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Reliability of Quay Walls

Master of Science Thesis
Probabilistic Finite Element Calculations of Quay Walls

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Reliability of Quay Walls

Calibration of partial safety factors for parameters in quay wall design by probabilistic Finite Element Calculations

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'This is our true state; this is what makes us incapable of certain knowledge and of absolute ignorance. We sail within a vast sphere, ever drifting in uncertainty, driven from end to end. When we think to attach ourselves to any point and to fasten to it, it wavers and leaves us; and if we follow it, it eludes our grasp, slips past us, and vanishes for ever. Nothing stays for us. This is our natural condition and yet most contrary to our inclination; we burn with desire to find solid ground and an ultimate sure foundation whereon to build a tower reaching to the Infinite.'

Blaise Pascal - Pensées

Management Summary

Motivation

A committee of Civieltechnisch Centrum Uitvoering Research en Regelgeving (CUR) is currently working on the update of the handbook Quay Walls (CUR 211). In the past the partial safety factors in this book were derived using a spring model, but new developments enable to use Finite Element Methods (FEM) to model sheet-pile structures and quay walls with relieving floor. The main objective is to use FEM (in this case PLAXIS) to calibrate the partial safety factors.

To fulfil this objective, level II probabilistic calculations (FORM) are performed using Prob2B (a toolbox from TNO) to define influence coefficients for the different parameters and reliability indices for the different failure mechanisms. This is done for two benchmark quay walls: an anchored sheet-pile and a quay wall with relieving floor. Both are designed according to the FEM design guidelines in CUR 166, the handbook on sheet-pile structures. Furthermore, the calculations require coefficients of variation for each stochastic parameter, correlation coefficients and definitions of Limit State Functions. These are all derived and presented in the report. With the output of the level II calculations, a level I method is used to define the partial safety factors and besides the design procedure of CUR 166 with FEM is evaluated and suggestions for adaptation for quay walls with relieving floor are presented.

Recommendations

Following this study, it is recommended to:

- Use the presented partial safety factors of Table 8-1until more research is done.
- Do more research with respect to the definition of coefficients of variation, especially the influence of spatial correlation. It can be reasonable to use the lower safety factors of Table 8-2.
- Lower the target reliability of 'failure of the passive soil wedge' and raise that of 'anchor failure in tension' in the fault tree for quay walls.
- Rewrite CUR 166 chapter 4 to be applicable on the design of quay walls with relieving floor. A suggestion is given at page 184.
- Make more probabilistic FEM calculations of Dutch quay walls to have better ground for drawing conclusions. Prob2B is suited to perform these calculations. A check with calculations using a higher order level II method would be worthwhile as well.
- Keep on the discussion to make probabilistic calculations (or sensitivity analyses) for each quay wall design that is made to define the most important layers and parameters. Based on these analyses the partial safety factors can be placed on the correct parameters and a more optimal design can be made. This is already possible with the sensitivity analysis in PLAXIS or in a more sophisticated way with Prob2B.

Arguments for the recommendations

The tables with partial safety factors are based on the output of the probabilistic PLAXIS calculations. Initially, no spatial variability for the soil parameters was taken into account. This implied relative high coefficients of variation for the internal angle of friction and stiffness of the soil compared to the values in NEN 6740. This directly influences the values of the partial safety factor: they are much higher than prescribed in the current editions of CUR 166 and CUR 211. When spatial correlations are 'crudely' included the partial safety factors are closer to the values in the codes, but still the factor for internal angle of friction needs to be increase from 1.20 to 1.35 and for the stiffness parameters changes need to be made as well. It is recommended to use different safety factors per failure mechanisms, in order not to overdesign elements.

Soil mechanical failure is the dominant failure mechanism in both benchmarks. It has the lowest reliability index. In the current fault tree, however, it has a higher target reliability index than the other main mechanisms (anchor failure and wall failure in bending). This also leads to a high factor on the internal friction angle. It is more efficient to lower its target reliability index in the fault tree. In that case the partial safety factor on the friction angel is reduced. Some of the failure space should be taken from the mechanism 'anchor failure in tension' and given to 'failure of the passive soil wedge'. Anchor failure has a higher reliability index according to the calculations and can be made even more more reliable by an additional factor on the anchor force, which is already prescribed in CUR 211.

Furthermore, from the study it appeared that the method to design quay walls with FEM as prescribed in CUR 166 is fine for anchored sheet-piles, except for the mechanism soil mechanical failure. However, for quay walls with relieving floor the method should be changed, because the target reliability indices for wall failure in bending and anchor failure are not reached when applying CUR 166 chapter 4.

Finally, it is clear that two (or three, when including the elongated sheet-pile as additional variant) modelled structures are not sufficient to draw strong conclusions about the partial safety factors. It is therefore recommended to model more existing Dutch structures and make probabilistic calculations. The toolbox Prob2B is well suited to perform these calculations. Actually, it is possible to perform these calculations for each future design of a quay wall. Making probabilistic calculations or sensitivity analyses would give insight into the important parameters and enables to make a more optimal design possible for each unique case.

The use of FORM is questionable, because the plastic deformation of soil elements is non-linear, whereas FORM assumes linear behaviour. The application of higher order methods (or even fully probabilistic calculations like Monte Carlo) would therefore be worthwhile to study as well.

Consequences

The main consequences are directly derived from from the recommendations:

- Higher partial safety factors on the internal angle of friction and soil stiffness parameters
- Lower partial safety factors on the geometrical parameters
- Mean values for cohesion, sheet-pile parameters and anchor diameter
- Changes in fault tree for quay wall with relieving floor
- Adaptation of the prescribed calculation method using FEM in CUR 166 for CUR 211
- Additional effort in the research and calculations of other quay walls
- Standardization of probabilistic calculations or sensitivity analysis in quay wall design

Preface

'Talking on the Quay Wall' is the title of the grey print of the photo on the cover of this Master of Science thesis[1]. The quay wall is barely recognizable on the photo, resembling the 'hidden' character of quay walls in general. A glimpse of the bollard at the foot of the person in the middle reveals the structure is a quay wall. Most people are ignorant of the structure and the complexity of modelling and designing it in its surrounding soil and water. Ninety percent of a quay wall is invisible, hidden between different soil layers and water. Furthermore, most large quay structures are located in port areas where most people do not come. Fortunately, for many people a quay wall is just a place where you can walk and talk. At some moments during my research I was jealous of those people. 'Ignorance is bliss' sometimes holds. However, most of the time I enjoyed the work and it satisfied me. I'm sure this thesis made it impossible for me to just walk and talk on the quay wall, as it is fascinating to think about the complexity that is below the talkers.

The talkers on the quay wall seem to discuss important matters. Discussion also forms an important element of this research. Modelling requests decisions. Decisions require arguments and different people come up with different arguments to argue towards their opinion. As a researcher, I discussed many topics with different people. Most people had their own view, from their own experience and background. Different answers came up to questions like: What elements do we include in the research and what elements we exclude? Which model uncertainties are relevant and which we can ignore? How should we validate results and are the results reliable? Can we implement the model results or are we afraid to change track? The interesting part is to filter these opinions and search for the answers that fit in the scope of the research.

A MSc thesis is like a discussion, a big Talk on the Quay Wall. Working with other people towards a goal and during the process the way of approaching the goal gets clearer and clearer. The left talker on the photo already points towards the goal and the way to approach it, or maybe he is misleading the other two? Sometimes a road comes to an end and a switch to another path is the only option. All these questions and discussions are part of the learning curve I climbed the last months. Sometimes I pointed to the wrong spot and focus on another was required. I upgraded both my knowledge about the subjects, as well as my skills to manage a project. However, I would not have succeeded without help of many people.

I would like to thank Dr. ir. J.G. de Gijt and Ir. A.A. Roubos for their assistance during my time at Gemeentewerken Rotterdam. Dr. ir. K.J. Bakker for his advices about geotechnical engineering, PLAXIS and the progress of my work. These three people supervised my work to a large extent. I am grateful to T. Schweckendiek, MSc for his tips and tricks about probabilistic calculations and the application of Prob2B. A special thanks to dr. ir. W.M.G. Courage from TNO as he made Prob2B available for me. Furthermore, he was always available for questions and additional programming work. I also would like to thank the other members of my MSc thesis committee for their critical feedback on my work: prof. drs. J.K. Vrijling, prof. ir. A.C.W.M. Vrouwenvelder and dr. ir. R.B.J. Brinkgreve. Last but not least I'm thankful to the colleagues from Gemeentewerken Rotterdam, they formed a great environment around me to do my work successfully.

Rotterdam, 7 September 2012
Herm-Jan Wolters

[1] © www.flickr.com. The photo is taken at a quay wall in Viareggio, Italy.

Index

List of Symbols

Probabilistic calculations

α_{Xi}	Influence coefficient of parameter
β	Reliability index
$E[X_i]$	Expected parameter value
γ_k	Partial safety factor over characteristic value
γ_μ	Partial safety factor over mean value
γ_R	Partial safety factor resistance
γ_S	Partial safety factor solicitation
Θ	Spatial Correlation length
μ_{Xi}	Mean parameter value
P_f	Probability of failure
$P(x > X)$	Probability of exceedance
$\rho_{Xi,Xj}$	Correlation coefficient between two parameters
R	Resistance
S	Solicitation
σ_X	Standard deviation of parameter
V_X	Coefficient of variation of parameter
$Var[X_i]$	Variance of parameter
$X_{k,i}$	Characteristic parameter value
$X_{d,i}$	Design parameter value
X^*_i	Design parameter value
Z	Limit state function

Soil

C	Cohesion	[kPa]
δ	Angle of wall friction	[°]
e	Pore number	[-]
E_{50}	Secant stiffness modulus at a 50% deviatoric stress	[kPa]
E_{oed}	Oedometric stiffness modulus	[kPa]
E_{ur}	Unloading reloading stiffness modulus	[kPa]
γ_{sat}	Specific saturated soil weight	[kN/m^3]
γ_{unsat}	Unsaturated soil weight	[kN/m^3]
k_h	Subgradation constant	[-]
k_x	Horizontal permeability	[m/day]
k_y	Vertical permeability	[m/day]
K_o	Coefficient of neutral earth pressure	[-]
K_a	Coefficient of active earth pressure	[-]
K_p	Coefficient of passive earth pressure	[-]
m	Amount of stress dependency (power)	[-]
MSF	Safety factor from φ-C reduction	[-]
n	Porosity	[-]
OCR	Over consolidation rate	[-]
p^{ref}	Reference soil pressure	[kPa]
q_c	Cone resistance (CPT)	[kPa]
ρ	Density	[kg/m^3]
R_{int}	Interface condition	[-]
σ'	Effective soil stress	[kPa]

σ_h	Horizontal soil stress	[kPa]
σ_v	Vertical soil stress	[kPa]
τ	Shear resistance	[kPa]
φ	Internal angle of friction	[°]
ψ	Angle of dilatancy	[°]

Structure

A	Cross-sectional area	[m^2]
δ	Deformation	[m]
d	Equivalent depth sheet-pile	[m]
D	Diameter	[m]
E	Young's modulus	[kPa]
EA	Axial stiffness	[kN]
EI	Flexural rigidity	[kPa]
$F_{max,anch}$	Maximum anchor force	[kN]
G	Shear modulus	[kPa]
G_{eq}	Equivalent shear modulus sheet-pile	[kPa]
γ_s	Specific weight steel	[kN/m^3]
I	Moment of inertia	[m^4]
$M_{max,wall}$	Maximum bending moment in wall	[kNm]
$N_{max,wall}$	Maximum normal force in wall	[kN]
v	Poisson's ratio	[-]
$f_{y,steel}$	Yield stress steel	[kPa]
t	Wall thickness	[m]
w	Specific weight plate element	[kN/m^3]
W	Moment of resistance	[m^3]

Loads

F_b	Bollard force	[kN]
F_h	Hawser force	[kN]
q	Surcharge load	[kN/m]

1 Problem Analysis

1.1 Introduction

In May 2003 a committee of Civieltechnisch Centrum Uitvoering Research en Regelgeving (CUR) presented the Handbook Quay Walls (CUR-publicatie 211 [CUR 211], 2003). This handbook contains the knowledge of quay walls in the Netherlands and its design procedures. In 2010 a new committee was formed that should update the handbook. Especially the chapter on the design of quay walls (chapter 6) is outdated and needs to be rewritten. Also an extension with new developments should be included and some additional examples will be presented in the new edition. (CUR Bouw & Infra, 2010)

An important part of chapter 6 on design of quay walls is the introduction of partial safety factors. These factors on the variables must guarantee a certain safety level of the structure. They were determined in 1996 in a semi-probabilistic way based on spring models for sheet-pile structures and adjusted for quay walls. The development of Finite Element Methods (FEM) enables to make probabilistic calculations in a more sophisticated manner. This gives the opportunity to calibrate the partial safety factors for the new edition of CUR 211. This opportunity forms the core of this research.

One of the concerns of the committee related to these partial safety factors is the uniformity between the Handbook Sheet-Pile Structures (CUR-publicatie 166 [CUR 166], 2005) and the Handbook Quay Walls. CUR 166 (chapter 3) as well as the national annex of Eurocode (EC7) prescribes partial safety factors for the soil parameters, whereas CUR 211 uses the characteristic values (without safety factors). The committee decided that the new edition of CUR 211 should correspond to CUR 166. This asks for reconsideration of the partial safety factors for quay walls with relieving floor (design procedure CUR 166, chapter 3) and the implementing of FEM in the new Handbook Quay Walls (design procedure CUR 166 part I, chapter 4).

1.2 Differences between CUR 211 and CUR 166

From the introduction it can be noted that the approach of CUR 211 differs from CUR 166. In the update of CUR 211 these approaches should be the same, in order to follow the EC and reduce the variety in approaches. In principal the approach in both guides is semi-probabilistic, but CUR 211 uses different starting points. This difference is caused by the considerable structural differences between a quay wall with relieving floor and a sheet-pile structure. The relieving floor provides a better distribution capacity for the horizontal loads. These considerations made the former CUR 211 committee decide to use the characteristic values for the soil shear strength and mean values for soil stiffness parameters instead of applying additional partial safety factors. The committee (2003) argued that in this way the real behaviour of the structure can be described more accurate than in the case with design values. This connects to the expectations of the engineer. To reach the required safety level, additional load factors are applied on the calculation results. This is not in accordance with CUR 166 chapter 3, where partial safety factors on the soil parameters are prescribed. (CUR 211, 2003)

Another difference between the two guidelines is the presentation of FEM in CUR 166 chapter 4 to make design calculations to check the global safety factor of the structure. Basically, a reduction of the soils shear strength with factor 1.15 or 1.2 (depending on the safety class) is applied in the FEM-model, which corresponds to the safety factors in chapter 3. FEM can also be used to design sheet-pile structures, according tot CUR166. CUR 211 mentions the possibilities of using FEM, but the method as presented in CUR 166 is not included. This should be done in the update of the guidelines. It can be tried to follow the approach of CUR 166 chapter 4 (FEM), but adaptations may be necessary as a quay wall with relieving floor considerably differs from a sheet-pile structure. It should be noted that chapter 4 of CUR 166 makes use of characteristic soil parameter values, which differs from the approach in chapter 3. However, for the stiffness parameters of the soil design values are used.

1.3 Recent research

As a first research on application of FEM in the new handbook, the CUR 211 committee has started an analysis on two benchmark quay walls (Bakker & Jaspers Focks, 2011). First a 'relative simple' anchored sheet-pile structure, then a heavier quay wall with relieving floor. This analysis includes calculations with MSheet and PLAXIS and shows results for Serviceability Limit State (SLS) and Ultimate Limit State (ULS). The main idea behind this report is to make a comparison between the output of MSheet and PLAXIS and to analyze whether FEM is useful in quay wall design. It is relatively difficult to model a relieving floor structure in MSheet as it is an one dimensional model. PLAXIS gives more straightforward procedures to model 2D cross-sections.

The authors concluded that the calculation method from CUR 166 (chapter 3) cannot be applied immediately on quay walls. The partial safety factors for sheet-piles can lead to insufficient development of the plasticity of the subsoil due to the larger stiffness of a quay wall in comparison to a lighter sheet-pile. This causes the interactive behaviour of the soil and retaining wall not to be fully developed. The authors advice to use FEM (PLAXIS) in quay wall design by applying the same approach as in CUR 166 chapter 4. However, it is not known what the reliability of the structure designed with PLAXIS eventually is. The problem is the feedback of the PLAXIS output to the reliability index (β). This problem is related to the required partial safety factors in order to reach a certain β when designing a quay wall structure in PLAXIS. This leads to the demand for probabilistic calculations in PLAXIS.

1.4 Development of software

There is also development in the FEM software. A couple of years ago Prob2B was developed. Prob2B is a probabilistic package of TNO (Courage & Steenbergen, 2007) which enables to do probabilistic calculations (including methods like FORM) in PLAXIS and other software packages. In this toolbox failure mechanisms (in form of a Limit State Function) can be included and also the application of the Hardening Soil model in PLAXIS is recently added to the options.

1.5 Objective and research questions

The developments in the handbook sheet-pile structures and the (probable) outdated partial safety factors from the handbook quay walls, combined with the developed possibilities of probabilistic calculations in PLAXIS, are the main reason for this research. During the last decades it is getting more important to quantify the reliability of structures in general. The focus has been changed from design on experience to a more scientific probabilistic design.

This research tries to contribute to this development with respect to the design of quay walls. By using a PLAXIS-FORM coupling, the output from PLAXIS calculations is linked to a reliability index (β) and influence factors (α_i) of the relevant parameters. This information eventually can be used to calibrate the partial safety factors that are now in the handbook. Furthermore it can be stated whether the approach from CUR 166 chapter 4 is applicable on quay walls with relieving floor (i.e. CUR 211). The required approach in FEM should be added to the handbook quay walls and the partial safety factors should be evaluated by checking the reliability of some structures by using the adjusted partial safety factors. Additionally, the analysis can provide new insights into required system reliability and the dependencies between the different mechanisms.

From this problem analysis the main objective of this research can be derived: '*Calibrating the partial safety factors from CUR 211*'

From the central question four research questions can be derived:

1. How can the reliability index of a quay wall be determined?
2. What are the reliability indices and parameter influence factors of the two benchmark quay walls?
3. Which partial safety factors can be derived from these reliability indices and influence factors?
4. How can the reliability indices and influence factors be used to optimize the target reliabilities per failure mode and to describe the dependencies between the mechanisms?

1.6 Work approach

The research questions of paragraph 1.5 form the core of this study and can be used to structure the research. This forms the work approach in order to achieve the objective of this research.

The first part of the research is a literature study and forms the theoretical framework that embeds this research. The theoretical framework contains several topics:

- Dutch quay wall design, i.e. the development of the quay walls and the technology of the commonly used relieving floor structure
- Relations between soil stress, stiffness and deformations related tot quay walls with relieving floor
- Design methods: Blum, Spring-Supported Beam and FEM (especially PLAXIS)
- Probabilistic design, especially FORM
- Development of CUR 166 and CUR 211
- PLAXIS-FORM method (Prob2B)

The second part includes the starting points of the research:

- Boundaries and limitations of model and analysis method
- Development of CUR 211 fault tree and possible improvements (system reliability, failure mechanisms, relationships and target reliabilities or failure space)
- Definitions of mean and characteristic parameter values
- Distinction of stochastic and deterministic parameters in PLAXIS-FORM calculations and the way to deduce the parameter values
- Definition of Limit State Functions (LSF)
- Analysis method, i.e. description of the determination of the reliability of quay walls (for several criteria) with PLAXIS-FORM (Prob2B)

The third part is about the actual calculations on quay walls and contains the following elements:

- Reliability of the two benchmark quay walls of Bakker and Jaspers Focks (2011)
- Derivation of partial safety factors

The fourth part includes the generalization of the derived partial safety factor

- Generalization of partial safety factors to be implemented in the handbook quay walls
- Comparison between the reliability indices from the calculations and the reliability indices from CUR 166 and CUR 211

An additional fifth part is the reconsidering of the system reliability and target reliabilities per mechanism using the calculation results. This is basically a reflection on the calculation method and starting points of the calculations.

- Analysis of current requirements on system reliability and target reliability per mechanism by using the calculation results. Are the prescribed reliability indices in CUR 211 realistic or not? And does CUR166 chapter 4 prescribe a reasonable approach for the design of quay walls?
- Analysis of dependency of mechanisms in fault tree based on calculation results

The approach to calibrate the partial safety factors starts with a PLAXIS model of a structure. The parameters in this model can be made stochastic by the toolbox Prob2B. LSF's should be defined for the relevant failure mechanism. This can be implemented in Prob2B and the toolbox uses FORM to search for the design values of each stochastic variable. The influences of the different parameters on the reliability with respect to a certain failure mechanism can be used to define partial safety factors. In principal the 'normal' PLAXIS calculations (i.e. Plastic Analysis) can be used. The resistance parameter values are lowered and the load parameters raised by Prob2B until the design values for all stochastic parameters are found. This is the closest failure point, i.e. the point with the largest failure probability.

Another approach would be the use of φ-C reduction as for instance is done in the benchmark calculations of Bakker and Jaspers Focks (2011). In that case it is presumed that the soil parameters φ and C are dominant and that a relation between the Safety Factor obtained from the φ-C reduction and the reliability index would sufficiently describe the total failure tree of the quay wall. Basically, it is presumed that the structure fails by soil mechanical failure as only the shear trength of the soil is reduced during the φ-C reduction. However, soil mechanical failure is not the only possible failure mechanism. It is also possible that the sheet-pile or the anchor fails. Additionally, the deformations can be to large causing the user function not to be fulfilled. This is not total failure of the structure (Ultimate Limit State) but failure of a function (Serviceability Limit State). φ-C reduction alone therefore possibly gives an incorrect view of the situation, as it presumes that φ and C are governing in each Limit State.

It is clear the structure can fail in several ways (see also the fault tree in section 3.3.2, Figure 3-2) and therefore it is better to execute the 'normal' PLAXIS calculations (i.e. Plastic analysis). If soil mechanical failure is dominant for each quay wall, it would be an option to focus on the relation between the Safety Factor from φ-C reduction and the reliability index. However, initially in this research it is assumed that the structures can fail in more ways and therefore the 'normal' PLAXIS calculations are performed. For the LSF of soil mechanical failure of course φ-C reduction can be used.

As the benchmark quay walls in this thesis are redesigned according to CUR 166, the reliability index still can be coupled to a Safety Factor (MSF) from a φ-C reduction. CUR 166 chapter 4 prescribes to design the anchor and wall according to an anchor force and bending moment at a situation where the φ and C are reduced by a factor (1.15 for safety class II and 1.2 for safety class III). The calculations therefore can lead to a statement about the applicability of the method of CUR 166 chapter 4 for quay walls.

1.7 Report

The structure presented in paragraph 1.6 forms the basis of the report. In chapter 2 and 3 the theory that frames this thesis is summarized. There is a chapter about quay walls: their history, soil mechanical aspects and design models. The second theoretical chapter is about probabilistic methods and the coupling with FEM. Chapter 4 presents the starting points of the probabilistic calculations that are performed. This is mainly about the study boundaries and different model parameters, their coefficients of variation and the derivation of the parameter values.

Chapter 5 and 6 present the results of the probabilistic calculations of benchmark 1, an anchored sheet-pile wall, and benchmark 2, a quay wall with relieving floor. Finally, in chapter 7 some topics are discussed that are relevant with respect to the analysis of chapter 5 and 6. The conclusions and recommendations of chapter 8 follow from the analyses and results of the benchmark calculations and they refer back to the questions in the problem analysis and the assumptions made in the chapter on starting points. The structure of the report is visualized in Figure 1-1.

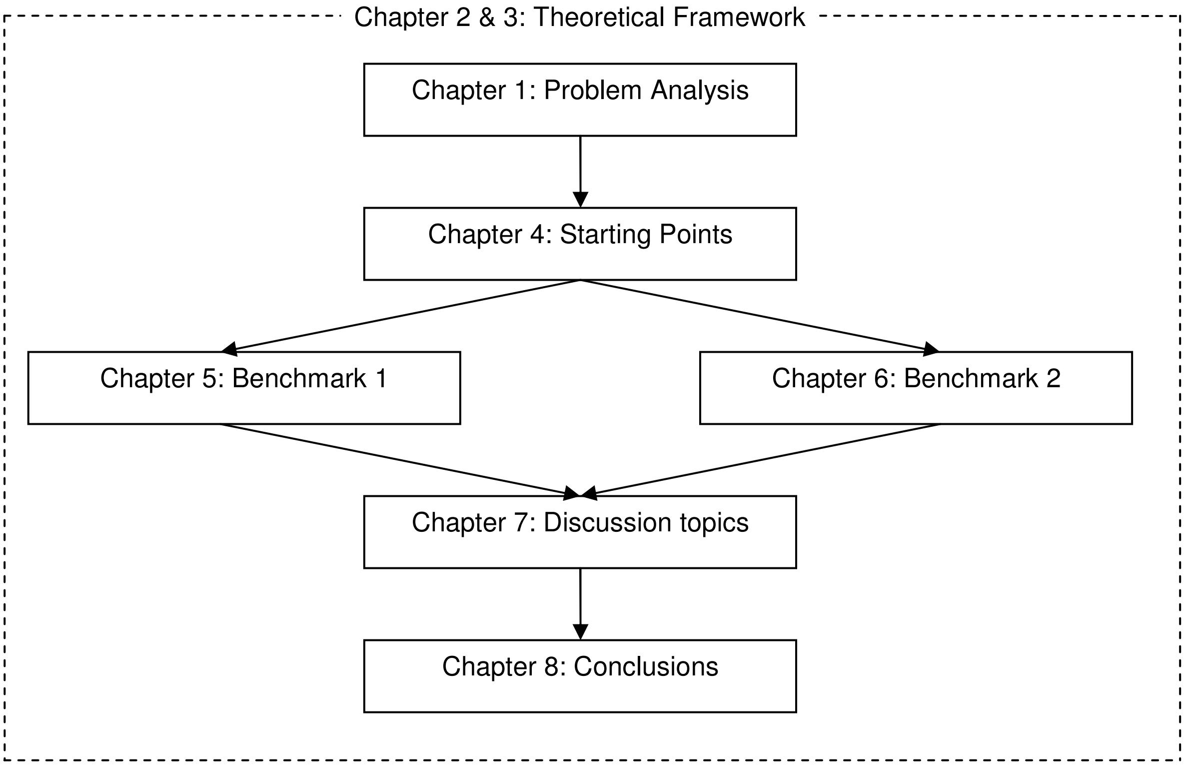


Figure 1-1 Report structure

2 Theoretical framework: Quay Walls

2.1 Introduction

This chapter introduces the background of the main concepts related to quay walls that are used in this research. This concerns three elements:

- A general description of quay walls in the Port of Rotterdam and their development and design aspects (2.2)
- Description of the relationships between soil stress, stiffness and deformations (2.3)
- An exposition of the design models and the possible use of software, especially Finite Element Methods (FEM), in quay wall design (2.4)

2.2 Development and design of quay walls in Port of Rotterdam

This section describes the development of quay walls in the Port of Rotterdam. It involves the history of the Port and the ideas behind the different type of developed quay walls.

2.2.1 History of the port of Rotterdam

The main driver in development of any harbour and its facilities is the intensity of shipping activity within the harbour area. This also holds for the Port of Rotterdam.

The origin of the port of Rotterdam can be found a few centuries ago in the centre of the present city. In the second half of the 19th century the expansion took place to the south side of the river and in western direction because of the larger water depth of the river. Around 1850 the first engines were used for propulsion and the ships were more often made of iron. Due to the increasing demand for transport of goods, the ships were developed further. This caused an excessive growth of the draught of the ships as visualised in Figure 2-1. (Tol & Gijt, 1999)

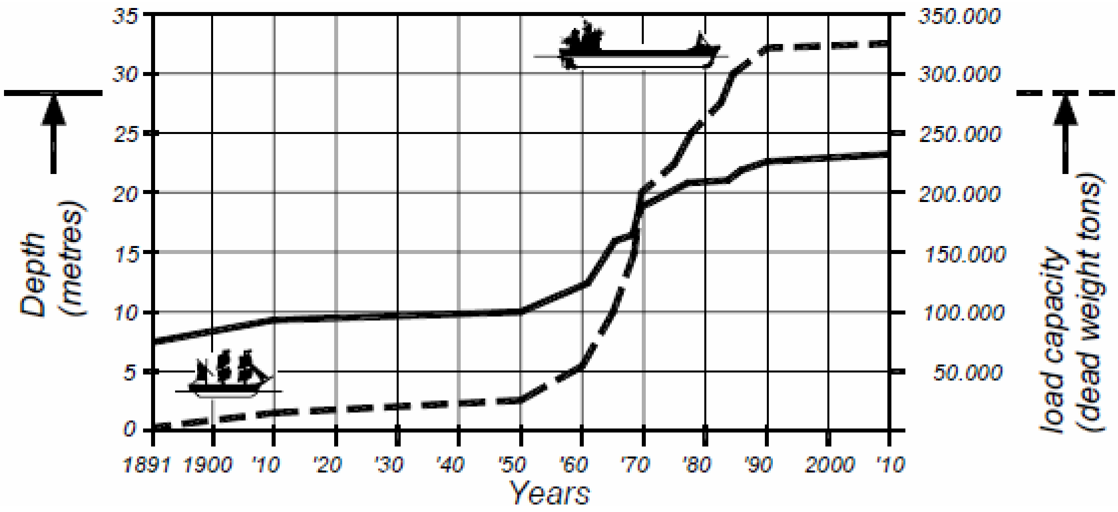


Figure 2-1 Development of ship dimensions over time (Tol & Gijt, 1999)

The design of the harbour is adapted to the growth of the ships. The draught of the ship determines the required depth of the quay wall. Other determining factors are the development in cargo handling facilities and the type and amount of surcharge on and behind the quay wall.

Due to the beneficial location of the port and the expansion possibilities, Rotterdam became the largest port in the world. The wet bulk goods are the most important cargo transports, but the container transport is large as well. Nowadays some ports in Asia are grown larger, but Rotterdam still is the largest port of Europe and a key player in the world. (CUR 211, 2003)

2.2.2 History of quay wall structures in Rotterdam

The Handbook Quay Walls (2003) describes the development of quay walls in Rotterdam. This analysis is briefly summarized here to understand the Dutch practice of quay wall design.

The oldest quay walls in Rotterdam date from 17th century. These quay walls were most often masonry walls standing on some wooden elements. The subsoil consisted mostly of dredged soil from the harbour basins, mainly weak clay. Problems with settlements and bearing capacity were common. The maximum retaining height in this time was 2.5 m which forced to moore the ships at some distance from the quay. Halfway the 19th century the first quay wall on piles was built in order to enable mooring directly to the quay.

The ships grew larger in the end of the 19th century and therefore the stability of the walls became problematic. One of the major progresses was the option of soil improvement. The weak layers were dredged away and replaced by soil with more bearing capacity. In the first years of the 20th century new materials were used to resist the impact of wet-dry-variations. Wooden elements were replaced by concrete elements. Furthermore, piles were replaced by concrete caisson-type structures, basically a shallow foundation. The caisson structure was the first prefabricated quay wall. Soil improvement was to a large extent required. The caisson wall was popular till the sixties, because the structure is structurally redundant and relatively cheap.

Alternative solutions were developed to avoid the high costs of soil improvements. An important milestone was the invention of steel sheet-piles, first applied in 1927. After second world-war it was clear that steel and concrete offer many opportunities. A structure with a system of tension and compression piles was designed to take the horizontal load. The superstructure is supported by the piles and is hollow in order to reduce the weight. An important development in soil conditions was the reduction of excessive water pressures by using vertical sand drains. This decreases the construction time.

Another aspect is the eccentrical hinge connection between the superstructure and the sheet-pile, which considerably reduces the bending moment in the sheet-pile. Also the floor was placed lower (as a relieving floor) in order to reduce the active soil pressure on the sheet-pile.

The decision to grant access to ships with a draught up to 24 m required more development. The relieving floor was placed lower till NAP -6.00 m, which reduced the height of the sheet-pile. Further optimization was done by introducing the combined sheet-pile consisting of Larssen hollow piles and three sheet-piles in between. They experienced that a dense pile system acts as a second sheet-pile screen. Therefore a system was developed with low stiffness and as open as possible. As tension pile the M.V.-pile (Müller Verfahren) was developed, consisting of a grouted H-profile of steel with a thickened end.

It can be concluded that the developments led to the quay wall design with combi-wall as shown in Figure 2-2. Since 1986, this type has been applied in the Rotterdam harbour. The basic structure consists of a concrete superstructure, supported by compression and tension piles and an inclined combi-wall. The relieving floor is deep and the steel tension piles are of the M.V. type and placed under an angle of 45 degrees. The required anchor force is reduced, because both the sheet-pile and the foundation piles are placed under an angle. The connection between combi-wall and superstructure is an eccentrically placed hinge. This basic structure should be adapted to account for local circumstances and specific requirements.

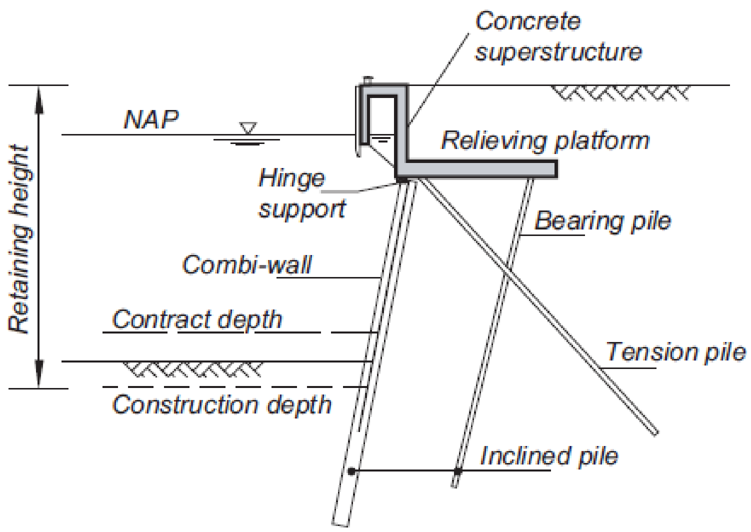


Figure 2-2 Quay wall as result of improvements in time (CUR 211, 2003)

2.2.3 Types of quay wall structures

Some types of quay wall structures have already been mentioned in the previous paragraph, but a general overview of the classification is presented here. Basically, there are four main types (CUR 211, 2003). They are briefly described here.

2.2.3.1 Gravity walls

A gravity wall performs its retaining function by it is own weight. The principle is to realize enough shear resistance in the soil to protect the structure from sliding or overturning. Due to the relative large self weight the soil bearing capacity should be relative large as well. Examples of gravity structures are the block wall, L-wall and caisson.

2.2.3.2 Sheet-pile walls

The sheet-pile can be constructed of timber, concrete, steel or plastics. Steel sheet-piles are most suitable in case of large retaining heights. The single, double or triple sheet-pile profiles are connected to each other. In case of larger retaining heights, combi-walls can be used consisting of heavy primary elements and secondary sheet-pile profiles in between. Other options for sheet-pile structures are diaphragm walls or cofferdams.

An important aspect of sheet-pile walls is the anchorage system. The options for this system are horizontal anchors, anchors with grout bodies and tension piles.

2.2.3.3 Structures with relieving platforms

This is the system explained in paragraph 2.2.2 and visualised in Figure 2-2.

2.2.3.4 Open berth quays

This structure type is different from the other types, because it does not use a vertical element to close the difference in height, but an inclined slope.

2.2.4 Design guidelines in Rotterdam

During the years the experience in quay wall design has been fixed in guidelines. These guidelines are developed and still several different guidelines are used for quay wall design in Rotterdam.

2.2.4.1 EAU

The design guidelines developed as EAU's (Empfehlungen des Arbeitsausschusses Ufereinfassungen) are based on the hisotrical design experiences. These are in fact 'practically proven' design codes. The EAU 1990 is based on a deterministic approach. The design needs to be tested on three defined load situations with different frequencies of occurrence using design values of the soil parameters. EAU 1996 uses a more fundamental semi-probabilistic safety approach in order to be compatible with Eurocode 7. The same load situations are used with different partial factors. The EAU 1990 is still very often applied, also in Rotterdam. The EAU 1996 is used seldom as the semi-probabilistic approach is developed in other guidelines.

2.2.4.2 CUR 166

The design guideline for sheet-pile structures is often applied in the Netherlands. It also follows a semi-probabilistic approach and uses design values for the soil parameters (in chapter 3, but not in chapter 4 on FEM). The partial factors are applied on material properties, geometrical variables and loads and depend on the safety class. The guideline corresponds to the Dutch standard NEN 6700, 6702, 6740 and 6743. CUR 166 is specifically meant for the design of sheet-pile structures.

2.2.4.3 CUR 211

The design guideline for quay walls is specifically used for quay walls with relieving floor. The approach is comparable to the approach of CUR 166, but in CUR 211 characteristic values of the soil parameters are used.

2.2.4.4 Eurocode (EC)

EC 7 treats the design aspects of retaining structures. The rules are based on a semi-probabilistic safety approach. In principal, it corresponds to the NEN-standards, but the safety of the structure needs to be assessed by using one of the three predefined approaches. The determination of the design values of soil pressures is different for each approach. The national annex decides which approach should be used.

2.3 Soil stress, deformations and stiffness

As the relation between soil stress, deformations and stiffness is complex, this section is dedicated to describe some aspects of these interactions. The final part of the section (2.3.5 and 2.3.6) is about the internal equilibrium of a sheet-pile structure and a quay wall with relieving floor. Note that the references in this section are mostly secondair, the primary sources are only metioned through the names of the founders of certain theories.

2.3.1 Horizontal soil pressure

When the soil is not forced to move, the horizontal soil pressure can be described by multiplying the vertical effective stress by the neutral earth pressure coefficient, described by:

$$K_0 = \frac{\sigma_h^{'}}{\sigma_v^{'}} = 1 - \sin(\varphi) OCR^{\sin(\varphi)}$$

In which σ'_h is the effective horizontal soil pressure, σ'_v the effective vertical soil pressure and φ is the internal angle of friction of the soil. OCR is the over consolidation rate, which is defined as the largest vertical stress ever in the soil, divided by the actual vertical stress. A high cone-resistance in a Cone Penetration Test (CPT) can therefore imply a normal consolidated well packed soil or a strong over consolidated limited packed soil. (Tol & Everts, 2006)

The stress situation strongly depends on the deformations in the soil. The limit states of the soil stress are the active and passive stresses. The value of the active and passive soil pressure coefficients can be determined in several ways. Rankine proposed a solution with straight slip planes (Verruijt, 2010):

$$K_a = \frac{1-\sin(\varphi)}{1+\sin(\varphi)} \text{ and } K_p = \frac{1+\sin(\varphi)}{1-\sin(\varphi)}$$

In theory straight slip planes only occur when wall friction is not relevant.

Müller-Breslau proposed a general solution for the horizontal soil pressure coefficients for soils without cohesion and straight slip planes with an angle in homogenous soil (including the wall friction angle (δ)) (Deltares, 2010):

$$K_a = \frac{\cos^2(\varphi)}{(1+\sqrt{\frac{\sin(\varphi)\sin(\varphi+\delta)}{\cos(\delta)}})^2} \text{ and } K_p = \frac{\cos^2(\varphi)}{(1-\sqrt{\frac{\sin(\varphi)\sin(\varphi+\delta)}{\cos(\delta)}})^2}$$

Kötter proposed solutions with curved slip planes, which is often a better description of reality. These equations assume an unloaded horizontal soil surface, homogeneous soil with a volumetric weight of zero and a slip plane consisting of a logarithmic spiral and straight part (Deltares, 2010):

$$K_a = \frac{1-\sin(\varphi)\sin(2\alpha+\varphi)}{1+\sin(\varphi)} \exp\left\{(-\frac{\pi}{2}+\varphi+2\alpha)\tan(\varphi)\right\}$$

with α: $\cos(2\alpha+\varphi-\delta) = \frac{\sin(\delta)}{\sin(\varphi)}$

and $K_p = \frac{1+\sin(\varphi)\sin(2\alpha'-\varphi)}{1-\sin(\varphi)} \exp\left\{(\frac{\pi}{2}+\varphi-2\alpha')\tan(\varphi)\right\}$

with α’: $\cos(2\alpha'-\varphi+\delta) = \frac{\sin(\delta)}{\sin(\varphi)}$

2.3.2 Deformations and stiffness

On one hand stresses are thus caused by deformations, but on the other hand deformations are caused by changes in stresses. Both statements are true as there is interaction between the two. Increase of the stress causes compression of the soil and decrease of stress causes the soil to swell. The compression velocity is slowed down by the pore pressure, which implies consolidation in time. Compression of the soil consists of elastic deformations of the soil skeleton and plastic deformation by change in the grain structure. (Tol & Everts, 2006)

In general, in case of small loads, deformations in the soil (especially sand) are relatively small and more or less elastic. At increasing loads the deformations not only increase, but they become plastic as well. The deformations of grain type material are determined by the change in grain stress. In a contact point between two grains normal forces and shear forces can occur. The normal forces introduce a depression of the grain. These deformations are in general very small compared to the deformations caused by the sliding of the grains with respect to each other.

When the material is compressed by normal forces a decrease in volume of the material occurs, but the form of the element is mainly unchanged. Due to irregularities in the packing of the grains, shear forces develop, which cause the grains to disconnect. The shear forces cause sliding of the grains and this sliding causes the number of points of contact between the grains to increase and therefore a denser material is created. This phenomenon causes the soil to behave stiffer under compression. When the grains are relieved the grain package will almost elastically bounce back without too much remaining deformations. When reloading again, less contact points between the grains are broken and therefore the material will react stiffer than during the first loading. This is noticed when looking at an over consolidated soil. (CROW, 2004)

2.3.3 Mobilized shear resistance

When describing the interactive process between deformation and soil stress, shear resistance is also an important issue. Shear resistance, the resistance against movement due to shear, is increasingly mobilized when deformations increase. When mobilization is maximal the horizontal earth pressure can be defined by multiplying the vertical effective stress by the horizontal earth pressure coefficients K_a and K_p as defined in section 2.3.1. These coefficients are only reached when the deformations are large enough. However, in pure compression, this mobilization does not occur. In case of a relieving floor the superstructure causes no pure compression of the soil, but the mobilization degree of the shear resistance can be considerably influenced. The shear resistance that counteracts deformations depends directly on the stress (and therefore the deformations) in the soil:

$\tau = c + \sigma \tan(\varphi)$ (Verruijt, 2010)

In layered soil packages the stiff layers will first reach the maximum shear resistance (stiff layers needs the least deformation to mobilize shear resistance). The deformations corresponding to this resistance are not large enough to develop the maximum shear resistance in the weaker clay and peat layers. When the maximum in these weak layers is reached, the shear resistance in the stiff layers along the slip plane is reduced by even larger deformations. This implies that in a layered soil package the maximum mobilized shear resistance in the entire slip plane is always smaller than the sum of the maximum shear resistances in the separate layers. Furthermore the slip plane will occur along the path of the least resistance. (Tol & Everts, 2006)

From this it is clear a sheet-pile structure needs to have deformation capacity in order to develop shear resistance. As a quay wall structure is often stiffer than a sheet-pile it is possible that not enough plasticity in the soil develops and therefore the interactive behaviour of the soil and retaining wall not fully develops. The deformation, required to develop the required shear resistance could be too small. In order to fully mobilize the shear resistance, larger deformations are needed and therefore a larger deformation capacity of the retaining wall is required.

This mobilization process of the shear resistance in passive and active wedge shows the interactive behaviour between soil and sheet-pile. Different methods are discussed in section 2.4. Method Blum simply ignores the interactive behaviour as fully mobilized passive and active shear resistance is assumed. MSheet and PLAXIS give more possibilities to describe this behaviour, especially the Hardening Soil model in PLAXIS.

2.3.4 SLS and ULS

Distinction should be made between the situation in Serviceability Limit State (SLS) and Ultimate Limit State (ULS). The SLS refers to the required state of the structure to perform its normal functions. For sheet-piles and quay walls this is related to the maximum deformations when the structure is used. ULS refers to failure or collapse. In general sheet-pile structures are designed according to a ULS and it is assumed that the SLS is implicitly satisfied as well. Basically, it is assumed that the deformations will not exceed the maximum value set at SLS. This is reasonable for sheet-piles as the shear resistance is often fully mobilized with limited deformations.

However, in case of a quay wall structure the situation can be different. Following the discussion of section 2.3.3 it is possible that larger deformations (and deformation capacity) are needed to mobilize the shear resistance. These deformations can be governing for the design, because the quay wall should not deform too much in SLS. Therefore, it is important to check the maximum deformations as well and assure in the design that these situations will not occur. This discussion can be found as well in the comparison calculations of quay walls in MSheet and PLAXIS (Bakker & Jasper Focks, 2011). The authors advice the designer to be prudent when applying the partial safety factors of CUR 166 in a quay wall design. Applying this method probably leads to problems in SLS.

2.3.5 Internal equilibrium of sheet-pile

The internal equilibrium of a sheet-pile is related to the situation where the anchor connection with the sheet-pile moves. In that situation a deep slip plane may occur in between the sheet-piles lower rotation point and the anchorage part of the anchor (see Figure 2-3). When this rotational motion of the wall occurs an active slip plane develops above the anchor and behind the sheet-pile. Above the anchor a ‘fake’ vertical wall is assumed. The equilibrium of the sliding plane can be evaluated by making a section between the sheet-pile and the soil mass behind, crossing the deep slip plane en the vertical fake wall. The resulting grain stresses and the anchor force are implemented as external forces. The relevant loads are:

- Water pressure
- The weight of the soil ‘G’ above and below water
- The active pressures E_d and E_1, calculated by the vertical soil pressure and the coefficient for the active soil pressure.
- The surcharge loads q_1 and q_2 when they are unfavourable with respect to equilibrium. q_1 will be counted for via E_1 and q_2 only will be taken into account when angle β is larger than the angle of internal friction.
- Resistance along the deep slip plane Q. It is practical to assume a straight plane between rotation point of the wall and the heart of the anchorage part of the anchor. In reality the line will be more curved, which can be accounted for in computer calculations like MSheet or PLAXIS. The total resistance Q and the anchor force A_n must make equilibrium with the known forces from weight and active soil pressures
- The possible anchor force. The directions of A_n and Q are known, but their magnitude unknown.
- The rotation point of the sheet-pile is the lowest point of the sheet-pile (simple support) or the point of maximum bending moment in the soil (fixed support).

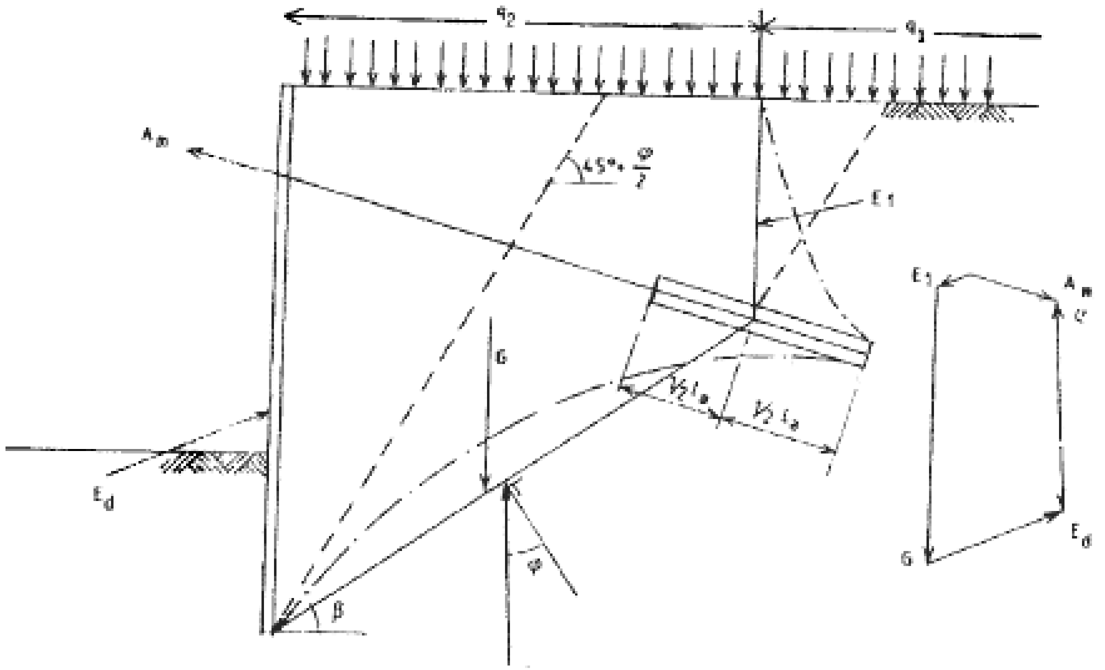


Figure 2-3 External forces on the soil element – Kranz (Tol & Everts, 2006)

2.3.6 Internal equilibrium of quay wall with relieving floor

If a relieving floor is applied, the active soil pressures on the upper part of the sheet-pile are reduced. This results in reduction of the bending moment in the wall and a higher bottom level of the wall. Figure 2-4 shows the working of the relieving floor. The influence of the surcharge load σ'_{k0} on the vertical soil stress at the axis of the sheet-pile is shown. The area of influence starts where the line under an angle φ intersects the axis of the sheet-pile. The influence is complete from the point where the composed line from the different active slip plane angles θ_a intersects the axis of the sheet-pile. θ_a depends on the angle of internal friction φ, the angle of wall friction δ, the slope of the ground level β and the inclination of the sheet-pile α.

$$\tan\theta_a = \frac{1 + \dfrac{1}{\cos\alpha}\sqrt{\dfrac{\sin(\varphi+\delta)\cdot\cos(\alpha+\beta)}{\cos(\delta-\alpha)\cdot\sin(\varphi-\beta)}}\cdot\sin\varphi}{\tan\alpha + \dfrac{1}{\cos\alpha}\sqrt{\dfrac{\sin(\varphi+\delta)\cdot\cos(\alpha+\beta)}{\cos(\delta-\alpha)\cdot\sin(\varphi-\beta)}}\cdot\cos\varphi}$$

It should be checked whether the shear resistance in the soil body below the relieving floor is sufficiently developed. When a weak clay layer or a stiff sheet-pile is applied this development can be too small. The control can be done by looking at the horizontal equilibrium of the soil body above the clay layer and between the sheet-pile and the vertical behind the relieving floor. (CUR 211, 2003)

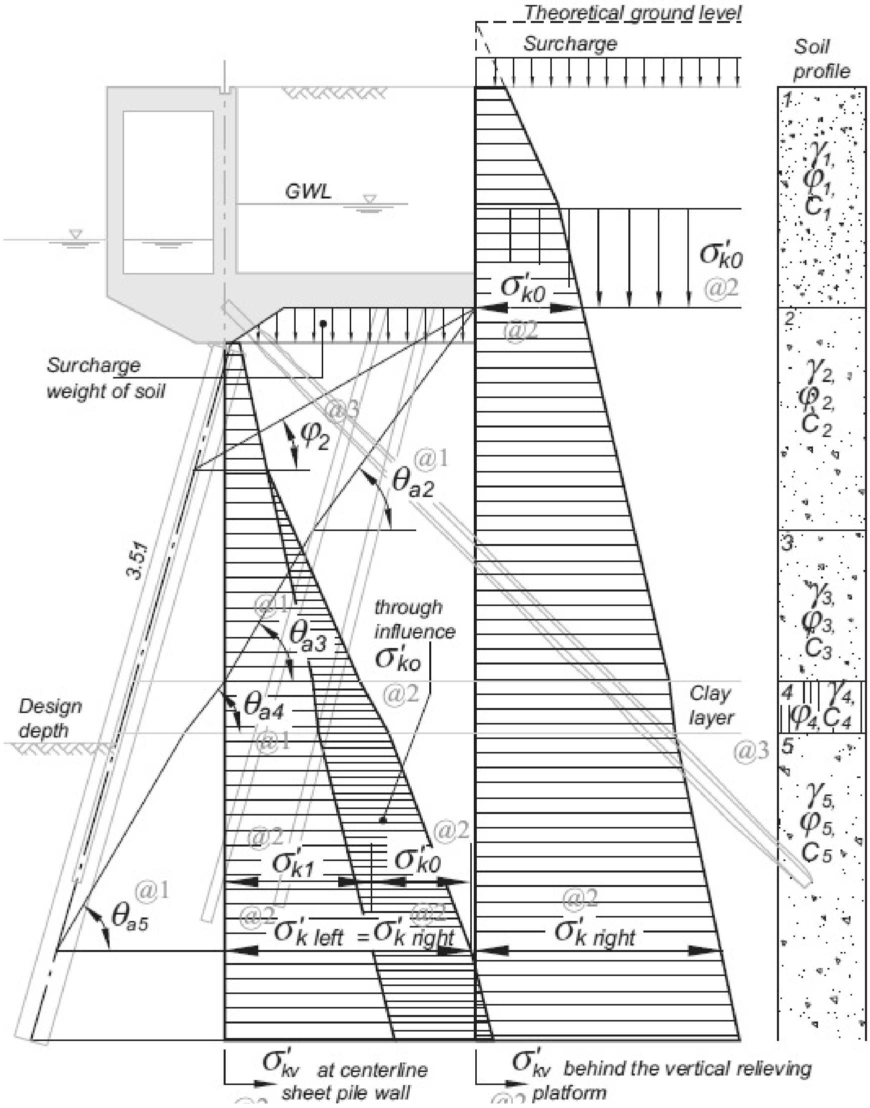


Figure 2-4 Working of a relieving floor (CUR 211, 2003)

In the calculation of a sheet-pile with a bearing function, the wall friction angle is often assumed to work favourable. When a sheet-pile settles due to high axial loads, the wall friction between wall and soil can change direction, causing an increase in active soil pressure. Therefore it needs to be checked whether the wall friction can be assumed to act favourable or not. To show this, Figure 2-5 presents the effect of the wall friction angle direction on the magnitude of the resultant active soil pressure. This is based on equilibrium of the active slip plane as shown before (Figure 2-3). (CUR 211, 2003)

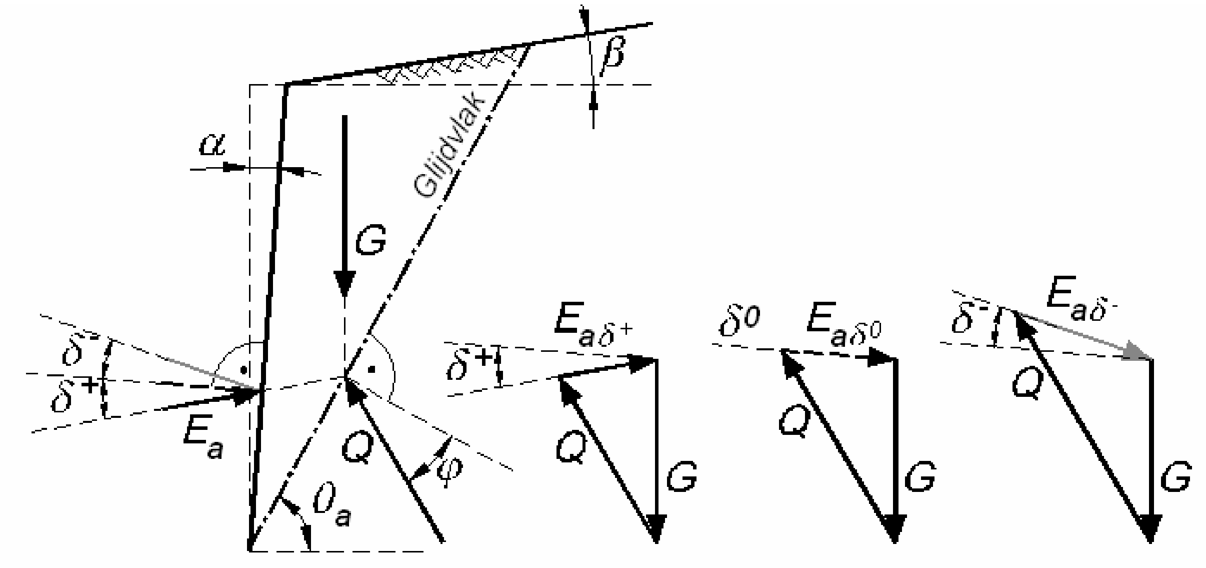


Figure 2-5 Effect of the direction of δ on E_a (glijdvlak = slip plane)

2.4 Design Models

Three basic design models for the design of sheet-pile structures and quay walls can be distinguished:

- Method of Blum
- Spring supported beam model
- Finite Element Method

These models are briefly described in this section. The description is mostly based on the information from the Handbook Quay Walls (CUR-publicatie, 2003).

2.4.1 Method of Blum

The classical design model is the method of Blum. The idea is to describe the sheet-pile as fixed, with a moment at the bottom of zero and a shear force at the bottom not equal to zero. This shear force is the resultant of the stresses below the fixing of the sheet-pile. The length of the sheet-pile is determined in a way that there is equilibrium between the active soil pressure on high side and passive soil pressure on the low side in such a way that the displacement at the anchor is zero. (Verruijt, 2010) The simplified situation is shown in Figure 2-6. To make analytical solutions possible the phreatic line is the same as the level of the ground.

