://commons.wikimedia.org/wiki/File:Liquefaction_at_Niigata.JPG
- Viana da Fonseca et al, A. (2019). The geotechnical test site in the greater Lisbon area for liquefaction.
- Vilhar, G., Brinkgreve, R., & Zampich, L. (2018). *PLAXIS The PM4Sand model 2018*. Plaxis.
- Visone, C., Bilotta, E., & Magistris, F. S. (2011). *Remarks on site response analysis by using Plaxis dynamic module*. Plaxis practice.
- Wood, C. G. (2012). *Mechanical Behaviour Of Soils Under Environmentally Cyclic Loads*.
- Youd, T., & Idriss, I. (2001). LIQUEFACTION RESISTANCE OF SOILS: Summary Report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of Liquefaction Resistance of Soils.