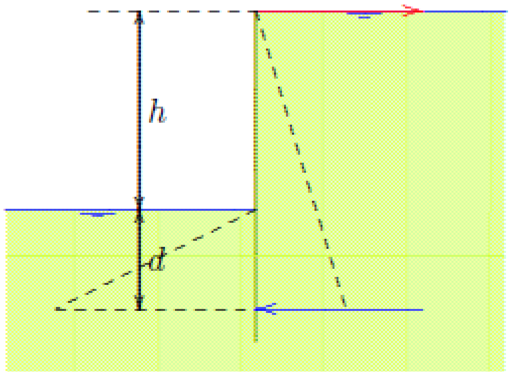


Figure 2-6 Active and passive soil pressure (Verruijt, 2010)

The method assumes the deformations in failure situation are large enough to develop maximum shear resistance. Therefore minimum active and maximum passive soil pressures can be used. The disadvantage is that the acting soil pressures can considerably deviate from the assumed values. This can result in too large fixing moments, too small field moments and too low anchor forces.

When more soil layers are present and water levels differ from ground level, solutions can be found in an iterative way, for instance in MatLab. Verruijt (2010) presents a simple computer program to implement this. The code can be found in Appendix F, because it is used later on as verification method in the analysis.

2.4.2 Spring supported beam model

This method uses elasto-plastic springs to schematize the soil. In case of sufficient deformations the plastic area of the soil spring is reached. Only in that case minimal active soil pressures and maximum passive soil pressures occur. When there is no deformation the soil pressure is neutral. Because of the interaction between soil and sheet-pile the calculations should be carried out iteratively. The available software (like MSheet) uses uncoupled springs, which means that arching of the soil is ignored. Different building and user stages can be defined. The history of the stresses is included in the next stages.

The software is one-dimensional. Two-dimensional superstructures can be implemented, but it takes some tricks to do so. I.e. the ground level can be adapted or an additional surcharge load can be applied to model the vertical load. Furthermore an external moment, on top of the sheet-pile can be used to describe the saddle structure of the superstructure. An example of MSheet input is shown in Figure 2-7.

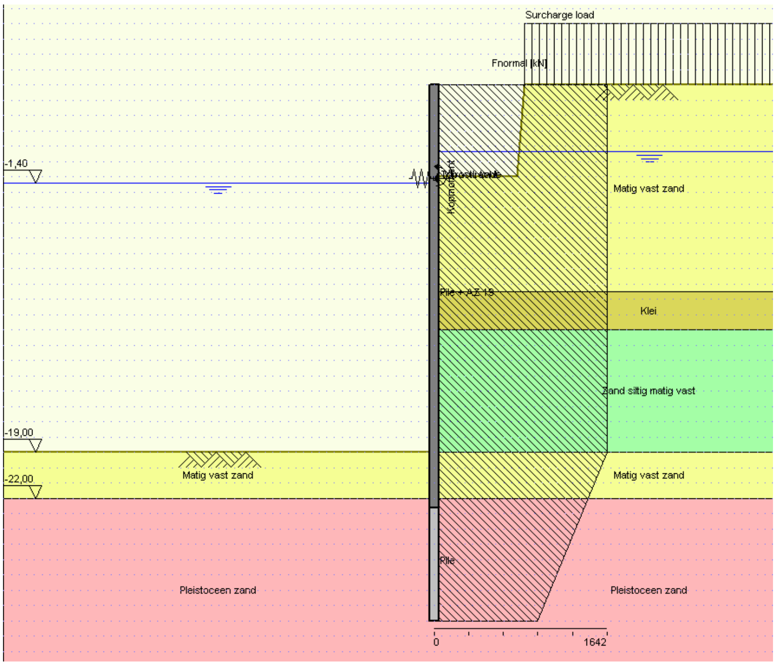


Figure 2-7 Spring-supported beam model MSheet

2.4.3 Finite Element Method (FEM)

This method allows integration of the behaviour of soil and structure. The material properties are implemented by the fundamental stress-strain relations. This method can be used to control the stability or deformations of a quay wall. Several other problems can also be analyzed with FEM:

- The horizontal deformations of the foundation in order to calculate bending moments in the piles and additional horizontal loads on the superstructure.
- Deformations of the superstructure in the different stages
- Vertical arching
- The control of the working of the relieving structure in case of a weak cohesive layer
- 3D problems

2.4.3.1 PLAXIS

One of the commonly used Finite Element software packages in geotechnical engineering is PLAXIS. *'PLAXIS is intended to provide a tool for practical analysis to be used by geotechnical engineers who are not necessarily numerical specialists'* (Brinkgreve & Broere, 2008). This paragraph describes the approach of PLAXIS and the required input to obtain the results. Furthermore the modules and soil models that are relevant for this research are discussed. These descriptions are based on the Manual of PLAXIS (Brinkgreve & Broere, 2008). PLAXIS is the main software used in this study, therefore it is described more extensively. PLAXIS 2D is used, but many parts of the explanation hold for PLAXIS 3D as well.

<u>Input</u>

In the input program the user must create a two-dimensional geometry model composed of points, lines and other components in the x- and y-plane. This also includes loads and prescribed displacements. The user needs to define all material properties of the different elements. Different soil behaviour models and their parameters can be selected, which will be discussed below. PLAXIS automatically generates an appropriate finite element mesh and boundary conditions. The user can add its preferences for better performance. Finally, the user needs to generate the water pressures and initial effective stresses. Also the active elements in initial stage can be selected here. An example of PLAXIS geometrical input is shown in Figure 2-8.

The behaviour of the soil can be modelled in several ways. The relevant models for this study are:

- Linear elastic model. This model represents Hooke's law of isotropic linear elasticity. The model involves two elastic parameters, i.e. Young's modulus (E) and Poisson's ratio (v). This model is primarily used for stiff structures in the soil
- Mohr-Coulomb (M-C) model. This is used as a first approximation of soil behaviour in general. The model involves the parameters: Young's modulus (E), Poisson's ratio (v), cohesion (C), internal friction angle (φ) and dilatancy angle (ψ).
- Hardening Soil (HS) model. This is an elasto-plastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. It also involves compression hardening to simulate irreversible compaction of the soil under primary compression. This model can be used to simulate behaviour of sands and gravel as well as softer soils. In addition to the M-C strength parameters, the HS model invloves three reference stiffness parameters for a given reference stress, i.e. the 50% secant stiffness (E_{50}), the oedometric stiffness (E_{oed}) and the unloading-reloading elasticity modulus (E_{ur}). These three parameters aim to describe the behaviour as discussed in section 2.3.2.

Furthermore the user should choose between three types of material behaviour:

- Drained behaviour. No excess pore pressures are generated. This is the case for dry soils and also for well drained soils due to a high permeability and/or a low rate of loading. This option is also suitable to simulate long-term soil behaviour.
- Undrained behaviour. This setting is used for a full development of excess pore pressures. The effective elastic parameters should be entered (E' and v'). In addition to the stiffness of the soil skeleton, PLAXIS automatically adds bulk stiffness for the water and distinguishes between effective stresses and excess pore pressures.
- Non-porous behaviour. Neither initial nor excess pore pressures are taken into account. This may be applied in the modelling of concrete or structural behaviour. This option is often used in the linear elastic model.

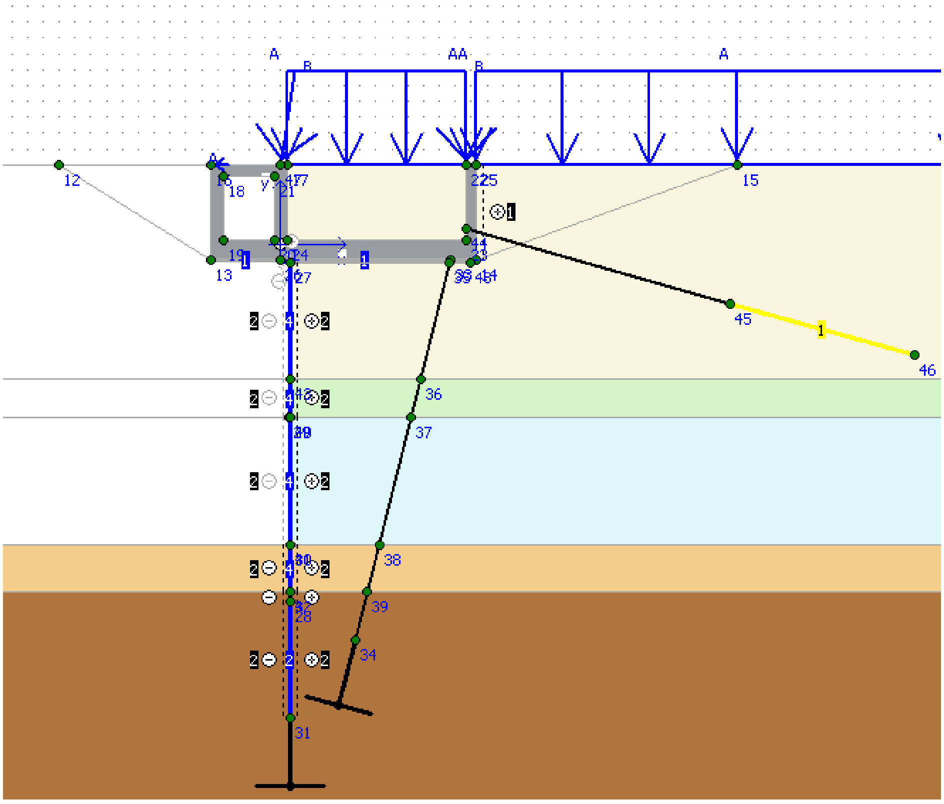


Figure 2-8 Example geometrical input PLAXIS

<u>Calculations</u>

In the calculation program the user can define the types of calculations to be performed and the types of loadings or construction stages to be activated during the calculations. The calculation program considers only deformation analysis and distinguishes between Plastic calculations, φ-c reduction, Consolidation analysis, and Dynamic calculations. The first two options are relevant for this study and explained more:

- A Plastic analysis is used to make elastic-plastic deformations analyses. The analysis does not take include the effect of decay of excess pore pressures in time. Small deformation theory is used and the stiffness matrix is based on the original undeformed geometry. This type of calculations is suitable for most practical geotechnical applications.
- A φ-C reduction is a safety analysis. Basically, the soil strength parameters (φ and C) of the soil are reduced stepwise. The method should be used when it is desired to calculate a global safety factor and soil mechanical failure is the only relevant mechanism. When using φ-C reduction in combination with advanced soil models (for example HS model) these models will actually behave as a Mohr-Coulomb model, because stress-dependent stiffness behaviour and hardening effects are excluded from the analysis. The total multiplier is used to define the value of the soil strength parameters at a given stage in the analysis:

$$\sum MSF = \frac{\tan \varphi_{input}}{\tan \varphi_{reduced}} = \frac{C_{input}}{C_{reduced}}$$

When a fully developed failure mechanism occurs, the factor of safety is given by:

$$SF = \frac{available\_strength}{strength\_at\_failure} = \frac{C_i + \sigma' \tan(\varphi_i)}{C_f + \sigma' \tan(\varphi_f)} = \sum MSF_{at\_failure}$$

In this thesis ΣMSF is referred to as MSF. When all stages are defined and the needed methods are selected, the calculations can be executed. PLAXIS automatically calculates all stages in the way the user has ordered and defined them.

<u>Output</u>

The Output program contains all facilities to view and list the results of generated input data and finite element calculations. The user has to select the model and an appropriate calculation phase or step number for which the results are to be viewed. Here the deformations, bending moments, shear forces, stresses, strains, etc. can be viewed per element, per soil cluster or of the total structure.

<u>Curves</u>

Finally there is a tool to create load-displacement curves and stress paths. These curves can be made in the points selected in the Calculation program.

3 Theoretical Framework: Reliability analysis and FEM

3.1 Introduction

This chapter introduces the three main concepts related to reliability analysis and the coupling with FEM:

- A limited overview of the available probabilistic methods (3.2)
- The development and (probabilistic) approaches of CUR 166 and CUR 211 (3.3)
- The development of probabilistic calculations with FEM (3.4)

3.2 Probabilistic methods

Originally, design was based on deterministic methods. The deterministic values of the parameters are in most cases conservative, based on experience. Deterministic methods do not make it possible to quantify the reliability of the structure. The reliability is based on experience.

In order to make the transition from design based on experience to a more scientific design, probabilistic design was developed, which takes the uncertainties in parameters into account. The calculations are done including specific parts (depending on the method) of the characteristics of the probability density function of a parameter, i.e. the shape of the distribution, mean value, standard deviation or higher order moments.

3.2.1 General

In the consideration of the reliability of an element, the central issue is the determination of the probability of failure. The reliability is the probability that failure does not occur. In order to describe this, a limit state (LS) is introduced, expressing the limit between failure and non-failure in terms of a reliability function:

$$Z = R - S$$

in which R is the resistance and S the load (solicitation). The probability of failure is the probability that the reliability function is smaller than zero.

$$P_f = P(Z \leq 0)$$

An analysis of the reliability in which the probability of failure of a structure is calculated is known as a structural reliability analysis.

The concepts of strength and loads can be described as random variables, because they are seldom exactly known. In structural domain it is common to handle a level-classification of calculation methods (CUR-publication 190 [CUR 190], 1997):

- Level III: These methods calculate the probability of failure by considering the probability density function of all strength and load variables. (Fully probabilistic)
- Level II: This level entails linearising the reliability in a design point. Furthermore, it approximates the probability distribution of each variable by a standard normal distribution. (Fully probabilistic with approximations)
- Level I: At this level no failure probabilities are calculated. It is a design method, which considers an element sufficiently reliable if a certain margin is present between the characteristic values of the strength and the loads. This margin is created by taking into account so-called partial safety factors in the design. (Semi-probabilistic)

The three levels are discussed here in more detail as well as some methods developed for the different levels. The descriptions are based on CUR 190 (1997).

3.2.2 Level III methods

The level III failure probability calculation is a mathematical formulation of the probability of failure, which involves failure (Z<0). When the joint probability density function ($f_{R,S}(R,S)$) of the strength and load is known, the probability of failure can be calculated by means of integration:

$$P_f = \iint_{Z<0} f_{R,S}(R,S)dRdS$$

This integral can be solved as a convolution-integral (under the information that Z<0 if R<S). Usually the strength and loads are functions of one or more random variables. Such that the reliability function can be written as:

$$Z = g(X_1, X_2, ..., X_n)$$

If the probability of failure needs to be found by integration, it seldom can be determined analytically. However, there are various numerical integration methods available.

A common method is the Monte Carlo (MC) analysis. This method is based on drawing random numbers (X_U) from a uniform probability density function between zero and one. The non-exceedance probability is in this case:

$$F_X(X) = X_U$$

And thus for the variable X:

$$X = F_X^{-1}(X_U)$$

If the inverse cummulatieve distribution function is known, a random number X can be generated from an arbitrary distribution $F_X(X)$ by drawing a number X_u from the uniform distribution between zero and one.

The same way base variables of a statistical vector can be drawn from a known joint probability density function. The joint probability distribution function must be formulated as the product of the conditional probability distributions of the base variables of the vector.

$$F_{\vec{X}}(\vec{X}) = F_{X1}(X_1) \cdot F_{X2|X1}(X_2 | X_1)...F_{Xm|X1,X2,...Xm-1}(X_m | X_1, X_2, ..., X_{m-1})$$

By taking m realisations of the uniform probability distribution between zero and one, a value can be determined for every variable X_i:

$$X_1 = F_{X1}^{-1}(X_{U1})$$

$$X_2 = F_{X2|X1}^{-1}(X_{U2} | X_1)$$

.

.

$$X_m = F_{Xm|X1,X2,...,Xm-1}^{-1}(X_{Um} | X_1, X_2, ..., X_{m-1})$$

The probability of failure is estimated by dividing the number of simulations for which Z<0 (n_f) by the total number of simulations (n):

$$P_f \approx \frac{n_f}{n}$$

The number of simulations to get a conficence of for instance 95% can be derived and is still dependent on the probability of failure:

$$n > 400(\frac{1}{P_f} - 1)$$

Systems with high target reliability require a large number of calculations. To reduce this number advanced methods are often used to estimate the probability distribution of the reliability function. Examples are Stratified Sampling, Quasi-Random Sampling and Importance Sampling. Schweckendiek (2006) gives a clear overview of these and other level III methods.

3.2.3 Level II methods

In level II methods a distinction can be made between linear and non-linear reliability functions. Basically the methods described are the First Order Second Moment (FOSM) method and the First Order Reliability Method (FORM). For other methods as Second Order Reliability Method (SORM) or Point Estimate Method (PEM) reference is made to Schweckendiek (2006).

3.2.3.1 First Order Second Moment (FOSM) Method

The first order approximation to the mean, variance and standard deviation of a function F, based on the first terms of a Taylor series expansion of Z, can be described. With uncorrelated variables the expressions are (Beacher and Christian, 2003):

$$E[Z] = \mu_Z \approx Z(X_1, X_2, ..., X_n)$$

$$Var[Z] = \sigma_Z^2 \approx \sum_{i=1}^{n}\sum_{j=1}^{n} \frac{\partial Z}{\partial x_i}\frac{\partial Z}{\partial x_j}\rho_{X_i X_j}\sigma_{X_i}\sigma_{X_j}$$

with $\rho_{Xi,Xj}$ is the correlation coefficient between X_i and X_j.

The reliability index can be calculated in six steps (Baecher & Christian, 2003):

1. Identify all variables that affect the mechanism that is researched.
2. Determine the best estimate (usually the mean value) of each variable, $E[X_i]$ and use these to calculate the best estimate of the function, E[Z].
3. Estimate the uncertainty in each variable and, in particular, its variance, $Var[X_i]$.
4. Perform sensitivity analyses by calculating the partial derivatives of Z with respect to each of the uncertain variables or by approximating each derivative by the divided difference $\Delta Z/\Delta X_{i.}$
5. Use the equation of Var[Z] to obtain the variance of the function Z.
6. Calculate the reliability index $\beta = \dfrac{E[Z]}{\sigma_Z}$.

Basically, the reliability index is obtained by approximating the derivative of the function with respect to certain variable by calculating two function values for two different values of the variable close to each other.

3.2.3.2 Hasofer-Lind Approach (First Order Reliability Method)

One assumption of FOSM is that the failure criterion can be estimated accurately enough by starting with the mean values of the variables and linear extrapolation. There is an implicit assumption it makes little difference where the partial derivatives are evaluated. A second assumption is that the form of the distribution of Z is known. In practice these assumptions are seldom valid. Hasofer and Lind proposed a different definition of the reliability index that leads to a geometric interpretation, also known as First Order Reliability Method (FORM). (Baecher & Christian, 2003)
The method is explained for linear and non-linear reliability functions.

<u>Linear reliability functions</u>

If the reliability function is linear and the covariance between the variables is known, the expected value and standard deviation of this function can be determined with:

$$Z = a_1 X_1 + a_2 X_2 + ... + a_n X_1 + b$$

$$\mu_Z = a_1 \mu_{X1} + a_2 \mu_{X2} + ... + a_n \mu_{Xn} + b$$

$$\sigma_z = \sqrt{\sum_{i=1}^{n}\sum_{j=1}^{n} a_i a_j Cov(X_i, X_j)}$$

If the base variables X_1, X_2,…, X_n are normally distributed, Z is also normally distributed. The mean and standard deviation have to be transformed from their physical space into variables in the standard normal space with μ_U=0 and σ_U=1. A deterministic value x_i, physically in normal space, is transformed into standard normal space by:

$$u_i = \frac{x_i - \mu_{xi}}{\sigma_{xi}}$$

However, the original distribution is not necessarily normal. Therefore, these variables first have to be transformed into their normal equivalents. Several transformation methods are available to do this (Hicks et all, 2011), not described here.

The probability that Z<0 can be determined using the standard normal cumulative distribution function is:

$$P(Z<0) = \Phi(-\frac{\mu_z}{\sigma_z}) = \Phi(-\beta)$$

The reliability index β is the distance from the origin to the failure space, which is described by a linear reliability function. The point with the closest distance to the origin is the point on the edge of the failure area with greatest joint probability density and therefore it is by definition the design point (Figure 3-1). It can be shown that the design point is given by:

$$R^* = \mu_R + \alpha_1 \beta \sigma_R$$

$$S^* = \mu_S + \alpha_2 \beta \sigma_S$$

If the reliability function is linear the influence coefficients α_i are than given by:

$$\alpha_i = \frac{-a_i \sigma_{xi}}{\sigma_z}$$

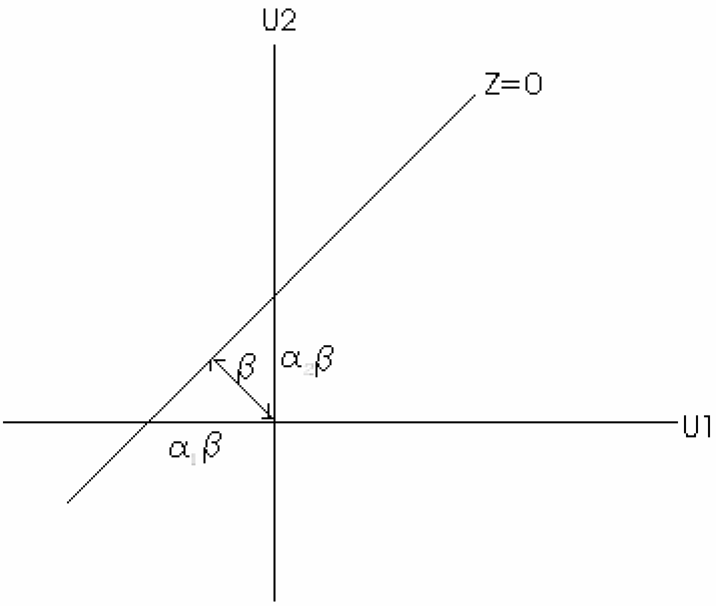


Figure 3-1 Design point and reliability index

Non-linear reliability functions

If the reliability function is a non-linear function, the function can be approximated by the first two terms of the Taylor-polynomial:

$$Z \approx g(\vec{X}_0) + \sum_{i=1}^{n} \frac{\partial g}{\partial X_i}(\vec{X}_0)(X_i - X_{0i})$$

This approximation is linear and normally distributed (Central Limit Theorem). The expected value can be approximated by:

$$\mu_Z \approx g(\vec{X}_0) + \sum_{i=1}^{n} \frac{\partial g}{\partial X_i}(\vec{X}_0)(\mu_{Xi} - X_{0i})$$

And the standard deviation:

$$\sigma_Z \approx \sqrt{\sum_{i=1}^{n} (\frac{\partial g}{\partial X_i}(\vec{X}_0)\sigma_{Xi})^2}$$

And the reliability index can be approximated by:

$$\beta = \frac{\mu_z}{\sigma_z}$$

If this reliability index needs to be found, linearization of the Z-function in a point can be applied. However, linearization in different points leads to different values for the approximation of the reliability index. The lowest value of β (Hasofer and Lind) for two variables is:

$\beta = \min_{Z=0}(\sqrt{U_1^2 + U_2^2})$ (U is the standard normal space)

The expression in case of more variables is: $\beta = \min_{Z=0}(\sqrt{\|\underline{U}\|}$

Looking for the design point is basically an optimisation problem. To find the design point many analytical and numerical approaches can be used. A relatively straightforward method to do this is by first guessing the design point to be the mean value. The obtained β-value is used to determine a new point, in which the reliability function is linearized. In this case the α_i-values are calculated with:

$$\alpha_i = \frac{\frac{\partial}{\partial X_i} g(\vec{X}^*)\sigma_{xi}}{\sqrt{\sum_{j=1}^{n} (\frac{\partial}{\partial X_j} g(\vec{X}^*)\sigma_{xj})^2}}$$

The new calculation point is determined by:

$$X_i^* = \mu_i - \alpha_i \beta \sigma_{xi}$$

After some iterations the design point is found and therewith the reliability index. In Appendix A a simple example with FORM is presented.

3.2.4 Level I methods

The essence of this method is the application of partial safety factors. In principal guidelines for structural design implement this method. A characteristic value of the strength (R_k) is divided by a factor (γ_R) and the characteristic value of the load (S_k) is multiplied by a factor (γ_S). The following criterion should hold:

$$\frac{R_k}{\gamma_R} > \gamma_S S_k$$

It is realistic to assume that for failure the values of the strength and the loads are close to the values of the design point. This results in expressions for the partial safety factors:

$$\gamma_R = \frac{R_k}{R^*} = \frac{1+k_R V_R}{1+\alpha_R \beta V_R}$$

$$\gamma_S = \frac{S^*}{S_k} = \frac{1+\alpha_S \beta V_S}{1+k_S V_S}$$

In which k=1.64 for a 5% reliability (see section 4.5). V is the coefficient of variation defined as $V = \frac{\sigma}{\mu}$

3.3 Approaches and development of CUR 166 and CUR 211

This section describes the development of CUR 166 and CUR 211. There is a history of improvement of the handbooks and the current update of CUR 211 is part of this. Furthermore, the differences between the approaches are analyzed.

3.3.1 Partial safety factors in CUR 166

One of the first documents that forms the base of the design philosophy in the first CUR 166 edition (1993) is the report 'veiligheid van damwandconstructies' (Calle & Spierenburg, 1991). Important in this document is the introduction of the fault tree for sheet-pile structures. The document describes that often there is a strong, but not full dependency between the probabilities of failure of the different mechanisms. The largest probability is fully counted and the other probabilities are counted half.

$$P(f_1 \cup f_2 \cup ....f_n) = \max(P(f_1), P(f_2),....P(f_n)) + \frac{1}{2}\sum_{i=1}^{N}(P(f_i) - \max(P(f_1)....P(f_n)))$$

The elements are working as a series system and therefore connected by OR-ports (i.e. when one element fails, the entire structure fails).

The three main failure modes were given an acceptable probability of failure 'p'. These are the mechanisms:

- Passive soil pressure is not enough to stabilize the sheet-pile
- Failure of the sheet-pile profile
- Failure of the anchor

For other mechanisms a lower acceptable failure probability (0.2p) was taken into account, as the safety with respect to these mechanisms is easier to improve. These are the mechanisms:

- Failure of the waling
- Failure of the anchor bar
- Kranz-failure

Overall instability and groundwater flow were assigned a higher failure space as well (p). The required overall target reliability index ($\beta_T = -\Phi(P_f)$ was defined by safety classes:

- Class I: simple structures, low risks (β=2.75)
- Class II: small economical risk, small probability of fatalities (β=3.75)
- Class III: large economical risk, relative large probability of fatalities (β=4.50)

The study in 'veiligheid van damwandconstructies' (1991) is limited to the mechanisms of soil mechanical failure, yielding of the sheet-pile profile and failure of the anchor.

The LS is found when the soil parameters C (cohesion) and φ (internal friction) are just able to maintain equilibrium (Z=0), when the Mohr-Coulomb model is applied. A level II analysis was carried out in DAMWAND (a spring model, see section 2.4.2) to obtain the influence factors (α) and reliability index (β). These values were used to define partial safety factors according to the level I method. Deterministic calculations in the design point of soil mechanical failure resulted in the design bending moment and the design anchor force. The margin between the reduced moment capacity/anchor capacity and the design values was translated to partial safety factors for the bending moment in the sheet-pile and the anchor force.

Safety margins for geometry parameters are different as the margin depends on the reference level. Therefore the partial safety factor for geometry parameters is written as:

$$\Delta_i = -\alpha_i \beta \sigma_i$$

The partial safety factors that are found in Calle & Spierenburg's research (1991) can be found in the newest version of CUR 166 as well (CUR 166, 2005, p.44). Havinga (2004) performed a study using MSheet-FORM to make probabilistic calculations to check whether the implementation of the partial safety factors lead to an acceptable reliability index. He concluded that in most cases the deviation from the required reliability is small and that therefore adaptation of the safety factors is not necessary.

3.3.2 Adjustments partial safety factors for CUR 211

Due to the differences between sheet-pile structures and quay walls Gemeentewerken Rotterdam (Huijzer, 1996) did research to adapt the results of CUR 166 to be applicable to quay walls.

First, the fault tree was evaluated and adapted according to the specific Rotterdam quay walls. Also the target reliabilities per failure mode over the variety of mechanisms were reviewed. The same approach as in 'veiligheid van damwandconstructies' was used as the partial safety factors are first calculated for the soil mechanical failure mechanism. Additional factors were applied on the other mechanisms. Alternatively it was checked whether it is better to use the failure mechanism yielding of the sheet-pile as main failure mode. It turned out to be the case and therefore this mechanism was used to derive partial safety factors.

The added mechanisms in the fault tree were:

- Failure of the anchorage connection, caused by failure of the steel or concrete
- Soil mechanical failure of the anchor, due to limited shear stress along the anchor or Kranz-stability

The adjustments in the traget reliabilities per failure mode were:

- Insufficient passive soil resistance reduced to 0.1p, because there is dealt with a clamped wall
- Anchorage connection failure gets 0.2p which is equally distributed over the steel and concrete
- Anchor failure still has 1.1p, although the soil mechanical failure is added. This is because either the anchor or the M.V. tension pile fails and not both

The final version of the fault tree is presented in Figure 3-2. This tree also includes failure of the superstructure and bearing piles, added later in CUR 211 (2003)

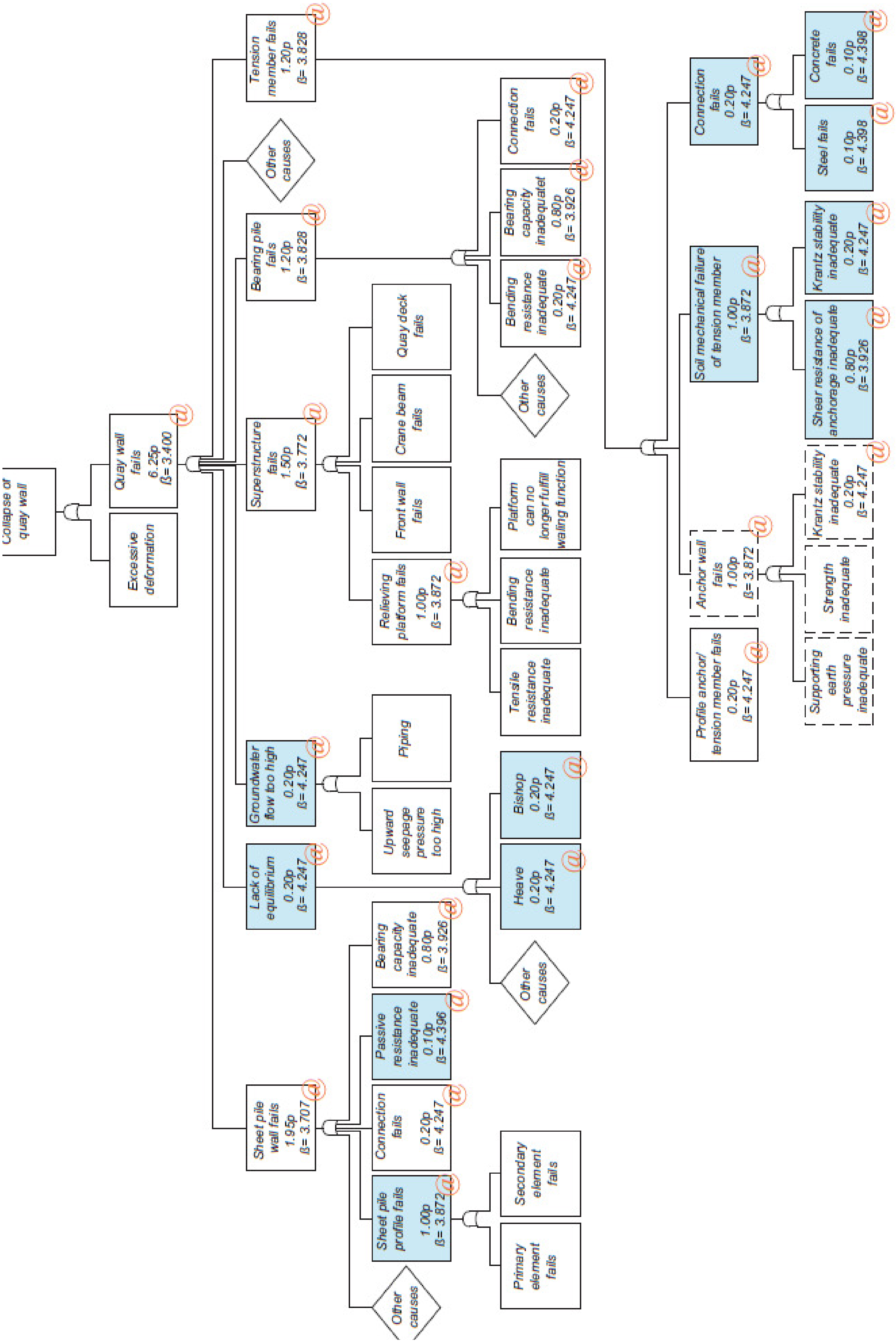


Figure 3-2 Fault tree of quay wall (CUR 211, 2003)

Grave (2002) performed a more detailed study to validate the results of the analysis. He tested four existing structures with the new partial safety factors to check whether the dimensions correspond to those obtained with the old method. He concluded that characteristic soil-parameter values can be used and that the new method can be applied to determine the wall thickness of the piles in the combi-walls in Rotterdam. Also for soil-mechanical strength of the anchor the new factors can be used. They are less conservative than the older overall factors.

Furthermore, Grave performed a probabilistic analysis on two of the redesigned structures, in order to see whether the desired safety level for the different failure mechanisms has been reached.
When he compared his achieved values of the reliability indices for the mechanisms with the standards in the fault tree, he showed that the safety-level comes close tot the desired standards. However, all research was restricted by many simplifications in the software (elasto-plastic spring model). He recommends to execute this research in a more accurate way, for instance with FEM calculations.

The obtained values are finally presented in CUR 211 (2003) and copied in Table 3-1 to Table 3-4. Note that Table 3-2 gives absolute factors as there is dealt with geometrical parameters that are case dependent. CUR 211 furthermore denotes that these absolute factors are added over nominal (or governing) geometric parameters. These are determined on the basis of expert insight and can be considered as representative values. Only if it can be shown that the change into design values will have only small effects on the structure the nominal value can be used. In all other cases the supplement Δ should be added. For the loads, CUR 211 gives a table that basically implies for safety class II a factor 1.2 on the permanent loads (0.9 when it works favorable) and 1.3 on the variable loads.

Parameter	Partial Safety factor (γ_m)	Parameter	Partial safety factor (γ_m)
γ*	1.00	k_h*	1.00
φ'	1.00	E*	1.00
c'	1.00	ν*	1.00
δ	1.00		
* Average values are used			

Table 3-1 Partial safety factors soil parameters (CUR 211, 2003)

Parameter	Partial Safety factor (Δ_{GE})
Bottom level	0.40
Ground water level	0.70
Water level (outside)	0.10

Table 3-2 Partial safety factors geometrical parameters (CUR 211, 2003)

Parameter	Partial Safety factor (γ_{GE})
Bending moment, Normal force, Shear force	1.30
Anchor force from sheet-pile calculation	1.20
Max. Pass. soil res. / Mob. Pass. soil res.	1.30

Table 3-3 Partial load factors on calculation results (CUR 211, 2003)

Failure mechanism anchorage or tensile element	Closing factors
Soil mechanical failure	1.00
Failure connection	1.50
Failure anchor	1.20

Table 3-4 Closing factors failure mechanisms (CUR 211, 2003)

3.3.3 Application of FEM

After the release of CUR 211 (2003) CUR 166 has been updated in 2005. As Havinga (2004) found out, the partial safety factors remained the same. New developments were added as well. One of the improvements is the addition of chapter 4 on application of FEM. It involves a test and design procedure of sheet-pile structures by using FEM. Two different calculation schemes are proposed:

- Calculation scheme A: Calculations with design values. The calculations are executed using the design values for the soil parameters, retaining height, water levels en stiffnesses of the structure. It uses two different soil stiffnesses. When using a high soil stiffness, the anchor force is generally relatively high. The advantage of this scheme is that it requires relative little effort. The disadvantage is that the deformations may be overestimated due to the use of design values in every stage. However, a designer is not interested in deformations when performing a ULS calculation.
- Calculation scheme B: Calculations with characteristic values. The calculations are executed using characteristic values of the soil strength (C' and φ'). Design values are used for the retaining height, water levels, external loads and stiffnesses of the soil. Here again two different soil stiffnesses are used. In the end of the governing stage, a φ-c reduction should be done (section 2.4.3.1). As the characteristic values for the soil parameters are used, the deformations are most probably smaller than in case of calculation scheme A. The safety margin lies in the fact that φ-C reduction (section 2.4.3.1) is imposed to obtain a safety factor of MSF=1.15 (class II) or MSF=1.2 (class III). This corresponds to the partial safety factors for φ and C in CUR 166 (table 3.7)

Due to the mentioned smaller displacements and the possibilities of a more economic design, scheme B is preferred according to CUR 166. On the contrary, it takes some additional effort to apply this scheme. (CUR 166, 2005)

3.3.4 Differences between CUR 211 and CUR 166

As already been discussed in the Problem Analysis (chapter 1) the approaches of CUR 211 and CUR 166 are slightly different. CUR 166 uses for the design rules in chapter 3 also partial safety factors on the soil parameters C and φ. However in chapter 4 it discusses that for FEM calculations it is better to use characteristic values of the soil parameters as 'reality' is described better in that case. CUR 211 uses the same reasoning to prescribe the characteristic values of the soil parameters. In this way it is for example avoided to take the wrong slip plane into account, argues CUR 211. The problem with this reasoning, however, is that there does not exist a 'wrong slip plane', because a design calculation is made in an unfavorable situation with a certain small probability of occurance. When this situation occurs, the calculated slip plane is correct.

Furthermore, CUR 211 uses average values of the specific weight of the soil, the subgradation constant, the soil stiffness and the Poisson ratio of the soil. This is based on the considerable structural differences between a quay wall with relieving floor and a sheet-pile structure. The relieving floor provides a better distribution capacity for the horizontal loads. To reach the required safety level, additional load factors are prescribed.

The new CUR 211 committee wants to follow the approach of CUR 166 chapter 4. Bakker en Jaspers Focks (2011) concluded that due to the higher stiffnesses of the quay wall in comparison to a lighter sheet-pile, the situation for quay walls with relieving floor is different. They advised not to use the partial safety factors of CUR 166 (table 3.7) as this leads to insufficient development of the plasticity in the subsoil. This can lead to insufficient development of the interactive behaviour between soil and retaining structure. A different procedure for quay walls should be developed, preferred to correspond to CUR 166 chapter 4.

3.4 Probabilistic calculations with FEM

Several attempts have been made in order to be able to make probabilistic calculations with FEM. This section describes some methods. PLAXIS-FORM is the method used in this study.

3.4.1 Method Bakker

H.L. Bakker performed a study and developed a model to make probabilistic calculations with Finite Element Methods (i.e. PLAXIS). This so-called Method Bakker has been applied a couple of times to calculate reliabilities of structures. The method uses rather complicated spreadsheets and is described in relative general terms in Appendix B as it is in principal not used in this research. The other methods appeared to provide more perspective.

3.4.2 PLAXIS-Prob2B

Schweckendiek (2006) made in cooperation with Courage (TNO) an attempt to asses the reliability of geotechnical structures (particular deep excavations). In order to do so he made a coupling between FEM software PLAXIS and reliability software Prob2B from TNO. Basically, this involves an interface between the two programs that is capable to exchange the data from the input and output parameters. A basic PLAXIS model has to be defined and Prob2B can read the input and output files of this model.

For every individual variable the mean value (μ_{Xi}), standard deviation (σ_{Xi}^2) and type of distribution of the probability density function ($f_{Xi}(x_i)$) should be defined. To create the joint probability density function $f_{\mathbf{X}}(\mathbf{X})$ cross-correlation coefficients ($\rho_{Xi,Xj}$) need to be provided. Finally, the user needs to define the reliability function per LS. Prob2B creates the calculation loop and evaluates the LS criteria based on the output from PLAXIS. In this way the probability of failure and reliability can be calculated by choosing a suited reliability method. Prob2B can therefore be used to couple PLAXIS with FORM.

3.4.3 PLAXIS-FORM

Hicks et all. (2011) worked on a module in PLAXIS to make it possible to carry out probabilistic calculations immediately in PLAXIS. The module uses FORM in combination with the Response Surface Method (RSM) (Lemaire et all, 2009). As input the basic PLAXIS set up is required, i.e. geometry, boundary conditions, material properties, etc. The package copies and modifies the project for the required number of permutations. A black box executes the PLAXIS calculation invoking the PLAXIS kernel for each of the directories. After the execution of the analysis the required results, presented as safety factors, are read back into the black box where it is used in the FORM analysis. In fact the PLAXIS-FORM implementation is a FORM-RS with quadratic Response Surface (RS). (Hicks, Nuttal & Arnold, 2011)

4 Starting Points

4.1 Introduction

This chapter contains the most important starting points of the study. The first section includes a view on the philosophical aspects of modelling followed by the starting points for the used model and analysis method, (Prob2B) in order to make FORM calculations in PLAXIS. Thereafter the fault tree is discussed, which forms the base of quay wall design. This is followed by a discussion on characteristic and mean values and the reliability index. In section 4.7 a description of the model parameters and the way they can be determined is presented. This section also defines the coefficients of variation and correlations for the relevant parameters in this research. Finally, the Limit State Functions are presented and a short introduction in Prob2B is given.

4.2 Model vs. reality

It is important to be aware that every model is just a description of reality with a specific goal. This is clearly stated by Figure 4-1 inspired by the artist Réne Magritte. Magritte reminded the spectator that his 'realistic' drawing of a pipe itself is not reality, but refers to an idea originated in human mind. The same holds for quay walls.

Figure 4-1 Brammenterminal (Inventec, 2012)

The theory of ideas is founded by ancient philosopher Plato. He argues the world consists of matter and non-material elements like human mind. Some things however cannot be categorized in these two classes. For instance the number π. Where does it come from? It exists as it has been always describing the relation between the radius of a circle and its area. Or the idea of righteousness; almost every human being somehow tries to describe this idea, through words, but mostly through deeds.

Human beings find it easy to make an imagination of ideas in mind. π has been meditated over and over, it is used in many formulas and thousands of decimals are learned by heart to describe it. But humans cannot grasp the core of the idea as it really is. We all know π exists, but our descriptions are limited. The same holds for righteousness. Even language is already a limitation in the description of this idea.

Reality seems to be inconceivable. But how should we relate to these ideas being a part of reality? What can we say about reality while we always fall short in describing it? This asks for a certain trust in reality as we perceive it. This is not a simple trust based on certain feelings, but trust embedded in scientific effort. Not only trust embedded in mathematics, biology and technology, but also in philosophy and theology. Trying to improve the accuracy of descriptions and gathering and eliminating the uncertainties in these descriptions. Working on reduction of uncertainties and deepening the knowledge about life, its origin and the earth on which we are living and building.

As geotechnical and hydraulic engineering is part of our description of reality, the same reasoning as above holds for its attempts to describe reality. A geotechnical model like PLAXIS visualises a representation of soil behaviour. However, the soil does not exactly behave like the model predicts. The model of a quay wall is not equal to a quay wall.

On the other hand a research like this needs assumptions because we cannot say anything without adopting starting points. A model is functioning as intermediary to create a bridge between reality and a certain goal. However this model is not equal to reality. The purpose of this thesis is described in chapter 1. The intermediary to represent reality in the required form is the Finite Element software PLAXIS. This model is used with the purpose of describing behaviour of quay wall structures. During the study the shortcomings of the model will become clearer and it can be analyzed whether these simplifications of reality are thought to be important or less important for the accuracy of the reality description.

While researching this topic the ideas about reality should be hold in mind. Working on a model is not equal to revealing the truth, unfortunately, but it is a powerful tool to embed our trust in reality when we know its strengths and limitations.

4.3 Model and calculation boundaries

The first step is the formulation of the boundaries and limitations of the used model and the limitations to the calculations that are performed. The most important statements about the model and the analysis are listed here:

<u>Model</u>

- The model used is PLAXIS-FORM. Alternatively, MSheet-FORM or Blum-FORM (or Blum-Monte Carlo) can be used to compare results. PLAXIS is more sophisticated than MSheet or Blum, but it is also more complex to obtain correct parameter values. Stiffness parameter values, for instance, can be obtained from CPT's, but only via relations with the cone resistance or relative density. Also variations in (ground) water levels and bottom level and the probable relations between them can be hard to define. PLAXIS-FORM can be successful, but finding the correct input might be difficult. FORM is the only reasonable probabilistic method. A Monte Carlo analysis or other level III approach would allow full probabilistic calculations, but the required number of calculations in PLAXIS is unrealistic high. This would require too much calculation time. FOSM has too many limitations as it is not an iterative procedure.
- The only workable tool to do PLAXIS-FORM calculations is Prob2B from TNO (Courage & Steenbergen, 2007). It has been tried via other ways, but Prob2B gives the most opportunities in defining LSF's, stochastic parameters and couplings with other software like MatLab and Excel.
- The TNO Prob2B software has the limitation that the loads, geometrical parameters and water levels cannot be made stochastic. The influence of variation in these parameters can be checked manually in the design point calculation. Another approach is to perform probabilistic Blum calculations to verify the importance of these parameters. Of course, Blum has its limitations as well.
- The hardening soil model is used in PLAXIS. This model includes shear hardening modelling irreversible strains due to primary loading. Decreasing stiffness of the soil leads to irreversible plastic strains. Furthermore the model includes compression hardening modelling irreversible plastic strains due to primary compression in oedometer loading and isotropic loading. This complex model gives opportunities, but it also requires more input, which can be complex to gather.

- At first it is assumed that spatial variability of parameters is averaged to a representative mean in the considered cross-section. In later stages this appeared not to be an entirely correct assumption, but this is discussed later on in section 7.2.
- The uncertainty of the model in its description of reality is taken into account via the obtained reliability index. It is not checked whether this is the best way to include this, but it is less complicated than when uncertainty must be spread out over all input parameters. Basically a mean β with a coefficient of variation is obtained in this case.

Calculations

- Not all failure mechanisms can be included in PLAXIS calculations. Only anchor failure (profile), wall failure in bending and soil mechanical failure are analyzed. They are the most important mechanisms as also discussed in 'veiligheid van damwandconstructies' (Calle & Spierenburg, 1991). Also a LS with respect to deformations can be defined. This means that only for these four mechanisms a reliability index can be determined. Other mechanisms, like piping, need to be analyzed with different models and therefore the reliability index of the top event cannot be determined with PLAXIS-FORM only.
- Plastic deformation of the structural elements is not included. The LS's use the yield stress. This implies that the plastic capacity of the steel is not used in the calculations.
- Both SLS and ULS situations can be analyzed. In SLS the deformations are checked and in ULS the mentioned failure mechanisms.
- Only a limited number of quay walls can be analyzed, because of the limited time that is reserved for this study. Only the two benchmark quay walls of Bakker en Jaspers Focks (2011) can be analyzed. More analyzed quay walls would make the foundation of the conclusions more reliable and clear. Note that both benchmark quay walls and their soil characteristics are specific for the Port of Rotterdam and therefore not immediately applicable for other ports in the Netherlands.

4.4 Fault tree

4.4.1 Introduction

An important aspect of probabilistic analysis is the fault tree. The fault tree gives a succession of all events that lead to one unwanted 'top event' (CUR 190, 1997). This tool can be used to calculate the overall failure probability by using the probabilities that failure mechanisms occur. Furthermore, it can be used to make design choices. The so-called budgeting implies that small failure probabilities are accepted for elements that are relatively easy (cheap) to strengthen. This gives more failure space to mechanisms that are more expensive to improve. The fault tree also gives information about the relationships between mechanisms; their contribution to the overall failure probability and their correlation to other mechanisms.

CUR 211 based its fault tree on CUR 166 and extended and adjusted it in order to apply it on relieving floor structures. As this fault tree forms the basis of probabilistic calculations it is important to analyze the current tree. This is done in this section. Furthermore, some recommendations for improvement are given.

4.4.2 Analysis

The original tree of CUR 166 is defined in 'Veiligheid van Damwandconstructies' (Calle & Spierenburg, 1991). Gemeentewerken Rotterdam (Huijzer, 1996) and the former CUR 211 committee extended this to the failure tree of Figure 3-2. It can be questioned whether this fault tree is compiled transparently and logically. This question holds both for the structure of the tree, as well as for the division of failure space and the relations between the different mechanisms.

The first aspect that draws the attention is the mechanism 'Excessive deformations'. This mechanism is characterized as top event. There is no failure space reserved for this event because in case of a sheet-pile structure the SLS is automatically fulfilled. However, in case of a quay wall structure the excessive deformations may be a real problem before the ULS is reached. Due to the larger stiffness of a quay wall structure in comparison to a sheet-pile the shear resistance mobilization may not fully develop at limited deformations. In that case the situation in SLS might be governing and it is important to quantify the failure space of this mechanism. Otherwise the failure probability of the entire structure might be higher than expected.

The second aspect is the division of failure space. In 'Veiligheid van Damwandconstructies' it is explained why certain mechanisms have a certain failure space. For instance the Kranz-stability is given 0.2 p as it is relative easy (and cheap) to improve the structure to obtain a smaller failure probability. However, in CUR 211 it is not explained why the added mechanisms have their specific failure space. Why is 0.8 p chosen for the mechanism 'shear resistance of anchorage inadequate'? Why 0.2 p for the mechanism connection fails (tension member, bearing piles and sheet-pile wall)? Why 0.8 p for the bearing capacity of sheet-pile walls?

The third aspect is about the relations between the mechanisms. In 'Veiligheid van Damwandconstructies' it is explained that an OR-port is used to describe the series system (if one mechanism occurs the entire structure fails). The combined probability of failure can be determined by counting the largest probability fully and the other probabilities half. This is mentioned as strong, but not fully dependent. It is not further elaborated why the relations are assumed to be like this, most probably due to lack of knowledge. CUR 211 however makes it less transparent. A direct quote from the appendix: '*For some failure mechanisms some correlation between the probabilities of occurrence is taken into account'* (CUR 211, 2003). It is unclear where, why and how large the applied correlations are. Therefore it is not transparent why the failure probability of the top event is 6.25 p. Even stranger is the fact there are no correlations between the mechanisms below the top layer, while in the original tree of sheet-pile structures there are correlations. Why did these correlations disappear?

4.4.3 Recommendations

The fault tree forms the core of the probabilistic design used in the normal guide lines. The partial safety factors are assigned in accordance to the appropriate failure spaces. Economic optimization of the design is possible. A smart division of the failure spaces leads to economical advantages, i.e. small failure space for a mechanism that is cheap to improve. Basically, it requires expertise or a thoroughly executed optimization study to get a maximum result. In case of CUR 211 the reasoning behind the fault tree is not clear. Is it expertise or is it simply using expertise of CUR 166? It is recommended to improve this in the update of the handbook and if possible it can be useful to do the budgeting of the fault tree in a more sophisticated and transparent way for later updates.

4.4.4 Applied fault tree

As it is difficult to estimate correlations and an optimal division of failure space without doing research, this study uses the fault tree of CUR 211 (2003) as starting point. This means that the required reliability indices for the different mechanisms are given in the failure tree of Figure 3-2. This study can probably provide some ideas to improve the fault tree. This analysis is made later on in section 7.5.

4.5 Mean and characteristic values

This section describes the starting point and definition of the used parameter values in the analyses. It is subdivided in solicitation and resistance parameters and the definition of loads.

4.5.1 Solicitation and Resistance

The current design philosophy in CUR 211, CUR 166 and Eurocode is characterized by the use of characteristic values of the parameters. The characteristic value of a parameter implies there is a 5% probability that the value is higher (solicitation) or lower (resistance). The definitions are shown in Figure 4-2 in which the mean value equals 0 and the standard deviation 1 (standard normal). The characteristic value for a normal distribution is 1.64σ higher than the mean value (In this case 1.64).

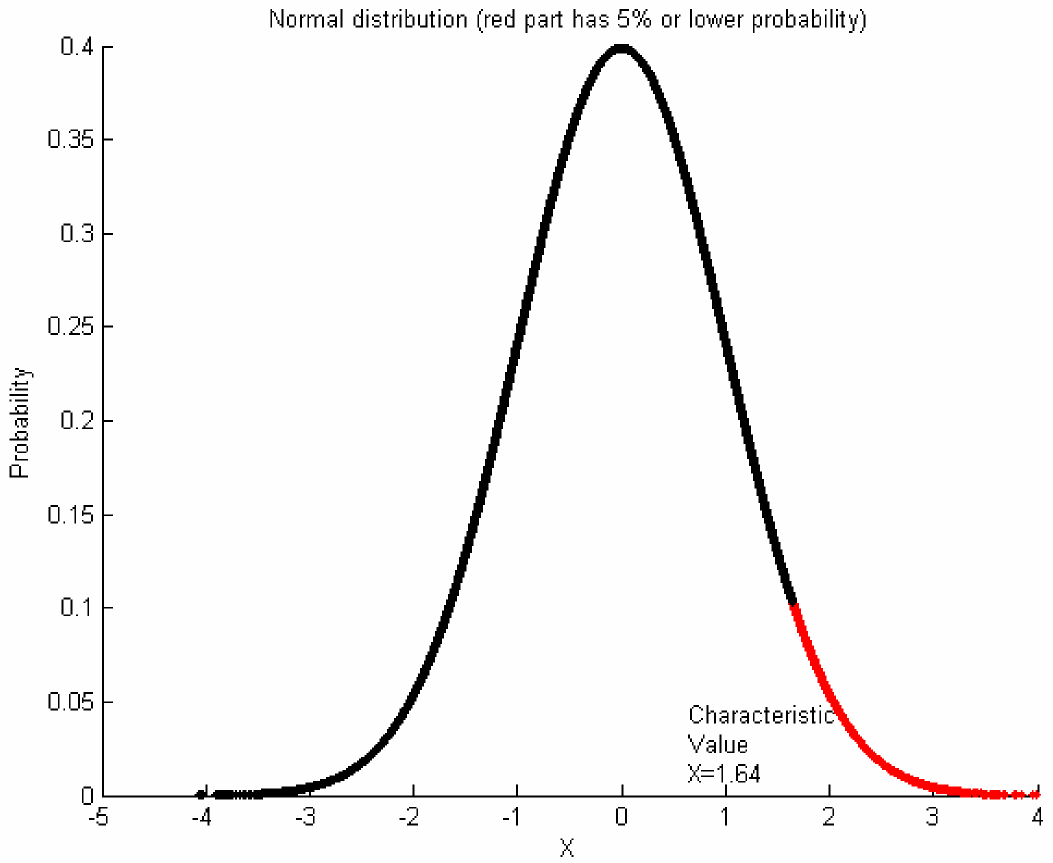


Figure 4-2 Mean and characteristic value

The general expression for a Limit State Function (LSF) is $Z = R - S$ in which R is the resistance and S the solicitation.

The values of the partial safety factors are additional factors over the characteristic values.

Solicitation: $\gamma_{k,S} = \dfrac{1+1.64V}{1-\alpha\beta V}$ Resistance: $\gamma_{k,R} = \dfrac{1-1.64V}{1-\alpha\beta V}$

Note that in principal α can be positive or negative, following from the probabilistic calculation. Positive implies resistance, negative implies solicitation. The formulas automatically give the correct sign.

With respect to the mean values the safety factor is in fact larger.

Solicitation: $\gamma_{\mu,S} = \frac{1}{1-\alpha\beta V}$ Resistance: $\gamma_{\mu,R} = \frac{1}{1-\alpha\beta V}$

The design values are:

Solicitation: $S_d = \frac{S_k}{\gamma_{k,S}} = \frac{S_\mu}{\gamma_{\mu,S}}$ Resistance $R_d = \frac{R_k}{\gamma_{k,R}} = \frac{S_\mu}{\gamma_{\mu,R}}$

The decision to use characteristic parameter values is in itself a cautious estimate. As this approach is extensively discussed over the years, the aim of this study is not to choose another path. The reasons to work with characteristic values are taken for granted. The use of characteristic values in probabilistic calculations however is problematic. As probabilistic calculations imply the implementation of mean values and standard deviations, the use of characteristic values as mean value would give an unrealistic result. The characteristic value in Figure 4-2 is then moved to the symmetry axis, which would completely change the curve.

The approach used in this study therefore differs from the standard approach. The probabilistic calculations are performed with the mean parameter values. Eventually this leads to partial safety factors that should logically be applied on the mean values. These factors are therefore higher than the original factors. To find connection to the normal design procedures the safety factors on the mean values can be transformed to safety factors on characteristic by the formulas:

Solicitation: $\gamma_{k,S} = \gamma_{\mu,S}(1+1.64V)$ Resistance: $\gamma_{k,R} = \gamma_{\mu,R}(1-1.64V)$

In this way the comparison between old and new values still can be made and the design approach is not changed.

4.5.2 Definition of characteristic load

For loads the definition of characteristic value is different. In practice the characteristic value is defined as the load that occurs once in the lifetime of a structure. An example is a surcharge load due to traffic. It is assumed that every minute two trucks pass the cross-section at the same time. This means there are 5000000 of these situations in the structures lifetime (33 years). The sum of the two trucks is normally distributed with a mean value of 500 kN and a standard deviation of 280 kN. To obtain the characteristic value of this load the maximum of 5000000 events needs to be found. This is the capital X in the following expression: $P(x > X) = 2 \cdot 10^{-7}$

The characteristic value of the load with a probability of exceedance of 1/5000000 is equal to 1679 kN. This is not clearly visual in Figure 4-3 as the probability of exceedance is quite small. The pink bar indicates the characteristic value. Note that the negative values of the load in the lower tail of the distribution are not realistic.

In the once in a lifetime load is also uncertainty present. It might be not exact the 1/500000 probability of exceedance. This can be accounted for by a coefficient of variation (CoV = standard deviation / mean value) over the obtained value. This is also done in the probabilistic calculations in this thesis.

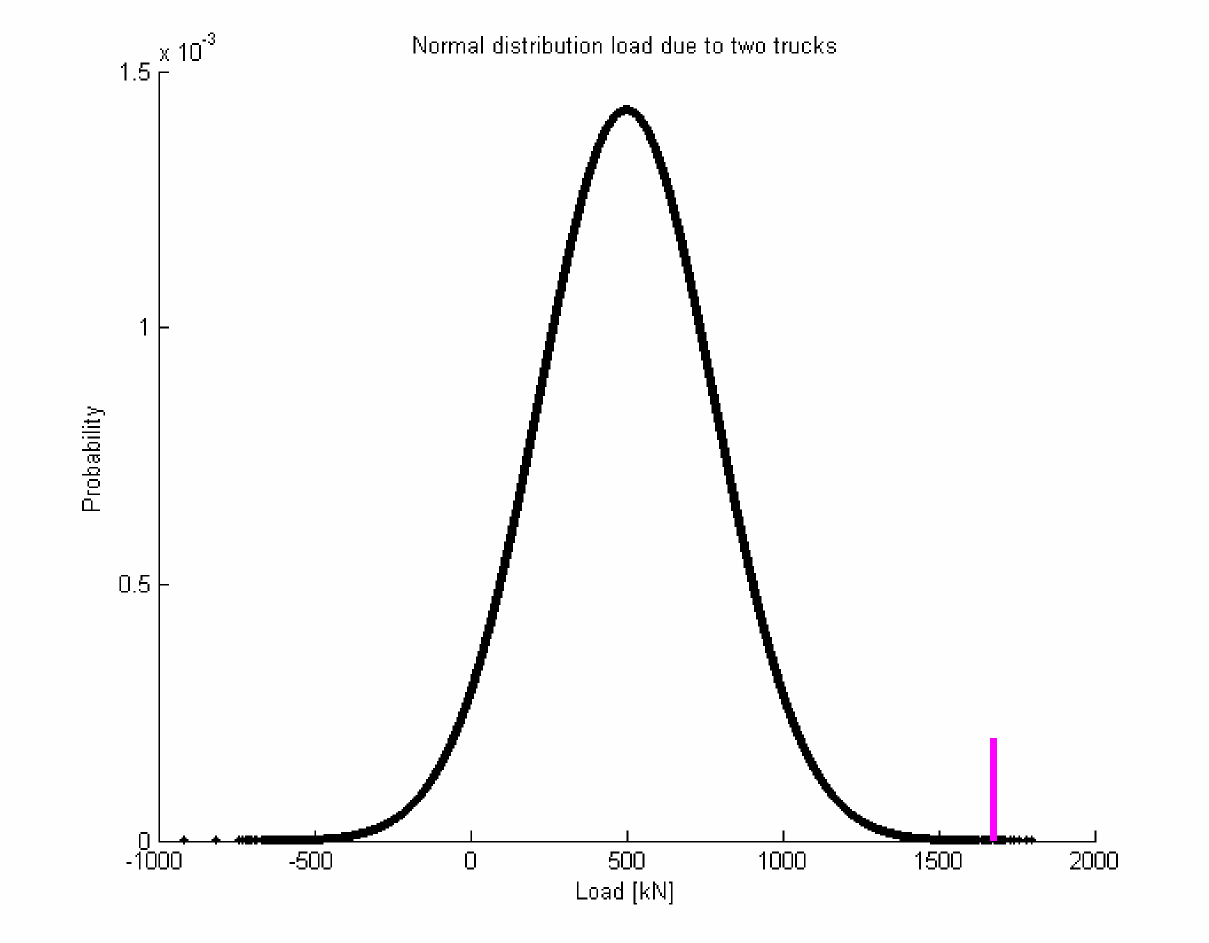


Figure 4-3 Normal distribution two trucks

4.6 Discussion on target reliability index β

The design codes CUR 166, CUR 211 and also Eurocode 7 prescribe a target reliability index for several types of structures and safety classes. This is a certain level of safety that should be reached when following the procedures in the code. Basically, the code includes all different uncertainties and guarantees a β over the lifetime of the structure. There are different sources of uncertainty that work through in β in different ways. Three groups can be distinguished:

- Material parameters. For these parameters the uncertainty is constant over the entire lifetime. The soil that is present at day one will also be there at the final day of lifetime. The daily β when only looking at these parameters, does not change over lifetime. Off course degradation of the material (for instance corrosion of the steel of the sheet-pile) should be taken into account as well, which makes the daily β decrease over time. In this research, however, material degradation is not taken into account and β with respect to material parameters is constant over time. This is a reasonable assumption when katodic protection is applied on the steel.
- Load parameters. A quay wall is designed according to a representative load, most often the load that will occur once in a lifetime (section 4.5.2). Most of the time the load is much lower than the representative load, but only one moment of time in the structures lifetime this load will be present. When the quay wall is designed according to this load, this uncertainty is in fact implicitly incorporated in the design. The only uncertainty that is left is the uncertainty in the value of the once in a lifetime load. This uncertainty influences the β of the structure, whereas the decision to choose a once in a lifetime load gives the boundary conditions for this β. In fact following this procedure a β is given for a certain load condition. It is therefore necessary to take the governing load condition, as only in that case β is high enough for each load case. In practice it is necessary to define several load combinations. However, in this reasearch it is assumed to have only one governing load condition.
- Geometrical parameters. For the retaining height and water levels basically the same idea holds as for the load parameters. The designer takes into account the most critical values of the retaining height and water levels with respect to failure of the structure. For the water levels this can be the once in a lifetime water level difference or for instance the case in which the drainage coffer does not work. This depends on what the code prescribes to be governing. For the retaining height this definition is slightly different as the bottom will not fluctuate during its lifetime. Therefore the lowest possible bottom height should be chosen. This can be based on possible scour holes, uncertainties in dredging, etc. In this way the required β is based on this governing situation. The uncertainty that is left is the uncertainty in the value of the once in a lifetime water levels and governing retaining height. It might be difficult to make exact predictions and therefore a standard deviation can be taken into account in a probabilistic calculation. This standard deviation influences the reliability of the structure in the 'way the material parameters' do.

The probabilistic calculations made in this research assume a governing load situation, water levels and bottom height. The β's obtained in the calculations are therefore bound to these governing situations. Basically this is the same approach as discussed in CUR 211. In this handbook it is said that a nominal situation should be found by the expertise of the engineer. The same is done for the extreme load, which is taken to be the characteristic/governing load in the design. (CUR 211, 2003)

The probabilistic calculations made in this research in PLAXIS-Prob2B cannot include uncertainties in load and geometrical parameters. Therefore the β's are only based on the material parameters. Manually, there are made variations in the load and geometrical parameters in order to estimate their influence on the reliability index. This approach is checked by making the less complex probabilistic Blum calculations in which these parameters can be included. Important, however, is that the influence of the load and geometrical parameters is only based on the uncertainty in their governing situation and not with respect to their ‘mean value’ or ‘daily present’ values.

When defining the partial safety factors it should be taken into account there is also some safety hidden in the design choices of retaining height, characteristic surcharge load and governing water level differences. The probabilistic calculations in this report are mainly focussed on the uncertainties in the material parameters combined with the variations in extreme values of the retaining height, water levels and loads. It is therefore questionable whether it is correct to apply the target β in the formula for the partial safety factor, as it might give too high partial safety factors as it does not include the incorporate safety in the design choices. It should be clear that the reliability indices and influence factors are based on these design choices and therefore it is difficult to compare the importance of the material factors with the load and geometrical parameters. This discussion returns in the discussion of the results in chapter 5 and 6 and also in section 7.3.

4.7 Parameter values, coefficient of variations and correlations

The hardening soil model and the structure in PLAXIS require the input of several variables. These parameters are presented in Table 4-1. This table also shows the sources of the parameter values and whether they are taken deterministic or stochastic in the probabilistic calculations.

Description	Symbol	Unit	Stochastic / Deterministic	Source of parameter value
Soil parameters				
Unsaturated soil weight	γ_{unsat}	[kN/m^3]	S	Soil sample
Saturated soil weight	γ_{sat}	[kN/m^3]	S	Soil sample
Internal angle of friction	φ	[°]	S	Triaxial test
Cohesion	C	[kPa]	S	Triaxial test
Dilatancy angle	ψ	[°]	S	0 (φ<30) or φ-30 (φ>30)
Secant elasticity modulus at a 50% deviatoric stress (at P_{ref} = 100 kPa)	E_{50ref}	[kPa]	S	CPT
Oedometer elasticity modulus (at P_{ref} = 100 kPa)	E_{oedref}	[kPa]	S	≈ E50ref
Unloading reloading elasticity modulus (at P_{ref}=100 kPa)	E_{urref}	[kPa]	S	≈ 4*E50ref à 5*E50ref
Amount of stress dependency (power)	m	[-]	S	0.5 for sand 1.0 for clay
Interface	R_{int}	[-]	S	0,9 for sand 0,67 for clay
Plate Parameters				
Axial stiffness	EA	[kN/m]	S	Steel producer
Flexural rigidity	EI	[kNm^2/m]	S	Steel producer
Specific weight	w	[kN/m]	S	Steel producer
Resisting moment	W	[m^3/m]	S	Steel producer
Yield stress steel	$f_{y,s}$	[kPa]	S	Steel producer
Anchor parameter				
Axial stiffness	EA	[kN]	S	Steel producer
Diameter	D	[m]	S	Steel producer
Yield stress steel	$f_{y,a}$	[kPa]	S	Steel producer
Spacing between anchors	$l_{spacing}$	[m]	D	Contractor
Level parameters (with respect to NAP)				
Soil layer depth	h_i	[m]	D	CPT
Ground level (land side)	GL	[m]	D	Contractor
Bottom level (water side)	BW	[m]	S	Contractor
Groundwater level (land side)	GWL	[m]	S	Boundary Conditions
Water level (water side)	WL	[m]	S	Boundary Conditions
Depth wall	d_{wall}	[m]	S	Contractor
Loads				
Surcharge load	q	[kN/m^2]	S	Boundary Conditions
Bollard force	F_b	[kN/m]	S	Boundary Conditions
Hawser force	F_h	[kN/m]	S	Boundary Conditions

Table 4-1 PLAXIS input parameters

All parameters from the table will be discussed in this section. Ideally, all soil parameters are obtained from many soil tests in situ. However, for most quay walls there are no extensive databases with results of soil investigations.

As a large source for general parameter values the database of Gemeentewerken Rotterdam [GW] (2003) is used, which contains over 2000 triaxial tests taken in the period 1999-2003. Also estimations of the CoV's can be derived from this database.

4.7.1 Soil parameters

4.7.1.1 Soil weight (γsat and γunsat)

The specific soil weight can be determined by dividing the total weight (W) of the soil by the volume (V). The dry weight equals $\gamma_{unsat} = \frac{W_d}{V} = (1-n)\rho_{grains} g$ [kN/m^3]

When a sample is dried in the oven the dry weight W_d and porosity n can be determined. The saturated weight follows from $\gamma_{sat} = n\rho_{water} g + (1-n)\rho_{grains} g$ [kN/m^3]

This determination of the specific soil weight can be done relatively accurate in comparison to other soil parameters (Verruijt, 2010).

The value of the CoV is taken as 0.05 based on NEN 6740 (2006) and a document of Joint Committee on Structural Safety [JCSS] (2002).

4.7.1.2 Internal angle of friction (φ) and cohesion (C)

The internal angle of friction and cohesion of a soil can be determined from a triaxial test. However, in most cases only a few soil layers are tested on only a few spots at the building location. Therefore it is hard to estimate the correct angle. It is more convenient to relate the internal angle of friction to the saturated soil weight. In this way a distribution of φ can be found for different saturated soil weight classes. φ and C are obtained from commonly used 2% strain tests from the database (GW, 2003). Figure 4-4 shows the relation with γ_{sat}. It is clear that there is a better correlation with φ than with C.

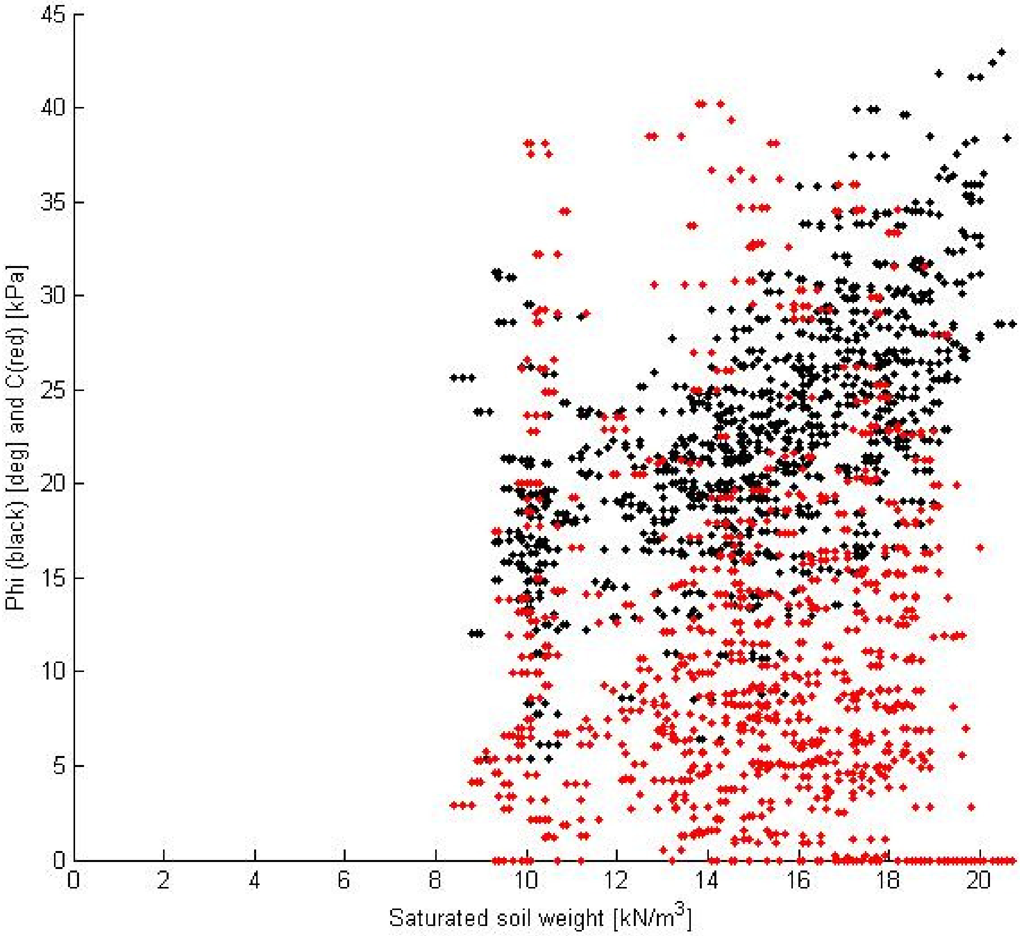
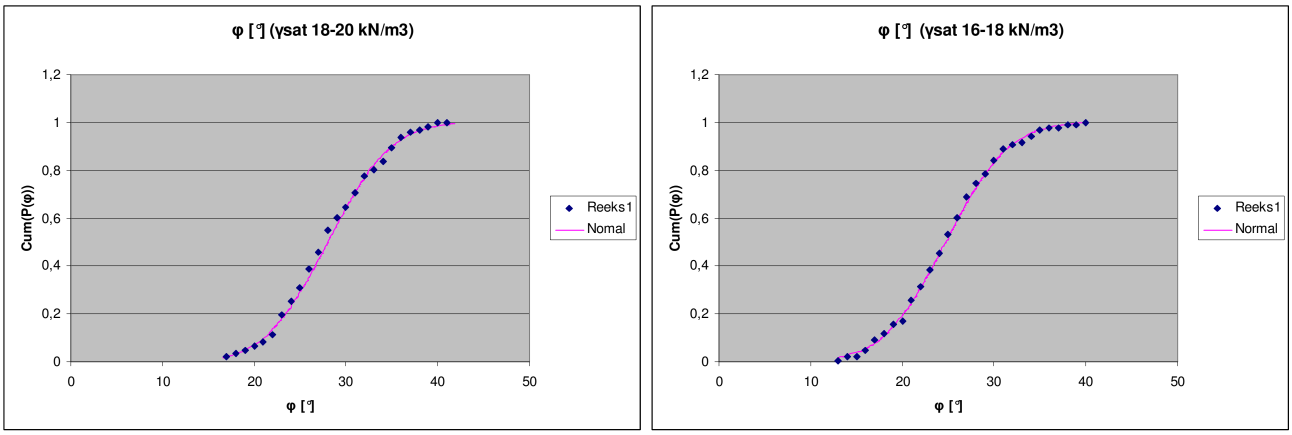


Figure 4-4 φ and C related to saturated soil weight

Therefore first for different γ_{sat} classes a cumulative probability plot of φ is made (Figure 4-5). These plots are compared to cumulative normal distribution plots (the pink line).



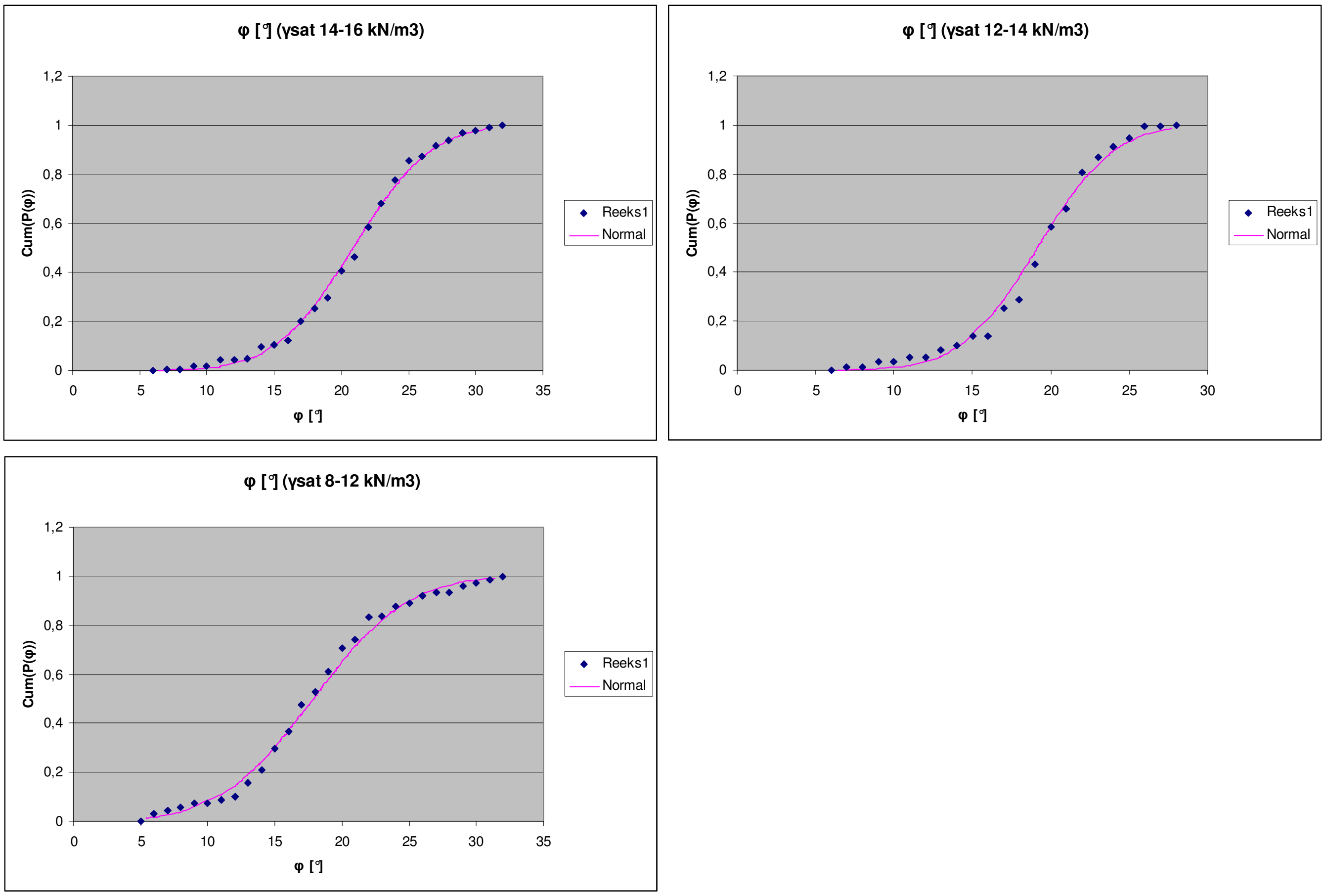


Figure 4-5 Probability density functions φ [°] 2% strain

It can be seen from the figure that the measurements approximate the shape of a normal distribution. The statistical parameters of the normal distribution per γ_{sat} class are presented in Table 4-2. As can be seen from the table a CoV of 0.2 is reasonable for each γ_{sat} class. The lowest γ_{sat} class is not often found. Therefore 0.2 is taken for all γ_{sat} classes. This is somewhat higher than the NEN 6740 (2006) prescribes and comparable to the value of JCSS (2002).

γ_{sat} [kN/m^3]	μ_φ [°]	σ_φ [°]	$V_\varphi=\sigma_\varphi/\mu_\varphi$
8-12	17.87	5.56	0.31
12-14	19.16	3.88	0.20
14-16	20.88	4.59	0.22
16-18	24.73	5.45	0.22
18-20	28.03	5.45	0.19

Table 4-2 Statistical parameters φ [°] 2% strain

The scatter of C is much larger than the scatter of φ (Figure 4-4). This is logical when looking at the procedure to determine these two parameters. The result of a Triaxial test is given in Figure 4-6. A small deviation in the blue line also leads to small changes in angle φ. However the C (intersection with vertical axis) can deviate considerably.

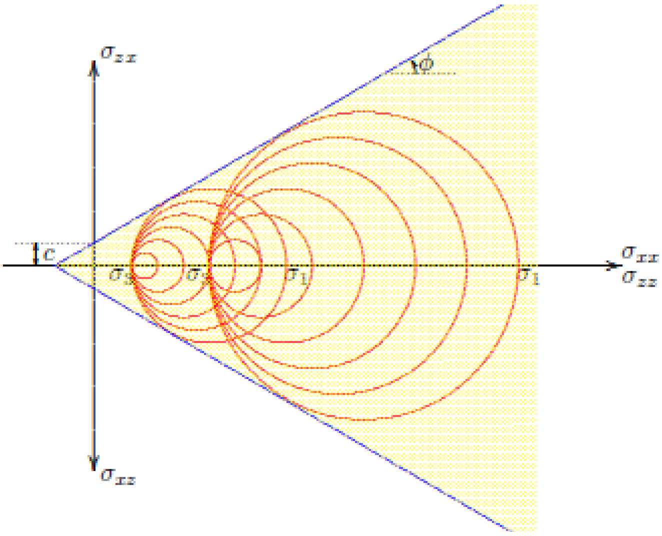


Figure 4-6 Triaxial test results (Verruijt, 2010)

Therefore the obtained CoV's for cohesion are larger. The values are presented in Table 4-3. The standards of NEN 6740 (2006) and JCSS (2002) prescribe lower values, between 0.1 and 0.5. The values from Table 4-3 are therefore slightly reduced till a general value of 0.8 for each γ_{sat} class. This is still higher than the standards, but it is expected that C does not influence the output by much as there is dealt with relative large retaining heights. The internal angle of friction will get more importance in these cases (Coulomb: $\tau = C + \sigma' \tan\varphi$). Only for relative shallow clay layers C can be relevant.

γ_{sat} [kN/m^3]	μ_C [kPa]	σ_C [kPa]	$V_C=\sigma_C/\mu_C$
8-12	12.6	10.7	0.85
12-14	11.5	10.6	0.93
14-16	13.3	10.3	0.77
16-18	12.1	9.2	0.76
18-20	8.6	9.6	1.04

Table 4-3 Statistical parameters C [kPa] 2% strain

C and φ are obtained from the same test and sample. Therefore there is a correlation between the two parameters. A large value of φ implies often a lower value of C. This is confirmed by the plot from Figure 4-7. The correlation coefficient between φ and C can be derived from the plot to -0.65.

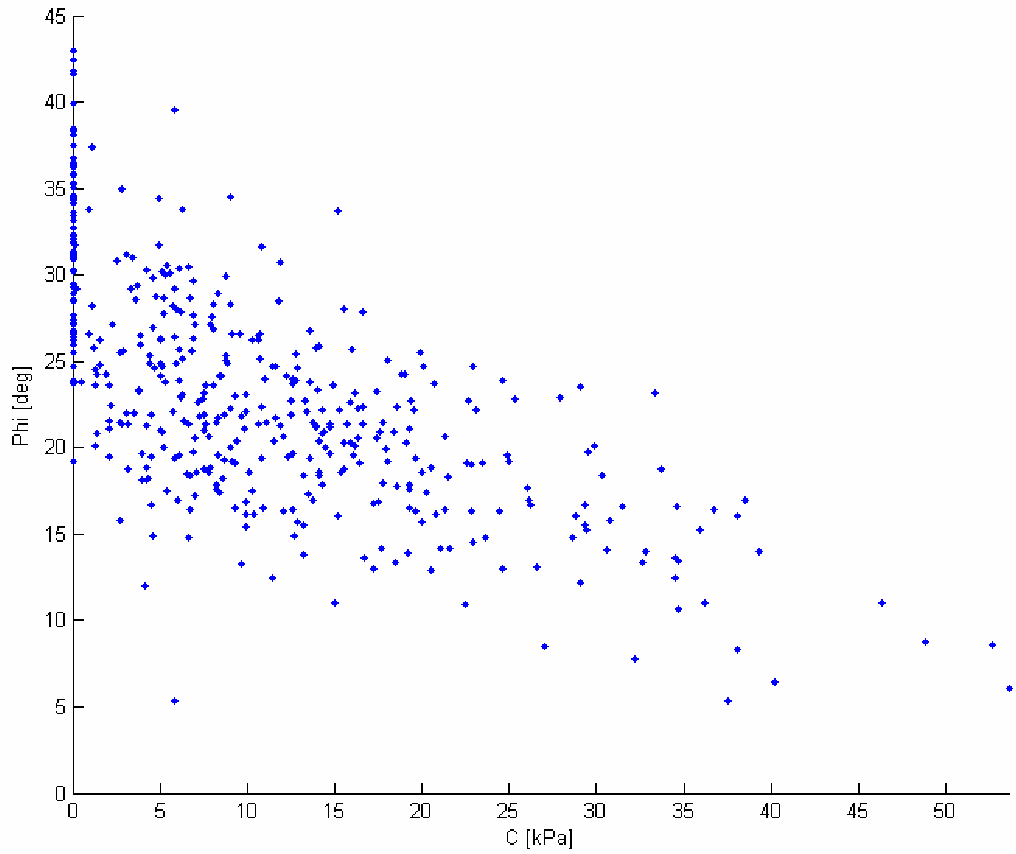


Figure 4-7 φ [°] vs. C [kPa] 2% strain

4.7.1.3 Dilatancy angle (ψ)

The dilatancy angle is related to the angle of internal friction. ψ gives the change in volume of the soil when shear deformations occur. The values are determined by experience (CUR C2003-7, 2003):

$$\psi = \begin{cases} \varphi - 30 & (\varphi > 30) \\ 0 & (\varphi < 30) \end{cases} [°]$$

Therefore, the CoV is taken the same as for φ (0.2).

4.7.1.4 Soil stiffness (E_{50}, E_{oed}, E_{ur})

The Hardening Soil model requires three soil stiffness parameters, all of them defined at a reference pressure of 100 kPa. E_{50} is the secant stiffness at a 50% deviatoric stress, shown in Figure 4-8, obtained from a triaxial test.

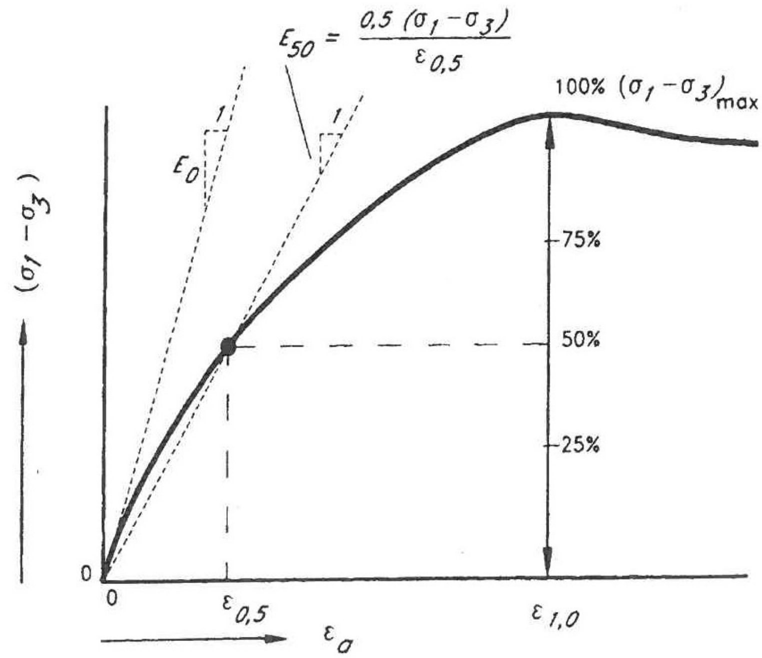


Figure 4-8 Determination of secant stiffness E_{50} (CUR C2003-7, 2003)

The oedometric stiffness parameter E_{oed} at a reference stress level can be determined from a compression test, according to Figure 4-9.

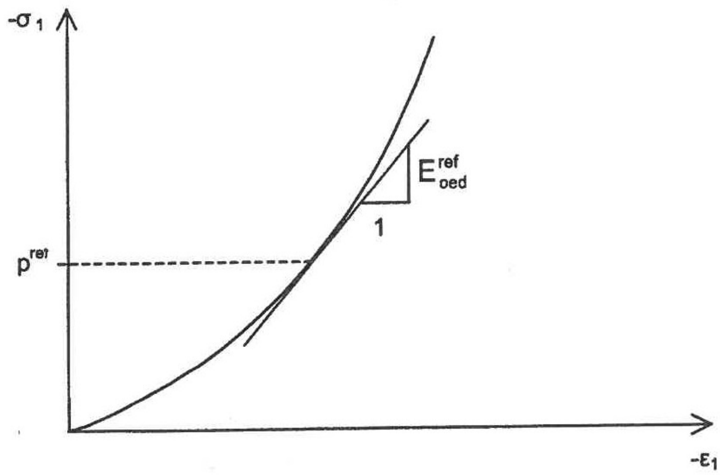


Figure 4-9 Determination of oedometer stiffness E_{oed} (CUR C2003-7, 2003)

The unloading reloading stiffness parameter E_{ur} can also be obtained from a triaxial test. This is shown in Figure 4-10.

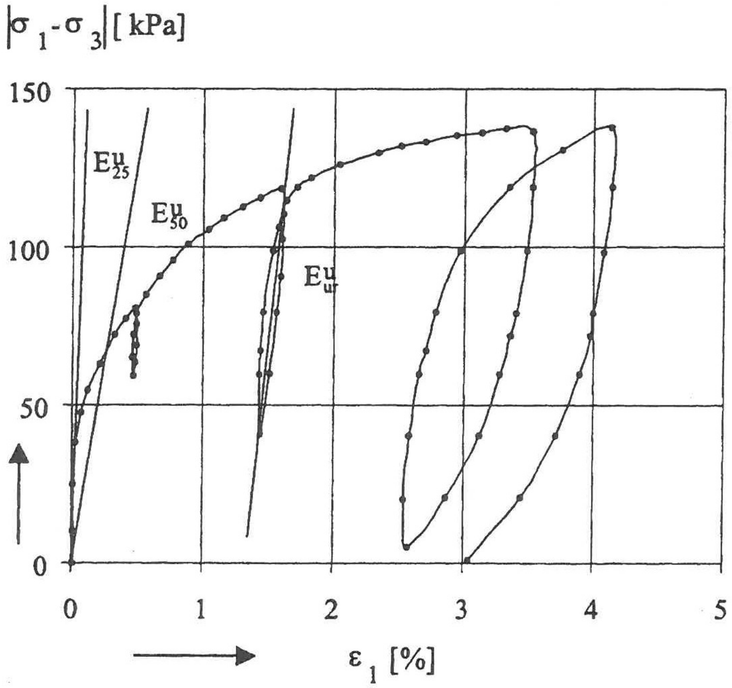


Figure 4-10 Determination of the unloading reloading stiffness E_{ur} (CUR C2003-7, 2003)

The soil stiffness can be related to the cone resistance (q_c) from a cone penetration tests (CPT). There are several empirical rules to derive the values. Basically there are two approaches. The first relates soil stiffness to relative density (Re), which can be done in several ways:

- Re is related to the pore number e: $\mathrm{Re} = \frac{e_{\max} - e}{e_{\max} - e_{\min}}$ (Verruijt, 2010) where $e = \frac{n}{n-1}$ (n is the porosity, e_{max} is the maximum pore number and e_{min} the minimum)
- Re is related to the cone resistance q_c using the empirical formula of Lunne:

  $$\mathrm{Re} = \frac{\ln\left(\frac{q_c}{61\sigma_v^{0.71}}\right)}{2.91}$$ (Lunne, Robertson & Powell, 1997)
- Via other empirical relations between Re and E_{50} or E_{oed}, for instance from Floss or Hunt (see CROW, 2004)
- Via triaxial tests and measurements. From databases from Gemeentewerken Rotterdam of for instance the Gevelco Brammenterminal (~2006) plots like Figure 4-11 can be made and the required E_{50} can be found.

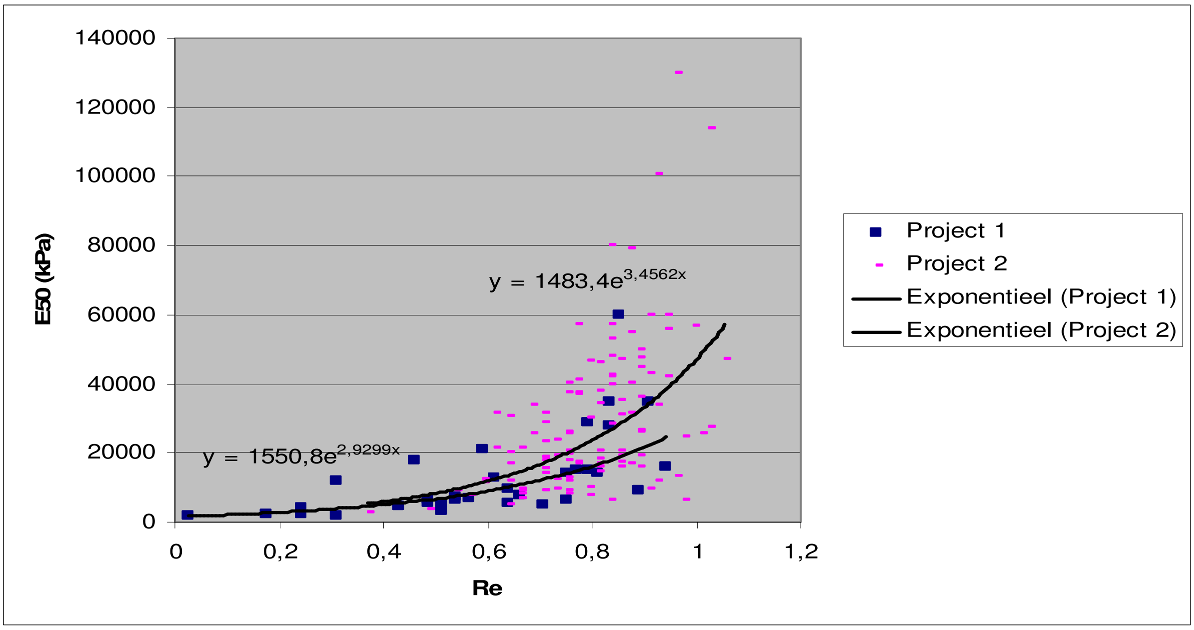


Figure 4-11 Relative density (Re) vs 50% stiffness modulus (E_{50}) (Brammenterminal)

Note that the determination of Re and its relation with the soil stiffness gives the characteristic value of the soil stiffness.

The second approach relates soil stiffness directly to cone resistance (q_c) obtained from the CPT. As the number of triaxial tests is often limited, this is a convenient way to find reasonable values for the stiffness parameters. The basic relation is in this case $E = \alpha q_c$ where α depends on the soil classification. Thooft (2007), TAW (1989), USACE (1990) all suggest certain values for this. Often used are the empirical relations of Trofimenkov (Thooft, 2007):

$E_{50} = 3.4q_c + 13$ for tertiary sand

$E_{50} = 7.8q_c + 2$ for quaternary sand and tertiary clay

Vermeer (CUR 162, 1990) presents a simplification of this formula:

$E_{50} = 3q_c$ [kN/m^2]

Note that the relations from Trofimenkov and Vermeer give the characteristic value of the parameter. As in this research the mean parameter values are used, it is decided to change the factor from Vermeer into a factor 4 between cone resistance and E-modulus. <u>The general simplified relation for this research therefore is the mean value</u>:

$$E_{50} = 4q_c$$

The relation with cone resistance is preferred above a relation with relative density, because the relation is more simple and often more CPT's are available than triaxial tests.

According to the CUR-document about geotechnical parameters (CUR C2003-7, 2003) the oedometer loading modulus ($E_{oed,ref}$) and unloading reloading modules ($E_{ur,ref}$) can be related to the 50% stiffness modulus ($E_{50,ref}$).

$E_{oed,ref} \approx E_{50,ref}$

$E_{ur,ref} \approx 4E_{50,ref}$ or $E_{ur,ref} \approx 5E_{50,ref}$

The PLAXIS manual (Brinkgreve & Broere, 2008) mentions a different relation: $E_{ur,ref} = 3E_{50,ref}$. This relation is applied in the benchmark quay walls (chapter 5 and 6), but in general often 4 or 5 times $E_{50,ref}$ can be applied.

Due to the given fixed relations, the correlation coefficients between the stiffness parameters are 1.0. There is also a correlation between the stiffness parameters and the saturated soil weight. This correlation coefficient is 0.5, which can be deduced from the scatter plot of Figure 4-12, made from the data of the database of GW (2003).

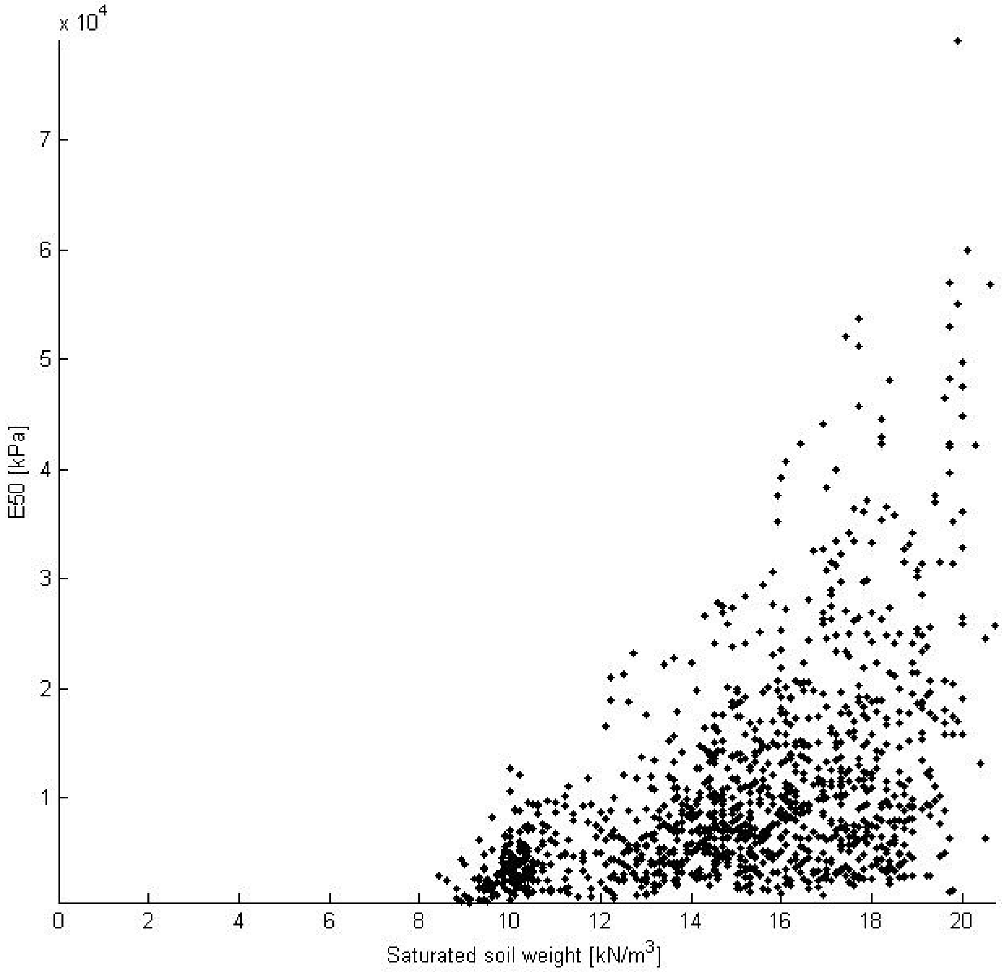


Figure 4-12 E_{50} [kPa] vs. saturated soil weight [kN/m^3]

Another important aspect is the CoV of the stiffness parameters. NEN 6740 (2006) prescribes 0.25, whereas JCSS (2002) is more unclear with a value between 0.2 and 1.0. It is decided to connect to the approach in CUR 166 (2005). In this document the distribution of a stiffness parameter is defined by the characteristic values, in equations:

$$E_{d;low} = \frac{E_{k;low}}{1.3} = \frac{E_{mean}}{2}$$

$$E_{d;high} = \frac{E_{k;high}}{1.0} = 1.5E_{mean}$$

The lower characteristic value represents the 5% lower limit and the upper characteristic value the 5% upper limit. By applying a Monte Carlo-analysis the variance used in CUR 166 can be deduced from the lognormal distribution. Figure 4-13 gives an example for the distribution of E with mean value 45 MPa. It can be shown that a CoV of 0.26 leads to the required 5% lower and upper limits (F(E) in Figure 4-13).

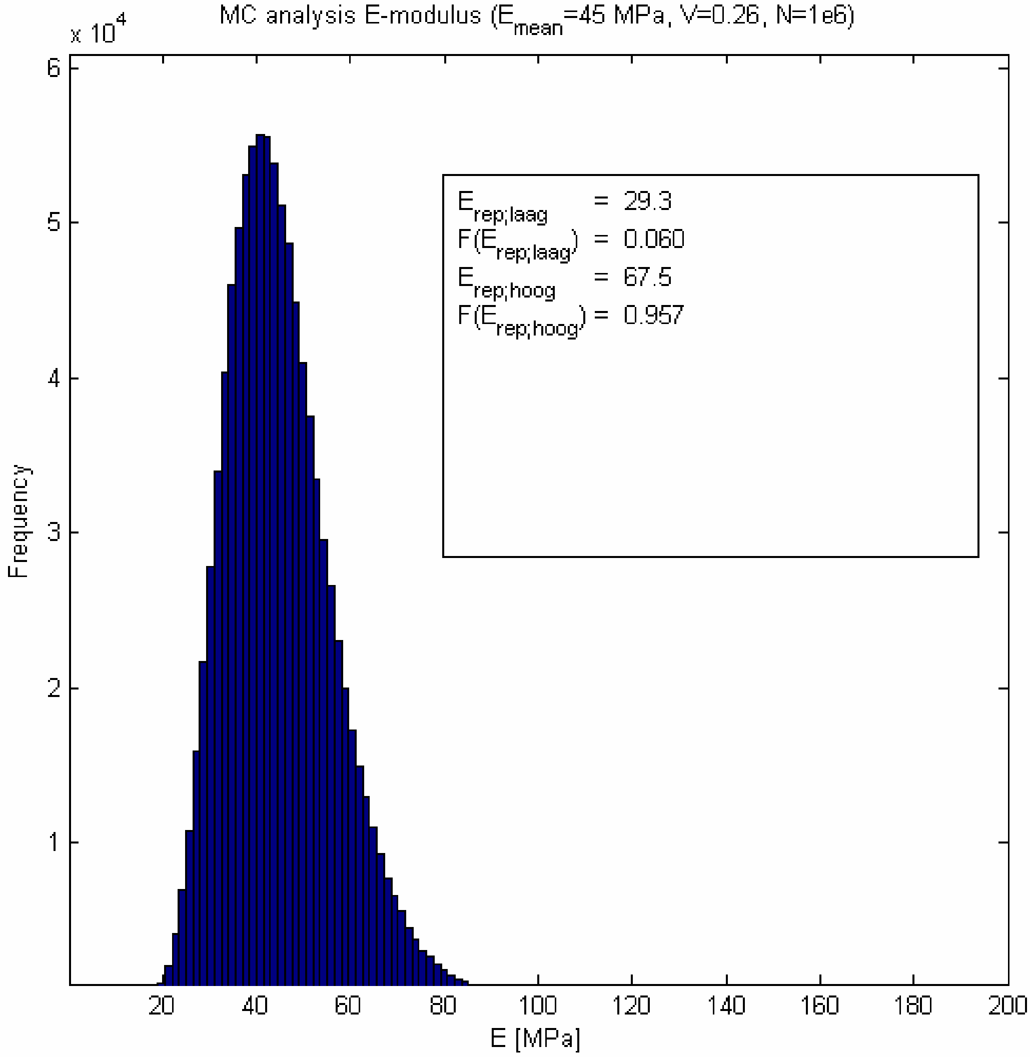


Figure 4-13 Histogram E-modulus

Therefore, a CoV 0.3 for the soil stiffness parameters is chosen, which is also in line with NEN 6740 (2006) and JCSS (2002).

4.7.1.5 Amount of stress dependency (power) (m)

The E_{50} is the confining stress dependent stiffness modulus for primary loading and is given by the equation (Brinkgreve & Broere, 2008): $E_{50} = E_{50}^{ref} \left(\frac{C\cos\varphi - \sigma_3^{'}\sin\varphi}{C\cos\varphi + p^{ref}\sin\varphi}\right)^m$

For power m in the stress dependency for clay a value of 1 is used and for sand 0.5. For clay-sand mixtures interpolation between these values can be applied. The values are based on CUR C2003-7 (2003). At first attempt 0.2 as CoV is used in the probabilistic calculations. There is not much information available about this coefficient, but this value is based on advice of the MSc thesis committee (2012).

4.7.1.6 *Interface (R_{int})*

As interface parameter for clay a value of 0.67 is used and for sand 0.9 (CUR C2003-7, 2003). For mixtures of sand and clay interpolation can be applied. The CoV is estimated to be 0.2, but no information about this coefficient is found.

4.7.1.7 *Summary of soil parameters*

Based on the above analysis and information from the JCSS (2002) and NEN 6740 (2006), the list of CoV's for soil parameters is presented in Table 4-4. In principal all parameters have normal distribution. However, for C lognormal is applied as otherwise negative values of C could occur in the lower tail of the normal distribution. C is relatively close to zero and has a large scatter. Also for the stiffness parameters a lognormal distribution is used, according to their derivation from CUR 166 (2005).

Parameter	**Symbol**	**Unit**	**Distribution**	**V=σ/μ**
Unsaturated soil weight	γ_{unsat}	[kN/m^3]	Normal	0.05
Saturated soil weight	γ_{sat}	[kN/m^3]	Normal	0.05
Internal angle of friction	φ	[°]	Normal	0.2
Cohesion	C	[kPa]	Lognormal	0.8
Dilatancy angle	ψ	[°]	Normal	0.2
50% stiffness modulus	E_{50}	[kPa]	Lognormal	0.3
Oedometer loading modulus	E_{oed}	[kPa]	Lognormal	0.3
Unloading reloading modulus	E_{ur}	[kPa]	Lognormal	0.3
Amount of stress dependency (power)	m	[-]	Normal	0.2
Interface	R_{int}	[-]	Normal	0.2

Table 4-4 CoV's soil parameters

The unloading reloading modulus E_{ur} is defined differently in Prob2B. G_{ref} is asked as input, which can be obtained by: $G_{ref} = \dfrac{E_{ur}}{2(1+\nu)}$, in which ν is the Poisson ratio of the soil. As the Poisson ratio is taken deterministic the CoV of G_{ref} is the same as for E_{ur} (0.3).

Furthermore, the correlation coefficients between the parameters are summarized in the matrix in Table 4-5. They are derived from the database of Gemeentewerken (2003). An example of this derivation is given in Figure 4-7.

	γ_{unsat}	γ_{sat}	φ	C	ψ	E_{50ref}	E_{oedref}	E_{urref}	m	R_{int}
γ_{unsat}		1.0	0.5	-0.09		0.5	0.5	0.5		
γ_{sat}	1.0		0.5	-0.09		0.5	0.5	0.5		
φ	0.5	0.5		-0,65		0.25	0.25	0.25		
C	-0.09	-0.09	-0.65			0.12	0.12	0.12		
ψ										
E_{50ref}	0.5	0.5	0.25	0.12			1.0	1.0		
E_{oedref}	0.5	0.5	0.25	0.12		1.0		1.0		
E_{urref}	0.5	0.5	0.25	0.12		1.0	1.0			
m										
R_{int}										

Table 4-5 Correlation matrix soil parameters

4.7.2 Plate and anchor parameters

The axial stiffness (EA), flexural rigidity (EI) and specific weight (w) of the plates and the axial stiffness (EA) of the anchor are composed of basic random variables. The CoV's of these random variables can be found in literature. JCSS (2002) presents an overview which is used to derive the CoV's of Table 4-6. Per quay wall these coefficients are used in a Monte Carlo simulation to compose the CoV of the specific PLAXIS input parameters and their correlations (Appendix C). All structural parameters are assumed to have a normal distribution.

Parameter	Symbol	Unit	Distribution	V=σ/μ
Yield stress steel	f_y	[kPa]	Normal	0.07
Young's modulus steel combi-wall/sheet-pile	E_c	[kPa]	Normal	0.03
Specific weight steel combi-wall/sheet-pile	γ_s	[kN/m^3]	Normal	0.01
Inertial moment sheet-pile	I_s	[m^4]	Normal	0.04
Cross-sectional area sheet-pile	A_s	[m^2]	Normal	0.04
Diameter tubular pile	D_p	[m]	Normal	0.032
Wall thickness tubular pile	t_p	[m]	Normal	0.07
Young's modulus steel anchor	E_a	[kPa]	Normal	0.03
Diameter anchor	D_a	[m]	Normal	0.032

Table 4-6 CoV's plate and anchor parameters (JCSS, 2002)

The specific PLAXIS input parameters for plates in Prob2B differ from the user interface. Instead of EI and EA, G_{eq} (equivalent shear modulus) and d (equivalent thickness) are used. Their derivation from the basic random variables can be found in Appendix D.

4.7.3 Level parameters

Depending on the conditions in which the quay wall is operational the outside water levels can fluctuate. The levels should be taken stochastic as there is uncertainty in the governing water height. The groundwater level can fluctuate as well. For each case first the governing situation needs to be defined and thereafter the variation (standard deviation) of these governing levels needs to be estimated. This especially depends on the type of quay wall and soil conditions.
However, as a first attempt governing water levels are chosen deterministic as determined in the design report of the quay wall. In a later stage the levels are manually varied to check the influence of these variations on the results.

Furthermore, the ground level and the contract depth are stochastic. This is even harder to implement in the model as it requires regeneration of the mesh in PLAXIS. Therefore, the ground and bottom level are initially taken deterministic (the governing situation). In a later stage the levels are manually varied to check the influence of these variations on the results.

The water levels and retaining height are also included in a probabilistic Blum calculation in order to check their influence in a more convenient way. In Blum they can be included from the start of the calculation. The manual variations in PLAXIS assume that the design point is still the same (i.e. the design values of the other parameters do not change). This is not necessarily the case, but there is no way to find the correct design values without doing probabilistic calculations including these geometrical parameters.

4.7.4 Loads

Permanent, variable and exceptional loads can be distinguished (TAW, 2003):

Permanent:

- Dead weight structural elements
- Soil pressure
- (Ground) water pressure

Variable:

- Pressure differences due to water level differences
- Pressure differences due to wind waves
- Currents
- Ship waves
- Ship currents
- Hawser forces
- Fender forces
- Terrain/traffic load
- Crane loads
- Temperature load
- Wind pressure

Exceptional:

- Ship collision
- Earth quake
- Explosion
- Ice load
- Vandalism

The permanent loads are implicitly taken into account via the soil parameters and level parameters. This also holds for the pressure differences due to water level differences. Currents, Ship waves and Ship currents can be included via a change in outside water level. However, it can be assumed that these influences are small in a protected port. Temperature load can be accounted for by a prescribed displacement in PLAXIS. Wind pressures can be added as distributed loads. The values and variation of all loads depend on the specific situation.

As a start the loads are taken deterministic in the analysis, because it is not possible in Prob2B to make them stochastic. The characteristic load is the once in a lifetime load. The influence of variations on the output can be checked manually as can be done for the level parameters. Furthermore the influence of the loads can obtained from the probabilistic Blum calculations.

4.8 Limit State Functions (LSF)

In principle four Limit States (LS) are analyzed: anchor failure, wall failure in bending and soil mechanical failure in ULS and excessive deformations in SLS.

The LSF's of three mechanisms are straightforward:

- ULS: Anchor failure: $Z = f_y - \frac{F_{anchor}}{A_{anchor}}$
- ULS: Wall failure in bending: $Z = f_y - (\frac{M_{\max,wall}}{W_{wall}} + \frac{N_{\max,wall}}{A_{wall}})$

  (Assuming that M_{max} and N_{max} are at the same location in the wall, which is a conservative estimate)

- SLS excessive deformations $Z = \delta_{\max} - \delta_{occured}$

However, for the LSF soil mechanical failure the situation is different. Basically, soil mechanical failure can be analyzed in several ways:

1. Using φ-C reduction and subsequently the LSF: Z = MSF – 1.0
2. Multiplying all φ and C with a factor 2.0 and using φ-C reduction. The LSF is: Z = MSF – 2.0.
3. Deriving a LSF based on the mobilized shear resistance $\tau_{mob} = \tau_{max}/\tau_{yield}$
4. Defining a LSF using the PLAXIS definition of a soil body collapse during calculations (option ValidCalc)

The methods and their difficulties are briefly described below.

1) MSF 1.0

The idea is that the point is found where the remaining safety in the soil shear strength is zero. This can be done by adding an additional phase (after the final phase) with a φ-C reduction. Prob2B needs to find the design values of the parameters where this MSF = 1.0.

The analysis with this method however is difficult as the φ-C reduction is applied after a certain building stage. When a soil body fails during a stage the calculation is aborted, which implies that no LS evaluation is executed. Calculating a reliability index for the LSF Z = MSF - 1.0 is therefore impossible as the soil will collapse during the plastic calculations while a design point is iteratively found. It is only usable when a higher MSF is required, such that the φ-C reduction is needed to cause soil mechanical failure (for instance Z = MSF - 1.1) and the soil does not collapse in an earlier building stage. It is however unknown what the difference in reliability is between the MSF = 1.1 and MSF = 1.0. There is a gap in between these values as there is still some safety left in the soil.

2) MSF 2.0

The idea of this method is to artificially increase the shear strength of the soil by a factor 2.0 (i.e. 2*C and 2*tan(φ)). As φ-C reduction only influences φ and C this factor can be translated to the required MSF of 2.0. This trick makes it possible to get results, without getting failure in the plastic calculations (i.e. MSF's lower than 2.0 give also results). Prob2B has less trouble in finding the failure probability. This method however is not implemented in this research as it came up in a later stage of the research and it appeared to be difficult to get convergence in the probabilistic calculations (most probably due to instabilities in the φ-C reduction). This is however a method that is worthwhile to analyze in further research.

3) Mobilized shear resistance

The basic idea is to define the structure 'failed' when somewhere in the model an element exceeds the yield shear resistance (τ_{yield}). This is basically the checking of the mobilized shear resistance $\tau_{mobilized}$. The analysis using $\tau_{mobilized}$ is difficult as the LS definition is not straightforward. Basically it involves three questions:

- Where (in which cluster or node) should the LS be evaluated? This is presumed in this method to be known, however soil mechanical failure can occur in many different ways which makes this prediction difficult.
- Is it preferable to use a cluster observation or a single point evaluation? In case of a cluster it is less complicated to define the relevant cluster (however still problematic as explained in point 1), but the output is averaged over the cluster, which can underestimate the situation.
- What should be the LSF? The application of the Hardening Soil model makes it difficult to define the τ_{yield}, the yield shear resistance, because aspects like compression hardening are included in this model. The Mohr-Coulomb model would make this definition easier, but this is not applicable here.

4) ValidCalc

Another option is the application of the option 'ValidCalc'. Basically Prob2B gives Z = 1.0 for a correct calculation and Z = -1.0 for a failed calculation. The main problem with this method is that FORM is not applicable as the Z-function is discontinuous (Z = 1.0 for correct calculation, Z = -1.0 in case of soil body collapse (invalid calculation)). It is not possible to derive influence coefficients, because the derivative cannot be calculated. This implies that a Monte Carlo based method should be used, where no derivatives are used. The most suited method to this is Directional Sampling (DS). In this method it is evaluated if a directional sample is worthwhile to evaluate or not, i.e. whether a direction is important. This is efficient in cases where the LS evaluation is time consuming. For explanation of DS reference is made to Schweckendiek (2006). The main drawback is the large calculation time in comparison to FORM. For two or three variables the method works quite well, but in case of more parameters the order of magnitude of required calculations increases fast. Furthermore the derivation of influence coefficients is difficult for this type of probabilistic methods.

Both option 1 and 2 are reasonable approaches to use. Option 3 is too complicated as explained and option 4 requires too many calculations and gives difficulties in the determination of influence coefficients. It is decided to focus on option 1 (with a target MSF of 1.1 instead of the impossible 1.0), as this was the only φ-C reduction based method that was known at the start of the project and that gives reasonable converged solutions. The method is analyzed in section 7.4 as well.

4.9 Prob2B

The toolbox of TNO, Prob2B, is used to make probabilistic calculations in PLAXIS. Basically, this toolbox sends input parameters to PLAXIS according to the used probabilistic method (or other internal or external models) and evaluates the LSF using the output from the model. First the model needs to be defined. Furthermore, additional variables can be defined, for instance the yield stress of steel ($f_{y,steel}$) to identify the maximum anchor force. The next step is to define a LSF. This function can be composed of model output parameters or user defined variables or values.

Afterwards the relevant parameters can be made stochastic. The user can define the distribution and subsequently the parameters that belong to that distribution. He can also set correlations between variables.

The next step is the choice of a probabilistic method. In this research in principle FORM is used. Different settings are available. They influence the way in which the toolbox is searching for the design point (Z=0). In each step for each parameter the derivative dZ/du (u is the parameter value in U-space) is determined as necessary for the FORM calculations. The relaxation value determines whether the next step is exactly the calculated point (Relaxation value = 1) or a cautious step in between the old and new value (0 < Relaxation value < 1). The convergence criterion for Z holds: $\left|\frac{Z}{\left|\frac{dZ}{du}\right|}\right| < criterion$.

The β criterion is defined as: $\beta_{n+1} - \beta_n < criterion$

The perturbation value determines which part of the standard deviation is used to calculate the derivative. The perturbation method defines whether this derivative is taken on one or two sides of the point.

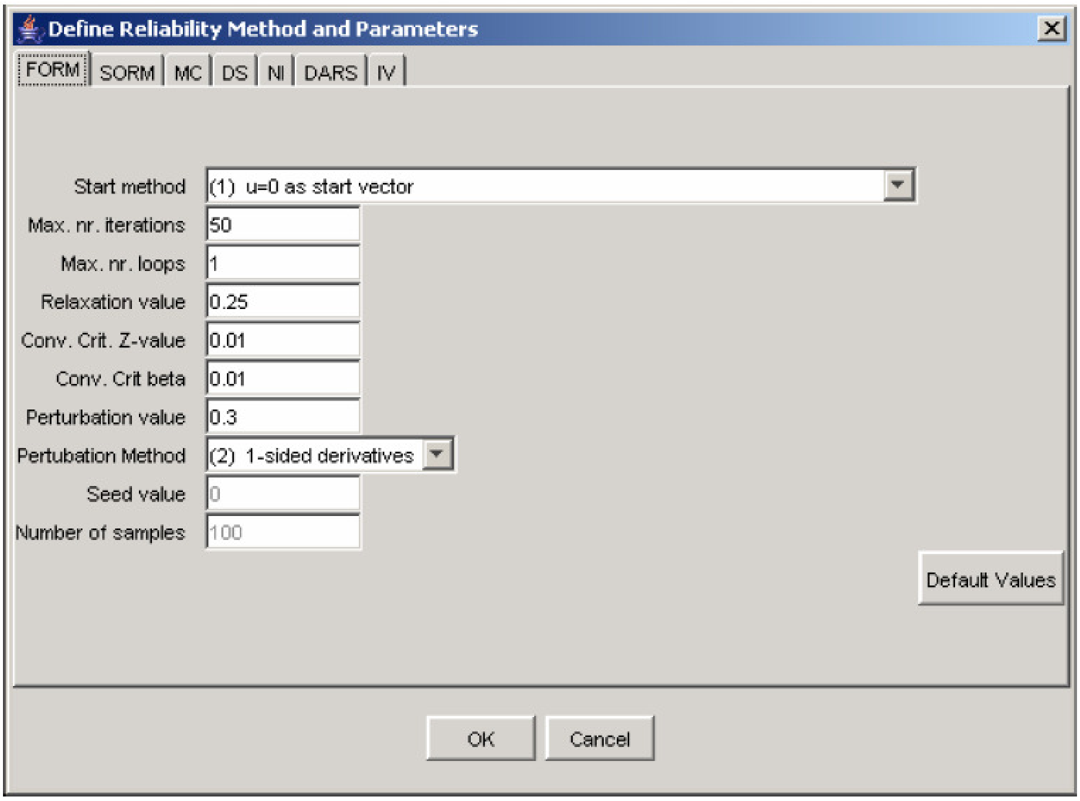


Figure 4-14 Settings FORM in Prob2B (Courage & Steenbergen, 2007)

In the Prob2B implementation the option ‘Evaluation Switch’ can be used. This is a ‘fake’ parameter which can be activated as parameter. If the user assigns value 1.0 it can be implemented as a trigger to search for the parameter value for which ValidCalc switches from 1.0 to -1.0, i.e. the parameter values for which the soil collapse just occurs. As explained before, this is just possible in combinations with DS.

When the value 2.0 is appointed to Evaluation Switch Prob2B evaluates the normal LSF in case ValidCalc = 1.0, otherwise it gives output -1.0. In this way FORM could still be used and wrong calculations are excluded. However there still might occur problems when 2 times -1.0 occurs in the FORM loop after each other, as the derivative cannot be calculated (dz = 0) in that case. However, the application of Evaluation Switch value 2.0 is useful when the design point of a mechanism (for instance wall failure in bending) is close to the design point of soil mechanical failure. The calculation will not immediately fail when a soil collapse accidentally occurs.

5 Anchored sheet-pile (benchmark 1)

5.1 Introduction

This chapter describes the analysis and results of the first benchmark quay wall, an anchored sheet-pile structure. This structure is first redesigned according to CUR 166 (5th edition) chapter 4 in section 5.3 in order to make comparisons between the design code and the calculation results possible. Furthermore, the parameter values are presented and the settings of Prob2B are discussed. As not all parameters can be included in the probabilistic analysis in PLAXIS, some calculations with Blum's method are performed to check the influence of these parameters on the reliability (section 5.6). In section 5.7 the eventual PLAXIS-Prob2B calculation results are presented for the different Limit States. From these results it turned out to be worthwhile to make calculations with an elongated wall. The results of these calculations are discussed in section 5.8.

5.2 Characteristics of the structure

The first benchmark quay wall is an anchored sheet-pile structure as shown in Figure 5-1. The design is presented in the report of Bakker and Jaspers Focks (2011). The structure has a reliability class II when following CUR 211 and a reliability class III according to CUR 166.

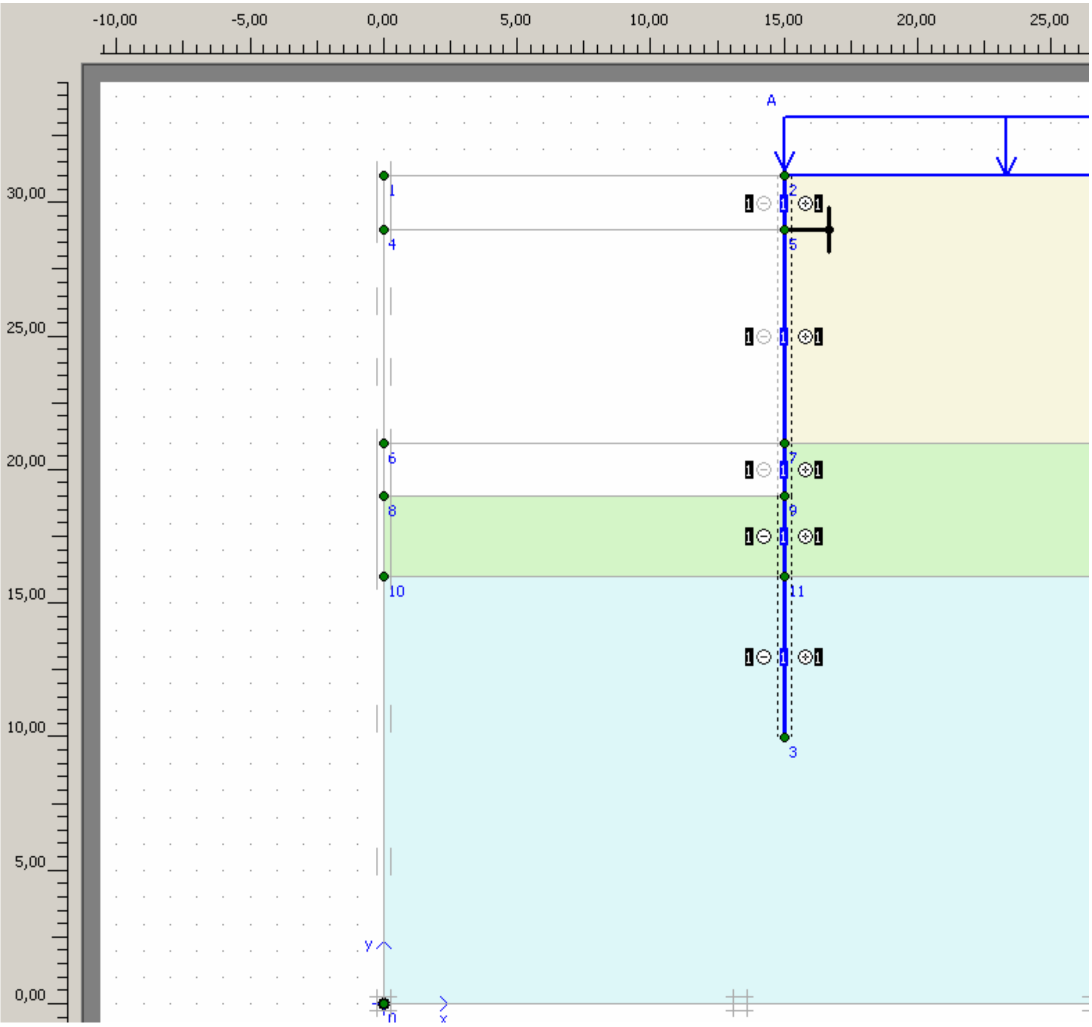


Figure 5-1 Benchmark 1: anchored sheet-pile

The characteristic soil parameter values are presented in Table 5-1. All soil stiffness parameters are determined at a reference level of 100 kPa. This is the case for all calculations in this chapter as PLAXIS uses this as standard input reference level.

Layer	**NAP**	**Ground level**	**Class**	**γ**	**C**	**φ**	**ψ**	**E_{50}**	**E_{oed}**	**E_{ur}**	**R_{int}**	**m**
[-]	*[m]*	*[m]*	*[-]*	*[kN/m³]*	*[kPa]*	*[°]*	*[°]*	*[MPa]*	*[MPa]*	*[MPa]*	*[-]*	*[-]*
1	0	0	moderately packed sand	17/19	1	32.5	2.5	45	45	135	0.9	0.5
2	-10	-10	moderate clay	16/16	10	22.5	0	5	3.4	10	0.67	1.0
3	-15	-15	densely packed sand	18/20	1	35	5	75	75	225	0.9	0.5

Table 5-1 Characteristic soil parameter values

The sheet-pile has an AZ48 profile and the anchor a diameter of 53 mm. The length of the sheet-pile is 21 m and the retaining height 12 m. The equivalent length of the fixed end anchor is 23 m (17 m of anchor element + half of the grout body (6 m)). On top of the structure and behind there is a surcharge load of 30 kN/m^2. Ground level is at NAP.

There are four building phases:

1. Flat ground level (initial water level is at NAP -4.0 m)
2. Placing the sheet-pile and excavation till NAP -2.0 m
3. Placing the anchor and excavation till NAP -12.0 m
4. Lowering the outside water level till NAP -6.0 m. The phreatric ground water level is at NAP -4.0 m. The deep sand layer has a hydraulic head of NAP -5.0 m. In the clay layer in between the sand layers the stress course is linear. Also the surcharge load becomes active.

However the last two stages are combined in the probabilistic calculations in order to reduce calculation time. It is confirmed that this adaptation has negligible influence on the PLAXIS output. The three building stages are therefore:

1. Flat ground level (initial water level is at NAP -4.0 m)
2. Placing the sheet-pile and excavation till NAP -2.0 m
3. Placing the anchor and excavation till NAP -12.0 m and lowering the outside water level till NAP -6.0 m. Also the surcharge load becomes active.

As first guess the configuration from Bakker and Jaspers Focks is used to perform the probabilistic calculations. However some problems occurred, that require a redesign of the structure. This is explained in the next section.

5.3 Redesigned anchored sheet-pile

The current configuration appeared not to be an optimal design with respect to material use and therefore costs. This became clear after performing the first probabilistic calculations in Prob2B. Wall failure in bending and anchor failure did never occur (i.e. their Limit States were never reached), as soil mechanical failure occurred before the Limit State was reached. Basically, the anchor and the wall are ‘too strong’. In practice a designer would not design the sheet-pile structure in this way as it is not the cheapest solution. Therefore the structure is redesigned in PLAXIS using the prescriptions from CUR 166 (2005). This gives the advantage that a comparison can be made between the reliability indices from the calculations and the indices that are prescribed in CUR 166.

CUR 166 chapter 4 prescribes to design the wall according to the maximum bending moment that is obtained from a φ-C reduction up to a MSF = 1.2 (safety class III). Subsequently the anchor has to be designed according to a maximum anchor force obtained at the same MSF = 1.2. Note that φ and C should have their characteristic value and the stiffness and geometrical parameters their design values. The geometrical design values are (according to CUR 166 table 3.7):

- Retaining height = NAP -12.35 m
- Ground water level low side = NAP -6.25 m
- Ground water level high side = NAP -3.95 m
- Surcharge load = 30 kN/m^2

For the soil stiffness parameters two options are given in CUR 166, a low and high design value:

$E_{d;low} = \dfrac{E_{k;low}}{1.3} = \dfrac{E_{\mu}}{2}$ and $E_{d;high} = \dfrac{E_{k;high}}{1.0} = 1.5E_{\mu}$

Both have been applied and the $E_{d,low}$ appeared to be governing for the bending moment in the wall and the anchor force. It is assumed that the sheet-pile parameters, anchor parameter, soil weight and dilatancy angle have their characteristic value as well, because CUR 166 defines a partial safety factor of 1.0 for these parameters. The input for the design calculation is given in Table 5-2.

Parameter	Type	Value	Unit
Soil			
$\gamma_{unsat,moderately\ packed\ sand}$	X_k	17	[kN/m^3]
$\gamma_{sat,moderately\ packed\ sand}$	X_k	19	[kN/m^3]
$\gamma_{sat,moderate\ clay}$	X_k	16	[kN/m^3]
$\gamma_{sat,densely\ packed\ sand}$	X_k	20	[kN/m^3]
$\varphi_{moderately\ packed\ sand}$	X_k	32.5	[°]
$\varphi_{moderate\ clay}$	X_k	22.5	[°]
$\varphi_{densely\ packed\ sand}$	X_k	35	[°]
$C_{moderate\ clay}$	X_k	10	[kPa]
$\psi_{moderately\ packed\ sand}$	X_k	2.5	[°]
$\psi_{densely\ packed\ sand}$	X_k	5	[°]
$E_{50,moderately\ packed\ sand}$	X_d	34.6	[MPa]
$E_{50,moderate\ clay}$	X_d	3.85	[MPa]
$E_{50,densely\ packed\ sand}$	X_d	57.7	[MPa]
$E_{oed,moderately\ packed\ sand}$	X_d	34.6	[MPa]
$E_{oed,moderate\ clay}$	X_d	2.62	[MPa]
$E_{oed,densely\ packed\ sand}$	X_d	57.7	[MPa]
$E_{ur,moderately\ packed\ sand}$	X_d	103.9	[MPa]
$E_{ur,moderate\ clay}$	X_d	7.69	[MPa]
$E_{ur,densely\ packed\ sand}$	X_d	173.1	[MPa]
AZ48			
EI_{AZ48}	X_k	2.4291*10^5	[kNm^2/m]
EA_{AZ48}	X_k	6.4365*10^6	[kN/m]
w_{AZ48}	X_k	2.406	[kN/m]
W_{AZ48}	X_k	4.800*10^{-3}	[m^3/m]
$f_{y,steel}$	X_k	355*10^3	[kN/m^2]
Anchor	X_k		
D_a	X_k	0.0528	[m]
EA_a	X_k	20000	[kN/m]
$f_{y,steel}$	X_k	355*10^3	[kN/m^2]

Table 5-2 Input parameters design calculation

When the deterministic calculation is made and φ-C reduction is applied the design bending moment in the wall at MSF = 1.2 is:

$M_d = 1117$ kNm/m

This implies that a sheet-pile profile needs to be chosen with a resisting moment of:

$$W = \frac{M_d}{\sigma_y} = \frac{1117}{355 \cdot 10^3} \approx 3.146 \cdot 10^{-3} \text{ m}^3$$

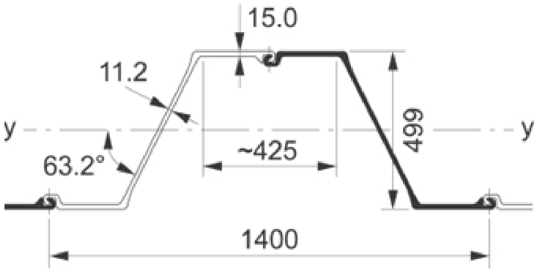


Figure 5-2 AZ36-700N profile (Arcelor Mittal, 2012)

Therefore a profile AZ36-700N (Figure 5-2) is taken with the following specifications:

Cross-sectional area	A	= 2.16*10^{-2}	m^2/m
Inertial moment	I	= 8.9610*10^{-4}	m^4/m
Resisting moment	W	= 3.590*10^{-3}	m^3/m
Weight	w	= 1.695	kN/m

This gives PLAXIS input parameters:

Flexural rigidity	EI	= 1.8816*10^5	kNm^2/m
Axial stiffness	EA	= 4.5339*10^6	kN/m
Weight	w	= 1.695	kN/m

The bending moment capacity, M_{max}, for this profile is 1274 kNm/m (without normal force and without plastic capacity of the steel).

The same can be done for the anchor. After φ-C reduction the design anchor force is found:

$F_{d,anchor} = 488$ kN/m

This implies that an anchor needs to be chosen with cross-sectional area:

$$A = \frac{F_{d,anchor}}{f_y} = \frac{488}{355 \cdot 10^3}$$

As the basic random variable is the anchor diameter, this can be translated to a required minimum diameter:

$$D = \sqrt{\frac{4A}{\pi}} \approx 0.0418 \text{ m}$$

An anchor diameter D_a of 42 mm is chosen.

This gives the PLAXIS input parameter:

Axial stiffness	EA_a[2]	=12650	kN/m

The maximum anchor force $F_{anchor,max}$ for this anchor is 492 kN/m.

In the probabilistic calculations however mean parameter values are used. A translation to mean values is therefore needed. For the soil parameters the coefficients of variation (CoV) in the design are based on NEN 6740 (2006), which implies that the mean value can be calculated from the formula:

[2] Actually this is the spring stiffness, which is k=EA/l , with l is the equivalent anchor length.

$$\mu_i = X_{i,k} + 1.64 \cdot V_{NEN6740,i} \cdot \mu_i$$

$$\mu_i = \frac{X_{i,k}}{1 - 1.64 V_{NEN6740,i}}$$

Except for the stiffness parameter in which the formula of CUR166 is used: $\mu_{Esoil} = \frac{2}{1.3} * E_{k;low,soil}$

For the structural parameters the CoV's are based on JCSS (2002) in combination with Monte Carlo simulations (Appendix C). The mean value follows from:

$$\mu_i = \frac{X_{i,k}}{1 - 1.64 V_{JCSS,i}}$$

All mean values are given in Table 5-3. Note that cohesion (C) of the sand layers is not included in this analysis, because their value (1.0) is in reality even smaller. It is therefore assumed that C of sand does not influence the probability of failure. In fact it is predictable that this assumption holds, because there is dealt with a relative large retaining height (12 m). This implies that the shear stress will be mainly determined by φ, because this parameter is stress dependent and therefore depth dependent ($\tau = C + \sigma' \tan(\varphi)$). C is only important in case of a small retaining height or a large C value (for instance in clay layers).

Parameter	**Unit**	$X_{char,i}$	$V_{NEN6740/JCSS}$	μ_i
Soil				
$\gamma_{unsat,moderately\ packed\ sand}$	[kN/m^3]	17	0.05	18.5
$\gamma_{sat,moderately\ packed\ sand}$	[kN/m^3]	19	0.05	20.7
$\gamma_{sat,moderate\ clay}$	[kN/m^3]	16	0.05	17.4
$\gamma_{sat,densely\ packed\ sand}$	[kN/m^3]	20	0.05	21.8
$\varphi_{moderately\ packed\ sand}$	[°]	32.5	0.1	38.9
$\varphi_{moderate\ clay}$	[°]	22.5	0.1	26.9
$\varphi_{densely\ packed\ sand}$	[°]	35	0.1	41.9
$C_{moderate\ clay}$	[kPa]	10	0.2	14.8
$\psi_{moderately\ packed\ sand}$	[°]	2.5	0.1	3.0
$\psi_{densely\ packed\ sand}$	[°]	5	0.1	6.0
$E_{50,moderately\ packed\ sand}$	[MPa]	45	CUR 166	69.2
$E_{50,moderate\ clay}$	[MPa]	5	CUR 166	7.69
$E_{50,densely\ packed\ sand}$	[MPa]	75	CUR 166	115.4
$E_{oed,moderately\ packed\ sand}$	[MPa]	45	CUR 166	69.2
$E_{oed,moderate\ clay}$	[MPa]	3.4	CUR 166	5.27
$E_{oed,densely\ packed\ sand}$	[MPa]	75	CUR 166	115.4
$E_{ur,moderately\ packed\ sand}$	[MPa]	135	CUR 166	207.7
$E_{ur,moderate\ clay}$	[MPa]	10	CUR 166	15.38
$E_{ur,densely\ packed\ sand}$	[MPa]	225	CUR 166	346.2
AZ36-700N				
$EI_{AZ36\text{-}700N}$	[kNm^2/m]	$1.8816*10^5$	0.05	$2.0497*10^5$
$EA_{AZ36\text{-}700N}$	[kN/m]	$4.5339*10^6$	0.05	$4.9389*10^6$
$w_{AZ36\text{-}700N}$	[kN/m]	1.695	0.04	1.8140

$W_{AZ36-700N}$	[m^3/m]	3.590*10^{-3}	0.04	3.842*10^{-3}
$f_{y,steel}$	[kN/m^2]	355*10^3	0.07	401*10^3
Anchor				
D_a	[m]	0.042	0.032	0.0443
EA_a	[kN/m]	12650	0.07	14291
$f_{y,a}$	[kN/m^2]	355*10^3	0.07	401*10^3

Table 5-3 Mean parameter values redesigned benchmark 1

For the interface condition R_{int} and power m it is assumed that the characteristic values equal the mean values, as it is not common to work with characteristic values for those parameters. The values are given in Table 5-4.

Parameter	Unit	μ
$R_{int,moderately\ packed\ sand}$	[-]	0.9
$R_{int,moderate\ clay}$	[-]	0.67
$R_{int,densely\ packed\ sand}$	[-]	0.9
$m_{moderately\ packed\ sand}$	[-]	0.5
$m_{moderate\ clay}$	[-]	1.0
$m_{densely\ packed\ sand}$	[-]	0.5

Table 5-4 Mean values R_{int} and m

5.4 Probabilistic input

The coefficients of variation (CoV) for soil parameters for the probabilistic calculations are obtained from Table 4-4 in section 4.7.1.7. However for the angle of internal friction (φ), Prob2B asks the input of sin(φ) instead of φ. When translating this by Monte Carlo (MC) simulations (Appendix C) sin(φ) has a CoV of 0.18, which is slightly lower than the 0.2 for φ. Another note is that Prob2B uses the shear modulus of the soil, G, instead of unloading-reloading stiffness modulus $E_{ur.}$. The relation between those parameters is: $G = \frac{E_{ur}}{2(1+\nu)}$, with ν is the Poisson ratio, which is assumed to be 0.2 for all soil layers. Also for the specific Prob2B structural parameters, G_{eq} and d (Appendix D) MC simulations are executed. Their CoV's are presented in Table 5-5.

Parameter	μ	Unit	V = σ / μ
Sheet-pile (AZ36-700N)			
$G_{eq,AZ36-700N}$	2691759	[kN/m^2/m]	0.07
$d_{AZ36-700N}$	0.706	[kN/m]	0.03
$w_{AZ36-700N}$	1.8140	[kN/m]	0.04
$W_{AZ36-700N}$	3.842*10^{-3}	[m^3/m]	0.04
$f_{y,steel}$	401*10^3	[kN/m^2]	0.07
Anchor			
D_a	0.0443	[m]	0.032
EA_a	14291	[kN/m]	0.07
$f_{y,steel}$	401*10^3	[kN/m^2]	0.07

Table 5-5 CoV's structural parameters

The correlations of the soil parameters can be found in Table 4-5 in section 4.7.1.7. Table 5-6 and Table 5-7 present the obtained correlations for the sheet-pile and anchor parameters.

	$G_{eq,AZ36\text{-}700N}$	$d_{AZ36\text{-}700N}$	$w_{AZ36\text{-}700N}$	$W_{AZ36\text{-}700N}$	$f_{y,steel}$
$G_{eq,AZ36\text{-}700N}$		-0.81	0.83	-0.29	0.00
$d_{AZ36\text{-}700N}$	-0.81		-0.69	0.71	0.00
$w_{AZ36\text{-}700N}$	0.83	-0.69		0.00	0.00
$W_{AZ36\text{-}700N}$	-0.29	0.71	0.00		0.00
$f_{y,steel}$	0.00	0.00	0.00	0.00	

Table 5-6 Correlation matrix sheet-pile parameters (AZ36-700N)

	EA	D_a	$f_{y,steel}$
EA_a		0.91	0.00
D_a	0.91		0.00
$f_{y,steel}$	0.00	0.00	

Table 5-7 Correlation matrix anchor parameters (D=0.042 m)

Variations in the retaining height, water level (inside and outside) and surcharge load cannot be included in the probabilistic calculation in Prob2B. These parameters are therefore included manually after the calculations in Prob2B. Variations of these parameters in Prob2B's design point are made in order to derive influence factors for these additional parameters. The CoV's are based on the research of Havinga (2004), who made probabilistic calculations for an anchored sheet-pile to calibrate safety factors for CUR 166 (2005). Their standard deviation is given in Table 5-8. Note that the standard deviation for retaining height and water levels are used instead of the CoV, because it is more convenient to use absolute changes instead of relative changes for these case-dependent parameters.

The parameter values in the governing situation are maintained as 'mean value (μ)' in the manual variations because these values are used during the probabilistic calculations as well. The manual variations can provide conclusions about the influence of the variation in the governing situation on the reliability and not of the influence of the parameter in general as explained in section 4.6. Their derived absolute factor therefore cannot be used to define a factor to design the structure. The factor is based on the governing situation as implemented in the model. The governing or nominal situation should be defined by experienced engineers or by standardized design rules.

Parameter	**μ**	**Unit**	**σ/V**
Retaining height	12	[m]	0.25 (σ)
Water level difference	2	[m]	0.2 (σ)
Surcharge load	30	[kN/m^3]	0.3 (V)

Table 5-8 Additional parameter values and standard deviations

In the design point variations are made to obtain influence factors (according to FORM procedure). For every Limit State three variations are made:

- Retaining height +0.35 m (lowering excavation depth)
- Water level inside +0.05 m and outside -0.25m
- Surcharge load +3 kN/m^2

These variations are based on the design values according to CUR 166 (table 3.7). In fact for the retaining height and water levels a variation of more than one standard deviation is applied, but this does not matter because the dZ/dx is used, which is independent of the step size. Off course there are some differences when taking different step sizes as linearization is applied, which is basically not correct, but unavoidable when using FORM. The process of plastic deformation in soil for instance is non-linear.

An important note is that it is implicitly assumed that the design point (i.e. the design values of the other parameters) does not change when varying the additional parameters. In fact it is not possible to show that this assumption is right, because the additional parameters should be included from the start of the probabilistic calculation and this is exactly the impossibility of Prob2B.

The reliability indices that are obtained in the probabilistic calculation are influenced by the addition of the extra parameters. The new reliability index can be approximated by the formula: $\beta_{NEW} = \beta_{OLD} \cdot \sqrt{1-\alpha_1^2-\alpha_2^2-...-\alpha_n^2}$ in which α_i are the influence coefficients of the additional parameters. These coefficients can be derived from the variation in governing water levels, retaining height and surcharge load. The probabilistic Blum calculations (section 5.6) are made in order to check the influence of the additional parameters as in these calculations all parameters can be included from the start. These results can be used as comparison results to check the relevance of the results of the manual variation in PLAXIS.

5.5 Practical aspects and settings Prob2B

The calculations are first splitted up in several calculations for each Limit State Function (LSF). This is due to the relative long calculation time when all possible parameters are made stochastic in the same calculation. Try-outs were made with different settings and therefore it is more pragmatic to split up the calculations in three parts. The most relevant parameter from the first calculation is included in the second calculation in order to compare the relevance of the other parameters with respect to the most important parameter. In this way several parameters are eliminated from the final calculation. For the elongated sheet-pile wall (section 5.8) mostly all parameters were included in one large calculation as experience was build up during the original benchmark 1 calculations. From this first large calculation several parameters are eliminated. The criterion for a parameter to be eliminated is an influence on the reliability of less than 5% (α_i^2*100), but there are some exceptions that are explained when they come into play.

For soil mechanical failure the calculation is even further split up (into five calculations) as the procedure with φ-C reduction is sometimes difficult to implement in Prob2B. The process of φ-C reduction is not completely stable, the final MSF value fluctuates during the additional calculation steps in PLAXIS. This makes it more difficult for Prob2B to find convergence in these calculations. It can happen that FORM sends the calculation in the 'wrong direction' when a sudden fluctuation occurs. As more try-out calculations had to be carried out for this LS the number of parameters per probabilistic calculation was reduced.

The soil stiffness parameters have an important connotation. The correlation between the three parameters is 1, so they are completely correlated. Prob2B however has problems to show this in the output of the results. It turned out to be the case that only one of the three stiffness parameters was picked out to get an influence factor larger than zero, whereas the other parameters get an influence factor zero. It is however easy to see that all stiffness parameters should get the same influence as they are 100% correlated in the same layer. This is why in the definition of the final partial safety factors all stiffness parameters get the same partial safety factor.

Prob2B requires some specific FORM settings that are visible in Figure 5-3. Depending on the calculation different relaxation values are used. The relaxation value determines whether the next step is exactly the calculated point (relaxation value = 1) or a cautious step in between the old and new value (0 < relaxation value < 1). Sometimes the calculation needs more manual steering in form of a user defined start U-vector and a low relaxation value. Other times Prob2B can find the design point from the zero U-vector and with a high relaxation value. Most of the time 0.3, which is the default setting for relaxation factor, is maintained.

An important note with respect to the accuracy of the calculations is that in the first elimination calculations (to see whether parameters can be excluded from the analysis) the convergence criteria are taken 0.1. However, in the final calculation per LS these criteria are stricter, i.e. 0.01. This latter criterion is not always met in the calculation. In that cases no convergence of the FORM calculation was found by Prob2B. It is mentioned in the report when another (less strict) criterion is applied.

The perturbation factor is the part of the standard deviation that is used as the variation in the parameter in order to calculate the derivative for influence factor α. The perturbation factor is kept on its default value 0.3, just as the setting to take a 1-sided derivative. Due to non-linearities these settings can influence the result and therefore it is better to keep the default value for all calculations.

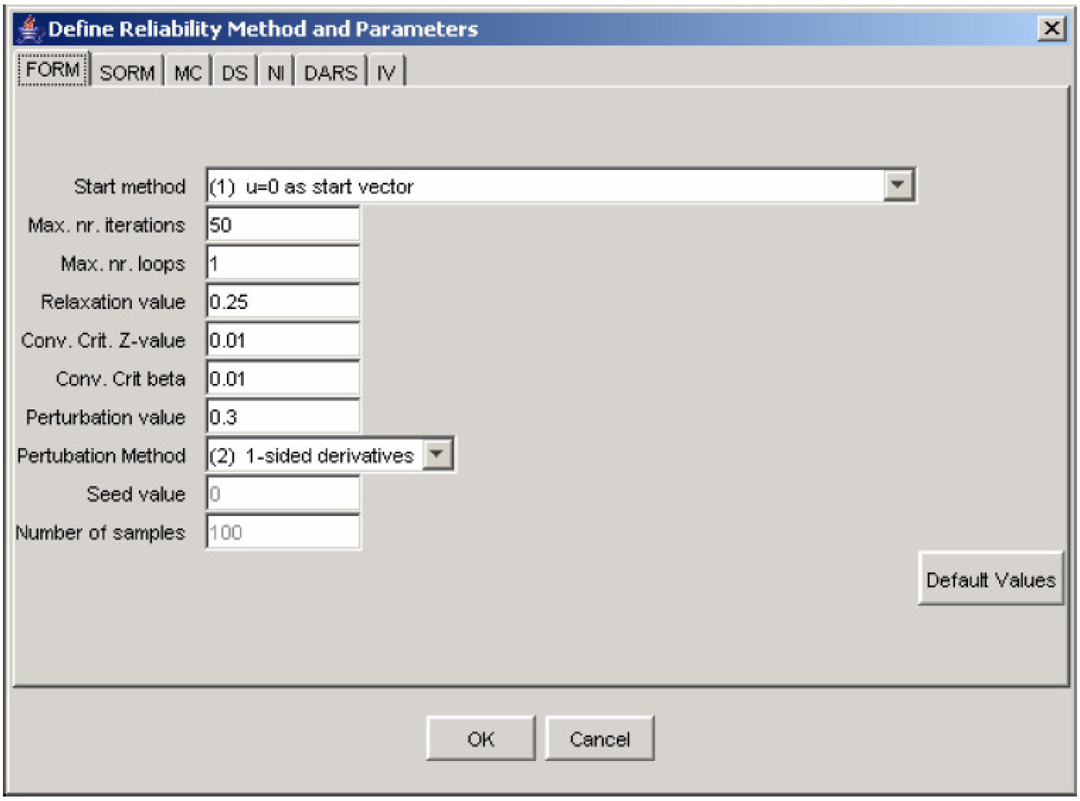


Figure 5-3 Settings FORM in Prob2B (Courage & Steenbergen, 2007)

5.6 Blum calculations

The main drawback of Prob2B in combination with PLAXIS is that the loads, geometrical parameters and water levels cannot be made stochastic in the probabilistic calculations. It is only possible to make some manual variations in the obtained design point, but it is not sure that the design point values for the other parameter stay the same. Therefore, a probabilistic calculation with Blum's method is done in order to make an estimate of the influence factors of the retaining height, water levels and surcharge load. This will be compared with the results of the probabilistic PLAXIS calculations.

The method of Blum also has its drawbacks. The bending stiffness of the wall and axial stiffness of the anchor are not included. Blum does not deal with elastical behaviour of the soil and assumes fully mobilized shear resistance in active and passive soil wedge. These are large simplifications compared to PLAXIS.

It is assumed that the additional parameters are normally distributed. The mean values, which are basically the values in the governing situation as explained before, are presented in Table 5-9. For the sake of completeness also the driving depth (sheet-pile length) is included.

Parameter	**μ**	**Unit**	**σ/V**
Retaining height	12	[m]	0.25 (σ)
Water level (outside)	-6	[m] NAP	0.2 (σ)
Water level (ground)	-4	[m] NAP	0.2 (σ)
Surcharge load	30	[kN/m^2]	0.1 (V)
Maximum length sheet-pile (driving depth)	21	[m]	0.25 (σ)

Table 5-9 mean value and standard deviations additional parameters

The calculations are executed using a 'computer script' presented in the soil mechanics book of Verruijt (2010). This is programmed in MatLab (Appendix F) and combined with the toolbox FERUM (Institut Français de Méchanique Advancé, 2012) to make probabilistic calculations. The same input is used as in the PLAXIS calculations. For the active and passive soil pressure coefficients the expressions of Kötter are used (section 2.3.1), which implies the use of curved slip planes.

Blum's method, however, follows an iterative procedure. The length of the wall is a variable and determined iteratively. In this benchmark calculation the wall length is in principal fixed to 21 m, but this is not possible in Blum's method. Therefore the reliability indices that are calculated can be overestimated as the calculated wall length may be longer than the actual presented length. It is however reasonable to assume that the influence factors will be more or less the same. The stochastic variable 'maximum length sheet-pile' therefore is only used in the LSF for soil mechanical failure. Three failure mechanisms are analyzed: wall failure in bending, anchor failure in tension and soil mechanical failure.

- Anchor failure in tension, LSF: $Z = \frac{1}{4}\pi D_a{}^2 f_y - F_{anch}$
- Wall failure in bending, LSF: $Z = Wf_y - M_{wall}$
- Soil mechanical failure, LSF: $Z = length_{sheet-pile,\max} - length_{sheet-pile,Blum}$

The iterative character of the wall length is used in the last Limit State. It is no problem when the sheet-pile is too long, but failure occurs when the sheet-pile is too short.

5.6.1 Calculation results

The calculation results can be found in Appendix G.

5.6.2 Conclusion

It can be concluded from the Blum calculations that the variations in the parameters that cannot be included in the PLAXIS-Prob2B analyses (i.e. bottom level, water level and load variations) are not relevant in the probabilistic calculations. This holds for the mechanisms anchor failure, wall failure in bending and soil mechanical failure. For the excessive deformations in SLS this cannot be verified as Blum is not suited to calculate deformations. It should be hold in mind that in fact the sheet-pile in the probabilistic calculations is too short, shown by the low reliability with respect to soil mechanical failure.

To make an additional check, the additional parameters are also manually verified in PLAXIS (in the design point). It should be kept in mind that the obtained reliability indices can be overestimated as Blum works with an iterative procedure that changes the sheet-pile length, while the benchmark sheet-pile has a fixed length.

The small influence of the additional parameters show that it is reasonable to use their 'governing value' in the probabilistic calculations, remembering that the obtained β is specifically related to the governing situation that is defined on beforehand. In order to define (absolute) partial factors for these parameters, the manual variations nevertheless are made in PLAXIS.

5.7 Limit state evaluations PLAXIS-Prob2B

The four relevant Limit States (LS) are evaluated in this section. Note that the first LS evaluation contains the most extensive explanation. The text in the other LS evaluations refers to these explanations.

5.7.1 Anchor failure (ULS)

The LSF for anchor failure in tension is: $Z = \frac{1}{4}\pi D_a{}^2 f_y - F_{anch,stage3}$

The total number of stochastic parameters equals 31. Therefore the probabilistic calculation is split up in three calculations in order to reduce the calculation time. These three calculation results can be found in Appendix H.

5.7.1.1 Final calculation

From the three calculations of Appendix H it can be concluded that not all parameters are relevant for the reliability of the structure with respect to anchor failure. The remaining parameters are:

1. D_a
2. $f_{y,steel}$
3. EA_a
4. $\sin(\varphi)_{moderately\ packed\ sand}$
5. $\sin(\varphi)_{densely\ packed\ sand}$
6. $E_{oed,moderately\ packed\ sand}$
7. $G_{densely\ packed\ sand}$
8. $\gamma_{unsat,\ moderately\ packed\ sand}$

D_a has a small influence (<5%) on the reliability, but is still included. This is actually not necessary, but it also does not influence the calculation by much. The output is given in Table 5-10 and Figure 5-4 gives the influence in % on the reliability (α_i^2*100). Normally convergence criteria for β and Z of 0.01 are applied for the calculations, but in this case 0.05 is used. The reason for this less strict criteria is the fact that Prob2B has troubles in finding convergence for this configuration.

Number of calculations (FORM): 251				
β: 3.597				
P_f: $1.610*10^{-4}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
D_a	0.032	0.06	0.0424	[m]
$f_{y,steel}$	0.07	0.31	369300	[kN/m^2]
EA_a	0.07	0.37	13030	[kN/m]
$E_{oed,moderately\ packed\ sand}$	0.3	0.20	67460	[kPa]
$G_{densely\ packed\ sand}$	0.3	0.23	114900	[kPa]
$\sin(\varphi)_{moderately\ packed\ sand}$	0.18	0.58	0.31	[-]
$\sin(\varphi)_{densely\ packed\ sand}$	0.18	0.19	0.56	[-]
$\gamma_{unsat,moderately\ packed\ sand}$	0.05	0.54	17.61	[kN/m^3]
calc.	Z-value			
1	318.70			
251	-0.11	*convergence criteria 0.05*		

Table 5-10 Output final calculation (LS Anchor)

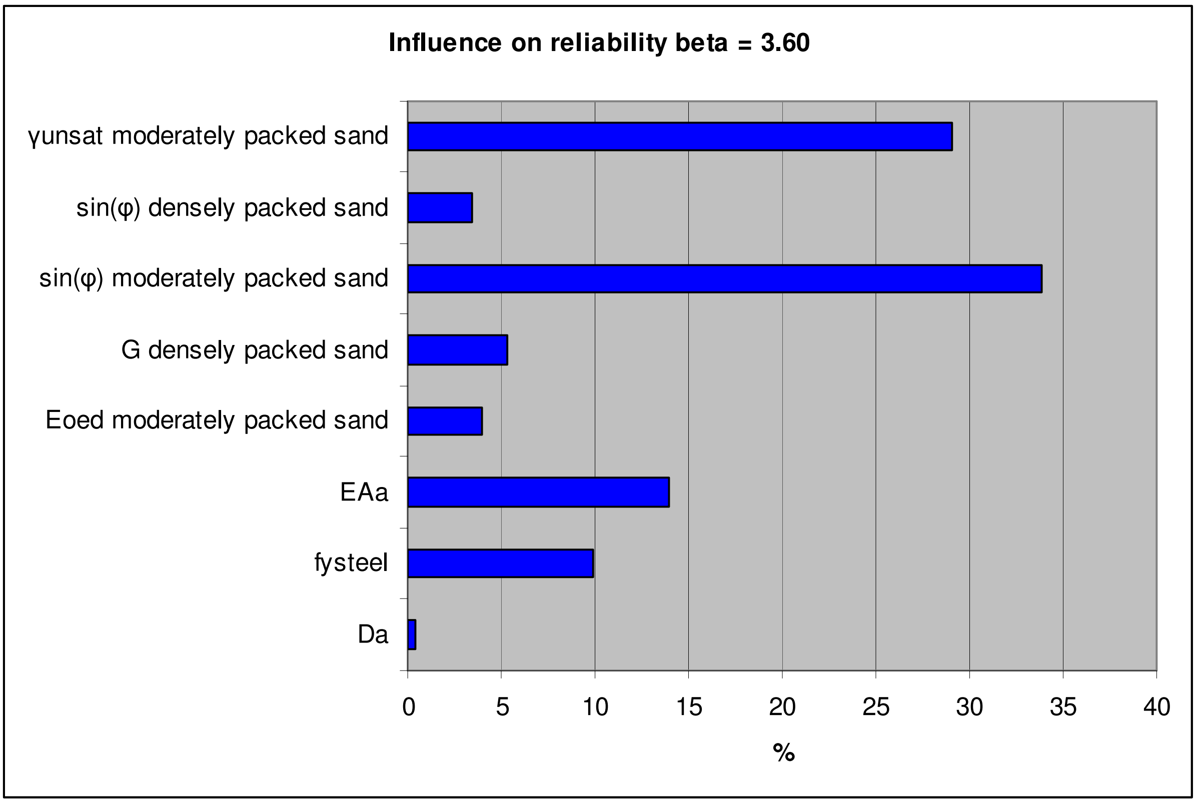


Figure 5-4 Influence in % from parameters on reliability final calculation (LS Anchor)

Figure 5-5 shows the plastic points in the design point calculation. The red blocks are points in the soil where local soil failure occurs due to the exceedance of the Mohr-Coulomb maximum shear stress criterion ($\tau = C + \sigma' \tan(\varphi)$). The red spots lay on the Mohr-Coulomb failure envelope. A blue spot (cap point) represents a state of normal consolidation where the preconsolidation stress is equivalent to the current stress state. The green and green/blue spots show the soil elements where shear hardening (according to the hardening soil model) occurs. The green and green/blue spots lay on the shear hardening envelope (mobilized friction envelope). The white elements are elements where the tension cut-off criterion (i.e. no tensile stresses allowed) is applied.

From the figure it can be seen that anchor failure is mainly induced due to failure of the soil in the moderately packed sand layer, i.e. the layer in which the anchor is anchored. Part of the anchor failure is the failure in the passive wedge behind the wall, which causes the wall to move and the anchor force to increase in order to maintain horizontal equilibrium. Furthermore the specific anchor parameters, axial stiffness and yield stress of the steel, reduce the maximum allowable anchor force. The relative importance of γ_{unsat} of the moderately packed sand layer with respect to anchor failure is caused by the correlation between φ and γ_{unsat} (0.57). When γ_{unsat} is reduced φ follows.

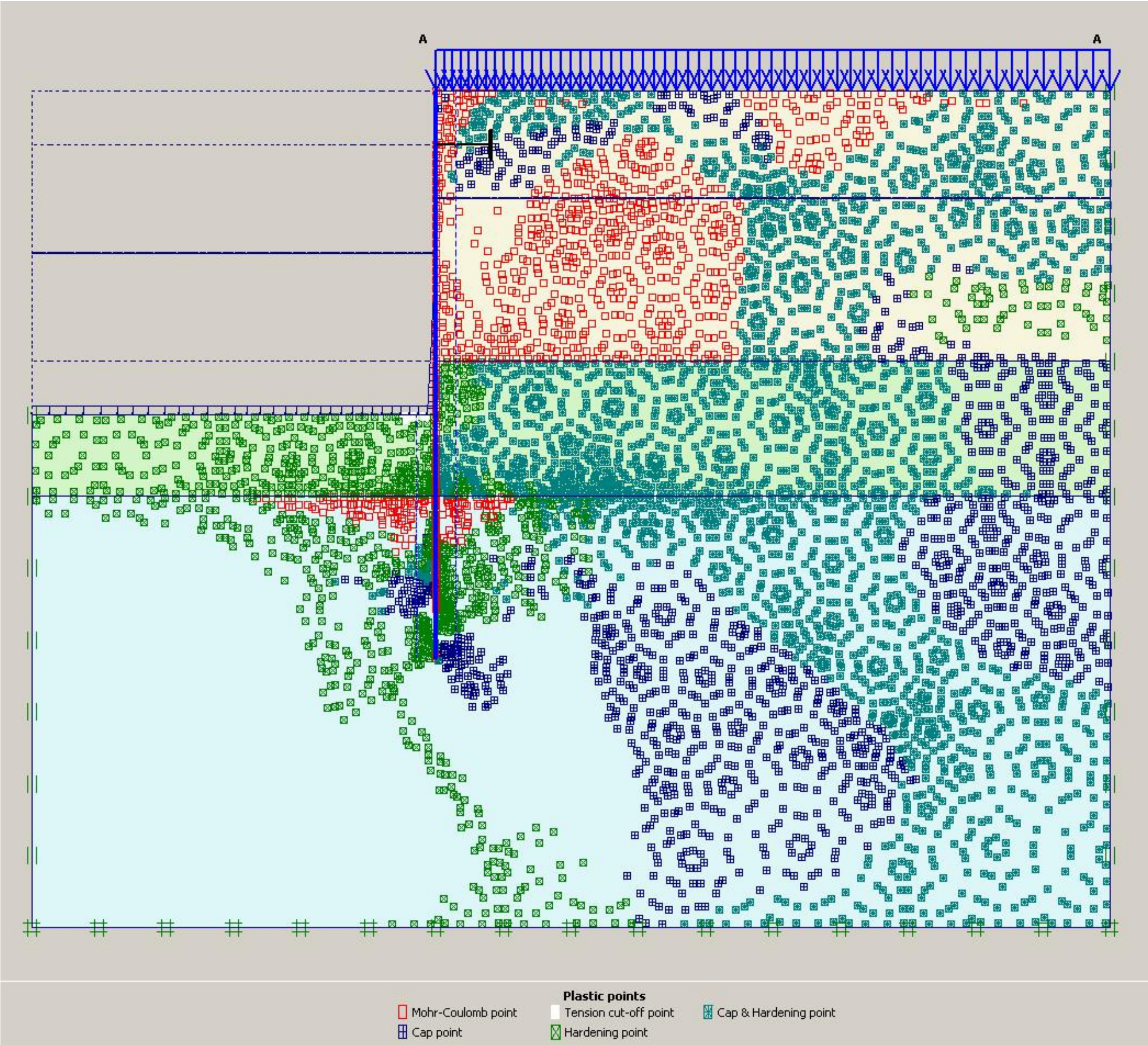


Figure 5-5 Plastic points in design point anchor failure

5.7.1.2 *Partial safety factors*

From these results partial safety factors can be derived, by applying the formula: $\gamma_{\mu,i} = \dfrac{\mu_i}{\mu_i - \alpha_i \beta \sigma_i}$

This is done both for the β obtained in the calculation as well as for the target β defined in the failure tree of CUR 211 (Figure 3-2, section 3.3.2).

Furthermore a translation is made to a safety factor over the characteristic value, by the following formula: $\gamma_{k,i} = \gamma_{\mu,i} * (1 - 1.64 V_{NEN6740/JCSS,i})$

(for the lognormal distributed soil stiffness parameters $\gamma_{k,i} = \gamma_{\mu,i} * (1.3/2)$)

In this formula the coefficient of variation from the guidelines is used again, as this was originally used to define the characteristic value in the document of Bakker and Jaspers Focks (2011).

It is important to note that due to non-linearity's the following equation <u>does not hold</u> for every parameter:

$$\frac{\mu_i}{X_i^*} = \frac{1}{1-\alpha_i \beta V_i}$$

In the FORM calculation it is assumed that the α coefficient can be obtained by linearizing in the design point. However, the process of plastic soil development is (highly) non-linear. Therefore the α factors are under- or overestimated, which leads to two different safety factors, one based on the left and one based on the right hand side of the formula. The difference is also caused by the correlations between parameters. There are two different safety factors, one is based on the left hand side of the equation and the other on the right hand side. The relevant safety factors are summarized in Table 5-11. Note that the sin(φ) is translated to the more commonly used φ.

As the use of FORM is a starting point of this research the safety factors with use of α are used to define the partial safety factors. This is important, because the partial safety factors need to be translated to a certain target reliability index. This is not possible with the situation specific formula μ/X^*. Table 5-11 however does show the troubles that occur when using FORM. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix I. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 3.597						
β_{CUR211} = 3.828						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
D_a	1.01	0.96	1.05	0.99	1.01	0.96
$f_{y,steel}$	1.09	0.97	1.09	0.96	1.09	0.98
EA_a	1.10	0.99	1.10	0.97	1.11	1.00
$E_{oed,moderately\ packed\ sand}$	1.28	0.83	1.03	0.67	1.30	0.84
$G_{densely\ packed\ sand}$	1.33	0.86	1.26	0.82	1.36	0.88
$\varphi_{moderately\ packed\ sand}$	1.72	1.48	2.15	1.79	1.80	1.55
$\varphi_{densely\ packed\ sand}$	1.15	0.99	1.24	1.03	1.17	1.00
$\gamma_{unsat,moderately\ packed\ sand}$	1.11	1.02	1.05	0.97	1.12	1.03
Retaining height (Δ)	0.11	0.11	-	-	0.12	0.12
Water level (Δ)	0.07	0.07	-	-	0.08	0.08
Surcharge load	1.04	1.04	-	-	1.04	1.04

Table 5-11 Partial safety factors anchor failure

5.7.1.3 Comparison with Blum calculations

Table 5-12 shows the calculation results for the two different methods, Blum and PLAXIS. The reliability index when using Blum is higher. This can be explained by the fact that the sheet-pile length in the Blum calculation can become larger than the original 21 m. The sheet-pile is more fixed and parameters need to be reduced more in order to get a high enough anchor force. The sheet-pile has a lower failure probability when it is longer.

In PLAXIS the design value of φ of the moderately packed sand layer is 16°, whereas in the Blum calculation φ is reduced till the unrealistic low value of 5 °. In PLAXIS such a low value is not possible due to internal equilibrium requirements. When a sand layer has a very low internal angle of friction the soil body will collapse internally and PLAXIS stops the calculation. This mainly explains the difference in α_φ.

$\alpha_{\gamma unsat,moderately\ packed\ sand}$ in the PLAXIS calculations is much higher than for Blum. This can be explained in the way the influence factors are deduced. Both methods include the correlation of 0.57 between γ and φ. PLAXIS-Prob2B increases the γ with 0.3σ to calculate the derivative dZ/dx and to obtain the influence factor. When γ is increased, φ is also increased and therefore the influence factor α_γ is relatively high because φ has a large influence on the reliability. γ has only influence via φ. In the

Blum probabilistic calculations in FERUM γ is also increased with 0.3σ to calculate the derivative dZ/dx and to obtain the influence factor. However, φ does not increase with γ. In the determination of α_γ the parameters are 'assumed to be uncorrelated'. The correlation in FERUM is only used in the calculation of the failure probability and the search for the design point and not in the determination of α.

The other parameters have quite similar influence factors for Blum and PLAXIS. Relatively small discrepancies occur due to the fact that in Blum the stiffness parameters of the soil and the axial stiffness of the anchor cannot be included. This implies that other parameters get more influence, i.e. the α factors are distributed differently. The Blum calculation results are only used as verification, whereas the PLAXIS calculation results are used to draw conclusions. From the comparison, however, it can be concluded that the general view on the situation is the same for PLAXIS and Blum calculations. φ of the upper sand layer is important and the variations in the governing situation of the level and load parameters are not very relevant with respect to the reliability.

Parameter	α_{BLUM}	α_{PLAXIS}
D_a	0.24	0.06
$f_{y,steel}$	0.28	0.31
EA_a	-	0.37
$E_{oed,moderately\ packed\ sand}$	-	0.20
$G_{densely\ packed\ sand}$	-	0.23
$\varphi_{moderately\ packed\ sand}$	0.88	0.58
$\varphi_{densely\ packed\ sand}$	0.16	0.19
$\gamma_{unsat,moderately\ packed\ sand}$	-0.19	0.54
Water level (outside)	0.08	0.12[3]
Water level (ground)	-0.05	0.12[3]
Surcharge load	0.06	0.10
Retaining height	0.05	0.11
	$\beta_{BLUM} = 4.49$	$\beta_{PLAXIS} = 3.60$

Table 5-12 Comparison Blum-PLAXIS

5.7.1.4 Conclusion

From the calculations for anchor failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads almost to the required reliability index of 3.872. However, the partial safety factors of chapter 3 to obtain this reliability index differ from the factors in the code. A comparison with CUR211 should be made, but both CUR 166 and CUR 211 are listed and compared with the partial safety factors in Table 5-13. It should be noted that the partial factors from the calculations are levelled up. All factors need to be applied over the characteristic value.

Parameter	**CUR166 (class III)**	**CUR211 (class II)**	**Calculations (anchor)**
D_a	1.00	1.10[4]	1.00
$f_{y,steel}$	1.00	1.00[4]	1.00
EA_a	1.00	1.20[4]	1.00
E_{50}	1.00	μ	0.85
E_{oed}	1.00	μ	0.85

[3] This is water level difference
[4] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force: ($F_{max} = A\sigma_y = 1/4\pi D_a^2\sigma_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on σ_y.

E_{ur}	1.00	μ	0.85
φ	1.20	1.00	1.55
C	1.10	1.00	μ
γ	1.00	μ	1.05
Water level difference (Δ)	0.30	0.80	0.10
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.15

Table 5-13 Partial safety factors for anchor failure and comparison with design codes

With respect to anchor failure therefore the partial safety factor on φ must increase to obtain the correct reliability. Furthermore, the soil stiffness parameters should increase to find the design value (characteristic value divided by 0.85). Some additional safety marge should be taken into account for the water levels and retaining height, 10 centimeter for the water level difference and 0.15 cm additional retaining height. These values are lower than CUR 166 and CUR 211 prescribe. The factor on the surcharge load lays in between the factor defined in CUR 166 and CUR 211.

5.7.2 Wall failure in bending (ULS)

The LSF for wall failure in bending is: $Z = Wf_y - M_{wall,stage3}$

Note that it is assumed there is no axial stress in the sheet-pile which is reasonable as there is no normal force working on the sheet-pile. The probabilistic calculation is again split up in three calculations in order to reduce the calculation time. These three calculation results can be found in Appendix H.

5.7.2.1 Final calculation

From the three calculations of Appendix H it can be concluded that not all parameters are relevant for the reliability of the structure with respect to wall failure. The remaining parameters are:

1. $\sin(\varphi)_{\text{moderately packed sand}}$
2. $\sin(\varphi)_{\text{densely packed sand}}$
3. $E_{\text{oed,moderately packed sand}}$
4. $G_{\text{densely packed sand}}$
5. $\gamma_{\text{sat,densely packed sand}}$
6. $\gamma_{\text{unsat,moderately packed sand}}$

The results are presented in Table 5-14 and Figure 5-6. 'Normally', convergence criteria for β and Z of 0.01 are applied for the calculations, but in this case 0.05 is used. The reason for this less strict criteria is the fact that Prob2B has troubles in finding convergence for this configuration. Most probably this is due to the fact that the failure mechanisms wall failure in bending and soil mechanical failure are closely related (i.e. both mechanisms occur due to large reduction of φ in the lower sand layer). It is difficult to find convergence when a soil collapse occurs around the design point of the bending moment.

Number of calculations (FORM): 211				
β: 3.777				
P_f: 7.925*10^{-5}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$E_{\text{oed,moderately packed sand}}$	0.3	0.15	63270	[kPa]
$G_{\text{densely packed sand}}$	0.3	0.50	97040	[kPa]
$\sin(\varphi)_{\text{moderately packed sand}}$	0.18	0.22	0.48	[-]
$\sin(\varphi)_{\text{densely packed sand}}$	0.18	0.72	0.25	[-]
$\gamma_{\text{sat,densely packed sand}}$	0.05	-0.33	19.41	[kN/m^3]
$\gamma_{\text{unsat,moderately packed sand}}$	0.05	0.24	17.90	[kN/m^3]

calc.	Z-value	
1	244,40	
211	-23,10	*convergence criteria 0.05*

Table 5-14 Output final calculation (LS Wall)

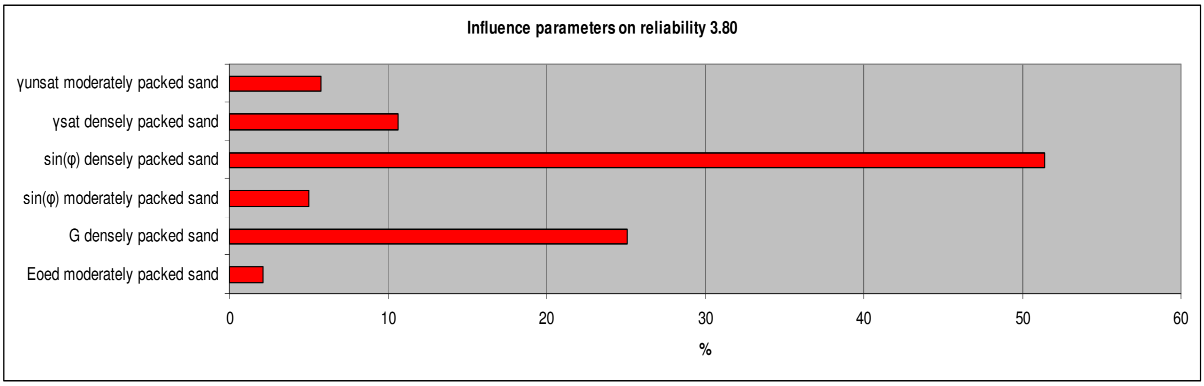


Figure 5-6 Influence in % from parameters on reliability final calculation (LS Wall)

Figure 5-8 shows the plastic points when the design values of the parameters with respect to wall failure in bending are used in the model. The main cause for the large bending moments in the wall is the reduction of the fixing moment at the bottom of the sheet-pile. This is caused by reduction of the internal angle of friction of the densely packed sand layer. When the fixing moment is reduced the field moment increases. Figure 5-7 shows the line of moments of the mean value calculation. Remember that the length of this line is 21 m (sheet-pile length). The maximum in the lower curved part of the line is the fixing moment and the maximum in the upper part the field moment. When the fixing moment is reduced by the fact that the pile is less clamped due to the reduction of the internal angle of friction, the field moment gets larger as the sheet-pile is more simply supported.

Additionally to this mechanism some soil failure in the upper two layers occurs, which increases the load that acts on the wall, causing higher bending moments in the wall.

Figure 5-7 Line of moments (mean values)

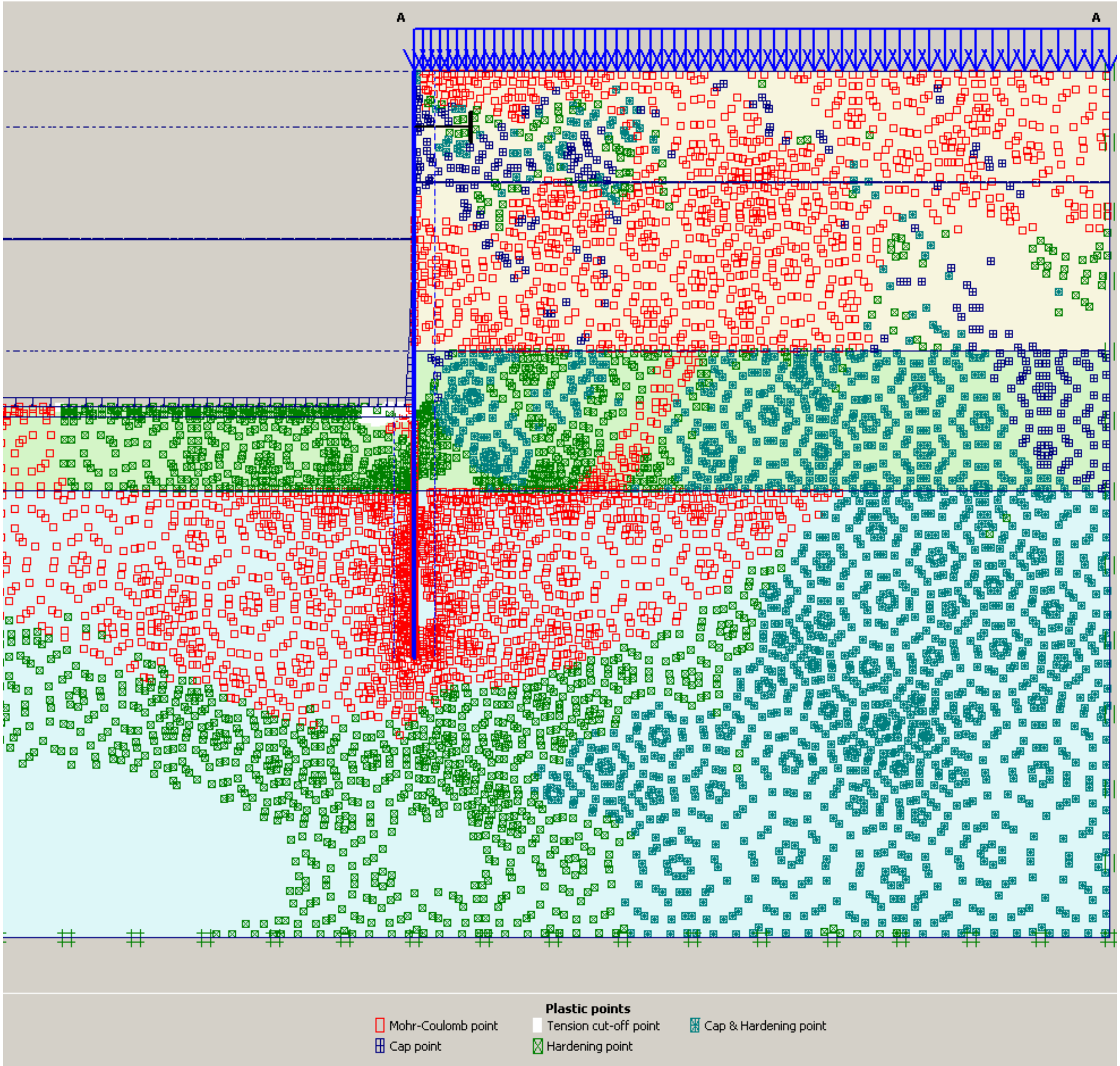


Figure 5-8 Plastic points design point wall failure

5.7.2.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 5.7.1.2. The different variants of safety factors are presented in Table 5-15. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix I. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 3.777						
β_{CUR211} = 3.872						
X_i	$\gamma_{\mu,i,calculation}$	$\gamma_{k,i,calculation}$	μ/X^*	X_k/X^*	$\gamma_{\mu,i,CUR211}$	$\gamma_{k,i,CUR211}$
$E_{oed,moderately\ packed\ sand}$	1.20	0.78	1.09	0.71	1.20	0.78
$G_{densely\ packed\ sand}$	2.31	1.50	1.49	0.97	2.39	1.55
$\varphi_{moderately\ packed\ sand}$	1.20	1.01	1.37	1.14	1.21	1.01
$\varphi_{densely\ packed\ sand}$	2.18	1.82	2.94	2.45	2.25	1.88
$\gamma_{sat,densely\ packed\ sand}$	0.94	0.86	1.12	1.03	0.94	0.86
$\gamma_{unsat,moderately\ packed\ sand}$	1.05	0.96	1.03	0.95	1.05	0.96
Retaining height (Δ)	0.05	0.05	-	-	0.05	0.05
Water level (Δ)	0.04	0.04	-	-	0.04	0.04
Surcharge load	1.02	1.02	-	-	1.02	1.02

Table 5-15 Partial safety factors wall failure

5.7.2.3 Comparison with Blum calculations

Table 5-16 shows the calculation results for the two different methods, Blum and PLAXIS. The reliability index when using Blum is lower. This is most probably due to the simplified model of Blum which does not take into account effects like arching which reduces the load on the wall and therefore the bending moment. This implies that for Blum the structure gets its maximum moment in an earlier stage than for PLAXIS.

$\alpha_{\varphi,densely\ packed\ sand}$ is in the PLAXIS calculations lower than in the Blum calculations. This is caused by the relative importance of the stiffness parameter G of the same soil layer. This parameter cannot be included in the Blum calculations and therefore more importance is going to φ. γ_{sat} has a larger influence due to its correlations with both G and φ of the same layer.

The other parameters have quite similar influence factors for Blum and PLAXIS. The Blum calculation results are only used as verification. The PLAXIS calculation results are used to draw conclusions. From the comparison, however, it can be concluded that the general view on the situation is the same for PLAXIS and Blum calculations. φ is important and the variations in the governing situation of the level and load parameters is not very relevant with respect to the reliability.

Parameter	α_{BLUM}	α_{PLAXIS}
$E_{oed,moderately\ packed\ sand}$	-	0.15
$G_{densely\ packed\ sand}$	-	0.50
$\varphi_{moderately\ packed\ sand}$	0.19	0.22
$\varphi_{densely\ packed\ sand}$	0.85	0.72
$\gamma_{sat,moderately\ packed\ sand}$	-0.01	-0.33
$\gamma_{sat,densely\ packed\ sand}$	0.05	0.24
$\gamma_{unsat,moderately\ packed\ sand}$	0.10	0.31
Water level (outside)	0.20	0.05[5]
Water level (ground)	-0.04	0.05[5]
Surcharge load	0.09	0.06
Retaining height	0.13	0.04
	β_{BLUM} = 3.15	β_{PLAXIS} = 3.78

Table 5-16 Comparison Blum-PLAXIS

[5] This is water level difference

5.7.2.4 Conclusion

From the calculations for wall failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads almost to the required reliability index of 3.828. However, the partial safety factors of chapter 3 to obtain this reliability index differ from this code. Basically, only a comparison with CUR 211 should be made, but for the sake of completeness both CUR 166 and CUR 211 are listed and compared with the partial safety factors in Table 5-17. It should be noted that the partial factors from the calculations are levelled up.

Parameter	CUR166 (class III)	CUR211 (class II)	Calculations (wall)
$EI_{sheet\text{-}pile}$	1.00	1.00	μ
$EA_{sheet\text{-}pile}$	1.00	1.00	μ
$w_{sheet\text{-}pile}$	1.00	1.00	μ
$W_{sheet\text{-}pile}$	1.00	1.00	μ
$f_{y,steel}$	1.00	1.00	μ
E_{50}	1.00	μ	1.55
E_{oed}	1.00	μ	1.55
E_{ur}	1.00	μ	1.55
φ	1.20	1.00	1.90
C	1.10	1.00	μ
γ	1.10	μ	0.95
Water level difference (Δ)	0.30	0.80	0.05
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.05

Table 5-17 Partial safety factors for wall failure comparison with design codes

With respect to wall failure therefore the partial safety factor on φ must increase to obtain the correct reliability. The same holds for the stiffness parameters of the soil. The sheet-pile parameters can have their mean value instead of the characteristic values. Some additional safety marge should be taken into account for the water levels and retaining height, both need an additional 5 cm. This is lower than the additional marges prescribed in CUR 166 and CUR 211. The factor on the surcharge load lays in between the factor defined in CUR 166 (1.00) and CUR 211 (1.20).

5.7.3 Soil mechanical failure (ULS)

The LSF for soil mechanical failure in its most correct form is: $Z = MSF - 1.0$

However, this LS is difficult to reach in Prob2B as it is very reasonable that soil collapse occurs in an earlier building stage (section 4.8). In that case the calculation is not finished, which causes the LS evaluation not to be performed. Therefore it is tried to define the LSF in a way that Prob2B gives results. Different options have been tried and the closest value of MSF that can be reached in the LS calculation appeared to be 1.1. Therefore the LSF is defined as: $Z = MSF - 1.1$

It is clear that the obtained β is in fact too low, there is some safety left in the difference between MSF = 1.1 and MSF = 1.0. It is assumed however that the distance between these two design points is small enough to have no influence on the influence factors α. Therefore still safety factors can be determined based on these calculations. In a later stage these assumptions are checked (section 7.4).

The probabilistic calculation is split up in five calculations in order to reduce the calculation time. The results of these first five calculations can be found in Appendix H.

5.7.3.1 Final calculation

From the five calculations in Appendix H it can be concluded that not all parameters are relevant for the reliability of the structure with respect to soil mechanical failure. The remaining parameters are:

1. $E_{50,\text{densely packed sand}}$
2. $E_{oed,\text{densely packed sand}}$
3. $G_{\text{densely packed sand}}$
4. $\sin(\varphi)_{\text{densely packed sand}}$
5. $\gamma_{sat,\text{densely packed sand}}$

The previous calculations showed that Prob2B presents that from the soil stiffness parameters only $G_{\text{densely packed sand}}$ is relevant, but it appeared to be necessary to include all stiffness parameters as they are correlated. Without the other stiffness parameters the calculations had problems in convergence. Two of the three stiffness parameters however still 'do not influence' the reliability according to the Prob2B output, because all stiffness parameters are fully correlated with each other.

The results are presented in Table 5-18 and Figure 5-9. Normally convergence criteria for β and Z of 0.01 are applied for the calculations, but in this case 0.05 is used. The reason for this less strict criteria is the fact that Prob2B has troubles in finding convergence for this configuration. Most probably this is due to the fact that the process of φ-C reduction is not exactly stable. When a certain MSF is reached the MSF in the steps afterwards is fluctuating around this value. Prob2B can have difficulties in finding a convergent solution for strict convergence criteria.

Number of calculations (FORM): 199				
β: 3.031				
P_f: 1.220*10^{-3}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$E_{50,densely\ packed\ sand}$	0.3	0.56	81930	[kPa]
$E_{oed,densely\ packed\ sand}$	0.3	0.00	81930	[kPa]
$G_{densely\ packed\ sand}$	0.3	0.00	102400	[kPa]
$\sin(\varphi)_{densely\ packed\ sand}$	0.18	0.82	0.32	[-]
$\gamma_{sat,\ densely\ packed\ sand}$	0.05	-0.13	19.26	[kN/m^3]

calc.	Z-value	
1	1.38	
199	0.05	*convergence criteria 0.05* **MSF = 1.15**

Table 5-18 Output final calculation (LS Soil)

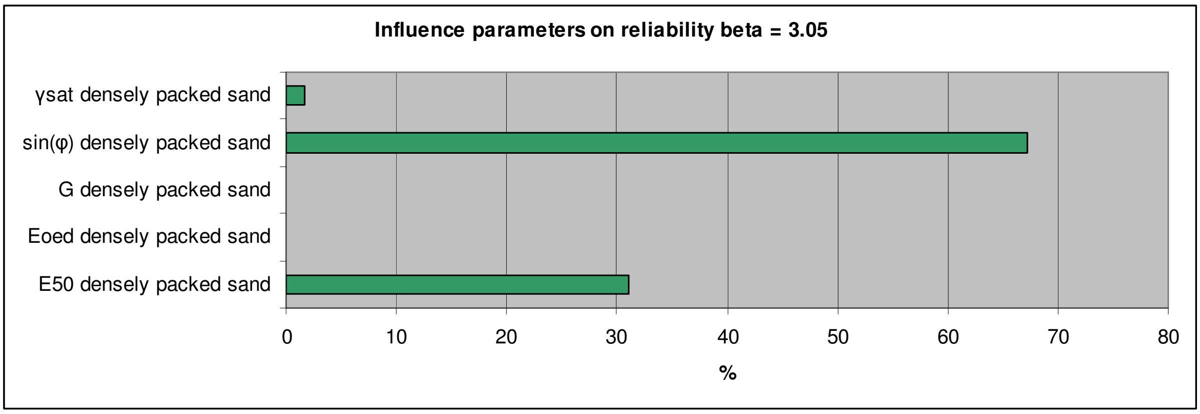


Figure 5-9 Influence in % from parameters on reliability final calculation (LS Soil)

Figure 5-10 shows the plastic points in the design point calculation for soil mechanical failure. There are only Mohr-Coulomb points, because PLAXIS uses the Mohr-Coulomb soil model in φ-C reduction. There is a clear slip plane visible. This slip plane is induced by the failure of the passive wedge. The large influence of E_{50} is 'strange' as the failure is completely caused by exceedance of the maximum shear resistance which implies a large influence of φ of the densely packed sand layer. The influence of E_{50} (and also of γ_{sat}) is caused by its correlation with φ.

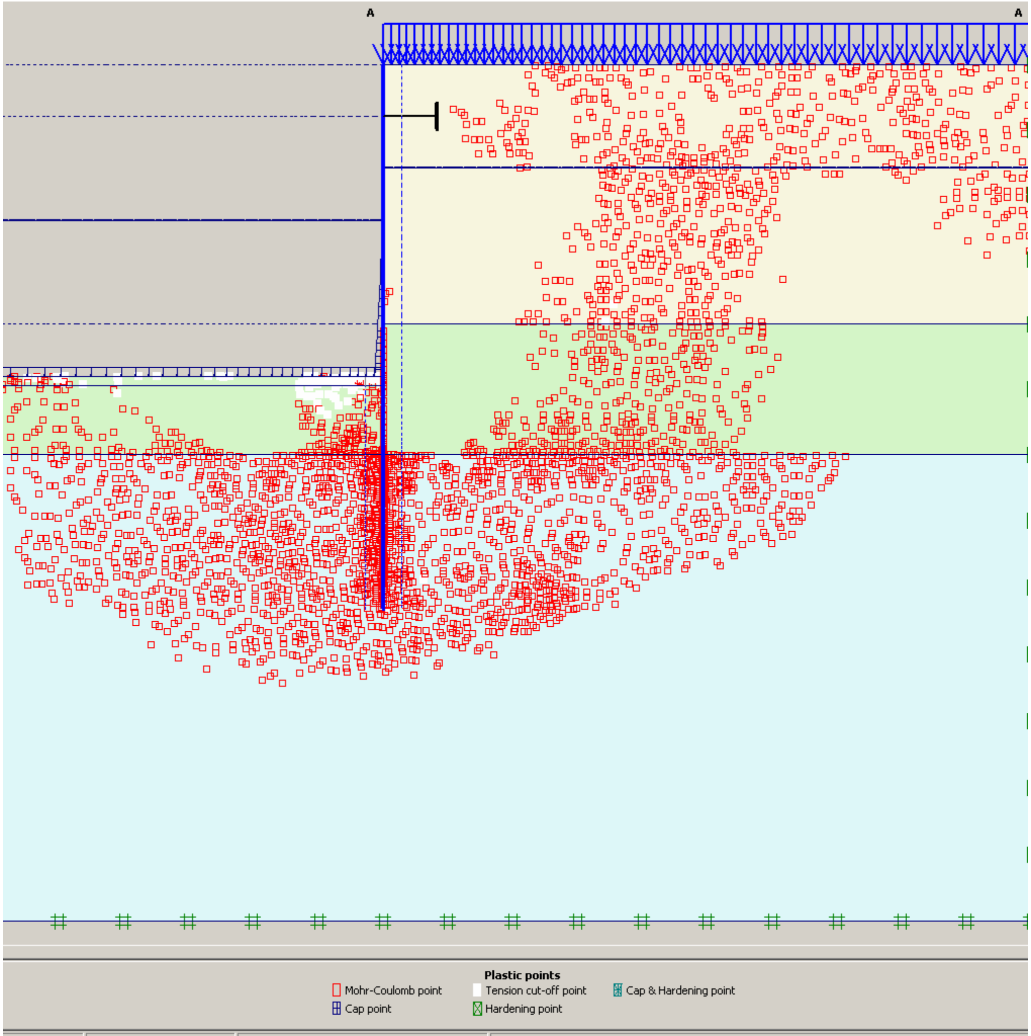


Figure 5-10 Plastic points design point soil mechanical failure

5.7.3.2 Partial safety factors

From these results partial safety factors can be derived according to the formulas of section 5.7.1.2. The different variants of safety factors are presented in Table 5-19. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix I. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 3.031						
β_{CUR211} = 4.396						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$E_{50,densely\ packed\ sand}$	2.03	1.32	1.41	0.92	3.78	2.46
$E_{oed,densely\ packed\ sand}$	1.00	0.65	1.41	0.92	1.00	0.65
$G_{densely\ packed\ sand}$	1.00	0.65	1.41	0.92	1.00	0.65
$\varphi_{densely\ packed\ sand}$	1.99	1.66	2.27	1.90	3.58	3.08
$\gamma_{sat,densely\ packed\ sand}$	0.98	0.90	1.13	1.04	0.97	0.90
Retaining height (Δ)	0.06	0.06	-	-	0.09	0.09
Water level (Δ)	0.04	0.04	-	-	0.06	0.06
Surcharge load	1.00	1.00	-	-	1.00	1.00

Table 5-19 Partial safety factors soil mechanical failure

5.7.3.3 Comparison with Blum calculations

Table 5-20 shows the calculation results for the two different methods, Blum and PLAXIS. The reliability index when using Blum is lower. This shows that in fact the sheet-pile is too short as explained earlier. The difference between the reliability index in PLAXIS and Blum is due to the model limitations of Blum, i.e. the interactive behaviour between soil and sheet-pile is disregarded. This leads to an underestimation of β with Blum.

The differences in influence factors can be explained by the fact that Blum includes no stiffness parameters for the soil, which gives the other parameters a slightly higher influence coefficient. However the results are comparable to each other. The PLAXIS calculation results are used to draw conclusions.

Parameter	α_{BLUM}	α_{PLAXIS}
$E_{50,\text{densely packed sand}}$	-	0.56
$E_{oed,\text{densely packed sand}}$	-	0.00
$G_{,\text{densely packed sand}}$	-	0.00
$\varphi_{\text{densely packed sand}}$	0.89	0.82
$\gamma_{sat,\text{densely packed sand}}$	0.10	-0.13
Retaining height	0.15	0.08
Water level (outside)	0.15	0.07[6]
Water level (ground)	0.00	0.07[6]
Surcharge load	0.08	-0.01
	β_{BLUM} = 1.78	β_{PLAXIS} = 3.03

Table 5-20 Comparison Blum-PLAXIS

5.7.3.4 Conclusions

From the calculations for soil mechanical failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4, by far not leads to the required reliability index of 4.396. Furthermore, the partial safety factors of chapter 3 to obtain this reliability index differ from th factors in this code. Both CUR 166 and CUR 211 are listed and compared with the partial safety factors in Table 5-21. Note that the partial factors from the calculations are levelled up.

Parameter	CUR166 (class III)	CUR211 (class II)	Calculations (soil)
E_{50}	1.00	μ	2.50
E_{oed}	1.00	μ	2.50
E_{ur}	1.00	μ	2.50
φ	1.20	1.00	3.10
C	1.10	1.00	μ
γ	1.00	μ	μ
Water level difference (Δ)	0.30	0.80	0.10
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.00
Retaining height (Δ)	0.35	0.40	0.10

Table 5-21 Partial safety factors for soil mechanical failure comparison with design codes

The partial safety factors for the stiffness parameters and internal angle of friction are extremely high. This is due to their large influence coefficient and also to the high required target reliability of 4.396. The influence factor is given and cannot be changed, but the target reliability index distribution over the different failure mechanisms is a design choice.

[6] This is water level difference

(Remember: $\gamma_{k,i} = \dfrac{1-1.64V_{NEN6740/JCSS,i}}{1-\alpha_i \beta V_i} \Rightarrow \gamma_{k,\varphi} \dfrac{1-1.64*0.10}{1-0.82*0.4396*0.2} \approx 3.08$. When for instance β=3.8 the partial safety factor becomes 2.22) In a later stage (section 7.5) this will be discussed in more detail and solutions to reduce this partial safety factor are suggested.

With respect to soil mechanical failure therefore the partial safety factor on φ, E_{50}, E_{oed} and E_{ur} must increase to obtain the correct reliability. 10 centimeter additional retaining height and 10 centimeter additional water level difference should be taken into account for the retaining height. The factor on the surcharge load lays in between the factor defined in CUR 166 (1.00) and CUR 211 (1.20).

5.7.4 Excessive deformations (SLS)

The LSF for excessive deformations in SLS is $Z = \delta_{\max} - \delta_{occured}$.

δ_{max} is the maximum deformation in the wall which is about halfway of the sheet-pile. δ_{max} is defined in the guidelines. In this case the design is made according to CUR 166. This guideline prescribes a maximum deformation of 1/200 of the retaining height for permanent structures and 1/100 of the retaining height for temporary structures. However, this requirement is not realistic for this case as 1/200*12 = 60 mm, which is close to the value that is obtained in a mean value calculation (54 mm). Therefore it is assumed that the requirement of 1/100 of the retaining height is valid in this case. This implies the LSF to be: $Z = 0.120 - \delta_{occured}$

A calculation with all stochastic parameters included is made, because this calculation is easier to perform (a less critical design point) than the previous calculations. The result can be found in Appendix H.

5.7.4.1 Final calculation

From the calculation in Appendix H it can be concluded that not all parameters are relevant for the reliability of the structure with respect to excessive deformations. The remaining parameters are:

1. $G_{\text{moderate clay}}$
2. $G_{\text{densely packed sand}}$
3. $\sin(\varphi)_{\text{moderately packed sand}}$
4. $\sin(\varphi)_{\text{densely packed sand}}$
5. $\gamma_{\text{sat, moderate clay}}$
6. $\gamma_{\text{sat, densely packed sand}}$

$G_{\text{moderate clay}}$ and $\sin(\varphi)_{\text{moderately packed sand}}$ are also included although their influence is less than 5%. This is for the fact that their design point value is somewhat further away from the mean value than the influence coefficient suggests (i.e. around 10% difference between mean and design value). The output of the final calculation is given in Table 5-22 and Figure 5-11.

Number of calculations (FORM): 73				
β: 2.761				
P_f: 2.879*10^{-3}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$G_{\text{moderate clay}}$	0.3	0.06	5917	[kPa]
$G_{\text{densely packed sand}}$	0.3	0.49	107600	[kPa]
$\sin(\varphi)_{\text{moderately packed sand}}$	0.18	0.11	0.61	[-]
$\sin(\varphi)_{\text{densely packed sand}}$	0.18	0.75	0.34	[-]
$\gamma_{\text{sat,moderate clay}}$	0.05	0.05	17.26	[kN/m^3]
$\gamma_{\text{sat,densely packed sand}}$	0.05	-0.42	20.16	[kN/m^3]

calc.	Z-value
1	0.05
73	0.00

Table 5-22 Output final calculation (LS Deformation)

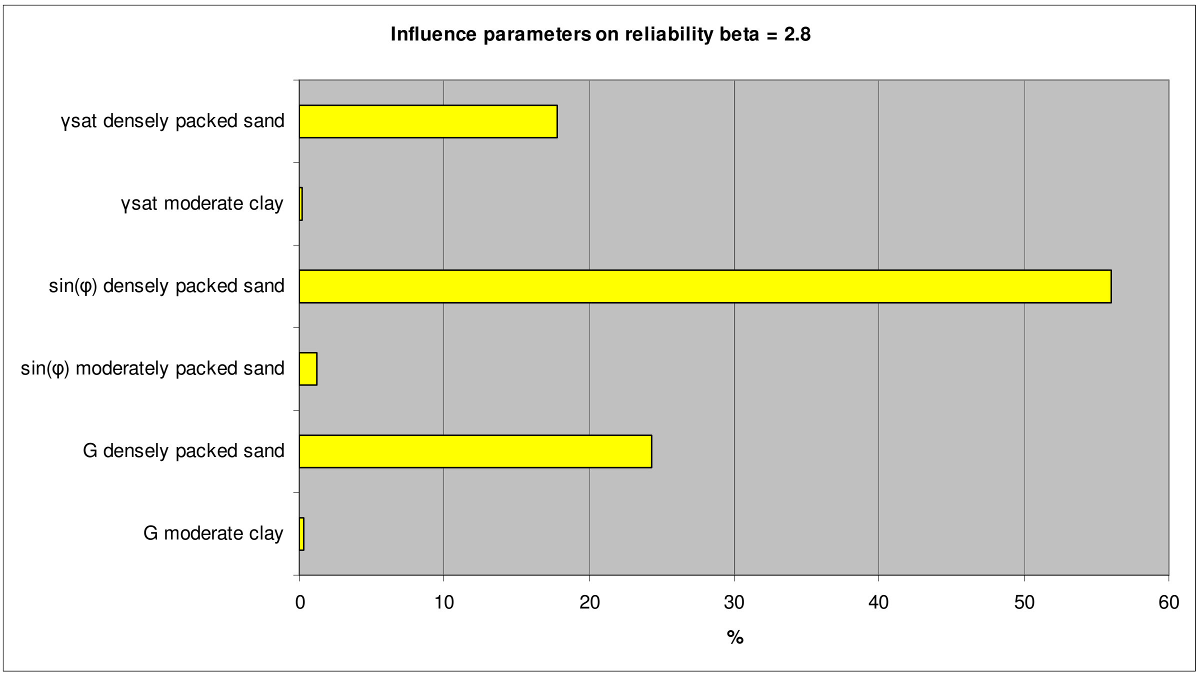


Figure 5-11 Influence in % from parameters on reliability final calculation (LS deformation)

Figure 5-12 shows the plastic points in the design point for the mechanism excessive deformations. The plot shows that deformations occur due to reduction of the fixing moment at the toe of the sheet-pile (shown by the Mohr-Coulomb envelope points (red)). Furthermore shear hardening occurs at many spots which implies that the plastic behaviour is important as well, which is also shown by the relative importance of E_{ur} of the densely packed sand. It can be seen that E_{ur} is the most relevant elastic parameters, since there is no cap hardening, which implies that the the average stress decreases.

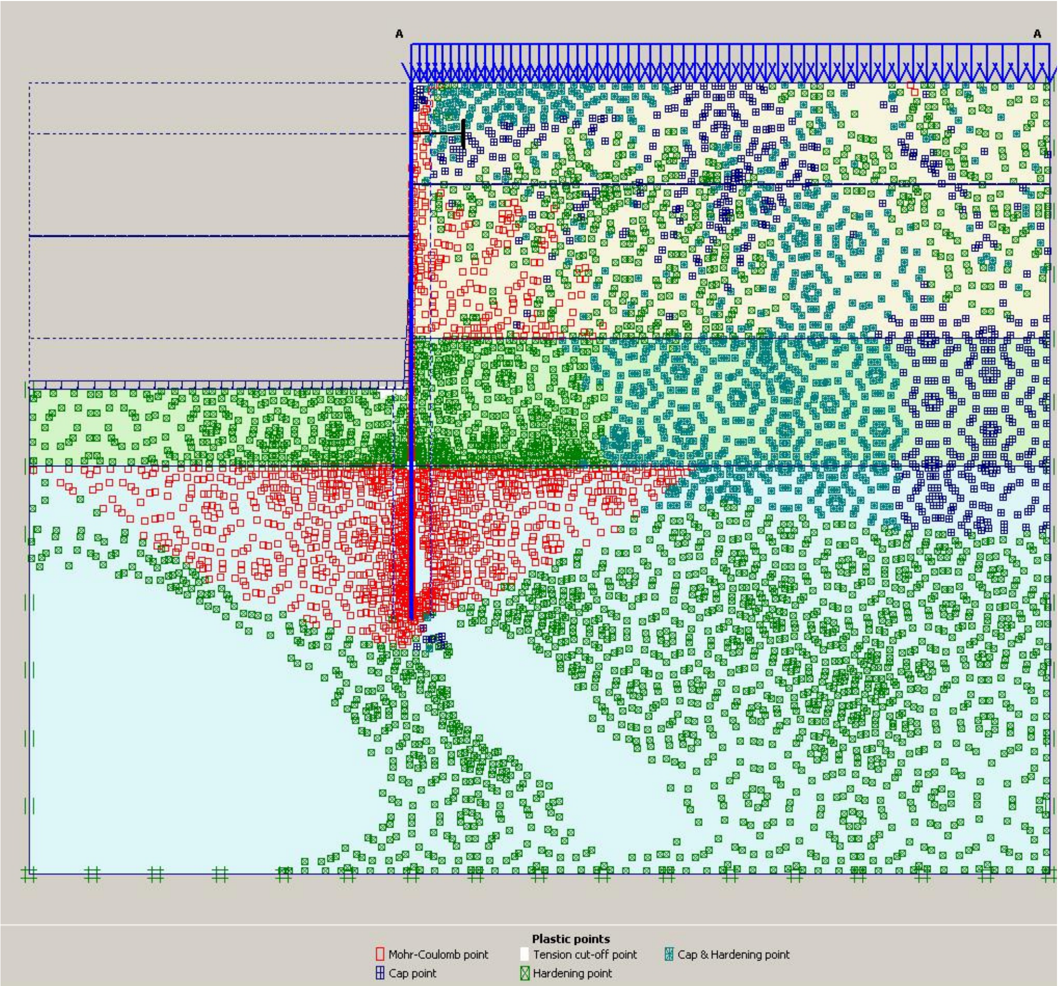


Figure 5-12 Plastic points design point excessive deformations

5.7.4.2 Partial safety factors

From these results partial safety factors can be derived, they are presented in Table 5-23. The target reliability of CUR 211 for excessive deformations is 1.800. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix I. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 2.761						
β_{CUR211} = 1.800						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$G_{moderate\ clay}$	1.05	0.68	1.08	0.70	1.03	0.67
$G_{densely\ packed\ sand}$	1.69	1.10	1.34	0.87	1.36	0.89
$\varphi_{moderately\ packed\ sand}$	1.07	0.92	1.04	0.87	1.04	0.89
$\varphi_{densely\ packed\ sand}$	1.70	1.46	2.11	1.76	1.37	1.18
$\gamma_{sat,moderate\ clay}$	1.01	0.93	1.01	0.93	1.00	0.93
$\gamma_{sat,densely\ packed\ sand}$	0.94	0.87	1.08	0.99	0.96	0.89
Retaining height (Δ)	0.12	0.12	-	-	0.08	0.08
Water level (Δ)	0.07	0.07	-	-	0.05	0.05
Surcharge load (Δ)	1.01	1.01	-	-	1.01	1.01

Table 5-23 Partial safety factors excessive deformations

5.7.4.3 Conclusion

As deformations cannot be calculated with the method of Blum, there cannot be made a comparison between the two design methods. From the calculations for excessive deformations it can be concluded that the design procedure, as it is given in CUR 166 chapter 4, leads to a higher value than the 1.800 prescribed in CUR 211. However, the partial safety factors of chapter 3 to obtain this reliability index differ from this code. Both CUR 211 as CUR 166 are listed and compared with the calculated partial safety factors in Table 5-24. It should be noted that the partial factors from the calculations are leveled up.

Parameter	CUR166 (class III)	CUR211 (class II)	Calculations (deformations)
E_{50}	1.00	μ	0.85
E_{oed}	1.00	μ	0.85
E_{ur}	1.00	μ	0.85
φ	1.20	1.00	1.20
C	1.10	1.00	μ
γ	1.00	μ	μ
Water level difference (Δ)	0.30	0.80	0.05
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.10

Table 5-24 Partial safety factors for deformations comparison with design codes

With respect to excessive deformations therefore the partial safety factor on φ must increase (with respect to CUR 211) to obtain the required reliability. Furthermore, the stiffness parameters should have a higher design value than the characteristic value instead (subdivision by 0.85) but lower than the mean value. 5 cm additional water level difference should be taken into account and 10 cm extra retaining height.

5.7.5 Conclusion

The partial safety factors are summarized and generalized in Table 5-25. As the partial factors are quite different for each different mechanism it is more optimal to define partial safety factors per mechanism (otherwise the anchor would for instance be too heavy). For the SLS mechanism excessive deformations no additional factors need to be defined, because the factors from Table 5-24 fall in between the boundaries of the other mechanisms. For the sake of completeness they are nevertheless presented in Table 5-25, but it can be seen that when the deformations are not exceeding the limit when checking 'anchor failure', the situation is already fine. When this is not fine, the deformations can be additionally checked by using the right column of the table.

An important notion is that application of these generalized parameters leads to a suboptimal design. This is because the partial safety factors are applied on each layer, whereas the probabilistic calculations often show that the uncertainty in one layer is dominant. For instance, for soil mechanical failure only factors are needed on the parameters of the lower sand layer. This also holds to a large extent for wall failure in bending. When placing factors on each layer, a heavier and longer wall is chosen and a too high reliability index is obtained.

It is however not practical to define partial safety factors per layer. Moreover, real situations are not equal to one of the benchmark quay walls. The most convenient way would be to do probabilistic calculations (or sensitivity analyses) to define the most important layers and parameters wehen designing a quay wall. Based on these analysis the partial safety factors can be placed on the correct parameters and a more optimal design can be made.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (generalized)			
			Anchor	Wall	Soil	Deformations
$EI_{sheet-pile}$	1.00	1.00	μ	μ	μ	μ
$EA_{sheet-pile}$	1.00	1.00	μ	μ	μ	μ
$w_{sheet-pile}$	1.00	1.00	μ	μ	μ	μ
$W_{sheet-pile}$	1.00	1.20	μ	μ	μ	μ
D_a	1.00	1.10[7]	1.00	μ	μ	μ
$f_{y,steel}$	1.00	1.00[7]	1.00	μ	μ	μ
EA_a	1.00	1.20[7]	1.00	μ	μ	μ
E_{50}	1.00	μ	0.85	1.55	2.50	0.85
E_{oed}	1.00	μ	0.85	1.55	2.50	0.85
E_{ur}	1.00	μ	0.85	1.55	2.50	0.85
φ	1.20	1.00	1.55	1.90	3.10	1.20
C	1.10	1.00	μ	μ	μ	μ
γ	1.00	μ	1.05	1.05	μ	μ
Water level difference (Δ)	0.30	0.80	0.10	0.05	0.10	0.05
Water level left (outside) (Δ)	0.25	0.70	-	-	-	-
Water level right (ground) (Δ)	0.05	0.10	-	-	-	-
Surcharge load	1.00	1.20	1.05	1.05	1.00	1.05
Retaining height (Δ)	0.35	0.40	0.15	0.05	0.10	0.10

Table 5-25 Summary of partial safety factor over characteristic value for anchored sheet-pile

It is advised to use three different design values for the soil stiffness parameters, the internal angle of friction and the specific soil weight.

$$E_{d,anchor-force} = \frac{E_k}{0.85},\ E_{d,bending-moment} = \frac{E_k}{1.55} \text{ and } E_{d,soil-collapse} = \frac{E_k}{2.50}$$

$$\varphi_{d,anchor-force} = \frac{\varphi_k}{1.55},\ \varphi_{d,bending-moment} = \frac{\varphi_k}{1.90} \text{ and } \varphi_{d,soil-collapse} = \frac{\varphi_k}{3.10}$$

$$\gamma_{d,anchor-force} = \frac{\gamma_k}{1.05},\ \gamma_{d,bending-moment} = \frac{\gamma_k}{0.95} \text{ and } \gamma_{d,soil-collapset} = \gamma_\mu$$

The additional water level difference can be generalized to 10 cm, the factor on the surcharge load to 1.05 and the absolute factor on the retaining height to 15 cm for all mechanisms. Furthermore for the sheet-pile parameters and Cohesion the mean values can be used and for the anchor parameters the characteristic values.

With respect to the reliability indices obtained in the probabilistic calculation it can be concluded that for anchor failure and wall failure the obtained reliability index is close to the prescribed target reliability index. For soil mechanical failure β is too low, but this is to some extent due to the fact that the sheet-pile is too short. Furthermore this β is underestimated due to the MSF = 1.1 instead of target MSF = 1.0 (in section 7.4 there is made an estimate of this underestimation). It can be concluded that the approach as presented in CUR 166 chapter 4 to design sheet-piles is applicable for anchored sheet-piles like benchmark 1. The reliability indices are summarized in Table 5-26.

[7] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force: ($F_{max} = A\sigma_y = 1/4\pi D_a^2\sigma_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on σ_y.

	β calculated	β CUR 211
Anchor failure	3.60	3.828
Wall failure	4.00	3.872
Soil mechanical failure	3.03	4.396
Deformations	2.76	1.800

Table 5-26 Reliability indices failure mechanisms

The Blum calculations have shown that the parameters that cannot be included in the probabilistic calculations do not influence the reliability index by much. Moreover, the manual variations in PLAXIS are accurate enough to estimate the influence factor. Therefore, no Blum probabilistic calculations are made for the next benchmark calculations.

As the wall appeared to be too short in section 5.8 additional calculations with a longer wall are made to check whether the results are influenced by this design error.

5.8 Anchored sheet-pile with elongated wall

From the first calculations it appeared that the wall in the first benchmark is actually not long enough. This is shown by the low reliability indices for soil mechanical failure, both in the Blum calculations and PLAXIS calculations. Therefore the required wall length is determined using Blum's iterative method (Appendix F). This method sounds 'old-fashioned' but the assumption of fully mobilized shear resistance is correct when the minimum sheet-pile length is determined. MSheet uses the same approach. In PLAXIS designing of the sheet-pile length is not possible with a straightforward procedure. The required wall length according to Blum is 22.8 m. Therefore the elongated wall is chosen to be 23 m.

5.8.1 Anchor failure (ULS)

Appendix J shows the result of the first calculation including all stochastic parameters.

5.8.1.1 Final calculation

From the first calculation in Appendix J it appeared that many parameters can be eliminated as they hardly influence the reliability. The remaining parameters for the final calculation are:

- $f_{y,steel}$
- EA_a
- $G_{densely\ packed\ sand}$
- $\sin(\varphi)_{moderately\ packed\ sand}$
- $\sin(\varphi)_{densely\ packed\ sand}$
- $\gamma_{sat,moderate\ clay}$
- $\gamma_{sat,moderately\ packed\ sand}$

Although $\sin(\varphi)_{densely\ packed\ sand}$ had a low influence coefficient in the first calculation (< 5%) it is nevertheless included in the final calculation, because the design value is far lower than the mean value (X^* = 0.30 and μ = 0.67). The results of the final calculation are given in Table 5-27 and Figure 5-13.

Number of calculations (FORM): 33				
β: 4.087				
P_f: $2.187 \cdot 10^{-5}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$f_{y,steel}$	0.07	0.42	352700	[kN/m^2]
EA_a	0.07	-0.05	14480	[kN/m]
$G_{densely\ packed\ sand}$	0.3	-0.29	149700	[kPa]
$\sin(\varphi)_{moderately\ packed\ sand}$	0.18	0.69	0.30	[-]
$\sin(\varphi)_{densely\ packed\ sand}$	0.18	0.13	0.53	[-]
$\gamma_{sat,moderate\ clay}$	0.05	0.10	17.20	[kN/m^3]
$\gamma_{sat,moderately\ packed\ sand}$	0.05	0.49	20.47	[kN/m^3]

calc.	Z-value
1	324.60
33	4.68

Table 5-27 Output final calculation (LS anchor - elongated)

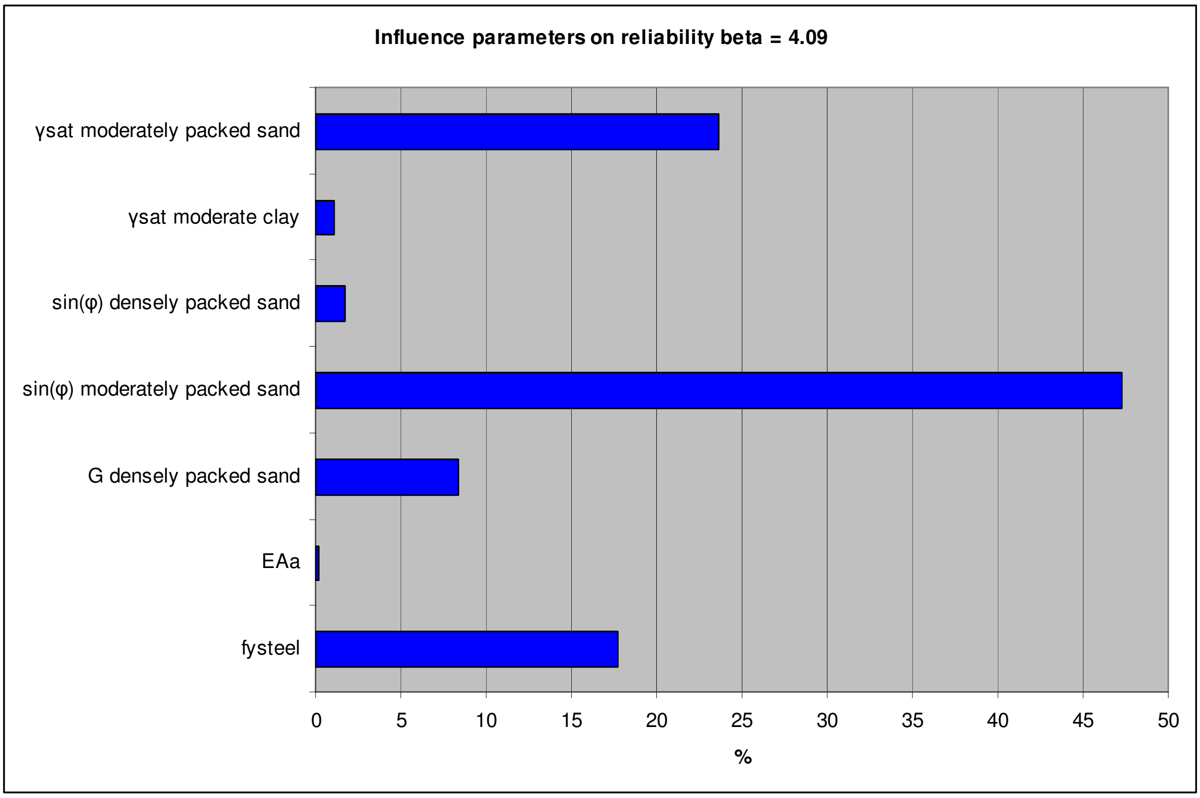


Figure 5-13 Influence in % on reliability final calculation (LS anchor - elongated)

Figure 5-14 shows the plastic point plot of the mechanism anchor failure. The plot is similar to the plot of the shorter sheet-pile (Figure 5-5). Anchor failure is mainly induced by failure of the soil in the upper sand layer and partly by the failure of the passive wedge behind the wall, combined with the decrease in yield stress of the steel.

The importance of $G_{densely\ packed\ sand}$ is increased by the fact that elastical behavior is important in the lower sand layer. The green blocks show the spots where shear hardening occurs which influences the interaction between the motion of the sheet-pile and the motion of the soil around it. The stiffer the soil the less the mobilization of the shear resistance (especially important at the passive side) and the higher the anchor force should be to guarantee horizontal equilibrium. This explains the negative value for $\alpha_{G\ densely\ packed\ sand}$, the stiffness is higher than the mean value in the design point.

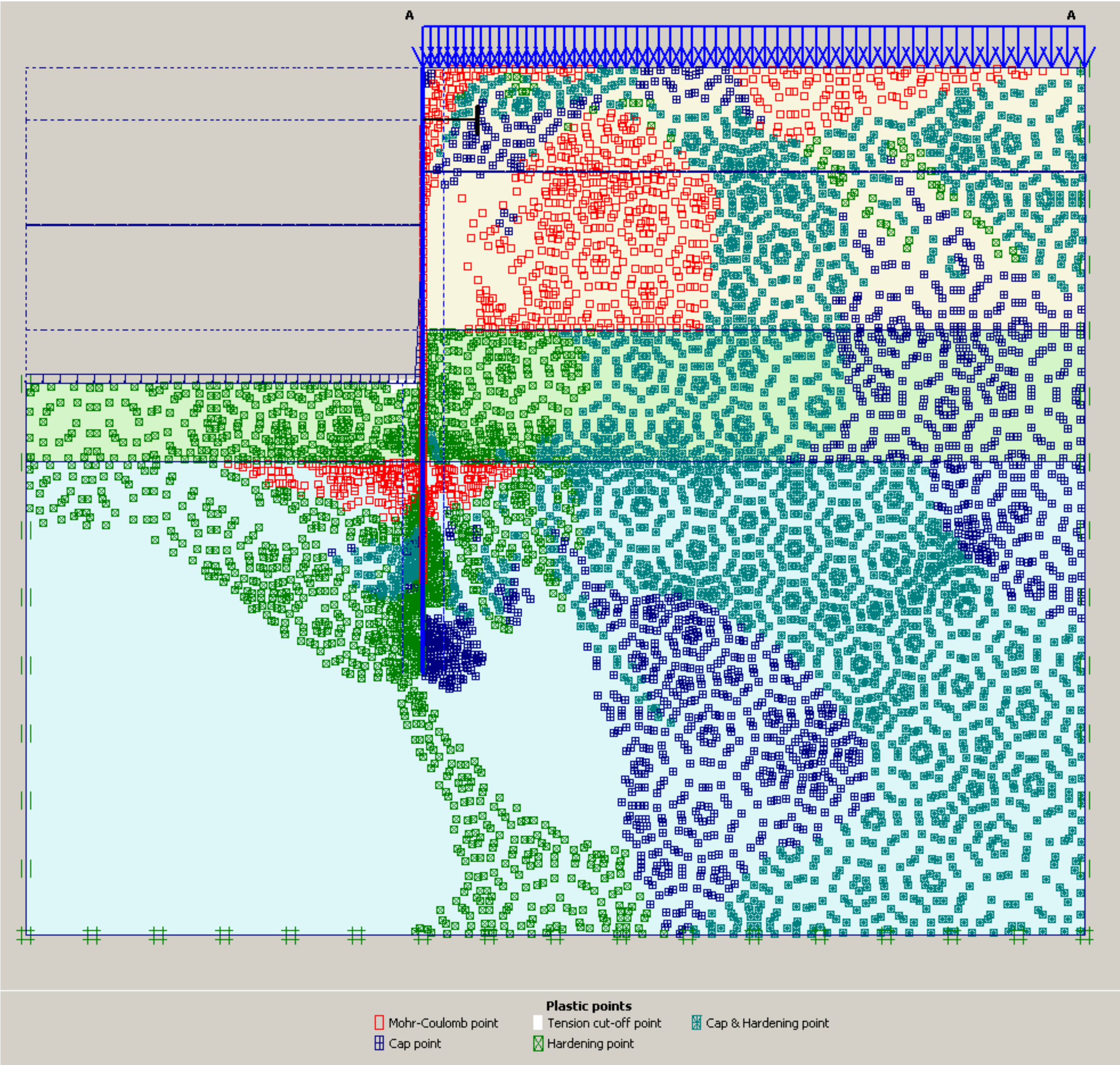


Figure 5-14 Plastic points in design point anchor failure - elongated

5.8.1.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 5.7.1.2. The different variants of safety factors are presented in Table 5-28. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix K. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 4.087						
β_{CUR211} = 3.828						
X_i	$\gamma_{\mu,i,calculation}$	$\gamma_{k,i,calculation}$	μ/X^*	X_k/X^*	$\gamma_{\mu,i,CUR211}$	$\gamma_{k,i,CUR211}$
$f_{y,steel}$	1.14	1.02	1.14	1.01	1.13	1.00
EA_a	0.99	0.89	0.99	0.87	0.99	0.87
$G_{densely\ packed\ sand}$	0.74	0.48	0.96	0.63	0.75	0.49
$\varphi_{moderately\ packed\ sand}$	1.95	1.67	2.21	1.85	2.11	1.76
$\varphi_{densely\ packed\ sand}$	1.10	0.95	1.31	1.10	1.11	0.93
$\gamma_{sat,moderate\ clay}$	1.02	0.94	1.01	0.93	1.02	0.94
$\gamma_{sat,moderately\ packed\ sand}$	1.11	1.03	1.01	0.93	1.10	1.01
Retaining height (Δ)	0.08	0.08	-	-	0.07	0.07
Water level (Δ)	0.07	0.07	-	-	0.06	0.06
Surcharge load	1.02	1.02	-	-	1.02	1.02

Table 5-28 Partial safety factors anchor failure – elongated

5.8.1.3 Conclusion

From the calculations for anchor failure for the elongated wall it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads to a reliability index that is slightly higher than the required 3.828. However, the partial safety factors to obtain this reliability index differ from the factors in the code. Both CUR 166 and CUR 211 are listed and compared with the partial safety factors in Table 5-29. It should be noted that the partial factors from the calculations are levelled up.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (anchor)
D_a	1.00	1.10[8]	μ
$f_{y,steel}$	1.00	1.00[8]	1.00
EA_a	1.00	1.20[8]	μ
E_{50}	1.00	μ	0.45
E_{oed}	1.00	μ	0.45
E_{ur}	1.00	μ	0.45
φ	1.20	1.00	1.80
C	1.10	1.00	μ
γ	1.00	μ	1.05
Water level difference (Δ)	0.30	0.80	0.10
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.10

Table 5-29 Partial safety factors for anchor failure comparison with design codes

With respect to anchor failure therefore the partial safety factor on φ must increase to obtain the correct reliability. The mean diameter and axial stiffness of the anchor can be used. Furthermore, the soil stiffness parameters should have a higher value than the mean value (and characteristic value). Some additional safety marge should be taken into account for the water levels and retaining height, 10 cm water level difference and 10 cm retaining height. The partial safety factor over the governing surcharge load should be 1.05.

[8] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force: ($F_{max} = A\sigma_y = 1/4\pi D_a^2\sigma_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on σ_y.

5.8.2 Wall failure (ULS)

Appendix J shows the result of the first calculation including all stochastic parameters.

5.8.2.1 Final calculation

From the calculation in Appendix J it can be concluded that not all parameters are relevant for the reliability calculations. In the final calculation therefore only the following parameters are included:

- $E_{50,\text{densely packed sand}}$
- $E_{\text{oed,densely packed sand}}$
- $G_{\text{densely packed sand}}$
- $\sin(\varphi)_{\text{moderately packed sand}}$
- $\sin(\varphi)_{\text{densely packed sand}}$
- $\gamma_{\text{sat,densely packed sand}}$

The previous calculations showed that Prob2B presents that from the soil stiffness parameters only $G_{\text{densely packed sand}}$ is relevant, but it appeared to be necessary to include all stiffness parameters of the layer as they are correlated. Without the other stiffness parameters the calculations had problems in convergence. Two of the three stiffness parameters however still do not influence the reliability as all stiffness parameters are fully correlated with each other.

The results are given in Table 5-30 and Figure 5-15. 'Normally', convergence criteria for β and Z of 0.01 are applied for the calculations, but in this case 0.05 is used. The reason for those less strict criteria is the fact that Prob2B has troubles in finding convergence for this configuration. Most probably this is due to the fact that the failure mechanisms wall failure in bending and soil mechanical failure are closely related (i.e. both mechanisms occur due to large reduction of φ in the lower sand layer). It is difficult to find convergence when a soil collapse occurs around the design point of the bending moment.

Number of calculations (FORM): 144				
β: 3.960				
P_f: $3.751 \cdot 10^{-5}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$E_{50,\text{densely packed sand}}$	0.3	0.54	77570	[kPa]
$E_{\text{oed, densely packed sand}}$	0.3	0.00	77570	[kPa]
$G_{\text{densely packed sand}}$	0.3	0.00	96970	[kPa]
$\sin(\varphi)_{\text{moderately packed sand}}$	0.18	0.05	0.62	[-]
$\sin(\varphi)_{\text{densely packed sand}}$	0.18	0.83	0.23	[-]
$\gamma_{\text{sat,densely packed sand}}$	0.05	0.12	19.52	[kN/m^3]

calc.	Z-value	
1	573.70	
144	194.60	*Convergence criteria 0.05*

Table 5-30 Output final calculation (LS wall - elongated)

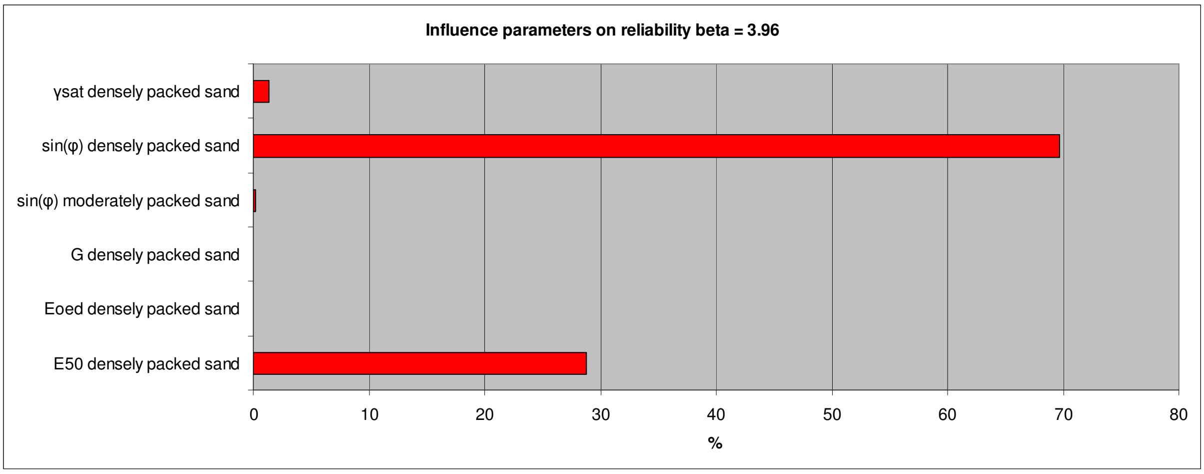


Figure 5-15 Influence in % on reliability final calculation (LS wall - elongated)

Figure 5-16 shows the plastic points in the design point calculation of wall failure in bending. The figure is comparable to the figure for the original benchmark calculation (Figure 5-8). The main cause for the large bending moments in the wall is the reduction of the fixing moment at the bottom of the sheet-pile. This is caused by reduction of the internal angle of friction of the densely packed sand layer. When the fixing moment is reduced the field moment increases.

Additionally to this mechanism some soil failure in the upper layers occurs, which increases the load that acts on the wall, causing higher bending moments in the wall.

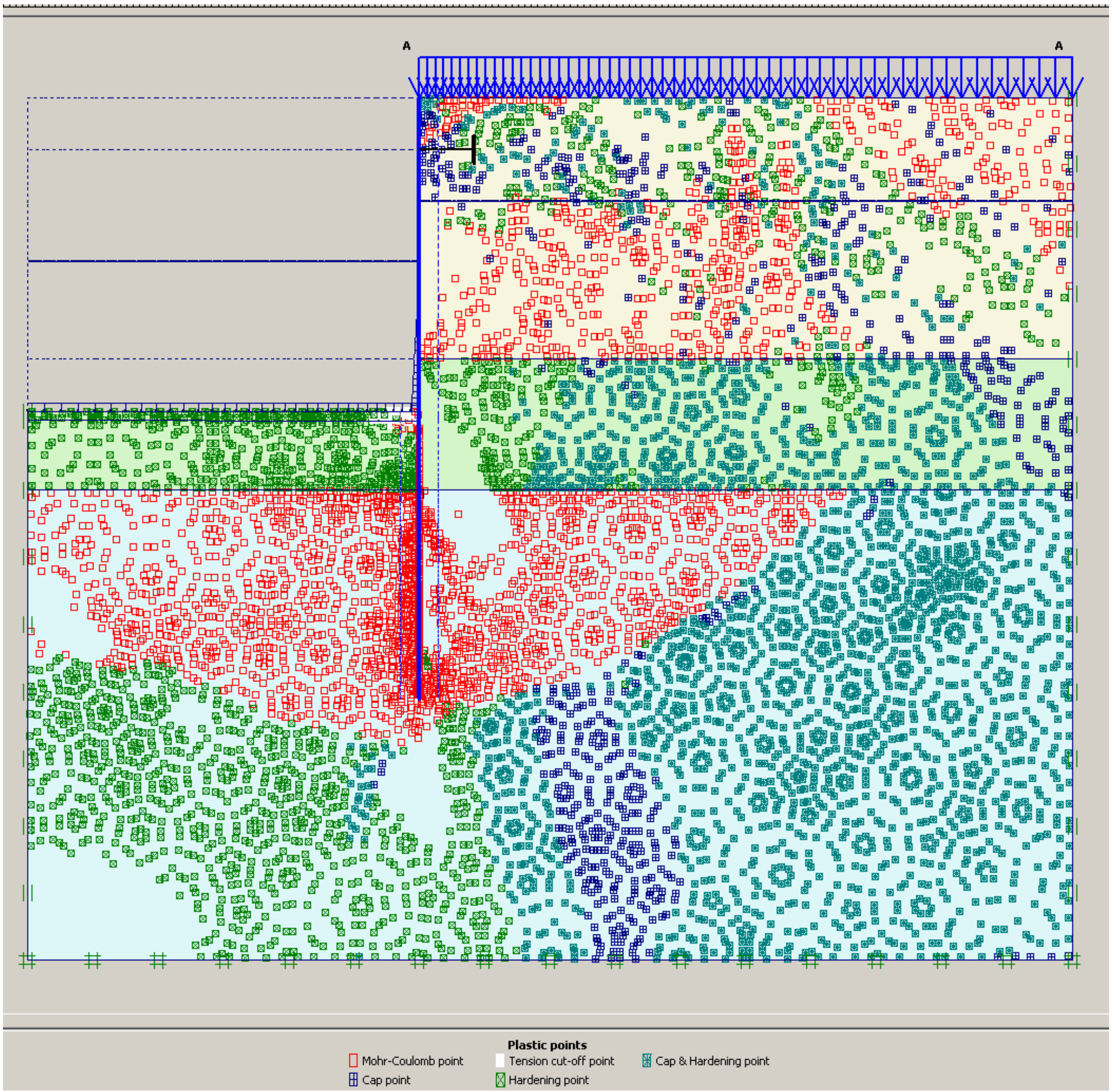


Figure 5-16 Plastic points in design point wall failure - elongated

5.8.2.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 5.7.1.2. The different variants of safety factors are presented in Table 5-31. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix K. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 3.960						
β_{CUR211} = 3.872						
X_i	$\gamma_{\mu,i,calculation}$	$\gamma_{k,i,calculation}$	μ/X^*	X_k/X^*	$\gamma_{\mu,i,CUR211}$	$\gamma_{k,i,CUR211}$
$E_{50,densely\ packed\ sand}$	2.75	1.79	1.49	0.97	2.65	1.72
$E_{oed,\ densely\ packed\ sand}$	1.00	0.65	1.49	0.97	1.00	0.65
$G_{densely\ packed\ sand}$	1.00	0.65	1.49	0.97	1.00	0.65
$\varphi_{moderately\ packed\ sand}$	1.04	0.89	1.02	0.86	1.04	0.87
$\varphi_{densely\ packed\ sand}$	2.95	2.53	3.13	2.62	2.83	2.36
$\gamma_{sat,densely\ packed\ sand}$	1.02	0.95	1.12	1.02	1.02	0.94
Retaining height (Δ)	0.11	0.11	-	-	0.11	0.11
Water level (Δ)	0.08	0.08	-	-	0.08	0.08
Surcharge load	1.00	1.00	-	-	1.00	1.00

Table 5-31 Partial safety factors wall failure – elongated

5.8.2.3 Conclusion

From the calculations for anchor failure for the elongated wall it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads to a reliability index that is slightly higher than the required 3.872. However, the partial safety factors to obtain this reliability index differ from the prescribed factors in the code. Both CUR 166 and CUR 211 are listed and compared with the calculated partial safety factors in Table 5-32. It should be noted that the partial factors from the calculations are levelled up.

Parameter	**CUR166 (class III)**	**CUR211 (class II)**	**Calculations (wall)**
$EI_{sheet-pile}$	1.00	1.00	μ
$EA_{sheet-pile}$	1.00	1.00	μ
$w_{sheet-pile}$	1.00	1.00	μ
$W_{sheet-pile}$	1.00	1.00	μ
$f_{y,steel}$	1.00	1.00	μ
E_{50}	1.00	μ	1.75
E_{oed}	1.00	μ	1.75
E_{ur}	1.00	μ	1.75
φ	1.20	1.00	2.40
C	1.10	1.00	μ
γ	1.00	μ	0.95
Water level difference (Δ)	0.30	0.80	0.10
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.00
Retaining height (Δ)	0.35	0.40	0.15

Table 5-32 Partial safety factors for wall failure comparison with design codes

With respect to wall failure in bending therefore the partial safety factor on φ must increase to obtain the correct reliability. Mean values can be used for the sheet-pile parameters. Furthermore, the soil stiffness parameters should have a lower value than the characteristic value. Some additional safety marge should be taken into account for the water levels and retaining height, 10 cm water level difference and 15 cm additional retaining height. For The governing surcharge load can be used.

5.8.3 Soil mechanical failure (ULS)

Appendix J shows the result of the four calculations that are made in order to check the influence of the stochastic parameters on the reliability.

5.8.3.1 Final calculation

From the four previous calculations from Appendix J it can be concluded that not all parameters are relevant for the reliability calculations. In the final calculation therefore only the following parameters are included:

- $G_{\text{densely packed sand}}$
- $\sin(\varphi)_{\text{densely packed sand}}$
- $\gamma_{\text{sat,densely packed sand}}$

The output is given in Table 5-33 and Figure 5-17.

Number of calculations (FORM): 277				
β: 3.381				
P_f: $3.618 \cdot 10^{-4}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$G_{\text{densely packed sand}}$	0.3	0.00	101600	[kPa]
$\sin(\varphi)_{\text{densely packed sand}}$	0.18	0.80	0.26	[-]
$\gamma_{\text{sat,densely packed sand}}$	0.05	-0.29	19.39	[kN/m^3]
calc.	Z-value			
1	0.057			
277	0.006	MSF = 1.11		

Table 5-33 Output final calculation (LS Soil - elongated)

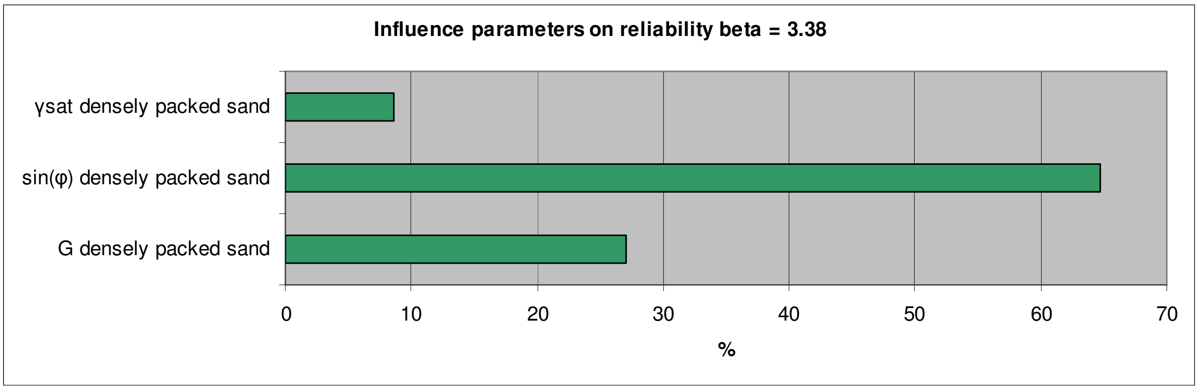


Figure 5-17 Influence in % on reliability final calculation (LS soil - elongated)

Figure 5-18 shows the plastic points in the design point for soil mechanical failure. There are only Mohr-Coulomb points, because PLAXIS uses the Mohr-Coulomb soil model in φ-C reduction. There is a clear slip plane visible, induced by the failure of the passive wedge. The figure is comparable to the figure of the shorter wall (Figure 5-10), however the area with plastic points behind the sheet-pile is larger due to the greater depth in the new case. The relative importance of G and γ_{sat} of the lower sand layer is caused by their correlation with φ.

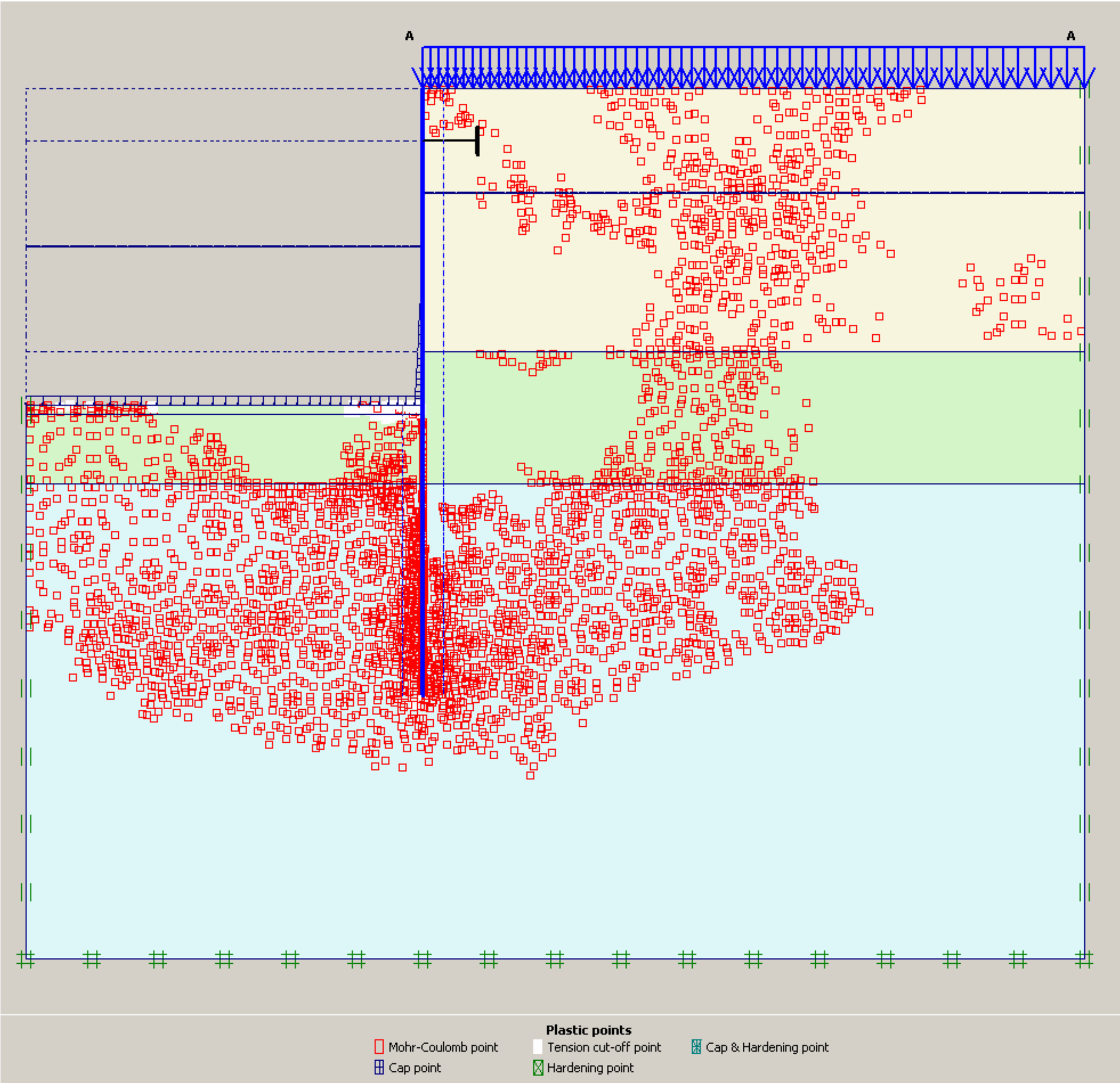


Figure 5-18 Plastic points in design point soil mechanical failure

5.8.3.2 Partial safety factors

From these results partial safety factors can be derived according to the formulas of section 5.7.1.2. The different variants of safety factors are presented in Table 5-34. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix K. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 3.381						
β_{CUR211} = 4.396						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$G_{densely\ packed\ sand}$	2.12	1.38	1.42	0.92	3.18	2.07
$\varphi_{densely\ packed\ sand}$	2.19	1.88	2.74	2.29	3.42	2.86
$\gamma_{sat,densely\ packed\ sand}$	0.95	0.88	1.12	1.03	0.94	0.86
Retaining height (Δ)	0.05	0.05	-	-	0.06	0.06
Water level (Δ)	0.00	0.00	-	-	-0.01	-0.01
Surcharge load (Δ)	1.01	1.01	-	-	1.02	1.02

Table 5-34 Partial safety factors soil mechanical failure - elongated

5.8.3.3 Conclusions

From the calculations for soil mechanical failure for the elongated wall it can be concluded that the design procedure as it is given in CUR 166 chapter 4 not leads to the required reliability index of 4.396. Furthermore, the partial safety factors to obtain the target reliability index differ from the partial safety factors in the code. Both CUR 166 and CUR 211 are listed and compared with the partial safety factors in Table 5-35. It should be noted that the partial factors from the calculations are levelled up.

Parameter	**CUR166 (class III)**	**CUR211 (class II)**	**Calculations (soil)**
E_{50}	1.00	μ	2.05
E_{oed}	1.00	μ	2.05
E_{ur}	1.00	μ	2.05
φ	1.20	1.00	2.90
C	1.10	1.00	μ
γ	1.00	μ	0.85
Water level difference (Δ)	0.30	0.80	0.00
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.10

Table 5-35 Partial safety factors for soil mechanical failure comparison with design codes

With respect to soil mechanical failure therefore the partial safety factor on E_{50}, E_{oed}, E_{ur} and φ must increase to obtain the correct reliability. γ should be increased by dividing it by 0.84. 10 centimeter additional safety marge should be taken into account for the retaining height and no additional water level difference is needed. The factor on the surcharge load is in between the factor defined in CUR 166 (1.00) and CUR 211 (1.20).

5.8.4 Excessive deformations (SLS)

Appendix J shows the result of the first calculation including all stochastic parameters.

5.8.4.1 Final calculation

From the calculation from Appendix J, it can be concluded that not all parameters are relevant for the reliability calculations. In the final calculation therefore only the following parameters are included:

- $E_{50,\text{ moderate clay}}$
- $E_{50,\text{ densely packed sand}}$
- $\sin(\varphi)_{\text{moderately packed sand}}$
- $\sin(\varphi)_{\text{densely packed sand}}$
- $\gamma_{\text{sat,moderate clay}}$
- $\gamma_{\text{sat,moderately packed sand}}$
- $\gamma_{\text{sat,densely packed sand}}$

$E_{50,\text{moderate clay}}$ has an influence smaller than 5%, but it is nevertheless included in the final calculation because the design point value is 17% lower than the original mean value. Just to be sure not to forget important parameters this stiffness parameter is also included. The output of the final calculations is shown in Table 5-36 and Figure 5-19.

Number of calculations (FORM):109				
β: 2.692				
P_f: $3.546 \cdot 10^{-3}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$E_{50,\text{moderate clay}}$	0.3	0.07	6356	[kPa]
$E_{50,\text{densely packed sand}}$	0.3	0.52	85380	[kPa]
$\sin(\varphi)_{\text{moderately packed sand}}$	0.18	0.14	0.59	[-]
$\sin(\varphi)_{\text{densely packed sand}}$	0.18	0.75	0.35	[-]
$\gamma_{\text{sat,moderate clay}}$	0.05	0.06	17.22	[kN/m^3]
$\gamma_{\text{sat,moderately packed sand}}$	0.05	0.08	20.69	[kN/m^3]
$\gamma_{\text{sat,densely packed sand}}$	0.05	-0.36	20.06	[kN/m^3]

calc.	Z-value
1	0.0653
109	-0.0006

Table 5-36 Output final calculation (LS deformations - elongated)

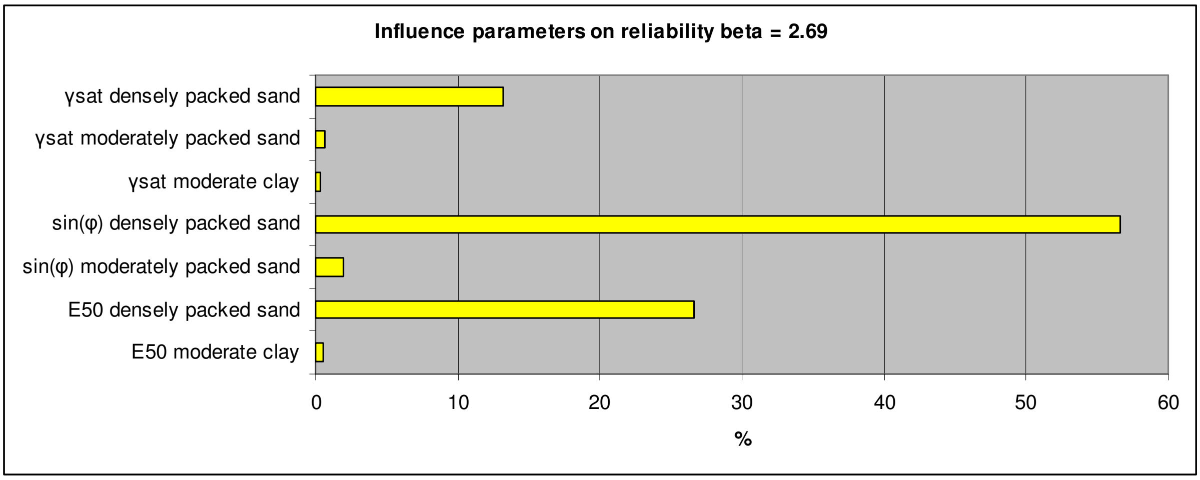


Figure 5-19 Influence in % on reliability final calculation (LS deformations - elongated)

Figure 5-20 (similar to Figure 5-12 for the shorter wall) shows the plastic points in the design point for the mechanism excessive deformations. The plot shows that deformations occur due to reduction of the fixing moment at the toe of the sheet-pile (shown by the Mohr-Coulomb envelope points (red)). Furthermore shear hardening occurs at many spots which implies that the plastic behaviour is important as well, which is also shown by the relative importance of E_{ur} of the densely packed sand. It can be seen that E_{ur} is the most relevant elastic parameters, since there is no cap hardening, which implies that the the average stress decreases.

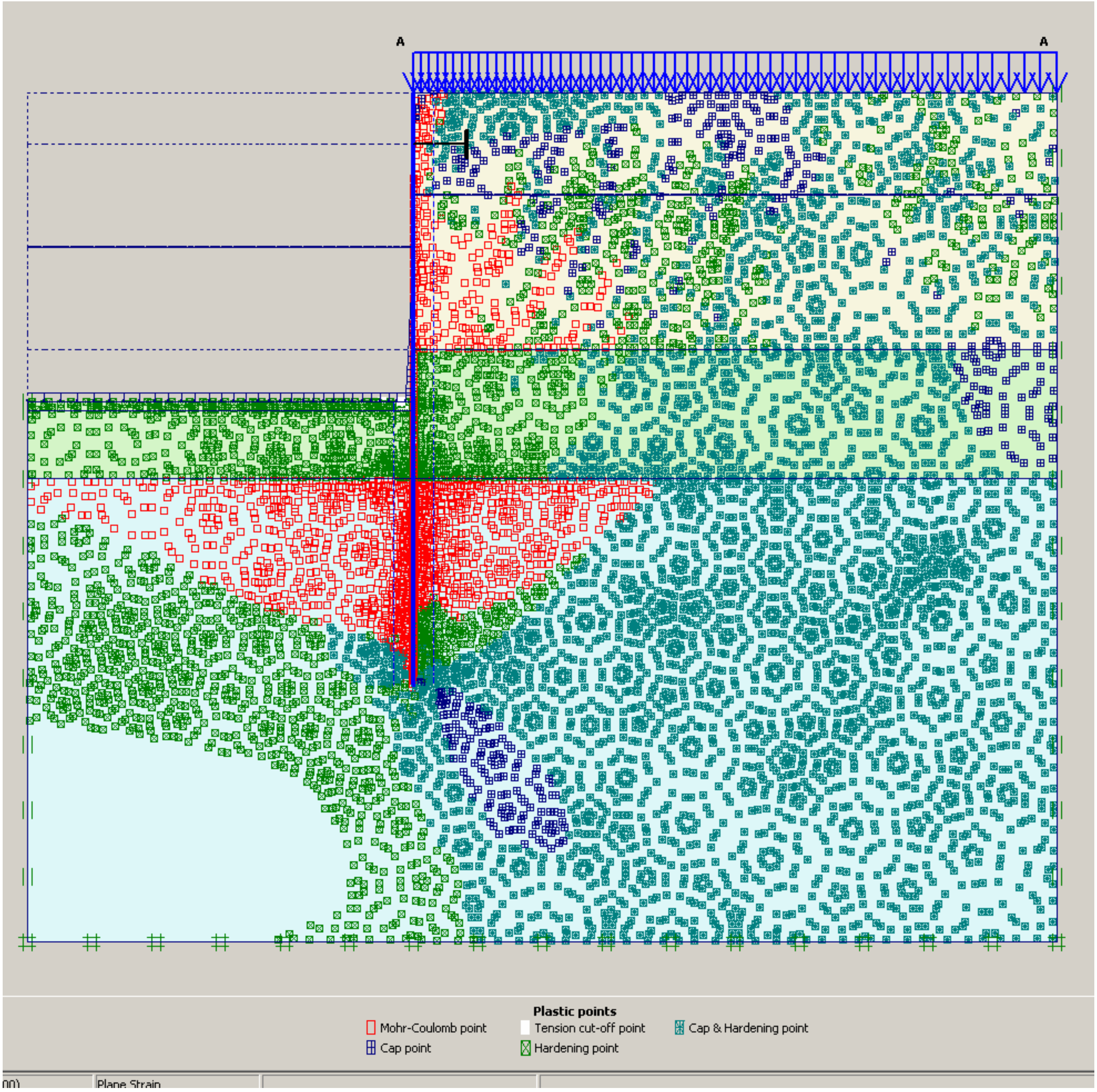


Figure 5-20 Plastic points in design point excessive deformations

5.8.4.2 Partial safety factors

From these results partial safety factors can be derived (according to the formulas of section 5.7.1.2) and they are presented in Table 5-37. The target reliability of CUR 211 for excessive deformations is 1.800. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix K. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 2.692						
β_{CUR211} = 1.800						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$E_{50,moderate\ clay}$	1.06	0.69	1.21	0.79	1.04	0.68
$E_{50,densely\ packed\ sand}$	1.71	1.11	1.35	0.88	1.39	0.90
$\varphi_{moderately\ packed\ sand}$	1.08	0.93	1.08	0.90	1.05	0.88
$\varphi_{densely\ packed\ sand}$	1.68	1.44	2.04	1.70	1.37	1.15
$\gamma_{sat,moderate\ clay}$	1.01	0.93	1.01	0.93	1.01	0.92
$\gamma_{sat,moderately\ packed\ sand}$	1.01	0.93	1.00	0.92	1.01	0.92
$\gamma_{sat,densely\ packed\ sand}$	0.95	0.88	1.09	1.00	0.97	0.89
Retaining height (Δ)	0.08	0.08	-	-	0.05	0.05
Water level (Δ)	0.06	0.06	-	-	0.04	0.04
Surcharge load (Δ)	1.00	1.00	-	-	1.00	1.00

Table 5-37 Partial safety factors excessive deformations – elongated

5.8.4.3 Conclusion

From the calculations for excessive deformations it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads to a higher value than the 1.800 prescribed in CUR 211. However, the partial safety factors to obtain this reliability index differ from the factors in the code. Both codes are listed and compared with the partial safety factors in Table 5-24. It should be noted that the partial factors from the calculations are leveled up.

Parameter	**CUR166 (class III)**	**CUR211 (class II)**	**Calculations (deformations)**
E_{50}	1.00	μ	0.90
E_{oed}	1.00	μ	0.90
E_{ur}	1.00	μ	0.90
φ	1.20	1.00	1.15
C	1.10	1.00	μ
γ	1.00	μ	μ
Water level difference (Δ)	0.30	0.80	0.05
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.00
Retaining height (Δ)	0.35	0.40	0.05

Table 5-38 Partial safety factors for excessive deformations comparison with design codes

With respect to excessive deformations therefore the partial safety factor on φ should be 1.15 to obtain the required reliability. Furthermore, the stiffness parameters should have a higher design value than the characteristic value instead (subdivision by 0.90). An additional 0.05 m on the governing retaining height and water level difference is required. The governing surcharge load can be used.

5.8.5 Conclusion

The partial safety factors are summarized and generalized in Table 5-39. As the partial factors are quite different for each different mechanism it is more optimal to define partial safety factors for each mechanism (otherwise the anchor would for instance be too heavy). For the SLS mechanism excessive deformations no additional factors need to be defined, because the factors from Table 5-24 fall in between the boundaries of the other mechanisms. When the deformations are not large when checking 'anchor failure', the situation is fine. When this is not fine, the deformations can be additionally checked by using the right column of the table.

An important notion is that application of these generalized parameters leads to a suboptimal design. This is because the partial safety factors are applied on each layer, whereas the probabilistic calculations often show that the uncertainty in one layer is dominant. For instance, for soil mechanical failure only factors need to be placed on the lower sand layer. This is similar for wall failure in bending. When placing factors on each layer, a heavier and longer wall is chosen and a too high reliability index is obtained.

It is however not practical to define partial safety factors per layer. Moreover, real situations are not equal to one of the benchmark quay walls. The most convenient way would be to do probabilistic calculations (or sensitivity analyses) to define the most important layers and parameters. Based on these analyses the partial safety factors can be placed on the correct parameters and a more optimal design can be made.

Parameter	**CUR166 (class III)**	**CUR211 (class II)**	**Calculations (generalized)**			
			Anchor	**Wall**	**Soil**	**Deformations**
$EI_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$EA_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$w_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$W_{sheet\text{-}pile}$	1.00	1.20	μ	μ	μ	μ
D_a	1.00	1.10[9]	μ	μ	μ	μ
$f_{y,steel}$	1.00	1.00[9]	1.00	μ	μ	μ
EA_a	1.00	1.20[9]	μ	μ	μ	μ
E_{50}	1.00	μ	0.45	1.75	2.05	0.90
E_{oed}	1.00	μ	0.45	1.75	2.05	0.90
E_{ur}	1.00	μ	0.45	1.75	2.05	0.90
φ	1.20	1.00	1.80	2.40	2.90	1.15
C	1.10	1.00	μ	μ	μ	μ
γ	1.00	μ	1.05	0.95	0.85	μ
Water level difference (Δ)	0.30	0.80	0.10	0.10	0.00	0.00
Water level left (outside) (Δ)	0.25	0.70	-	-	-	-
Water level right (ground) (Δ)	0.05	0.10	-	-	-	-
Surcharge load	1.00	1.20	1.05	1.00	1.05	1.05
Retaining height (Δ)	0.35	0.40	0.10	0.15	0.10	0.05

Table 5-39 Generalized partial safety factor anchored sheet-pile over the characteristic value

Based on Table 5-39 it is advised to use different design values per mechanism for the soil stiffness parameters, the internal angle of friction and the specific soil weight.

[9] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force: ($F_{max} = A\sigma_y = 1/4\pi D_a^2\sigma_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on σ_y.

$$E_{d,anchor-force} = \frac{E_k}{0.45}, E_{d,bending-moment} = \frac{E_k}{1.75} \text{ and } E_{d,soil-collapse} = \frac{E_k}{2.05}$$

$$\varphi_{d,anchor-force} = \frac{\varphi_k}{1.80}, \varphi_{d,bending-moment} = \frac{\varphi_k}{2.40} \text{ and } \varphi_{d,soil-collapse} = \frac{\varphi_k}{2.90}$$

$$\gamma_{d,anchor-force} = \frac{\gamma_k}{1.05}, \gamma_{d,bending-moment} = \frac{\gamma_k}{0.95} \text{ and } \gamma_{d,soil-collapset} = \frac{\gamma_k}{0.85}$$

The additional water level difference can be generalized to 10 cm, the factor on the surcharge load to 1.05 and the absolute factor on the retaining height to 10 cm for all mechanisms. Furthermore for the sheet-pile parameters, cohesion and the anchor parameters mean values can be used. This is except for the yield stress of steel of the anchor, for which the characteristic value needs to be used.

The trend in partial safety factors for the original benchmark (Table 5-25) and the elongated benchmark (Table 5-39) is the same, but the partial factors are different. The factors for the elongated wall are higher for anchor and wall failure, but lower for soil mechanical failure. Basically, it follows from the factors that the structure is less sensitive to soil mechanical failure when the wall is elongated. However, due to this elongation the stiffness parameters and internal angle of friction of the densely packed sand layer get higher influence factors in the LS for anchor and wall failure. As the wall is longer more reduction of the internal angle of friction is necessary to get the wall into motion or to reduce the fixing moment, which causes the influence coefficient to increase.

This is most clear for the mechanism anchor failure, where the stiffness of the soil is increased more (division by 0.45). This means that less mobilization of the shear resistance occurs, which gives a higher horizontal resultant force that is counteracted by the anchor force. Both the results for the original and elongated anchored sheet-pile are used to draw final conclusions, i.e. the most extreme values are included in the final table, combined with the results of benchmark 2.

The reliability indices are summarized in Table 5-40. With respect to the reliability indices obtained in the probabilistic calculation it can be concluded that for anchor failure and wall failure the required reliability index is close to the prescribed reliability index. For soil mechanical failure β is still too low, unless the elongation of the wall. However, this β underestimated due to the target value of MSF = 1.1 (in section 7.4 there is made an estimate of this underestimation). It can be concluded that the approach as presented in CUR 166 chapter 4 to design sheet-piles is applicable for anchored sheet-piles like benchmark 1, but that still some solution needs to be found for the too low reliability index for soil mechanical failure. This is further discussed in the conclusion of benchmark 2 (section 6.6.6) and later on as additional discussion topic (section 7.4).

	β calculated	**β CUR 211**
Anchor failure	4.09	3.828
Wall failure	3.96	3.872
Soil mechanical failure	3.38	4.396
Deformations	2.69	1.800

Table 5-40 Reliability indices failure mechanisms

When the reliability indices from the original benchmark calculation (Table 5-26) and the elongated benchmark (Table 5-40) are compared it is clear that the reliability for soil mechanical failure is increased by the elongation of the wall. Furthermore the reliability of the anchor is increased, whereas the indices for wall failure and excessive deformations remain almost the same.

6 Quay wall with relieving floor (benchmark 2)

6.1 Introduction

This chapter describes the analysis and results of the second benchmark quay wall, a quay wall with relieving floor. This structure is first redesigned according to CUR 166 chapter 4 in section 6.3, to enable to make comparisons between the design code and the calculation results. Furthermore, the parameter values are presented and the settings of Prob2B are discussed. In section 6.6 the eventual PLAXIS-Prob2B calculation results are presented for the different Limit States.

6.2 Characteristics of the structure

The second benchmark quay wall is a quay wall structure with relieving floor as shown in the drawing of Figure 6-2. The design is presented in the report of Bakker and Jaspers Focks (2011). The quay wall is derived from the EMO quay in the port of Rotterdam (Figure 6-1). The design is changed by the authors of the report. The combi-wall is placed vertically, whereas the original design has a combi-wall under an angle. This change made it necessary to stiffen the structure artificially by an additional fixed end anchor. Otherwise the deformations of the top exceeded 50 mm, which is not acceptable for the crane on top of the structure. The additional fixed end anchor is however not a realistic solution, because it is a PLAXIS trick to counter excessive deformations. In Figure 6-3 the PLAXIS model of the structure is shown.

Figure 6-1 Impression of EMO quay wall (Huitema, 2011)

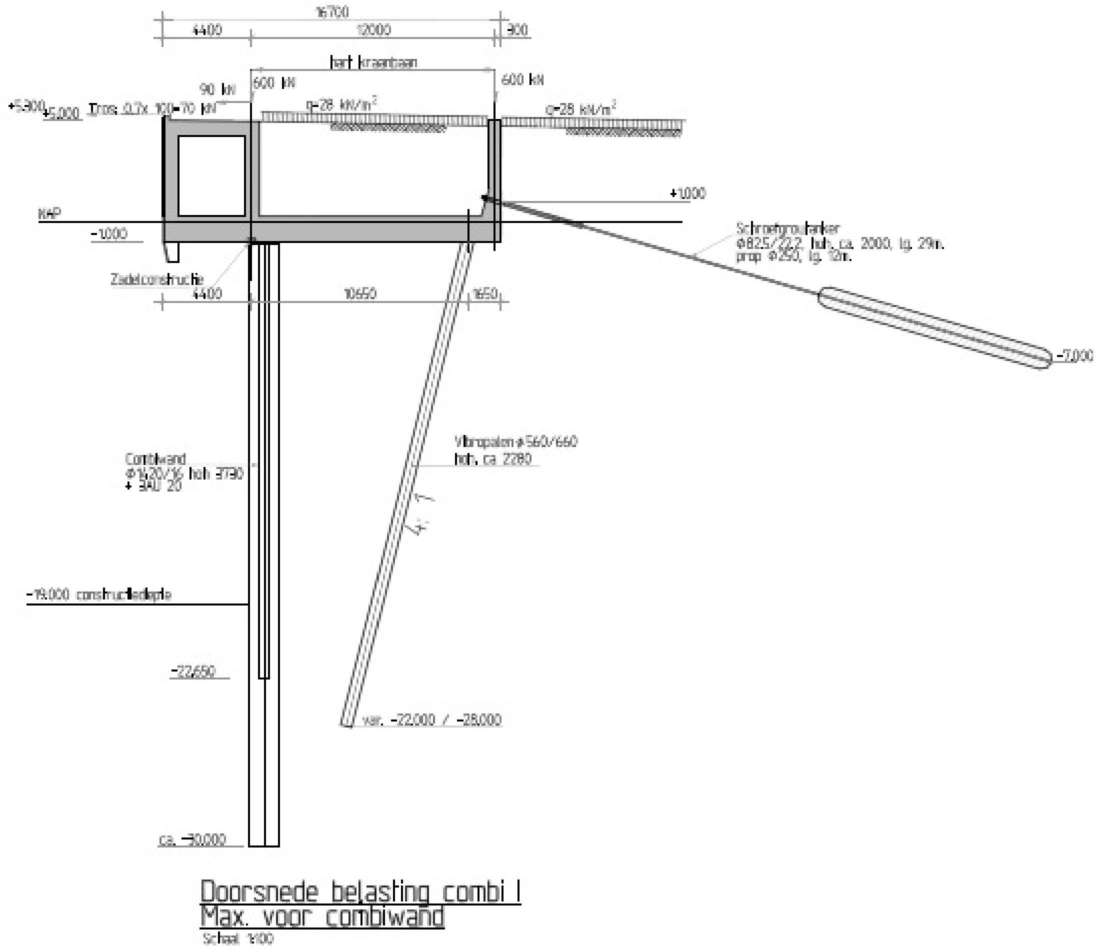


Figure 6-2 Drawing cross-section EMO quay wall (vertical wall)

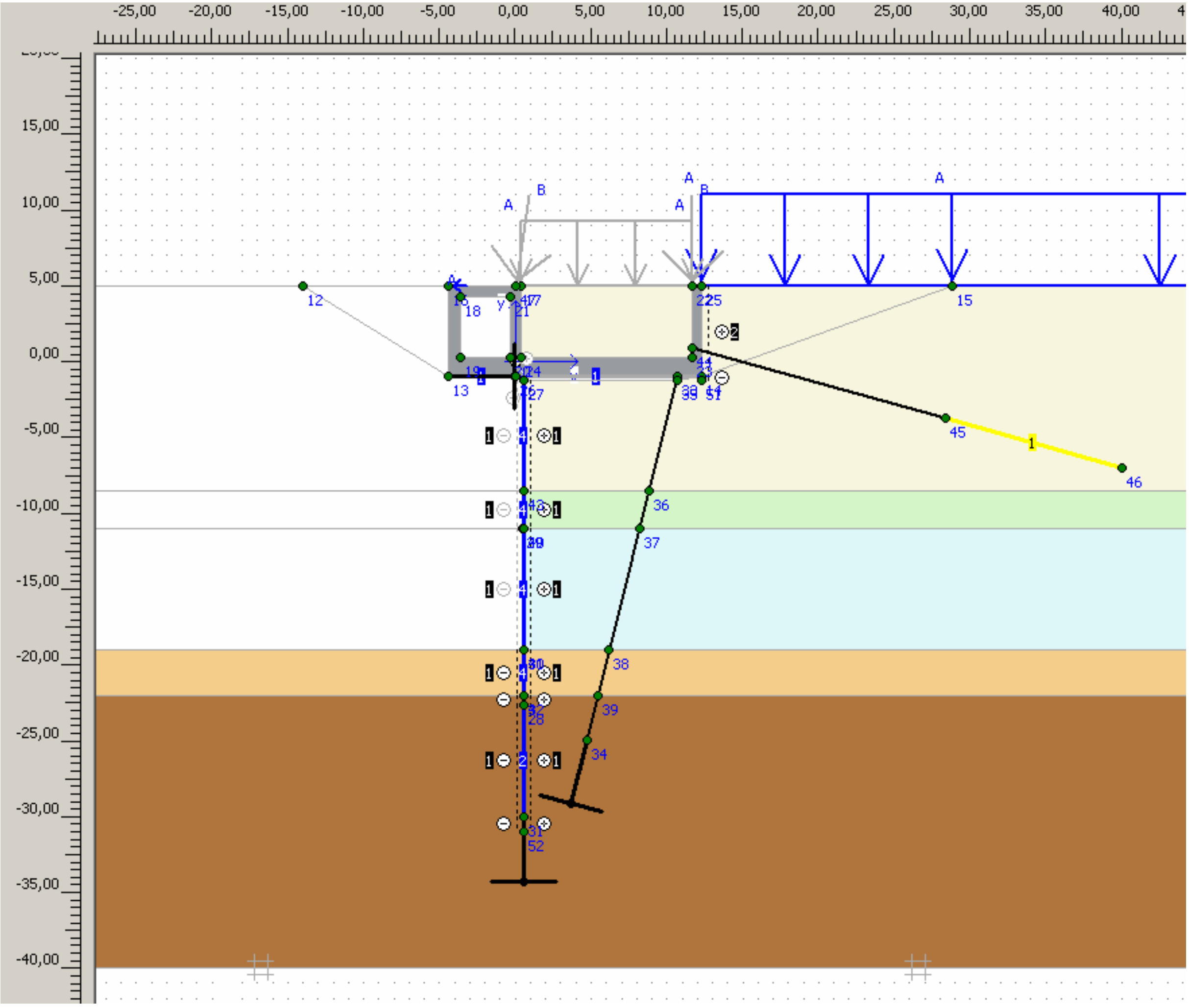


Figure 6-3 Quay wall with relieving floor

The characteristic soil parameter values are presented in Table 6-1. All soil stiffness parameters are determined at a reference level of 100 kPa. This is the case for all calculations in this chapter as PLAXIS uses this as standard input reference level. Another important note is that the properties of the vibropile and the concrete superstructure are not changed during the different (design) calculations. Their parameter values can be found in the report of Bakker and Jaspers Focks (2011).

Layer	NAP	Ground level	Class	γ	C	φ	ψ	E_{50}	E_{oed}	E_{ur}	R_{int}	m	OCR
[-]	[m]	[m]	[-]	[kN/m^3]	[kPa]	[°]	[°]	[MPa]	[MPa]	[MPa]	[-]	[-]	[-]
1	+5	0	moderately packed sand	18/20	1	32.5	2.5	40	40	120	0.9	0.5	1.1
2	-8.5	-13.5	clay 1	17/17	10	22.5	0	10	5	50	0.67	1.0	1.1
3	-11	-16	silty moderately packed sand	19/19	1	30	0	20	20	80	0.9	0.5	1.1
4	-19	-24.5	clay 2	16/16	10	20	0	8	4	32	0.67	1.0	1.1
5	-22	-27	pleistocene sand	19/20	1	36	6	60	60	210	0.9	0.5	1.1

Table 6-1 Characteristic soil parameter values

The combi-wall consists of 1420/16 piles and 3 sheet-pile profiles AU20 in between. The anchor has a diameter of 43 mm. The depth of the relieving floor is NAP -1 m which is -6 m below ground level (NAP + 5.0 m) and the length of the combi-wall is 29 m. The retaining height is 24 m. The length of the anchor is 29 m of which 12 m is grout. The vibropile is modelled as node to node anchor. Bakker and Jaspers Focks (2011) tried also the plate to model the screen effect that reduces the soil pressure on the wall. However the two authors advice to use the modelling of the node tot node anchor, which implies that the screen effect is not included in this model.

There are six building phases:
1. Flat ground level, K0-procedure in PLAXIS (initial water level is at NAP)
2. Excavating the building pit and placing the structure (combi-wall, vibropile, superstructure and anchor)
3. Backfilling the superstructure and prestressing the anchor till 300 kN
4. Excavation till NAP -19 m.
5. Adaptation of water levels. The ground water level is at NAP + 0.65 m and the outside water level is at NAP -1.4 m. The deep sand layer has a hydraulic head of NAP and in the clay layer between NAP -19 and -22 the stress course is linear
6. Applying loads. There are two Load Conditions (LC), but only LC2 is included as this one appeared to be governing in the calculations from Bakker and Jaspers Focks.

LC1:
- Surcharge load on super structure and behind quay wall (starting from x=0, which is the horizontal coordinate of the combi-wall) of 28 kN/m^2
- Crane load of 600 kN/m vertical at x=0 and x= 12 m
- Surcharge load on and behind superstructure of 90 kN/m^2 starting from x=0
- Bollard force of 70 kN/m at x = -4.4 m.

LC2:
- Surcharge load behind superstructure (starting from x = 12.3 m) of 40 kN/m^2.
- Bollard force of 70 kN/m at x = -4.4 m.

However some stages are combined in the probabilistic calculations in order to reduce calculation time. It is confirmed that this adaptation has a negligible influence on the PLAXIS output. The four building stages are therefore:
1. Flat ground level, K0-procedure in PLAXIS (initial water level is at NAP)
2. Excavating the building pit, placing the structure (combi-wall, vibropile, superstructure and anchor), backfilling the superstructure and prestressing the anchor till 300 kN
3. Excavation till NAP -19 m.
4. Adaptation of the ground water level to NAP + 0.65 m and the outside water level to NAP -1.4 m and applying the load conditions LC2.

6.3 Redesigned quay wall with relieving floor

The current configuration has been redesigned in order to have a design according to CUR 166 chapter 4 (FEM). It can be checked whether the method of CUR 166 is applicable in CUR 211. Furthermore it is important to remove the trick with the artificial quay anchor. This anchor is replaced by an additional realistic anchor.

CUR 166 chapter 4 prescribes to design the wall according to the maximum bending moment that is obtained from a φ-C reduction up to MSF = 1.2 (safety class III). Subsequently the anchor has to be designed according to a maximum anchor force obtained at the same MSF = 1.2. Note that the φ and C parameters should have their characteristic value and the stiffness and geometrical parameters their design values. CUR 211 prescribes an additional factor 1.2 on the anchor design force. This prescription is also taken into account in the redesign, to check whether this design rule is useful. To find the new anchor configuration the requirement of a maximum 50 mm displacement of the top of the structure is taken into account.

Several solutions with new anchors have been tried. The most successful is the option with a second anchor that is placed under an angle of 45 degrees and a length of 42.6 m of which 14.1 m is grout. Both the original anchor as well as the additional anchor must be prestressed with a prestress force of 600 kN. In that case the anchor diameter of both anchors can be the same, because the forces are relatively well distributed over the two anchors after a phi/c reduction up to MSF = 1.2. The new lay out is shown in Figure 6-4. The upeer anchor is called ‘anchor 1’ and the lower ‘anchor 2’.

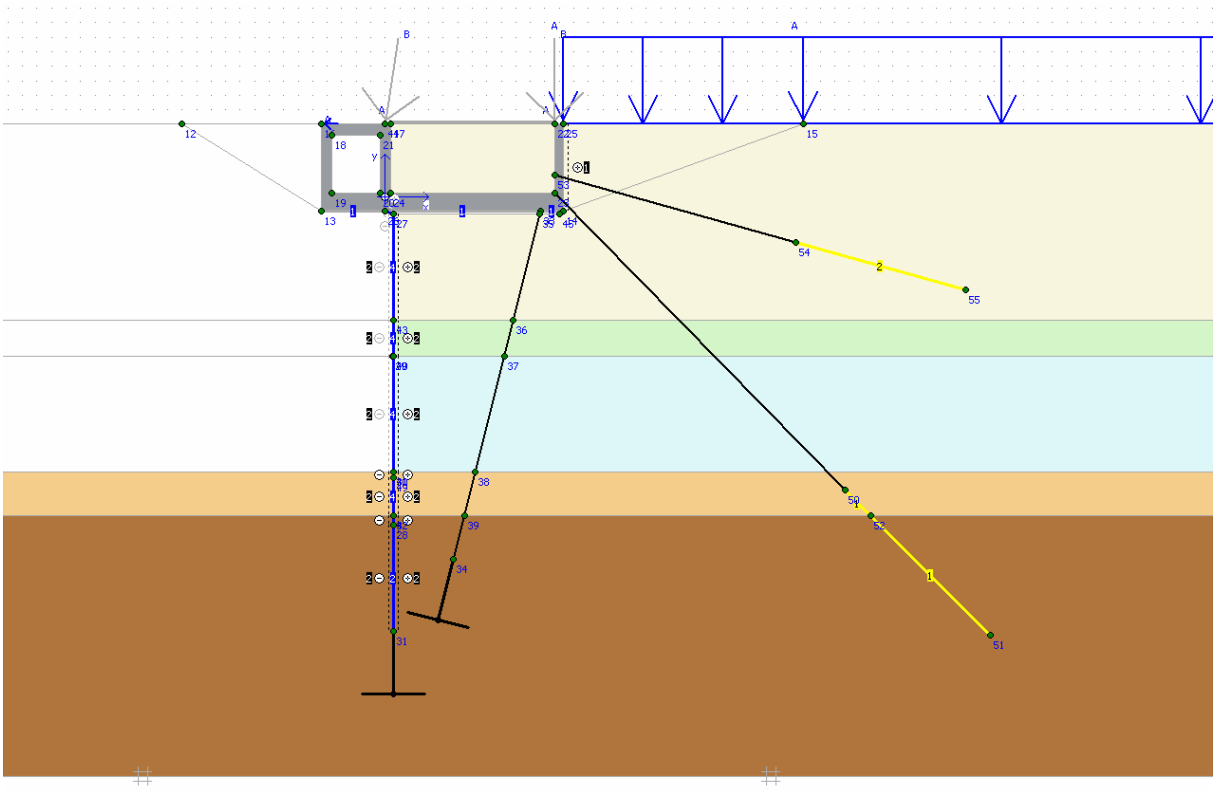


Figure 6-4 New lay-out quay wall with relieving floor

The geometrical design values are (according to CUR 166 table 3.7):

- Retaining height = NAP -19.35 m
- Ground water level low side = NAP -1.65 m
- Ground water level high side = NAP +0.7 m
- Surcharge load = 40 kN/m^2
- Bollard force = 70 kN/m

For the soil stiffness parameters two options are given in CUR 166, a low and high design value:

$$E_{d;low} = \frac{E_{k;low}}{1.3} = \frac{E_{mean}}{2} \text{ and } E_{d;high} = \frac{E_{k;high}}{1.0} = 1.5E_{mean}$$

Both have been applied and the $E_{d,low}$ appeared to be governing for both the bending moment in the wall and the anchor force.

It is assumed that the sheet-pile parameters, anchor parameter, soil weight and dilatancy angle have their characteristic value as well, because CUR 166 defines a partial safety factor of 1.0 for these parameters. The input for the design calculation is given in Table 6-1.

Parameter	**Type**	**Value**	**Unit**
Soil			
$\gamma_{unsat,moderately\ packed\ sand}$	X_k	18	[kN/m^3]
$\gamma_{sat,moderately\ packed\ sand}$	X_k	20	[kN/m^3]
$\gamma_{sat,\ clay\ 1}$	X_k	17	[kN/m^3]
$\gamma_{sat,silty\ moderately\ packed\ sand}$	X_k	19	[kN/m^3]
$\gamma_{sat,\ clay\ 2}$	X_k	16	[kN/m^3]
$\gamma_{sat,pleistocene\ sand}$	X_k	20	[kN/m^3]
$\varphi_{moderately\ packed\ sand}$	X_k	32.5	[°]
$\varphi_{clay\ 1}$	X_k	22.5	[°]
$\varphi_{silty\ moderately\ packed\ sand}$	X_k	30	[°]
$\varphi_{clay\ 1}$	X_k	20	[°]
$\varphi_{pleistocene\ sand}$	X_k	36	[°]
$C_{clay\ 1}$	X_k	10	[kPa]
$C_{clay\ 1}$	X_k	10	[kPa]
$\psi_{moderately\ packed\ sand}$	X_k	2.5	[°]
$\psi_{pleistocene\ sand}$	X_k	6	[°]
$E_{50,moderately\ packed\ sand}$	X_d	30.8	[MPa]
$E_{50,clay\ 1}$	X_d	7.7	[MPa]
$E_{50,silty\ moderately\ packed\ sand}$	X_d	15.4	[MPa]
$E_{50,clay\ 2}$	X_d	6.2	[MPa]
$E_{50,pleistocene\ sand}$	X_d	46.2	[MPa]
$E_{oed,moderately\ packed\ sand}$	X_d	30.8	[MPa]
$E_{oed,clay\ 1}$	X_d	3.8	[MPa]
$E_{oed,silty\ moderately\ packed\ sand}$	X_d	15.4	[MPa]
$E_{oed,clay\ 2}$	X_d	3.1	[MPa]
$E_{oed,pleistocene\ sand}$	X_d	46.2	[MPa]
$E_{ur,moderately\ packed\ sand}$	X_d	92.3	[MPa]
$E_{ur,clay\ 1}$	X_d	38.5	[MPa]
$E_{ur,silty\ moderately\ packed\ sand}$	X_d	61.5	[MPa]
$E_{ur,clay\ 2}$	X_d	24.6	[MPa]
$E_{ur,pleistocene\ sand}$	X_d	161.5	[MPa]
1420/16 3AU20			
$EI_{1420/16\ 3AU20}$	X_k	$1.031*10^6$	[kNm^2/m]
$EA_{1420/16\ 3AU20}$	X_k	$6.058*10^6$	[kN/m]
$w_{1420/16\ 3AU20}$	X_k	2.265	[kN/m]
$W_{1420/16\ 3AU20}$	X_k	$7.699*10^{-3}$	[m^3/m]
$f_{y,steel}$	X_k	$355*10^3$	[kN/m^2]
Anchor 1 and 2			

D_a	X_k	0.0428	[m]
EA_a	X_k	$302*10^3$	[kN/m]
$f_{y,steel}$	X_k	$355*10^3$	[kN/m^2]

Table 6-2 Input parameters design calculation

When the deterministic calculation is made, after φ-C reduction till MSF = 1.2 the design bending moment is:

$M_d = 2656$ kNm/m

And the design maximum normal force in the plate:

$N_d = 1132$ kN/m

A combination of pile diameter and wall thickness and sheet-pile profile should be chosen in which the yield stress of the steel is not exceeded at the design bending moment combined with the design axial force. The maximum moment and maximum axial force are assumed to be present at the same place in the wall. This is a conservative estimate, because the maximum normal force is most probably lower in the wall (closer to the toe) than the maximum bending moment. The following equation should hold:

$$f_{y,steel} = \frac{M_d}{W_{combi-wall}} + \frac{N_d}{A_{combi-wall}} \text{ kN/m}^2$$

If this equation is filled in for the current combi-wall the required σ_{steel} is:

$$\sigma_{steel} = \frac{2656}{7.699 \cdot 10^{-3}} + \frac{1132}{0.02885} = 384217 \text{ kN/m}^2$$

This stress exceeds the yield stress of 355000 kN/m^2. Therefore the pile is made 2 mm thicker which gives a combi-wall 1420/18 3AU20, with specifications:

Cross-sectional area	A	= $3.127*10^{-2}$	m^2
Inertial moment	I	= $5.485*10^{-3}$	m^4
Resisting moment	W	= $8.491*10^{-3}$	m^3
Weight	w	= 2.559	kN/m

With this combi-wall the required σ_{steel} is:

$$\sigma_{steel} = \frac{2656}{8.491 \cdot 10^{-3}} + \frac{1132}{3.127 \cdot 10^{-2}} = 349003 \text{ kN/m}^2$$

This is smaller than the yield stress and therefore this combi-wall is applied in the design. This design is very close to the original design, which is logical as the quay wall is derived from the actual built EMO quay wall.

This gives PLAXIS input parameters:

Flexural rigidity	EI	= $1.1518*10^6$	kNm^2
Axial stiffness	EA	= $6.5660*10^6$	kN
Weight	w	= 2.559	kN/m

For the bottom part of the combi-wall (only the piles and no sheet-pile profiles in between) this configuration gives more than enough capacity to resist the combination of bending moment and normal force. If the field moment in the combi-wall increases the fixing moment in the bottom part will reduce, this implies that no problems will occur in the bottom part. For this part therefore no additional (probabilistic) calculations are done.

The same can be done for the anchor. After φ-C reduction till MSF = 1.2 the design anchor forces are found and multiplied by the factor 1.2 from CUR 211:

$F_{d,anchor1} = 617 * 1.2 = 740$ kN/m

$F_{d,anchor2} = 635 * 1.2 = 762$ kN/m

The difference between those forces is small and therefore there is searched for two anchors with the same diameter.
The anchors need to have a cross-sectional area of:

$$A = \frac{F_{d,anchor2}}{f_y} = \frac{762}{355 \cdot 10^3}$$

As the basic random variable is the anchor diameter, this can be translated to a required minimum diameter:

$$D = \sqrt{\frac{4A}{\pi}} \approx 0.0523 \text{ m}$$

An anchor diameter D_a of 53 mm is chosen.

This gives the PLAXIS input parameter:

Axial stiffness	EA	=463299	kN/m

The maximum anchor force $F_{anchor,max}$ for this anchor is 783 kN/m.

When redoing the φ-C reduction with the new piles and anchors the following design values are found:

$M_d = 2631$ kNm/m

$N_d = 1154$ kN/m

$$\sigma_{steel,d} = \frac{2631}{8.491 \cdot 10^{-3}} + \frac{1154}{3.127 \cdot 10^{-2}} = 334929 \text{ kN/m}^2$$

$F_{d,anchor1} = 618 * 1.2 = 742$ kN/m

$F_{d,anchor2} = 641 * 1.2 = 769$ kN/m

In the probabilistic calculations however mean values are used. For the soil parameters the coefficients of variation in the design are based on NEN 6740 (2006), which implies that the mean value can be calculated from the formula:

$$\mu_i = X_{k,i} + 1.64 \cdot V_{NEN6740_i} \cdot \mu_i$$

$$\mu_i = \frac{X_{k,i}}{1 - 1.64 V_{NEN6740,i}}$$

Except for the stiffness parameters, because for these the formula of CUR 166 is used:

$$\mu_{Esoil} = \frac{2}{1.3} * E_{k;low,soil}$$

For the structural parameters the CoV's are based on JCSS (2002) in combination with MC simulations (Appendix C) and the mean value follows from:

$$\mu_i = \frac{X_{k,i}}{1-1.64V_{JCSS,i}}$$

All mean values are given in Table 6-3. Note that cohesion (C) of the sand layers is not included in this analysis, because their value (1.0) is in reality even smaller. It is therefore assumed that C of sand does not influence the probability of failure. In fact it is predictable that this assumption holds, because there is dealt with a relative large retaining height (24 m). This implies that the shear resistance will be mainly determined by φ, because this parameter is stress dependent and therefore depth dependent ($\tau = C + \sigma' \tan(\varphi)$). C is only important in case of a small retaining height or a large C value (for instance in clay layers).

Parameter	**Unit**	**X_k**	**$V_{NEN6740/JCSS}$**	**μ**
Soil				
$\gamma_{unsat,moderately\ packed\ sand}$	[kN/m^3]	18	0.05	19.61
$\gamma_{sat,moderately\ packed\ sand}$	[kN/m^3]	20	0.05	21.79
$\gamma_{sat,\ clay\ 1}$	[kN/m^3]	17	0.05	18.52
$\gamma_{sat,silty\ moderately\ packed\ sand}$	[kN/m^3]	19	0.05	20.70
$\gamma_{sat,\ clay\ 2}$	[kN/m^3]	16	0.05	17.43
$\gamma_{sat,pleistocene\ sand}$	[kN/m^3]	20	0.05	21.79
$\varphi_{moderately\ packed\ sand}$	[°]	32.5	0.1	38.88
$\varphi_{clay\ 1}$	[°]	22.5	0.1	26.91
$\varphi_{silty\ moderately\ packed\ sand}$	[°]	30	0.1	35.89
$\varphi_{clay\ 2}$	[°]	20	0.1	23.92
$\varphi_{pleistocene\ sand}$	[°]	36	0.1	43.06
$C_{clay\ 1}$	[kPa]	10	0.2	14.88
$C_{clay\ 2}$	[kPa]	10	0.2	14.88
$\Psi_{moderately\ packed\ sand}$	[°]	2.5	0.1	2.99
$\Psi_{pleistocene\ sand}$	[°]	6	0.1	7.18
$E_{50,moderately\ packed\ sand}$	[MPa]	40	CUR 166	61.5
$E_{50,clay\ 1}$	[MPa]	10	CUR 166	15.4
$E_{50,silty\ moderately\ packed\ sand}$	[MPa]	20	CUR 166	30.8
$E_{50,clay\ 2}$	[MPa]	8	CUR 166	12.3
$E_{50,pleistocene\ sand}$	[MPa]	60	CUR 166	92.3
$E_{oed,moderately\ packed\ sand}$	[MPa]	40	CUR 166	61.5
$E_{oed,clay\ 1}$	[MPa]	5	CUR 166	7.7
$E_{oed,silty\ moderately\ packed\ sand}$	[MPa]	20	CUR 166	30.8
$E_{oed,clay\ 2}$	[MPa]	4	CUR 166	6.2
$E_{oed,pleistocene\ sand}$	[MPa]	60	CUR 166	92.3
$E_{ur,moderately\ packed\ sand}$	[MPa]	120	CUR 166	184.6
$E_{ur,clay\ 1}$	[MPa]	50	CUR 166	76.9
$E_{ur,silty\ moderately\ packed\ sand}$	[MPa]	80	CUR 166	123.1
$E_{ur,clay\ 2}$	[MPa]	32	CUR 166	49.2
$E_{ur,pleistocene\ sand}$	[MPa]	210	CUR 166	323.1
1420/18 3AU20				
$EI_{1420/18\ 3AU20}$	[kNm^2/m]	1.1518*10^6	0.1	1.3777*10^6
$EA_{1420/18\ 3AU20}$	[kN/m]	6.5660*10^6	0.04	7.0269*10^6
$w_{1420/18\ 3AU20}$	[kN/m]	2.559	0.03	2.692

$W_{1420/18\ 3AU20}$	[m^3/m]	$8.491*10^{-3}$	0.07	$9.592*10^{-3}$
$A_{1420/18\ 3AU20}$	[m^2/m]	$3.127*10^{-2}$	0.03	$3.289*10^{-2}$
$f_{y,steel}$	[kN/m^2]	$355*10^3$	0.07	$401*10^3$
Anchor 1 and 2				
D_a	[m]	0.053	0.032	0.0559
EA_a	[kN/m]	463299	0.07	523383
$f_{y,steel}$	[kN/m^2]	$355*10^3$	0.07	$401*10^3$

Table 6-3 Mean parameter values redesigned benchmark 2

For the interface condition R_{int} and power m it is assumed that the characteristic values equal the mean values, as it is not common to work with characteristic values for these parameters. The values are given in Table 6-4.

Parameter	Unit	μ
$R_{int,moderately\ packed\ sand}$	[-]	0.9
$R_{int,clay\ 1}$	[-]	0.67
$R_{int,silty\ moderately\ packed\ sand}$	[-]	0.9
$R_{int,clay\ 2}$	[-]	0.67
$R_{int,pleistocene\ sand}$	[-]	0.9
$m_{moderately\ packed\ sand}$	[-]	0.5
$m_{clay\ 1}$	[-]	1.0
$m_{silty\ moderately\ packed\ sand}$	[-]	0.5
$m_{clay\ 2}$	[-]	1.0
$m_{pleistocene\ sand}$	[-]	0.5

Table 6-4 Mean values R_{int} and m

6.4 Probabilistic input

The coefficients of variation (CoV) for the probabilistic calculations for soil parameters are obtained from Table 4-4 in section 4.7.1.7. However for the angle of internal friction, Prob2B requires the input of sin φ instead of φ. This leads to a CoV of 0.18, which is slightly lower than the 0.2 for φ (based on Monte Carlo (MC) simulation, see Appendix C). Another note is that Prob2B uses the shear modulus of the soil, G, instead of unloading-reloading stiffness modulus $E_{ur.}$. The relation between those parameters is: $G = \frac{E_{ur}}{2(1+\nu)}$, with v is Poisson's ratio, assumed to be 0.2 for all soils.

The correlations of the soil parameters can be found in Table 4-5 in section 4.7.1.7. Also for the specific Prob2B structural parameters, G_{eq} and d (Appendix D) MC simulations are executed. The CoV's for structural parameters are presented in Table 6-5.

Parameter	μ	Unit	V = σ / μ
Combi-wall (1420/18 3AU20)			
$G_{eq,1420/18\ 3AU20}$	1761994	[kN/m^2/m]	0.07
$d_{1420/18\ 3AU20}$	1.534	[kN/m]	0.03
$w_{1420/18\ 3AU20}$	2.692	[kN/m]	0.03
$W_{1420/18\ 3AU20}$	$9.592*10^{-3}$	[m^3/m]	0.07
$A_{1420/18\ 3AU20}$	$3.289*10^{-2}$	[m^2/m]	0.03
$f_{y,steel}$	$401*10^3$	[kN/m^2]	0.07
Anchor 1 and 2			
D_a	0.0559	[m]	0.032
EA_a	523383	[kN/m]	0.07
$f_{y,steel}$	$401*10^3$	[kN/m^2]	0.07

Table 6-5 CoV's structural parameters

Table 6-6 and Table 6-7 present the obtained correlations for the sheet-pile and anchor parameters.

	$G_{eq,1420/18\ 3AU20}$	$d_{1420/18\ 3AU20}$	$w_{1420/18\ 3AU20}$	$W_{1420/18\ 3AU20}$	$A_{1420/18\ 3AU20}$	$f_{y,steel}$
$G_{eq,1420/18\ 3AU20}$		-0.35	0.23	-0.11	0.24	0.00
$d_{1420/18\ 3AU20}$	-0.35		-0.62	0.93	0.65	0.00
$w_{1420/18\ 3AU20}$	0.23	-0.62		0.84	0.96	0.00
$W_{1420/18\ 3AU20}$	-0.11	0.93	0.84		0.87	0.00
$A_{1420/18\ 3AU20}$	0.24	0.65	0.96	0.87		0.00
$f_{y,steel}$	0.00	0.00	0.00	0.00	0.00	

Table 6-6 Correlation matrix sheet-pile parameters (1420/18 3AU20)

	EA	D_a	$f_{y,steel}$
EA_a		0.91	0.00
D_a	0.91		0.00
$f_{y,steel}$	0.00	0.00	

Table 6-7 Correlation matrix anchor parameters (D=0.053 m)

Variations in the retaining height, water level (inside and outside) and surcharge load cannot be included in the probabilistic calculation in Prob2B. These parameters are therefore included manually after the calculations in Prob2B. Variations in the design point (output Prob2B) are made in order to derive influence factors for these additional parameters. Their standard deviations are based on the research of Havinga (2004), who made probabilistic calculations for an anchored sheet-pile to calibrate safety factors for CUR 166 (2005). These standard deviations for the additional parameters are given in Table 6-8.

The parameter values in the governing situation are maintained as 'mean value (μ)' in the manual variations, because the values of Table 6-8 are used during the probabilistic calculations as well. The manual variations can provide conclusions about the influence of the variation in the governing situation on the reliability and not of the influence of the parameter in general as explained in section 4.6. Their derived absolute factor therefore cannot be used to define a factor to design the structure. The factor is based on the governing situation as implemented in the model. Note that the variation in bollard force is not taken into account.

Parameter	μ	Unit	σ/V
Retaining height	24	[m]	0.25 (σ)
Water level difference	2.05	[m]	0.2 (σ)
Surcharge load	40	[kN/m^2]	0.4 (V)

Table 6-8 Additional parameter values and standard deviations

In the design point variations are made to obtain influence factors (according to the FORM procedure). For every LS the following variations are made:

- Retaining height + 0.35 m (lowering excavation depth)
- Water level inside +0.05 m and outside -0.25m
- Surcharge load +4 kN/m^2

These variations are based on the design values according to CUR 166 (table 3.7).

An important note is that it is implicitly assumed that the design point (i.e. the design values of the other parameters) does not change when varying the additional parameters. In fact it is not possible to show that this assumption is right, because the additional parameters should be included from the start of the probabilistic calculation and this is exactly the impossibility of Prob2B. The reliability indices that are obtained in the probabilistic calculation are influenced by the addition of the extra parameters. This influence can be approximated by the formula: $\beta_{NEW} = \beta_{OLD} \cdot \sqrt{1-\alpha_1^2-\alpha_2^2-...-\alpha_n^2}$ in which α_i are the influence coefficients of the additional parameters. These coefficients can be derived from the variation in governing water levels, retaining height and surcharge load.

6.5 Practical aspects and settings Prob2B

As there is learnt from the previous benchmark probabilistic calculations, all parameters are included in one first calculation for each LSF. This implies longer but less calculations. Problems (for instance non-convergent calculations) that often occurred in the first benchmark are encountered due to the enlarged experience. The most important parameters are included in a final calculation. The criterion for a parameter to be eliminated is an influence on the reliability of less than 5%, but there are some exceptions that are explained when they come into play.

The soil stiffness parameters have an important connotation. The correlation between the three parameters is 1, so they are completely correlated. Prob2B however has problems to show this in the output of the results. It turned out to be the case that only one of the three stiffness parameters was picked out to get an influence factor larger than zero, whereas the other parameters get an influence factor zero. It can however be concluded that all stiffness parameters should get the same influence as they are 100% correlated in the same layer. This is why in the definition of the final partial safety factors all stiffness parameters get the same partial safety factor.

Prob2B requires some specific FORM settings that are visible in Figure 6-5. Depending on the calculation different relaxation values are used. Sometimes the calculation needs more manual steering in form of a user defined start U-vector and a low relaxation value. Sometimes Prob2B can find the design point from the zero U-vector and with a high relaxation value. Most of the time 0.3, which is the default setting for relaxation factor, is maintained. When convergence is difficult to reach, a lower value is chosen.

An important note with respect to the accuracy of the calculations is that in the first elimination calculation (to see whether parameters can be excluded from the analysis) the convergence criteria are taken 0.1. However, in the final calculation per LS these criteria are stricter, i.e. 0.01. This latter criterion is not always met in the calculation. In that cases no convergence of the FORM calculation was found by Prob2B. It is mentioned in the report when another (less strict) criterion is taken into account.

The perturbation factor is the part of the standard deviation that is used as the variation in the parameter in order to calculate the derivative for influence factor α. The perturbation factor is kept on its default value 0.3, just as the setting to take an 1-sided derivative. Due to non-linearities these settings can influence the result and therefore it is better to keep the default value for all calculations.

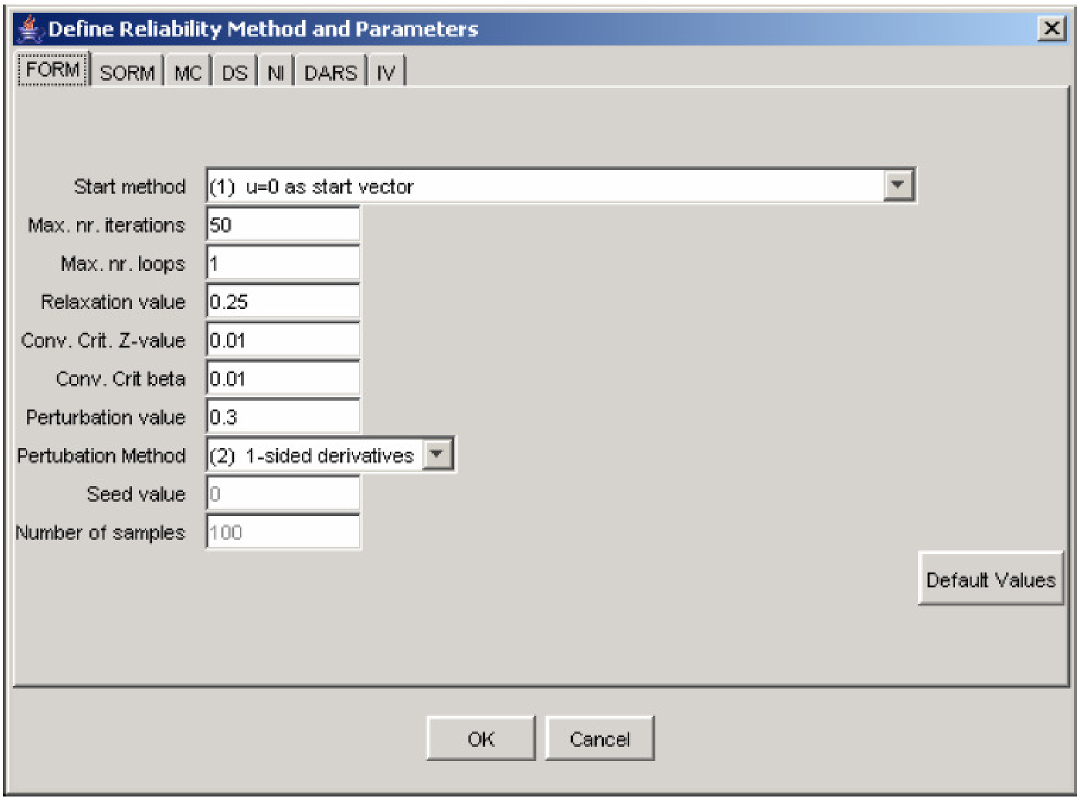


Figure 6-5 Settings FORM in Prob2B (Courage & Steenbergen, 2007)

6.6 Limit State evaluations PLAXIS-Prob2B

The five relevant Limit States (LS) are evaluated in this section. Note that the first LS evaluation contains the most extensive explanation. The text in the other LS evaluations refers to these explanations.

6.6.1 Anchor 1 failure (ULS)

The LSF for anchor 1 failure in tension is: $Z = \frac{1}{4}\pi D_a^{\ 2} f_y - F_{anch1,stage4}$

First there is made a calculation with all stochastic parameters included. The output is presented in Appendix L.

6.6.1.1 Final calculation

When the irrelevant parameters are eliminated, the following parameters are included in the final calculation:

- D_a
- $f_{y,steel}$
- $EA_{anchor1}$
- $G_{moderately\ packed\ sand}$
- $G_{silty\ moderately\ packed\ sand}$
- $G_{pleistocene\ sand}$
- $\sin(\varphi)_{silty\ moderately\ packed\ sand}$
- $\sin(\varphi)_{pleistocene\ sand}$
- $\gamma_{sat,silty\ moderately\ packed\ sand}$
- $\gamma_{sat,pleistocene\ sand}$
- $\gamma_{unsat,moderately\ packed\ sand}$

In this calculation many parameters did not reach the 5% influence in the first calculation (this holds for D_a, $G_{moderately\ packed\ sand}$, $G_{pleistocene\ sand}$, $\sin(\varphi)_{silty\ moderately\ packed\ sand}$ and γ_{sat} of the three sand layers). These parameters are nevertheless included in order to assure not to underestimate the reliability index too much. When many parameters have influence of about 3-4% per parameter in total their influence is larger and when all of them are excluded the reliability index can be underestimated.

The output is given in Table 6-9 and Figure 6-6. Normally convergence criteria for β and Z of 0.01 are applied for the calculations, but in this case 0.05 is used. The reason for this less strict criteria is the fact that Prob2B has troubles in finding convergence for this configuration.

Number of calculations (FORM): 118				
β: 4.400				
P_f: $5.412 \cdot 10^{-6}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
D_a	0.032	0.11	0.0521	[m]
$f_{y,steel}$	0.07	0.61	325900	[kN/m^2]
$EA_{anchor1}$	0.07	0.47	453300	[kN/m]
$G_{moderately\ packed\ sand}$	0.3	-0.09	80280	[kPa]
$G_{silty\ moderately\ packed\ sand}$	0.3	-0.19	23910	[kPa]
$G_{pleistocene\ sand}$	0.3	-0.47	119900	[kPa]
$\sin(\varphi)_{silty\ moderately\ packed\ sand}$	0.18	0.16	0.50	[-]
$\sin(\varphi)_{pleistocene\ sand}$	0.18	0.23	0.39	[-]
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.05	-0.23	20.69	[kN/m^2]
$\gamma_{sat,pleistocene\ sand}$	0.05	0.00	20.83	[kN/m^2]
$\gamma_{unsat,moderately\ packed\ sand}$	0.05	-0.03	19.96	[kN/m^2]
calc.	Z-value			

1	483	
118	13.05	*convergence criteria 0.05*

Table 6-9 Output final calculation (LS Anchor 1)

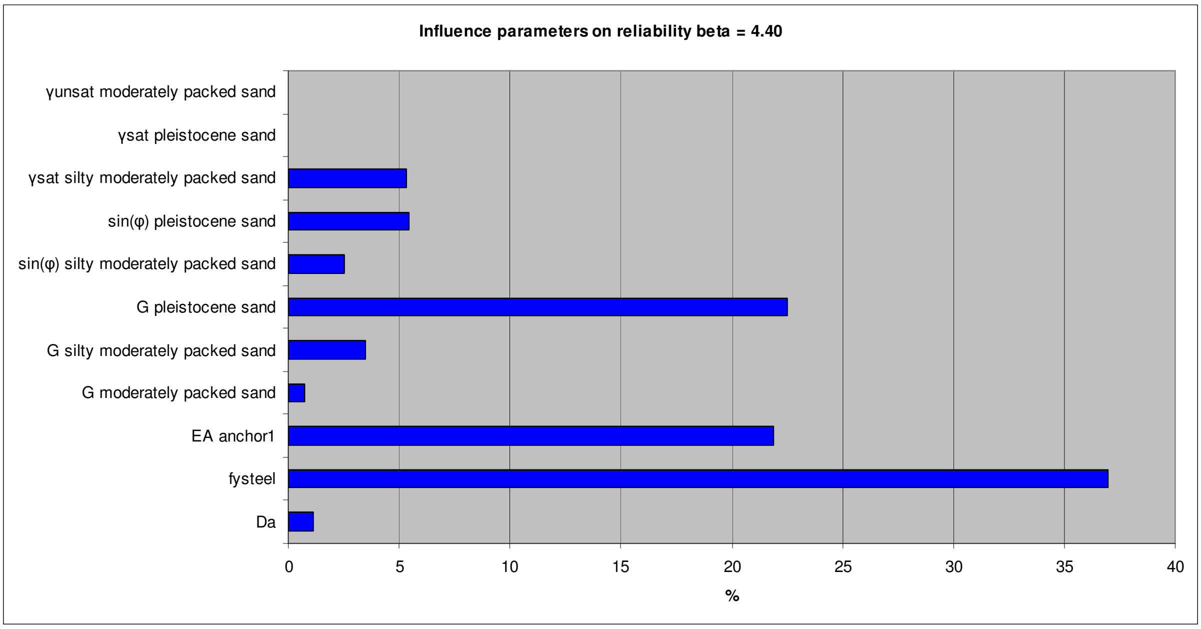


Figure 6-6 Influence in % from parameters on reliability final calculation (LS Anchor 1)

Figure 6-7 shows the plastic points when the design values are given to the parameters. The red blocks are points in the soil where local soil failure occurs due to the exceedance of the Mohr-Coulomb maximum shear resistance criterion ($\tau = C + \sigma' \tan(\varphi)$). The red spots lay on the Mohr-Coulomb failure envelope. A blue spot (cap point) represents a state of normal consolidation where the preconsolidation stress is equivalent to the current stress state. The green and green/blue spots show the soil elements where shear hardening (according to the hardening soil model) occurs. The green and green/blue spots lay on the shear hardening envelope (mobilized friction envelope). The white elements are elements where the tension cut-off criterion (i.e. no tensile stresses allowed) is applied.

Anchor 1 failure is mostly induced by the reduction of the anchor stiffness and strength. Furthermore failure of the soil in the lower sand layer (pleistocene) is important as there are many red spots in that area. Basically the fixing of the wall is reduced and the structure moves forward which causes the upper anchor to react on this motion. This leads to an increased anchor force. The importance of $\varphi_{\text{pleistocene sand}}$ is not following from Figure 6-6, but it can be seen from the value in the design point, which is 0.39 for sin(φ), whereas the mean value is 0.67. The influence parameters however are influenced by the correlations between the soil parameters in the same soil layer. This is why $G_{\text{pleistocene sand}}$ gains large importance as it influences φ by its correlation (0.25). The importance of $G_{\text{pleistocene sand}}$ is increased by the fact that elastic and plastic behavior is important in the lower sand layer. The green blocks show the spots where shear hardening occurs which influences the interaction between the motion of the sheet-pile and the motion of the soil around it. The stiffer the soil the less the mobilization of the shear resistance (especially important at the passive side) and the higher the anchor force should be to guarantee horizontal equilibrium. This explains the negative value for $\alpha_{\text{G pleistocene sand}}$, the stiffness is higher than the mean value in the design point.

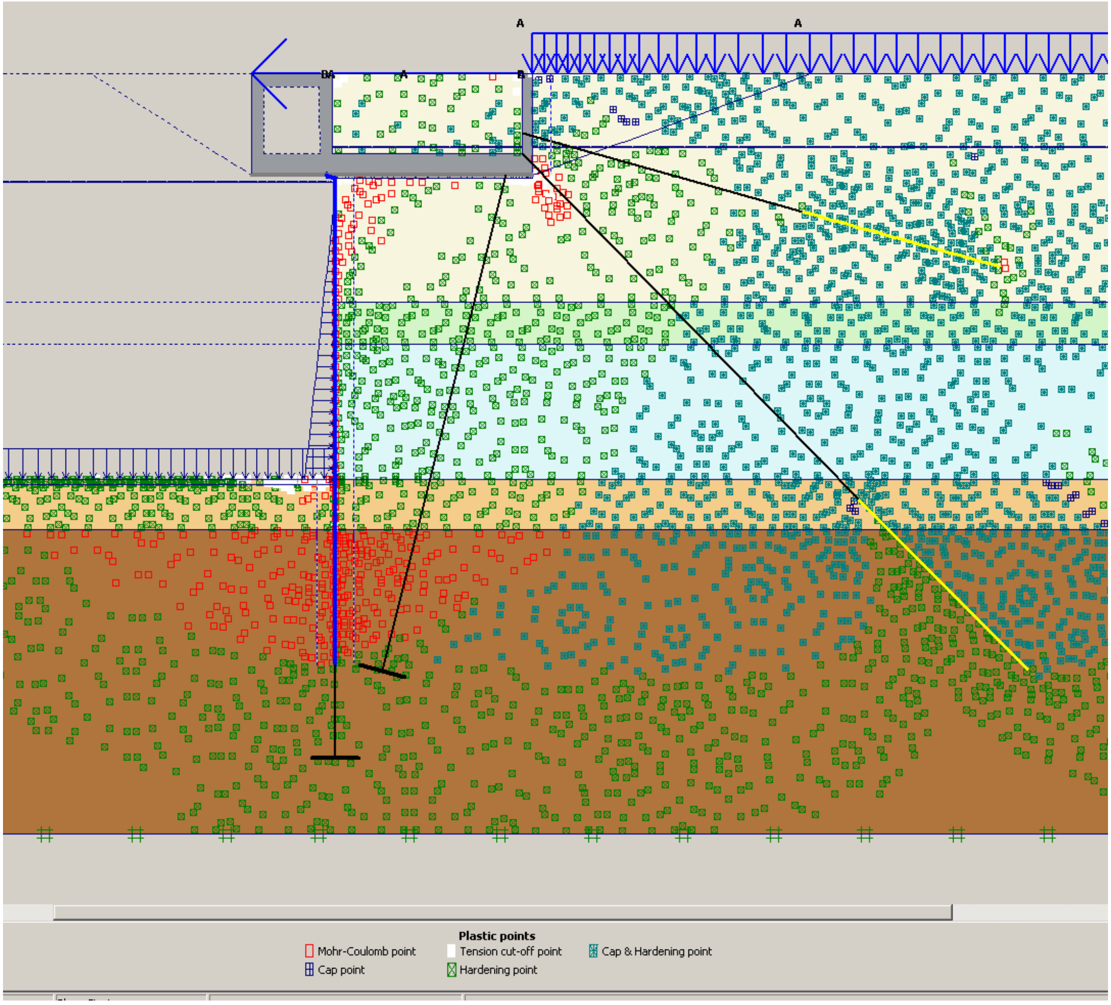


Figure 6-7 Plastic points in design point anchor 1 failure

6.6.1.2 Partial safety factors

From these results partial safety factors can be derived, by applying the formula: $\gamma_{\mu,i} = \dfrac{\mu_i}{\mu_i - \alpha_i \beta \sigma_i}$

This can be done for both the β obtained in the calculation as well for the target β defined in the failure tree of CUR 211 (Figure 3-2 in section 3.3.2).

Furthermore there can be made a translation to a safety factor over the characteristic value, by the following formula: $\gamma_{k,i} = \gamma_{\mu,i} * (1 - 1.64 V_{NEN6740/JCSS,i})$

(for the lognormal distributed soil stiffness parameters $\gamma_{k,i} = \gamma_{\mu,i} * (1.3/2)$)

In this formula the CoV from the guidelines is used again, as this was originally used to define the characteristic value in the document of Bakker and Jaspers Focks (2011).

It is important to note that due to non-linearities the following equation does not hold for every parameter:

$$\frac{\mu_i}{X_i^*} = \frac{1}{1 - \alpha\ \beta V_i}$$

In the FORM calculation it is assumed that the α factor can be obtained by linearizing in the design point. However, the process of plastic soil development is non-linear. Therefore the α factors are under- or overestimated, which leads to two different safety factors. The difference is enlarged due to the correlations between parameters. There are two different safety factors, one is based on the left hand side of the equation and the other on the right hand side. The relevant safety factors are summarized in Table 6-10. Note that the sin(φ) is translated to the more commonly used φ. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix M. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 4.400						
β_{CUR211} = 3.828						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
D_a	1.02	0.96	1.07	1.02	1.01	0.96
$f_{y,steel}$	1.23	1.09	1.23	1.09	1.19	1.06
$EA_{anchor1}$	1.17	1.03	1.15	1.02	1.14	1.01
$G_{moderately\ packed\ sand}$	0.90	0.58	0.96	0.62	0.91	0.59
$G_{silty\ moderately\ packed\ sand}$	0.80	0.52	2.14	1.39	0.82	0.54
$G_{pleistocene\ sand}$	0.62	0.40	1.12	0.73	0.65	0.42
$\varphi_{silty\ moderately\ packed\ sand}$	1.16	0.97	1.21	1.01	1.14	0.95
$\varphi_{pleistocene\ sand}$	1.26	1.05	1.87	1.57	1.22	1.02
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.95	0.87	1.00	0.92	0.96	0.88
$\gamma_{sat,pleistocene\ sand}$	1.00	0.92	1.05	0.96	1.00	0.92
$\gamma_{unsat,,moderately\ packed\ sand}$	0.99	0.91	0.98	0.90	1.00	0.91
Retaining height (Δ)	-0.04	-0.04	-	-	-0.04	-0.04
Water level (Δ)	-0.02	-0.02	-	-	-0.02	-0.02
Surcharge load	1.02	1.02	-	-	1.02	1.02

Table 6-10 Partial safety factors anchor 1 failure

6.6.1.3 Conclusion

From the calculations for anchor 1 failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads to a higher reliability index than the required 3.828. However, the partial safety factors to obtain this reliability index differ from the factors from the code (CUR 166, chapter 3). Both CUR 166 and CUR 211 are listed and compared with the calculated partial safety factors in Table 6-11. It should be noted that the partial factors from the calculations are levelled up.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (anchor 1)
D_a	1.00	1.10[10]	μ
$f_{y,steel}$	1.00	1.00[10]	1.10
EA_a	1.00	1.20[10]	1.05
E_{50}	1.00	μ	0.40
E_{oed}	1.00	μ	0.40
E_{ur}	1.00	μ	0.40
φ	1.20	1.00	1.05
C	1.10	1.00	μ
γ	1.00	μ	μ
Water level difference (Δ)	0.30	0.80	0.00
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.00

Table 6-11 Partial safety factors for anchor 1 in comparison with design codes

With respect to anchor 1 failure therefore the partial safety factor on E_{50}, E_{oed} and E_{ur} should be 0.4, which implies an increase of the stiffness parameter value. Furthermore, φ must decrease by dividing it by 1.05. No absolute geometrical factors on the geometrical parameters are needed and a factor of 1.05 on the surcharge load is sufficient.

[10] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force:
($F_{max} = A\sigma_y = 1/4\pi D_a^2 f_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on $f_{y,steel}$.

6.6.2 Anchor 2 failure (ULS)

The LSF for anchor 2 (lower anchore) failure in tension is: $Z = \frac{1}{4}\pi D_a^{\ 2} f_y - F_{anch2,stage4}$

First there is made a calculation with all stochastic parameters included. The output is presented in Appendix L.

6.6.2.1 Final calculation

When the irrelevant parameters are eliminated the following parameters are included in the final calculation:

- $f_{y,steel}$
- $EA_{anchor2}$
- $E_{oed,silty\ moderately\ packed\ sand}$
- $G_{moderately\ packed\ sand}$
- $G_{silty\ moderately\ packed\ sand}$
- $G_{pleistocene\ sand}$
- $\sin(\varphi)_{moderately\ packed\ sand}$
- $\sin(\varphi)_{silty\ moderately\ packed\ sand}$
- $\sin(\varphi)_{pleistocene\ sand}$
- $\gamma_{sat,silty\ moderately\ packed\ sand}$
- $\gamma_{sat,pleistocene\ sand}$
- $\gamma_{unsat,moderately\ packed\ sand}$

In this calculation many parameters did not reach the 5% influence in the first calculation (this holds for $G_{moderately\ packed\ sand}$, $\sin(\varphi)_{silty\ moderately\ packed\ sand}$, $\sin(\varphi)_{pleistocene\ sand}$, $\gamma_{sat,silty\ moderately\ packed\ sand}$, $\gamma_{sat,pleistocene\ sand}$ and $\gamma_{unsat,moderately\ packed\ sand}$). These parameters are still included in order to assure not to underestimate the reliability index. When many parameters have influence of about 3-4% per parameter in total their influence is larger and when all of them are excluded the reliability index can be underestimated. The output is given in Table 6-12 and Figure 6-8.

Number of calculations (FORM): 659				
β: 2.991				
P_f: 1.390*10^{-3}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$f_{y,steel}$	0.07	0.03	398900	[kN/m^2]
$EA_{anchor2}$	0.07	0.00	523400	[kN/m]
$E_{oed,silty\ moderately\ packed\ sand}$	0.3	0.15	31080	[kPa]
$G_{moderately\ packed\ sand}$	0.3	0.05	71570	[kPa]
$G_{silty\ moderately\ packed\ sand}$	0.3	-0.30	20690	[kPa]
$G_{pleistocene\ sand}$	0.3	-0.91	118100	[kPa]
$\sin(\varphi)_{moderately\ packed\ sand}$	0.18	0.07	0.60	[-]
$\sin(\varphi)_{silty\ moderately\ packed\ sand}$	0.18	-0.04	0.55	[-]
$\sin(\varphi)_{pleistocene\ sand}$	0.18	0.14	0.33	[-]
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.05	-0.16	20.70	[kN/m^3]
$\gamma_{sat,pleistocene\ sand}$	0.05	0.00	19.75	[kN/m^3]
$\gamma_{unsat,moderately\ packed\ sand}$	0.05	0.03	19.51	[kN/m^3]
calc.	Z-value			
1	461			
659	33.27			

Table 6-12 Output final calculation (LS Anchor 2)

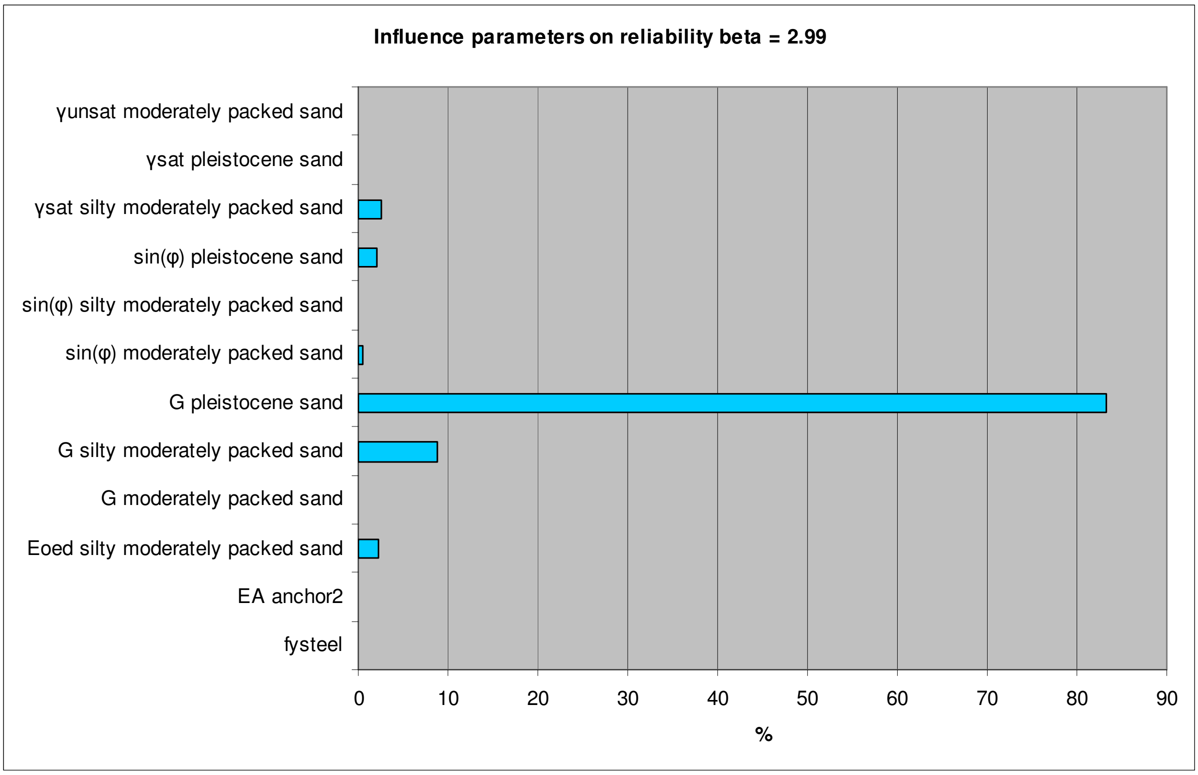


Figure 6-8 Influence in % from parameters on reliability final calculation (LS Anchor 2)

Figure 6-9 shows the plastic points in case the design values are given to the parameters. Anchor 2 failure is mostly induced by failure of soil in the lower sand layer (pleistocene) as there are many red spots in that area. Basically the fixing of the wall is reduced and the structure moves forward which causes the lower anchor to react on this motion. This leads to an increased anchor force. The importance of $\varphi_{\text{pleistocene sand}}$ is not following from Figure 6-8, but it can be seen from the value in the design point, which is 0.33 for sin(φ), whereas the mean value is 0.67. The influence parameters however are influenced by the correlations between the soil parameters in the same soil layer. This is why $G_{\text{pleistocene sand}}$ gains large importance.

The importance of $G_{\text{pleistocene sand}}$ is increased by the fact that plastic behavior is important in the lower sand layer. Shear hardening occurs at many spots which implies that the plastic behaviour is important as well, which is also shown by the relative importance of E_{ur} of the densely packed sand. It can be seen that E_{ur} is the most relevant elastic parameters, since there is no cap hardening, which implies that the the average stress decreases. Furthermore it is clear that the stiffer the soil the less the mobilization of the shear resistance (especially important at the passive side) and the higher the anchor force to guarantee horizontal equilibrium. This explains the negative value for $\alpha_{G\text{ pleistocene sand}}$, the stiffness is higher than the mean value in the design point.

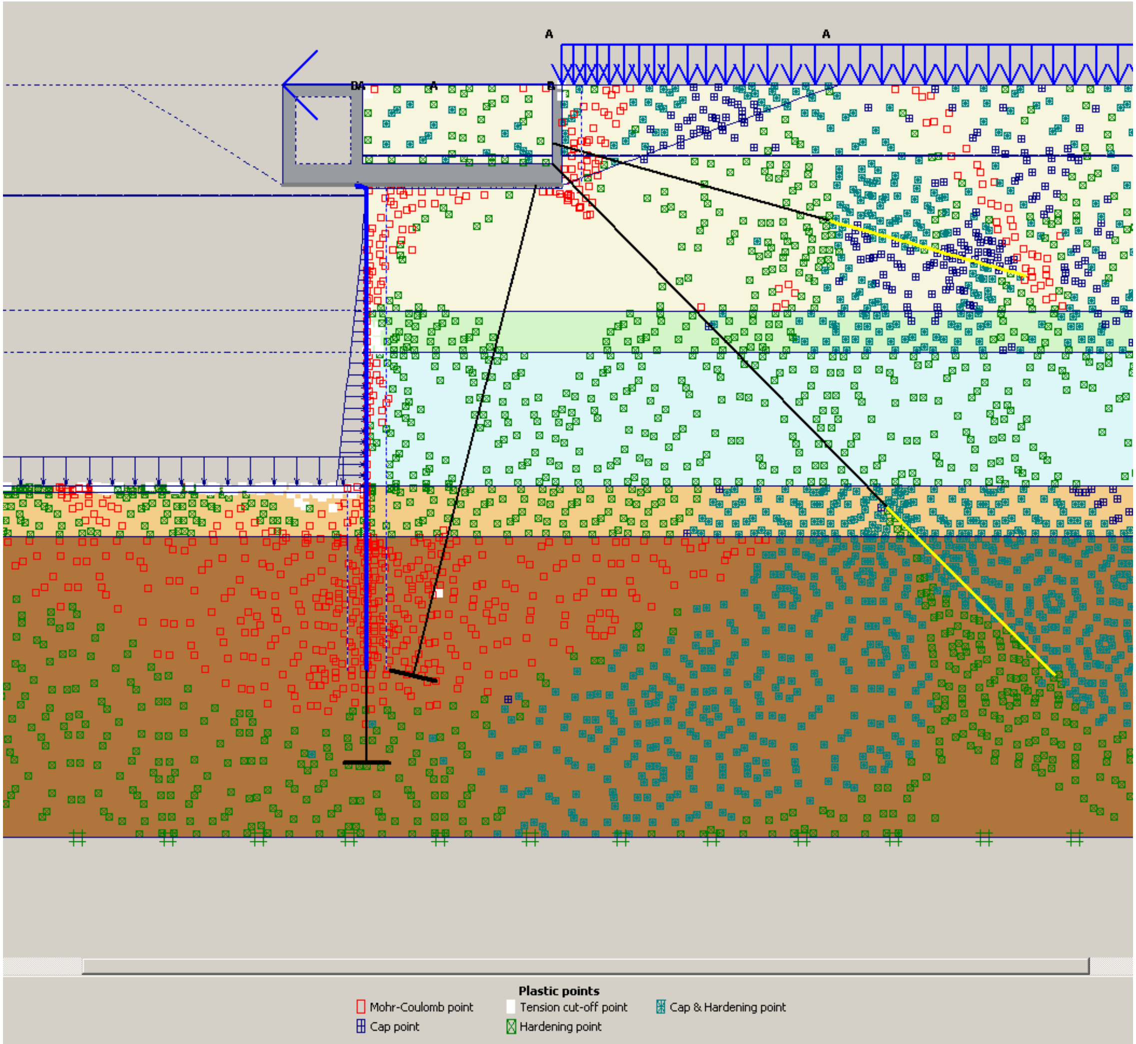


Figure 6-9 Plastic points in design point anchor 2 failure

6.6.2.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 6.6.1.2. The different variants of safety factors are presented in Table 6-13. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix M. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 2.991						
β_{CUR211} = 3.828						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$f_{y,steel}$	1.01	0.89	1.01	0.89	1.01	0.89
$EA_{anchor2}$	1.00	0.89	1.00	0.89	1.00	0.89
$E_{50,silty\ moderately\ packed\ sand}$	1.16	0.75	0.99	0.64	1.21	0.79
$G_{moderately\ packed\ sand}$	1.04	0.68	1.07	0.70	1.06	0.69
$G_{silty\ moderately\ packed\ sand}$	0.79	0.51	2.48	1.61	0.75	0.48
$G_{pleistocene\ sand}$	0.55	0.36	1.14	0.74	0.49	0.32
$\varphi_{moderately\ packed\ sand}$	1.04	0.87	1.06	0.89	1.06	0.88
$\varphi_{silty\ moderately\ packed\ sand}$	0.98	0.82	1.08	0.90	0.97	0.81
$\varphi_{pleistocene\ sand}$	1.09	0.91	2.23	1.86	1.12	0.94
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.98	0.90	1.00	0.92	0.97	0.89
$\gamma_{sat,pleistocene\ sand}$	1.00	0.92	1.10	1.01	1.00	0.92
$\gamma_{unsat,moderately\ packed\ sand}$	1.00	0.92	1.01	0.92	1.01	0.92
Retaining height (Δ)	0.01	0.01	-	-	0.01	0.01
Water level (Δ)	0.01	0.01	-	-	0.02	0.02
Surcharge load	1.00	1.00	-	-	1.00	1.00

Table 6-13 Partial safety factors anchor 2 failure

6.6.2.3 Conclusion

From the calculations for anchor 2 failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads not to the required reliability index of 3.828. Furthermore, the partial safety factors to obtain this reliability index differ from the factors in the code. Both design codes are listed and compared with the partial safety factors in Table 6-14. It should be noted that the partial factors from the calculations are levelled up.

Parameter	**CUR 166 (class III)**	**CUR 211 (class II)**	**Calculations (anchor 2)**
D_a	1.00	1.10[11]	μ
$f_{y,steel}$	1.00	1.00[11]	μ
EA_a	1.00	1.20[11]	μ
E_{50}	1.00	μ	0.30
E_{oed}	1.00	μ	0.30
E_{ur}	1.00	μ	0.30
φ	1.20	1.00	0.90
C	1.10	1.00	μ
γ	1.00	μ	1.00

[11] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force: ($F_{max} = A\sigma_y = 1/4\pi D_a^2\sigma_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on σ_y.

Water level difference (Δ)	0.30	0.80	0.05
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.00
Retaining height (Δ)	0.35	0.40	0.05

Table 6-14 Partial safety factors for anchor 2 in comparison with design codes

With respect to anchor failure therefore the partial safety factor on E_{50}, E_{oed} and E_{ur} should be 0.3, which implies an increase of the stiffness parameter value. Furthermore, φ must increase as well by dividing it by 0.9. The geometrical factors on the geometrical parameters do not need to be as high as CUR 211 prescribes (in case of wall failure). For surcharge load the governing value can be used.

6.6.3 Wall failure (ULS)

The LSF for yielding of the wall profile is: $Z = f_y - (\frac{M_{\max,wall,stage4}}{W_{1420/18AU20}} + \frac{N_{\max,wall,stage4}}{A_{1420/18AU20}})$

First there is made a calculation with all stochastic parameters included. The output is presented in Appendix L.

6.6.3.1 Final calculation

When the irrelevant parameters are eliminated the following parameters are included in the final calculation:

- $G_{\text{silty moderately packed sand}}$
- $G_{\text{pleistocene sand}}$
- $\sin(\varphi)_{\text{silty moderately packed sand}}$
- $\sin(\varphi)_{\text{pleistocene sand}}$
- $\gamma_{\text{sat,silty moderately packed sand}}$

$G_{\text{silty moderately packed sand}}$ has in the first calculation an influence smaller than 5%, but the design point is further away (11%) from the mean value and therefore it still is included in this calculation. The output of the final calculation is given in Table 6-15 and Figure 6-10.

Number of calculations (FORM): 78				
β: 2.646				
P_f: 4.068*10^{-3}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$G_{\text{silty moderately packed sand}}$	0.3	-0.12	43310	[kPa]
$G_{\text{pleistocene sand}}$	0.3	0.48	106400	[kPa]
$\sin(\varphi)_{\text{silty moderately packed sand}}$	0.18	0.67	0.48	[-]
$\sin(\varphi)_{\text{pleistocene sand}}$	0.18	0.46	0.38	[-]
$\gamma_{\text{sat,silty moderately packed sand}}$	0.05	-0.30	20.15	[kN/m^3]

calc.	Z-value
1	170400
78	8110

Table 6-15 Output final calculation (LS wall)

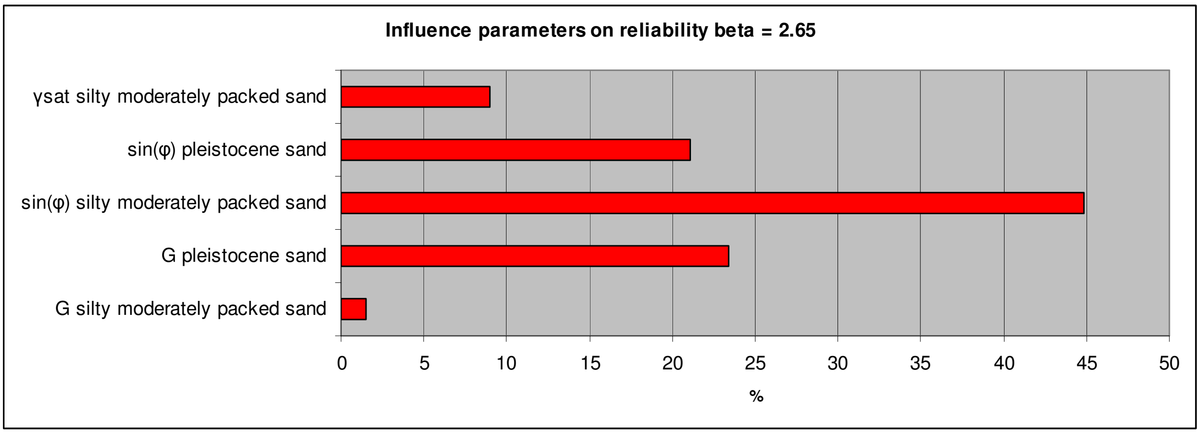


Figure 6-10 Influence in % from parameters on reliability final calculation (LS Wall)

Figure 6-11 shows the plastic points when the design values are given to the parameters. Wall failure is mostly induced by failure of soil in the lower sand layer (pleistocene) as there are many red spots in that area. Basically the fixing of the wall (and therewith the fixing moment) is reduced and field moment is increased. Along the wall red elements can be found which shows that soil failure increases the load on the wall which leads to an increase in the bending moment in the wall. The stiffness parameters mostly gain importance due to their correlation with the internal angle of friction.

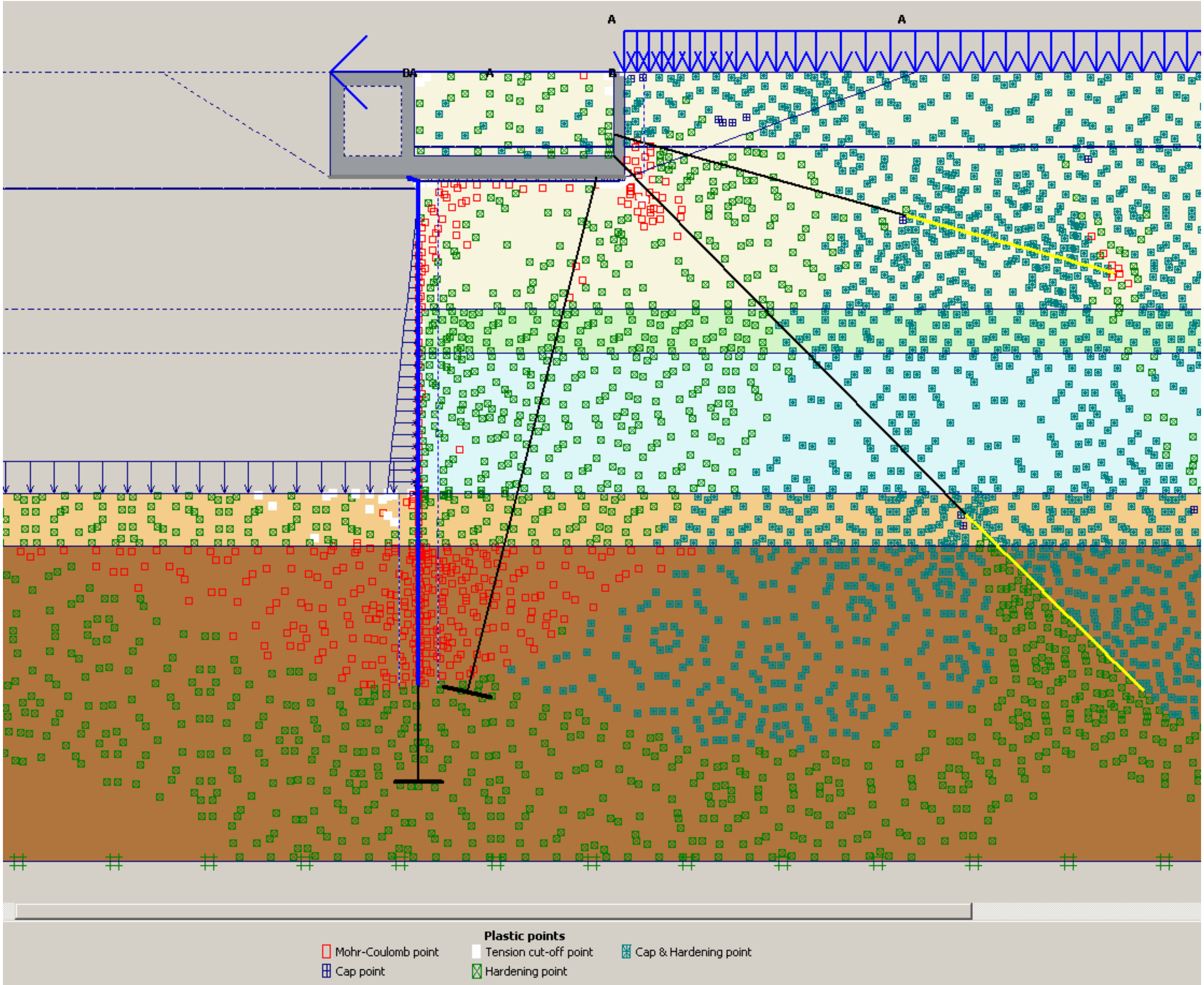


Figure 6-11 Plastic points in design point wall failure

6.6.3.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 6.6.1.2. The different variants of safety factors are presented in Table 6-16. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix M. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 2.646						
β_{CUR211} = 3.872						
X_i	$\gamma_{\mu,i,calculation}$	$\gamma_{k,i,calculation}$	μ/X^*	X_k/X^*	$\gamma_{\mu,i,CUR211}$	$\gamma_{k,i,CUR211}$
$G_{silty\ moderately\ packed\ sand}$	0.91	0.59	1.18	0.77	0.87	0.57
$G_{pleistocene\ sand}$	1.59	1.03	1.27	0.82	2.28	1.48
$\varphi_{silty\ moderately\ packed\ sand}$	1.52	1.27	1.25	1.04	2.08	1.74
$\varphi_{pleistocene\ sand}$	1.31	1.09	1.91	1.59	1.55	1.30
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.96	0.88	1.03	0.94	0.95	0.87
Retaining height (Δ)	0.06	0.06	-	-	0.09	0.09
Water level (Δ)	0.07	0.07	-	-	0.10	0.10
Surcharge load	1.00	1.00	-	-	1.00	1.00

Table 6-16 Partial safety factors for LS wall failure

6.6.3.3 Conclusion

From the calculations for wall failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads not to the required reliability index of 3.872. Furthermore, the partial safety factors to obtain this required reliability index differ from the factors in the code (CUR 166, chapter 3). Both CUR 211 and CUR 166 are listed and compared with the obtained partial safety factors in Table 6-17. It should be noted that the partial factors from the calculations are levelled up.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (wall)
$EI_{combi\text{-}wall}$	1.00	1.00	μ
$EA_{combi\text{-}wall}$	1.00	1.00	μ
$w_{combi\text{-}wall}$	1.00	1.00	μ
$W_{combi\text{-}wall}$	1.00	1.00	μ
$f_{y,steel}$	1.00	1.00	μ
E_{50}	1.00	μ	1.50
E_{oed}	1.00	μ	1.50
E_{ur}	1.00	μ	1.50
φ	1.20	1.00	1.75
C	1.10	1.00	μ
γ	1.00	μ	0.85
Water level difference (Δ)	0.30	0.80	0.10
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.00
Retaining height (Δ)	0.35	0.40	0.10

Table 6-17 Partial safety factors for wall failure comparison with design codes

With respect to wall failure therefore the partial safety factor on φ must increase to obtain the correct reliability. Furthermore, the stiffness parameters should have lower values than the mean values. The characteristic value should be subdivided by 1.50. The absolute additional differences on the geometrical parameters do not need to be as high as CUR 211 prescribes (in case of wall failure).

6.6.4 Soil mechanical failure (ULS)

The LSF for soil mechanical failure in its most correct form is: $Z = MSF - 1.0$

However, this Limit State is difficult to reach in Prob2B as it is very reasonable that soil collapse occurs in an earlier building stage. In that case the calculation is not finished, which causes the LS evaluation not to be performed. Therefore it is tried to define the LSF in a way that Prob2B gives results. Different options have been tried and the closest value of MSF that can be reached in the LS calculation appeared to be 1.1. This is a pragmatic approach and therefore the LSF is defined as: $Z = MSF - 1.1$

It is clear that the obtained β is in fact too low, there is some safety left in the difference between MSF = 1.1 and MSF = 1.0. It is assumed however that the distance between these two design points is small enough to have no influence on the influence factors α. Therefore still safety factors can be determined based on these calculations.

First there is made a calculation with all stochastic parameters included. The output is presented in Appendix L.

6.6.4.1 Final calculation

When the irrelevant parameters are eliminated the following parameters are included in the final calculation:

- $E_{oed,pleistocene\ sand}$
- $\sin(\varphi)_{pleistocene\ sand}$
- $\gamma_{sat,pleistocene\ sand}$

The output is given in Table 6-18 and Figure 6-12.

Number of calculations (FORM): 53				
β: 2.779				
P_f: 2.730*10^{-3}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$E_{oed,pleistocene\ sand}$	0.3	0.48	70360	[kPa]
$\sin(\varphi)_{pleistocene\ sand}$	0.18	0.80	0.34	[-]
$\gamma_{sat,pleistocene\ sand}$	0.05	-0.36	19.99	[kN/m^3]

calc.	Z-value	
1	1.20	
53	0.01	**MSF = 1.11**

Table 6-18 Output final calculation (LS soil)

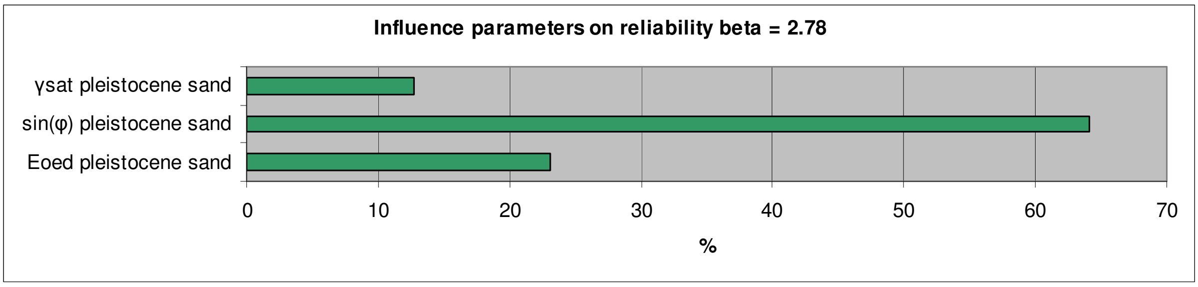


Figure 6-12 Influence in % from parameters on reliability final calculation (LS soil)

Figure 6-13 shows the plastic points in the design point of soil mechanical failure. There are only red spots (Mohr-Coulomb) because a φ-C reduction was applied after the final building stage. φ-C reduction uses in fact the Mohr-Coulomb model. There is a slip plane visible and the failure of the lower sand layer induces the failure of the structure due to soil mechanical processes. γ_{sat} and the stiffness parameters of the lower sand layer gain importance due to their correlation with the internal angle of friction.

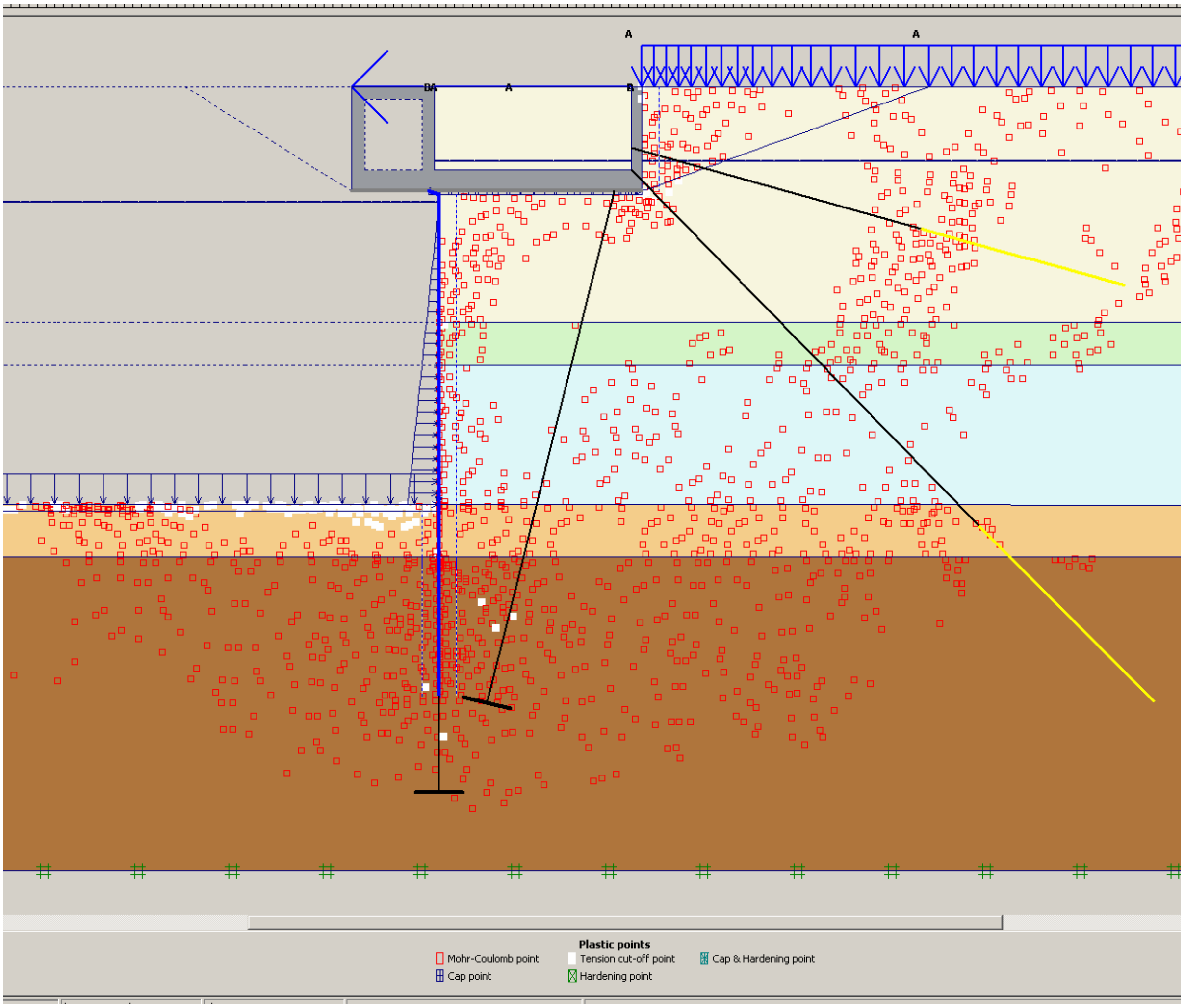


Figure 6-13 Plastic points in design point soil mechanical failure

6.6.4.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 6.6.1.2. The different variants of safety factors are presented in Table 6-19. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in

PLAXIS, described in Appendix M. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 2.779						
β_{CUR211} = 4.396						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$E_{oed,pleistocene\ sand}$	1.67	1.09	1.31	0.85	2.74	1.78
$\varphi_{pleistocene\ sand}$	1.80	1.51	2.16	1.80	3.38	2.82
$\gamma_{sat,pleistocene\ sand}$	0.95	0.87	1.09	1.00	0.93	0.85
Retaining height (Δ)	0.03	0.03	-	-	0.04	0.04
Water level (Δ)	-0.01	-0.01	-	-	-0.01	-0.01
Surcharge load	1.01	1.01	-	-	1.02	1.02

Table 6-19 Partial safety factors for LS soil mechanical failure

6.6.4.3 Conclusion

From the calculations for soil mechanical failure it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads not to the required reliability index of 4.396. To be clearer, this value is by far not reached. Furthermore, the partial safety factors to obtain this reliability index differ from the factors in the code. Both design codes are listed and compared with the partial safety factors in Table 6-20. It should be noted that the partial factors from the calculations are levelled up.

Parameter	**CUR 166 (class III)**	**CUR 211 (class II)**	**Calculations (soil)**
E_{50}	1.00	μ	1.80
E_{oed}	1.00	μ	1.80
E_{ur}	1.00	μ	1.80
φ	1.20	1.00	2.85
C	1.10	1.00	μ
γ	1.00	μ	0.85
Water level difference (Δ)	0.30	0.80	0.00
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.05

Table 6-20 Partial safety factors for soil mechanical comparison with design codes

With respect to soil mechanical failure therefore the partial safety factor on φ must increase to obtain the correct reliability (division by 2.85). Furthermore, the stiffness parameters should have lower values than the mean values. The characteristic value should be subdivided by 1.80. The surcharge load must have a factor of 1.05 and 5 cm additional retaining height should be taken into account.

6.6.5 Excessive Deformations (SLS)

The only criterion that is set with respect to deformations is the limitation to the displacement of the top of the structure. This displacement should be smaller than 50 mm, because otherwise the crane on top of the quay wall can not perform its function.

The LSF for excessive deformations therefore is: $Z = 0.05 - \delta_{topofstructure}$

First there is made a calculation with all stochastic parameters included. The output is presented in Appendix L.

6.6.5.1 Final calculation

When the irrelevant parameters are eliminated the following parameters are left over in the final calculation:

- $G_{\text{silty moderately packed sand}}$
- $G_{\text{pleistocene sand}}$
- $\sin(\varphi)_{\text{silty moderately packed sand}}$
- $\sin(\varphi)_{\text{pleistocene sand}}$
- $\gamma_{\text{sat,silty moderately packed sand}}$

The output is given in Table 6-21 and Figure 6-14.

Number of calculations (FORM): 97				
β: 2.439				
P_f: 7.373*10^{-3}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$G_{\text{silty moderately packed sand}}$	0.3	-0.19	46310	[kPa]
$G_{\text{pleistocene sand}}$	0.3	0.42	108400	[kPa]
$\sin(\varphi)_{\text{silty moderately packed sand}}$	0.18	0.65	0,49910	[-]
$\sin(\varphi)_{\text{pleistocene sand}}$	0.18	0.48	0.41	[-]
$\gamma_{\text{sat,silty moderately packed sand}}$	0.05	-0.36	20.48	[kN/m^3]

calc.	Z-value
1	0.0296
97	-0.0002

Table 6-21 Output final calculation (LS deformations)

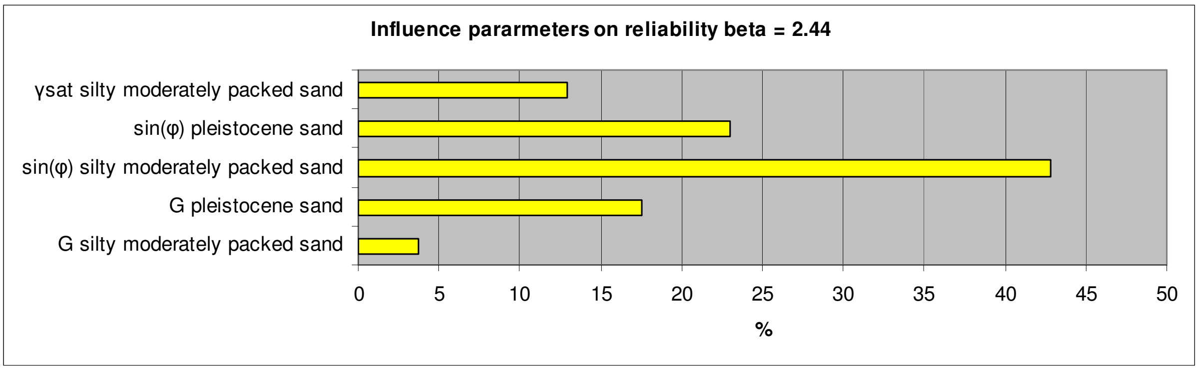


Figure 6-14 Influence in % from parameters on reliability final calculation (LS deformations)

Figure 6-15 shows the plastic points in the design point for the mechanism excessive deformations. The plot shows that deformations occur due to reduction of the fixing moment at the toe of the sheet-pile (shown by the Mohr-Coulomb envelope points (red)). Furthermore shear hardening occurs at many spots which implies that the plastic behaviour is important as well, which is also shown by the relative importance of E_{ur} of the pleistocene sand (and therewith the other stiffness parameters of this layer as well) and some relevance for the stiffness of the middle sand layer. It can be seen that E_{ur} is the most relevant elastic parameters, since there is no cap hardening, which implies that the the average stress decreases.

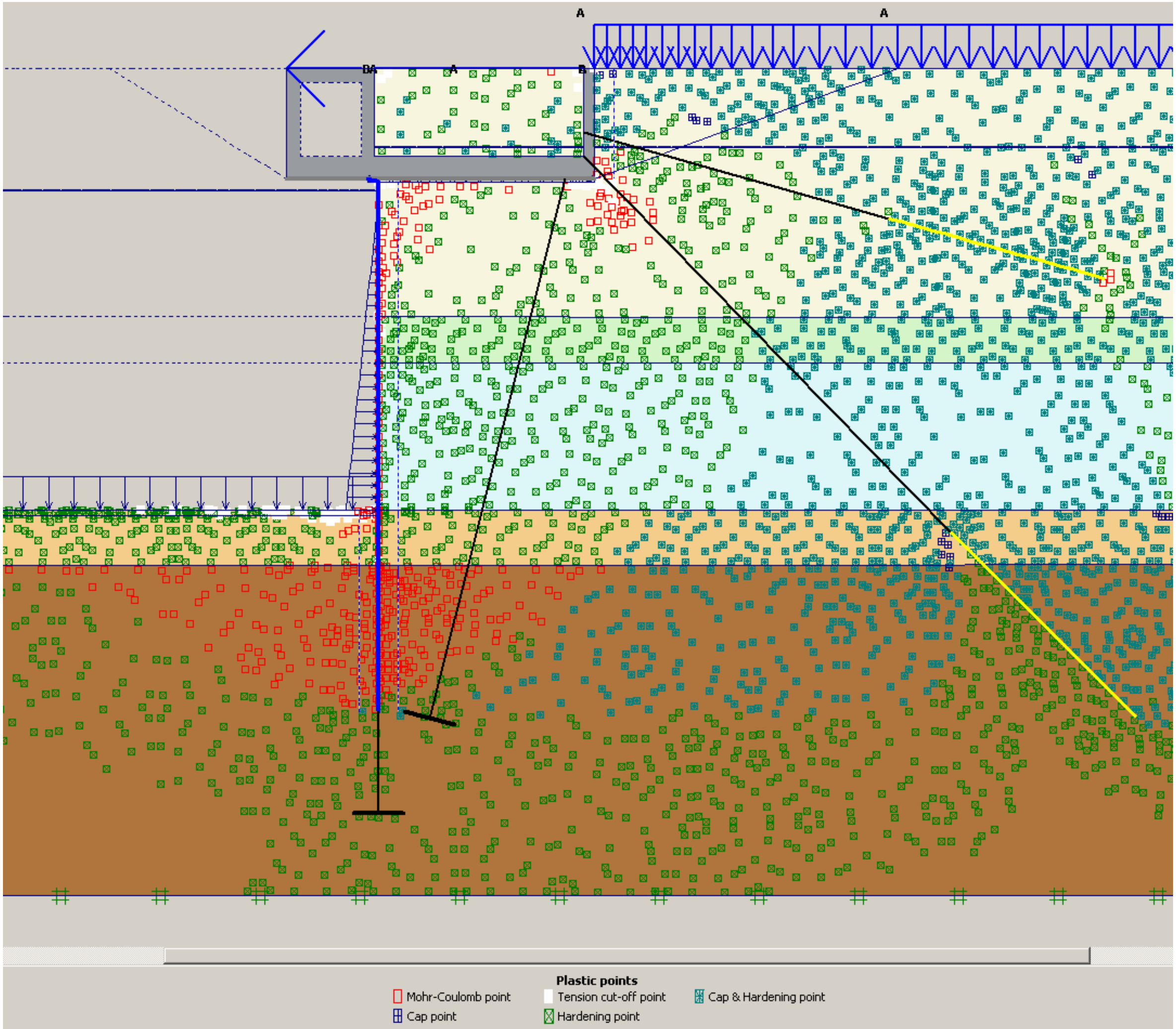


Figure 6-15 Plastic points in design point deformations

6.6.5.2 Partial safety factors

From the final results partial safety factors can be derived according to the formulas of section 6.6.1.2. The different variants of safety factors are presented in Table 6-22. Furthermore, the table includes the partial safety factors for the three additional parameters. They are based on a manual variation in PLAXIS, described in Appendix M. The level parameters do not get a partial safety factor but an absolute difference in meters (Δ). The change in reliability index by the addition of the extra parameters is very small and therefore not taken into account in the results.

$\beta_{calculation}$ = 2.439						
β_{CUR211} = 1.800						
X_i	**$\gamma_{\mu,i,calculation}$**	**$\gamma_{k,i,calculation}$**	**μ/X^***	**X_k/X^***	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$G_{silty\ moderately\ packed\ sand}$	0.88	0.57	1.11	0.72	0.91	0.59
$G_{pleistocene\ sand}$	1.44	0.94	1.24	0.81	1.29	0.84
$\varphi_{silty\ moderately\ packed\ sand}$	1.47	1.23	1.20	1.00	1.31	1.09
$\varphi_{pleistocene\ sand}$	1.30	1.09	1.80	1.51	1.21	1.01
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.96	0.88	1.01	0.93	0.97	0.89
Retaining height (Δ)	0.05	0.05	-	-	0.04	0.04
Water level (Δ)	0.06	0.06	-	-	0.05	0.05
Surcharge load	1.01	1.01	-	-	1.01	1.01

Table 6-22 Partial safety factors for LS excessive deformations

6.6.5.3 Conclusion

From the calculations for excessive deformations it can be concluded that the design procedure as it is given in CUR 166 chapter 4 leads to a higher reliability index than the required 1.800. The partial safety factors to obtain this target reliability index differ from the factors in the code. Both codes are listed and compared with the calculated partial safety factors in Table 6-23. It should be noted that the partial factors from the calculations are levelled up.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (deformation)
E_{50}	1.00	μ	0.80
E_{oed}	1.00	μ	0.80
E_{ur}	1.00	μ	0.80
φ	1.20	1.00	1.05
C	1.10	1.00	μ
γ	1.00	μ	0.85
Water level difference (Δ)	0.30	0.80	0.05
Water level left (outside) (Δ)	0.25	0.70	-
Water level right (ground) (Δ)	0.05	0.10	-
Surcharge load	1.00	1.20	1.05
Retaining height (Δ)	0.35	0.40	0.05

Table 6-23 Partial safety factors for excessive deformations comparison with design codes

With respect to excessive deformations therefore the partial safety factor on φ can be decreased to obtain the correct reliability. Furthermore, the stiffness parameters should have higher values than the mean values. The characteristic value should be subdivided by 0.80. An additional 5 cm retaining height and water level difference should be included.

6.6.6 Conclusion

The partial safety factors are summarized and generalized in Table 6-24. As the factors are quite different for each different mechanism it is more optimal to define partial safety factors for each mechanism (otherwise the anchor would for instance be too heavy). For the SLS mechanism excessive deformations no additional factors need to be defined, because the factors from Table 6-23 fall in between the boundaries of the other mechanisms. For the sake of completeness they are nevertheless presented in Table 6-24, but it can be seen that when the deformations are not large when checking 'anchor failure', the situation is already fine. When this is not fine, the deformations can be additionally checked by using the right column of the table.

An important notion is that application of these generalized parameters leads to a suboptimal design. This is because the partial safety factors are applied on each layer, whereas the probabilistic calculations often show that the uncertainty in one layer is dominant. For instance, for soil mechanical failure only factors need to be placed on the lower sand layer. When placing factors one each layer, a heavier and longer wall is chosen and a too high reliability index is obtained.

It is however not practical to define partial safety factors per layer. Moreover, real situations are not equal to one of the benchmark quay walls. The most convenient way would be to do probabilistic calculations (or sensitivity analyses) for each quay wall design to define the most important layers and parameters. Based on these analyses the partial safety factors can be placed on the correct parameters and a more optimal design can be made.

Parameter	**CUR166 (class III)**	**CUR211 (class II)**	**Calculations (generalized)**			
			Anchor	**Wall**	**Soil**	**Deformations**
$EI_{combi\text{-}wall}$	1.00	1.00	μ	μ	μ	μ
$EA_{combi\text{-}wall}$	1.00	1.00	μ	μ	μ	μ
$w_{combi\text{-}wall}$	1.00	1.00	μ	μ	μ	μ
$W_{combi\text{-}wall}$	1.00	1.20	μ	μ	μ	μ
D_a	1.00	1.10[12]	μ	μ	μ	μ
$f_{y,steel}$	1.00	1.00[12]	1.10	μ	μ	μ
EA_a	1.00	1.20[12]	1.05	μ	μ	μ
E_{50}	1.00	μ	0.30	1.50	1.80	0.80
E_{oed}	1.00	μ	0.30	1.50	1.80	0.80
E_{ur}	1.00	μ	0.30	1.50	1.80	0.80
φ	1.20	1.00	1.05	1.75	2.85	1.05
C	1.10	1.00	μ	μ	μ	μ
γ	1.00	μ	1.00	0.85	0.85	0.85
Water level difference (Δ)	0.30	0.80	0.05	0.10	0.00	0.05
Water level left (outside) (Δ)	0.25	0.70	-	-	-	-
Water level right (ground) (Δ)	0.05	0.10	-	-	-	-
Surcharge load	1.00	1.20	1.05	1.00	1.05	1.05
Retaining height (Δ)	0.35	0.40	0.05	0.10	0.05	0.05

Table 6-24 Generalized partial safety factor anchored sheet-pile over the characteristic value

[12] CUR 211 prescribes an additional factor of 1.20 on the maximum anchor force: ($F_{max} = A\sigma_y = 1/4\pi D_a^2\sigma_y$). This gives a factor of around 1.10 for D_a and 1.20 for EA_a. It is also possible to put the safety factor of 1.20 on σ_y.

Based on Table 6-24 it is advised to use different design values for the soil stiffness parameters, the internal angle of friction and the specific soil weight.

$E_{d,anchor-force} = \dfrac{E_k}{0.30}$, $E_{d,bending-moment} = \dfrac{E_k}{1.50}$ and $E_{d,soil-collapse} = \dfrac{E_k}{1.80}$

$\varphi_{d,anchor-force} = \dfrac{\varphi_k}{1.05}$, $\varphi_{d,bending-moment} = \dfrac{\varphi_k}{1.75}$ and $\varphi_{d,soil-collapse} = \dfrac{\varphi_k}{2.85}$

$\gamma_{d,anchor-force} = \dfrac{\gamma_k}{1.00}$, $\gamma_{d,bending-moment} = \dfrac{\gamma_k}{0.85}$ and $\gamma_{d,soil-collapset} = \dfrac{\gamma_k}{0.85}$

The reliability indices are summarized in Table 6-25 and compared with the prescribed reliability indices from CUR 211. CUR 211 contains the failure tree which should be used for quay wall design. It is clear that one and the same approach for two different types of anchors leads to completely different reliability indices. Anchor 1 is ‘too strong’ whereas anchor 2 is ‘too weak’. The design approach of CUR 166 chapter 4 (i.e. the φ-C reduction up to a value MSF = 1.2) leads to higher or lower reliability indices depending on the anchor. The procedure also leads to an overestimation of the reliability indices for wall and soil mechanical failure. For the deformations the procedure is fine. It can therefore be concluded that it is wise to adapt the design procedure of CUR 166 for quay walls with relieving floor. The design of the wall should be done at a higher MSF value.

With respect to soil mechanical failure it is worthwhile to reconsider the target reliability of 4.396. This required reliability index is relatively high as soil mechanical failure is a mechanism that has a higher probability of occurrence than for instance anchor failure. This also followed from the first benchmark calculations (chapter 5), where even in the elongated version of the anchored sheet-pile β was much too low. Basically, these types of structures are most sensitive to soil mechanical failure. This problem is discussed in more detail in section 7.5.

	β calculated	**β CUR 211**
Anchor 1 failure	4.40	3.828
Anchor 2 failure	2.99	3.828
Wall failure	2.65	3.872
Soil mechanical failure	2.78	4.396
Deformations	2.44	1.800

Table 6-25 Reliability indices failure mechanisms

7 Additional discussion topics

7.1 Introduction

In this chapter some additional topics that came up during the research are discussed. It forms the basis for further discussion and recommendations. Section 7.2 discusses the spatial correlation of soil parameters and its probable influence on the calculation results. Section 7.3 briefly recapitulates the discussion on geometrical parameters and loads. In section 7.4 it is discussed whether the assumptions in the Limit State Function for soil mechanical failure are reasonable and what possible consequences these assumptions have on the results. Finally, in section 7.5 the model uncertainty consisting of different components is discussed.

7.2 Spatial correlation

The coefficients of variation (CoV) of the soil parameters in the foregoing probabilistic calculations are based on the database of Gemeentewerken Rotterdam (2003). The CoV's are derived based on point measurements over different soil layers and at different locations. These different point measurements are combined to find the standard deviation (see for example Figure 4-5 in section 4.7.1.2). However, in reality the different points in a soil layer are correlated. The point variability is reduced by the variability over the layer. For slip planes and other geotechnical analysis it is more convenient to look at the soil layer averaged CoV. K.J. Bakker (2012) describes a procedure to include this spatial information and to derive a reasonable estimate of the spatial averaged CoV for the quay walls in this research.

To translate point variability to spatial variability the spatial correlation should be involved in the analysis, which can be expressed in the correlation length Θ. It can be shown that a continuous system under specific circumstances can be seen as a discrete system with n uncorrelated components when this system is subdivided in elements with length (L*):

$$L^* = \sqrt{\pi} \cdot \Theta \text{ where } n = \frac{L}{L^*} = \frac{L}{\sqrt{\pi} \cdot \Theta}$$

In case of parallel system the CoV of the average $V(\bar{r})$ is given by:

$$V(\bar{r}) = \frac{V(r_i)}{\sqrt{n}} = \frac{V(r_i)}{\sqrt{\frac{L}{\sqrt{\pi} \cdot \Theta}}}$$

This is in one dimension. In practice it is assumed that spatial variability in x, y and z direction are uncorrelated:

$$V(\bar{r}) = \frac{V(r_i)}{\sqrt{n}} = \frac{V(r_i)}{\sqrt{\frac{L_x}{\sqrt{\pi} \cdot \Theta_x} \cdot \frac{L_y}{\sqrt{\pi} \cdot \Theta_y} \cdot \frac{L_z}{\sqrt{\pi} \cdot \Theta_z}}}$$

Literature gives values for Θ_x and Θ_y of 30 till 200 m. For Θ_z these values are in between 0.3 and 2.0 m. When the structure is subdivided in horizontal direction into parts larger than the correlation length the reduction in CoV due to the spatial correlation is negligible. Only the vertical spatial correlation remains relevant.

The vertical correlation length is determined from the average distance between two high cone resistance values in the cone resistance graph. It can be assumed that reduction in variability is mainly determined by the large variations that are given by the division in soil layers. When a representative Dutch 'Maasvlakte' quay wall is chosen with 7 soil layers and 18 m retaining height, the correlation length is: $\Theta_x \approx 2.5$. Therefore the number of uncorrelated components is:

$$n = \frac{18}{\sqrt{\pi} \cdot 2.5} \approx 4.1$$

Which implies that the spatial averaged CoV equals:

$$V(\bar{r}) = \frac{V(r_i)}{\sqrt{n}} = \frac{V(r_i)}{\sqrt{4.1}} \approx 0.5V(r_i)$$

It is reasonable to reduce the point CoV with a factor 2 when situations like those are analyzed (for instance benchmark 1 and 2). It is also possible to define the correlation length in one single layer based on cone resistance values. This can give even lower spatial averaged CoV, but the $0.5V(r_i)$ is taken as conservative estimate for the quay walls in this research. Furthermore it better approaches the values in NEN6740.

7.2.1 Consequences

When applying these lower CoV values in the cases of this thesis the partial safety factors for the soil parameters will decrease as the CoV is directly related to the partial safety factor:

$$\gamma_i = \frac{1}{1 - \alpha_i \beta \mathbf{V}_i}$$

Furthermore there will be a shift in division of influence factors α_i which reduces the partial safety factor even further. A test calculation for the elongated first benchmark is executed with the CoV's of Table 7-1. Note that the CoV for specific soil weight (γ) remains 0.05 as it is not recommended to take a value lower than the value prescribed in NEN 6740.

Parameter	μ	Unit	V = σ / μ
$\gamma_{unsat,moderately\ packed\ sand}$	18.5	[kN/m^3]	0.05
$\gamma_{sat,moderately\ packed\ sand}$	20.7	[kN/m^3]	0.05
$\gamma_{sat,moderate\ clay}$	17.4	[kN/m^3]	0.05
$\gamma_{sat,densely\ packed\ sand}$	21.8	[kN/m^3]	0.05
$\varphi_{moderately\ packed\ sand}$	38.9	[-]	0.10
$\varphi_{moderate\ clay}$	26.9	[-]	0.10
$\varphi_{densely\ packed\ sand}$	41.9	[-]	0.10
$C_{moderate\ clay}$	14.8	[kPa]	0.40
$\psi_{moderately\ packed\ sand}$	3.0	[-]	0.10
$\psi_{densely\ packed\ sand}$	6.0	[-]	0.10
$E_{50,moderately\ packed\ sand}$	69.2	[MPa]	0.15
$E_{50,moderate\ clay}$	7.69	[MPa]	0.15
$E_{50,densely\ packed\ sand}$	115.4	[MPa]	0.15
$E_{oed,moderately\ packed\ sand}$	69.2	[MPa]	0.15
$E_{oed,moderate\ clay}$	5.27	[MPa]	0.15
$E_{oed,densely\ packed\ sand}$	115.4	[MPa]	0.15
$G_{moderately\ packed\ sand}$	86.5	[MPa]	0.15
$G_{moderate\ clay}$	6.31	[MPa]	0.15
$G_{densely\ packed\ sand}$	144.2	[MPa]	0.15
$R_{int,moderately\ packed\ sand}$	0.9	[-]	0.2
$R_{int,moderate\ clay}$	0.67	[-]	0.2

$R_{int,densely\ packed\ sand}$	0.9	[-]	0.2
$m_{moderately\ packed\ sand}$	0.5	[-]	0.2
$m_{moderate\ clay}$	1.0	[-]	0.2
$m_{densely\ packed\ sand}$	0.5	[-]	0.2
<u>Sheet-pile (AZ36-700N)</u>			
$G_{eq,AZ36\text{-}700N}$	2691759	[kN/m^2/m]	0.07
$d_{AZ36\text{-}700N}$	0.706	[kN/m]	0.03
$w_{AZ36\text{-}700N}$	1.8140	[kN/m]	0.04
$W_{AZ36\text{-}700N}$	$3.842*10^{-3}$	[m^3/m]	0.04
$f_{y,steel}$	$401*10^3$	[kN/m^2]	0.07
<u>Anchor</u>			
D_a	0.0443	[m]	0.032
EA_a	14291	[kN/m]	0.07
$f_{y,steel}$	$401*10^3$	[kN/m^2]	0.07

Table 7-1 Coefficients of variation including spatial correlation

The output of the calculation for the LSF for anchor failure is presented in Table 7-2.

Number of calculations (FORM): 577				
β: 5.031				
P_f: $2.447*10^{-7}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
D_a	0.032	0.11	0.04	[m]
$f_{y,steel}$	0.07	0.59	317600	[kN/m^2]
EA_a	0.07	0.39	12520	[kN/m]
$C_{moderate\ clay}$	0.4	0.04	9.10	[kPa]
$E_{50,moderate\ clay}$	0.15	0.00	6542	[kPa]
$E_{50,moderately\ packed\ sand}$	0.15	0.00	69940	[kPa]
$E_{50,densely\ packed\ sand}$	0.15	0.00	100200	[kPa]
$E_{oed,moderate\ clay}$	0.15	0.00	4908	[kPa]
$E_{oed,moderately\ packed\ sand}$	0.15	0.00	69940	[kPa]
$E_{oed,densely\ packed\ sand}$	0.15	0.17	100200	[kPa]
$G_{moderate\ clay}$	0.15	0.00	6015	[kPa]
$G_{moderately\ packed\ sand}$	0.15	0.00	87430	[kPa]
$G_{densely\ packed\ sand}$	0.15	-0.16	125300	[kPa]
$m_{moderate\ clay}$	0.2	0.08	1.00	[-]
$m_{moderately\ packed\ sand}$	0.2	0.19	0.49	[-]
$m_{densely\ packed\ sand}$	0.2	0.06	0.52	[-]
$R_{int,moderate\ clay}$	0.2	0.23	0.57	[-]
$R_{int,moderately\ packed\ sand}$	0.2	0.12	0.58	[-]
$R_{int,densely\ packed\ sand}$	0.2	0.02	0.90	[-]
$\sin(\varphi)_{moderate\ clay}$	0.09	0.09	0.45	[-]
$\sin(\varphi)_{moderately\ packed\ sand}$	0.09	0.43	0.53	[-]
$\sin(\varphi)_{densely\ packed\ sand}$	0.09	0.08	0.60	[-]
$\sin(\psi)_{moderately\ packed\ sand}$	0.09	-0.12	0.05	[-]
$\sin(\psi)_{densely\ packed\ sand}$	0.09	0.18	0.07	[-]
$\gamma_{sat,moderate\ clay}$	0.05	0.20	16.86	[kN/m^3]
$\gamma_{sat,moderately\ packed\ sand}$	0.05	0.18	20.92	[kN/m^3]

$\gamma_{sat,densely\ packed\ sand}$	0.05	0.05	20.85	[kN/m^3]
$\gamma_{unsat,moderately\ packed\ sand}$	0.05	-0.02	18.72	[kN/m^3]
$d_{AZ36-700N}$	0.03	0.05	0.70	[kN/m]
$w_{AZ36-700N}$	0.04	-0.02	1.82	[kN/m]
$G_{eq,AZ36-700N}$	0.07	0.04	2693000	[kN/m^2/m]

calc.	Z-value	
1	308.20	
577	3.69	

Table 7-2 Output calculation 1 including spatial correlation (LS anchor - elongated)

The first important consequence is the higher reliability index. The 5.031 is much larger than the original 3.539 (Appendix J). The second consequence is the shift in influence factors. Figure 7-1 compares the influence (α^2*100) of the different parameters for the calculation with the original CoV's for the soil parameters and the calculation including spatial correlation (reduced CoV's). From the figure it is clear that the internal angle of friction of the upper sand layer has less influence when its CoV is reduced. Especially the anchor parameters (EA_a and $f_{y,steel}$) gain more importance in the new situation.

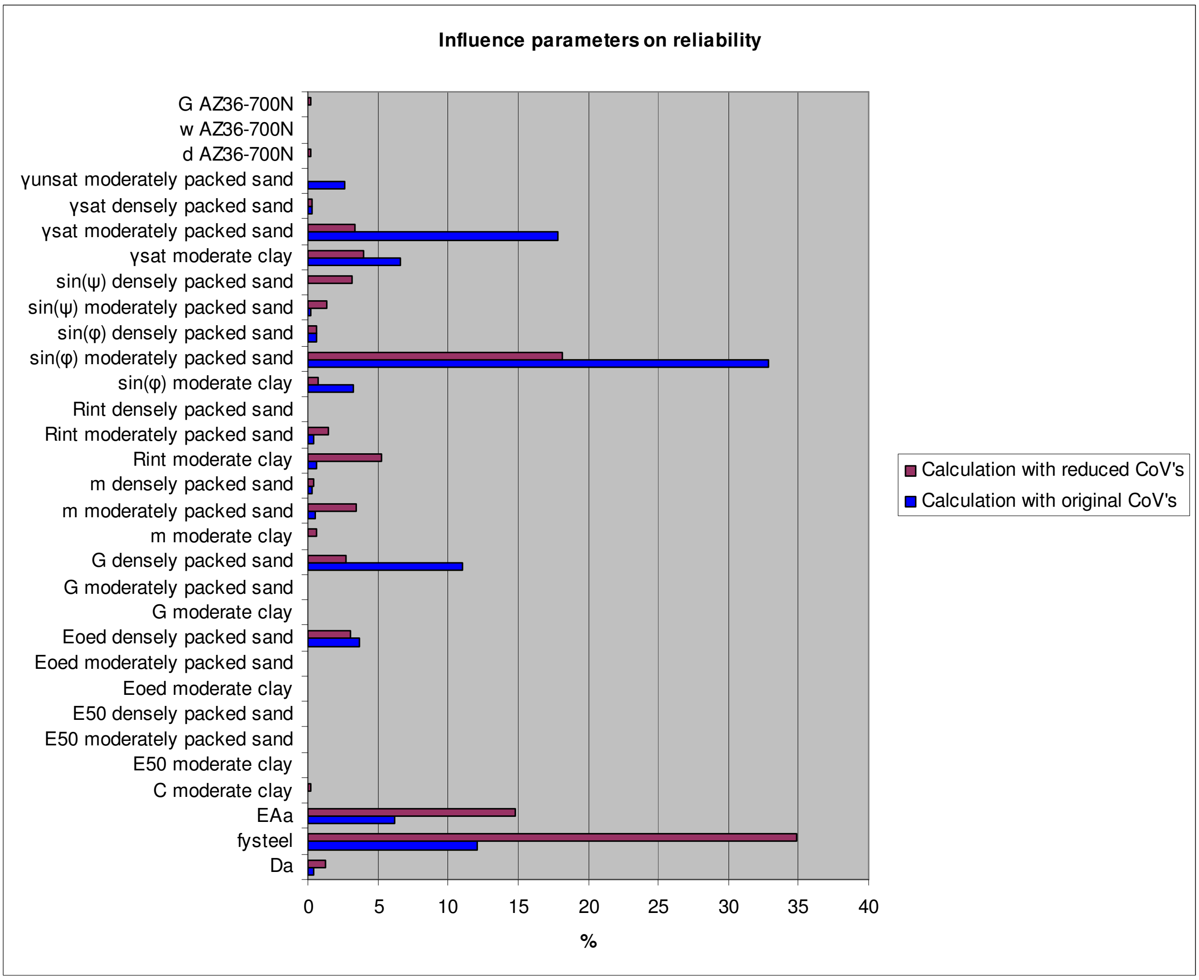


Figure 7-1 Comparison of parameter influence on reliability with different CoV's calculation 1

For the final calculation with the limited number of parameters also a calculation is made. The output is given in table. Figure shows the same trend as Figure 7-1.

Number of calculations (FORM): 97				
β: 6.294				
P_f: 1.550*10^{-10}				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$f_{y,steel}$	0.07	0.87	246800	[kN/m^2]
EA_a	0.07	-0.01	14360	[kN/m]
$G_{densely\ packed\ sand}$	0.3	-0.16	143000	[kPa]
$\sin(\varphi)_{moderately\ packed\ sand}$	0.18	0.40	0.50	[-]
$\sin(\varphi)_{densely\ packed\ sand}$	0.18	0.06	0.61	[-]
$\gamma_{sat,moderate\ clay}$	0.05	0.09	17.11	[kN/m^3]
$\gamma_{sat,moderately\ packed\ sand}$	0.05	0.20	20.76	[kN/m^3]

calc.	Z-value
1	320,30
97	4,51

Table 7-3 Output final calculation including spatial correlation (LS anchor - elongated)

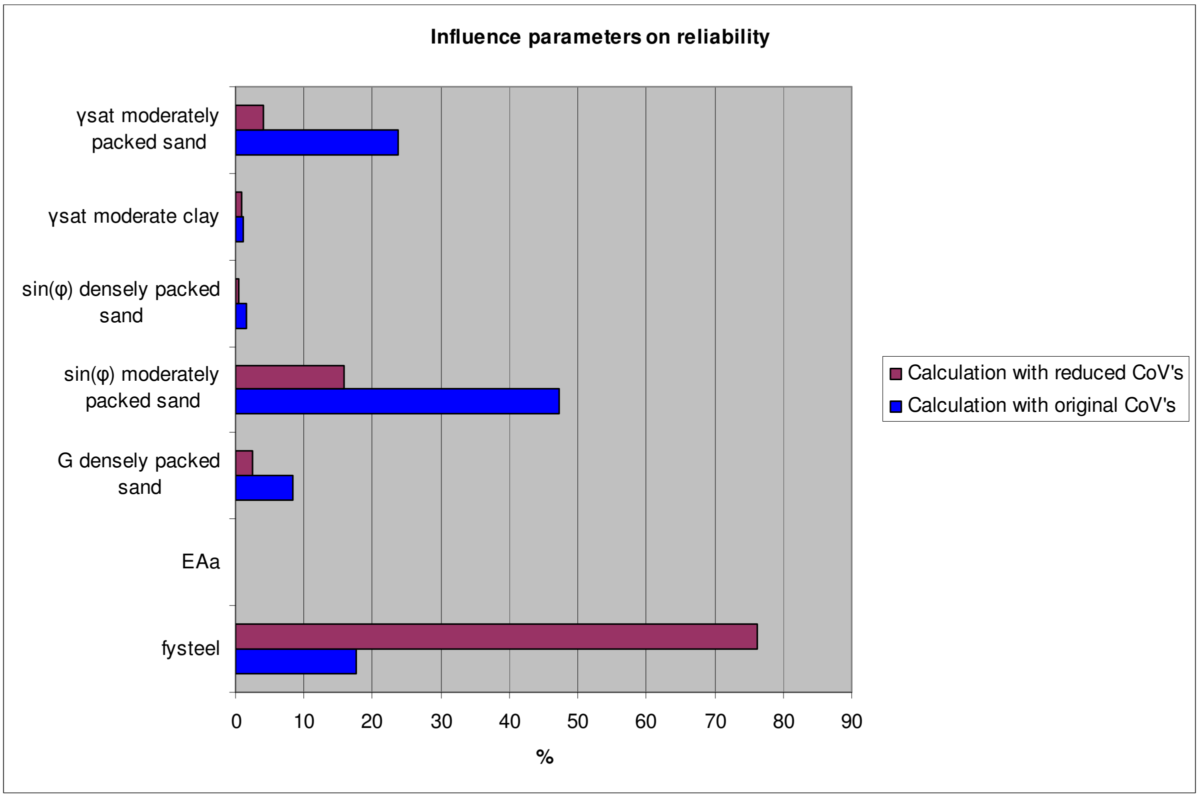


Figure 7-2 Comparison of parameter influence on reliability with different CoV's final calculation

This shift in influence coefficients and reduction of coefficients of variations has logically impact on the partial safety factors. The comparison between partial safety factors from the original calculation and the calculation from this chapter is shown in Table 7-4.

	Original calculation		Calculation with reduced CoV	
	$\beta_{calculation}$ = 4.087		$\beta_{calculation}$ = 6.294	
	β_{CUR211} = 3.828		β_{CUR211} = 3.828	
X_i	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**	**$\gamma_{\mu,i,CUR211}$**	**$\gamma_{k,i,CUR211}$**
$f_{y,steel}$	1.13	1.00	1.31	1.16
EA_a	0.99	0.87	1.00	0.88
$G_{densely\ packed\ sand}$	0.75	0.49	0.92	0.54
$\varphi_{moderately\ packed\ sand}$	2.11	1.76	1.18	0.99
$\varphi_{densely\ packed\ sand}$	1.11	0.93	1.03	0.86
$\gamma_{sat,moderate\ clay}$	1.02	0.94	1.02	0.93
$\gamma_{sat,moderately\ packed\ sand}$	1.10	1.01	1,04	0,96

Table 7-4 Comparison partial safety factors

For this research however it is too much work to redo all calculations and include spatial correlations. For now it is therefore assumed that the influence factors stay the same, but the coefficients of variation of the soil parameters are reduced by a factor 2 in the determination of the partial safety factors. This is for the soil parameters a conservative estimate, but the influence of the other (structural) parameters is underestimated (shown by Figure 7-1 and Figure 7-2). This change in partial safety factors due to the reduced CoV is applied to get an idea about the order of magnitude of the partial safety factor for the dominant parameters of soil stiffness and internal angle of friction. The previous analysis showed it is possible that for instance the anchor parameters get a slightly higher partial safety factor, but this effect is not taken into account in the tables below. The partial safety factors for the mechanisms of benchmark 1 (original) are given in Table 7-5.

Parameter	**CUR 166 (class III)**	**CUR 211 (class II)**	**Calculations (generalized)**			
			Anchor	**Wall**	**Soil**	**Deformations**
$EI_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$EA_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$w_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$W_{sheet\text{-}pile}$	1.00	1.20	μ	μ	μ	μ
D_a	1.00	1.10	1.00	μ	μ	μ
$f_{y,steel}$	1.00	1.00	1.00	μ	μ	μ
EA_a	1.00	1.20	1.00	μ	μ	μ
E_{50}	1.00	μ	0.70	0.95	1.05	0.75
E_{oed}	1.00	μ	0.70	0.95	1.05	0.75
E_{ur}	1.00	μ	0.70	0.95	1.05	0.75
φ	1.20	1.00	1.15	1.20	1.35	1.20
C	1.10	1.00	μ	μ	μ	μ
γ	1.00	μ	1.05	0.95	μ	μ
Water level difference (Δ)	0.30	0.80	0.10	0.05	0.10	0.05
Water level left (outside) (Δ)	0.25	0.70	-	-	-	-
Water level right (ground) (Δ)	0.05	0.10	-	-	-	-
Surcharge load	1.00	1.20	1.05	1.05	1.00	1.05
Retaining height (Δ)	0.35	0.40	0.15	0.05	0.10	0.10

Table 7-5 Partial safety factors original benchmark 1 with spatial correlation

The partial safety factors for the mechanisms of benchmark 1 (elongated) are given in Table 7-6.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (generalized)			
			Anchor	Wall	Soil	Deformations
$EI_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$EA_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$w_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ	μ
$W_{sheet\text{-}pile}$	1.00	1.20	μ	μ	μ	μ
D_a	1.00	1.10	μ	μ	μ	μ
$f_{y,steel}$	1.00	1.00	1.00	μ	μ	μ
EA_a	1.00	1.20	μ	μ	μ	μ
E_{50}	1.00	μ	0.55	0.95	1.00	0.75
E_{oed}	1.00	μ	0.55	0.95	1.00	0.75
E_{ur}	1.00	μ	0.55	0.95	1.00	0.75
φ	1.20	1.00	1.15	1.25	1.30	1.00
C	1.10	1.00	μ	μ	μ	μ
γ	1.00	μ	1.05	0.95	0.85	μ
Water level difference (Δ)	0.30	0.80	0.10	0.10	0.00	0.00
Water level left (outside) (Δ)	0.25	0.70	-	-	-	-
Water level right (ground) (Δ)	0.05	0.10	-	-	-	-
Surcharge load	1.00	1.20	1.05	1.00	1.05	1.05
Retaining height (Δ)	0.35	0.40	0.10	0.15	0.10	0.05

Table 7-6 Partial safety factors elongated benchmark 1 with spatial correlation

The partial safety factors for the mechanisms of benchmark 2 are given in Table 7-7.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (generalized)			
			Anchor	Wall	Soil	Deformations
$EI_{combi\text{-}wall}$	1.00	1.00	μ	μ	μ	μ
$EA_{combi\text{-}wall}$	1.00	1.00	μ	μ	μ	μ
$w_{combi\text{-}wall}$	1.00	1.00	μ	μ	μ	μ
$W_{combi\text{-}wall}$	1.00	1.20	μ	μ	μ	μ
D_a	1.00	1.10	μ	μ	μ	μ
$f_{y,steel}$	1.00	1.00	1.10	μ	μ	μ
EA_a	1.00	1.20	1.05	μ	μ	μ
E_{50}	1.00	μ	0.40	0.90	0.95	0.70
E_{oed}	1.00	μ	0.40	0.90	0.95	0.70
E_{ur}	1.00	μ	0.40	0.90	0.95	0.70
φ	1.20	1.00	0.90	1.15	1.30	0.95
C	1.10	1.00	μ	μ	μ	μ
γ	1.00	μ	1.00	0.85	0.80	0.85
Water level difference (Δ)	0.30	0.80	0.05	0.10	0.00	0.05
Water level left (outside) (Δ)	0.25	0.70	-	-	-	-
Water level right (ground) (Δ)	0.05	0.10	-	-	-	-
Surcharge load	1.00	1.20	1.05	1.00	1.05	1.05
Retaining height (Δ)	0.35	0.40	0.05	0.10	0.05	0.05

Table 7-7 Partial safety factors benchmark 2 with spatial correlation

It is clear that the partial safety factors for the soil parameters, especially for the stiffness parameters and internal angle of friction are significantly reduced when spatial correlation is included.

7.3 Geometrical parameters and loads

It is discussed before that it is difficult to include loads and geometrical parameters (soil layer levels, water levels, bottom height and height of the ground level) in probabilistic PLAXIS calculations. This is mainly due to the fact that regeneration of the PLAXIS mesh is necessary which makes it impossible to let Prob2B steer the calculation. When regeneration of the mesh is applied in fact a new model is introduced with different nodes and elements. Comparison with previous calculation results is therewith difficult. The approach that is used in this research is convenient in a way that the possible variations in the governing situation can be varied manually. Including the verification with Blum this gives a good insight into the relative importance of those parameters. A better approach, however, would be the inclusion of these parameters from the start of the probabilistic calculation. The uncertainty in surcharge load, retaining height and water levels is however not dominant, which makes it reasonable to assume the error in the manual variations to be small. It is however recommended not to include the absolute partial factors in the new edition of CUR 211, because of those shortcomings of the model, unless more research is done.

Furthermore the definition of this ‘governing’ situation and the probability that this situation occurs could not be analyzed in this thesis. The obtained results are therefore only applicable for the specific governing situation, a certain load, retaining height and water level difference between out- and inside.

It is however true that in the design choices that are made to define the design retaining height, water levels and surcharge load (or other loads) additional safety is incorporated in the design. It depends on the moment of time how large the reliability is. If it is a normal day, without large water level differences and with a small surcharge load, the reliability index will be higher than calculated in this report. The calculated β is only there in the most extreme case (as defined by experience), in all other cases it is higher and the structure is in fact more reliable. The calculated β is a minimum, but the exact β fluctuates from day to day. The minimum always is governing and determining for the life cycle of the structure and therefore defined as the lifetime reliability index.

7.4 Limit State soil mechanical failure Z = MSF – 1.1

As discussed in section 4.8, the only pragmatic approach to define influence coefficients and a reliability index for the failure mechanism soil mechanical failure is the use of φ-C reduction with a target value of MSF = 1.1. The actual failure, however, occurs when the value of MSF = 1.0. Therefore, there is a certain underestimation of the reliability index. In the calculations it is furthermore assumed that the influence factors are the same for MSF = 1.1 and MSF = 1.0. In this section some calculation results are presented in order to check this assumption and to quantify the underestimation of β.

For the second benchmark quay wall with relieving floor a calculation is made with a target value of MSF = 1.2. The results are presented in Table 7-8.

Number of calculations (FORM): 369				
β: 2.361				
P_f: $9.110 \cdot 10^{-3}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$EA_{anchor1}$	0.07	-0.05	524500	[kN/m]
$EA_{anchor2}$	0.07	-0.03	521800	[kN/m]
$C_{clay\ 1}$	0.8	-0.03	12.10	[kPa]
$C_{clay\ 2}$	0.8	0.10	11.68	[kPa]

$E_{50,clay\ 1}$	0.3	0.00	15570	[kPa]
$E_{50,clay\ 2}$	0.3	0.00	11360	[kPa]
$E_{50,moderately\ packed\ sand}$	0.3	0.06	60180	[kPa]
$E_{50,silty\ moderately\ packed\ sand}$	0.3	0.11	28530	[kPa]
$E_{50,pleistocene\ sand}$	0.3	0.01	73860	[kPa]
$E_{oed,clay\ 1}$	0.3	0.00	7785	[kPa]
$E_{oed,clay\ 2}$	0.3	0.02	5725	[kPa]
$E_{oed,moderately\ packed\ sand}$	0.3	-0.03	60190	[kPa]
$E_{oed,silty\ moderately\ packed\ sand}$	0.3	0.06	28540	[kPa]
$E_{oed,pleistocene\ sand}$	0.3	0.46	73860	[kPa]
$G_{clay\ 1}$	0.3	-0.09	32400	[kPa]
$G_{clay\ 2}$	0.3	0.00	18930	[kPa]
$G_{moderately\ packed\ sand}$	0.3	0.00	75280	[kPa]
$G_{silty\ moderately\ packed\ sand}$	0.3	0.00	47530	[kPa]
$G_{pleistocene\ sand}$	0.3	0.00	107700	[kPa]
$m_{clay\ 1}$	0.2	-0.03	1.00	[-]
$m_{clay\ 2}$	0.2	-0.01	1.02	[-]
$m_{moderately\ packed\ sand}$	0.2	0.03	0.49	[-]
$m_{silty\ moderately\ packed\ sand}$	0.2	-0.03	0.51	[-]
$m_{pleistocene\ sand}$	0.2	-0.03	0.50	[-]
$R_{int,clay\ 1}$	0.2	0.04	0.65	[-]
$R_{int,clay\ 2}$	0.2	0.06	0.68	[-]
$R_{int,moderately\ packed\ sand}$	0.2	-0.05	0.92	[-]
$R_{int,silty\ moderately\ packed\ sand}$	0.2	-0.04	0.93	[-]
$R_{int,pleistocene\ sand}$	0.2	-0.03	0.91	[-]
$\sin(\varphi)_{clay\ 1}$	0.18	0.07	0.45	[-]
$\sin(\varphi)_{clay\ 2}$	0.18	-0.02	0.39	[-]
$\sin(\varphi)_{moderately\ packed\ sand}$	0.18	0.02	0.63	[-]
$\sin(\varphi)_{silty\ moderately\ packed\ sand}$	0.18	0.05	0.56	[-]
$\sin(\varphi)_{pleistocene\ sand}$	0.18	0.79	0.40	[-]
$\sin(\psi)_{moderately\ packed\ sand}$	0.18	0.00	0.05	[-]
$\sin(\psi)_{pleistocene\ sand}$	0.18	-0.05	0.12	[-]
$\gamma_{sat,clay\ 1}$	0.05	0.00	18.59	[kN/m^3]
$\gamma_{sat,clay\ 2}$	0.05	0.01	17.34	[kN/m^3]
$\gamma_{sat,moderately\ packed\ sand}$	0.05	0.00	21.85	[kN/m^3]
$\gamma_{sat,silty\ moderately\ packed\ sand}$	0.05	-0.08	20.62	[kN/m^3]
$\gamma_{sat,pleistocene\ sand}$	0.05	-0.29	20.22	[kN/m^3]
$\gamma_{unsat,moderately\ packed\ sand}$	0.05	0.02	19.66	[kN/m^3]
$d_{1420/18\ AU20}$	0.03	-0.03	1.53	[kN/m]
$w_{1420/18\ AU20}$	0.03	-0.01	2.69	[kN/m]
$G_{eq,1420/18\ AU20}$	0.07	0.01	1763000	[kN/m^2/m]

calc.	Z-value	
1	1.01	
369	0.06	**MSF = 1.26**

Table 7-8 Output calculation 1 LS soil mechanical failure Z = MSF - 1.2

Figure 7-3 shows the comparison between the influence factors for MSF = 1.1 and MSF = 1.2. It is clear that these differences are small.

The reliability index is lower than in case of MSF = 1.1. It reduced from 2.7 till 2.4, which implies that with a 0.1 change in MSF, β changes with 0.3. However, it is still difficult to draw strict conclusions from these calculation, as the step in β between MSF = 1.2 and 1.1 is not necessarily the same as the step between MSF = 1.1 and 1.0. This is due to the non-linearities that occur when the soil is plastically deforming when it approaches failure. The 0.3 underestimation of β when applying the Z = MSF – 1.1 is therefore an uncertain estimate in itself.

With respect to the influence factors the reasoning from Figure 7-3 can be translated to the difference between results for MSF = 1.1 and MSF = 1.0. It is clear that the same parameters are important during the process in the direction of failure. It is reasonable to assume that this process continues in the same way until MSF = 1.0, which implies that the same parameters are important.

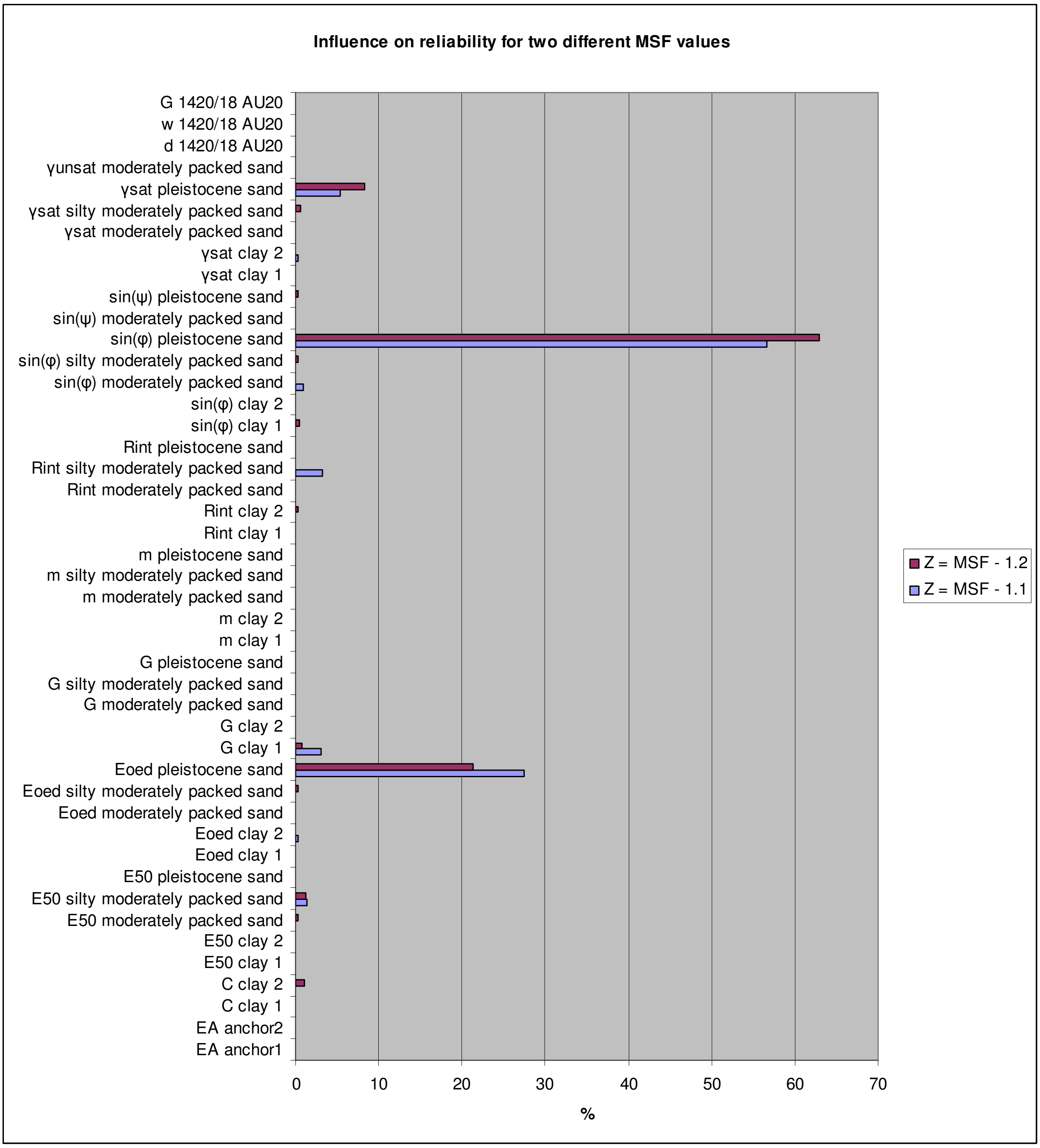


Figure 7-3 Influence in % on reliability for two different MSF values calculation 1

The same conclusions can be drawn when comparing the final calculation results of Table 7-9 and Figure 7-4. The difference in reliability index is in this case (2.78 – 2.49) also around 0.3. The influence parameters in case of MSF = 1.2 are almost identical to the influence parameters in the case of MSF = 1.1.

Number of calculations (FORM) : 57				
β: 2.493				
P_f: $6.336 \cdot 10^{-3}$				
Parameter (X)	**V = σ / μ**	**α**	**X* (design point)**	**Unit**
$E_{oed,pleistocene\ sand}$	0.3	0.46	73280	[kPa]
$\sin(\varphi)_{pleistocene\ sand}$	0.18	0.82	0.38	[-]
$\gamma_{sat,pleistocene\ sand}$	0.05	-0.34	20.17	[kN/m^3]

calc.	Z-value	
1	0.19	
57	0.00	**MSF = 1.20**

Table 7-9 Output final calculation LS soil mechanical failure Z = MSF - 1.2

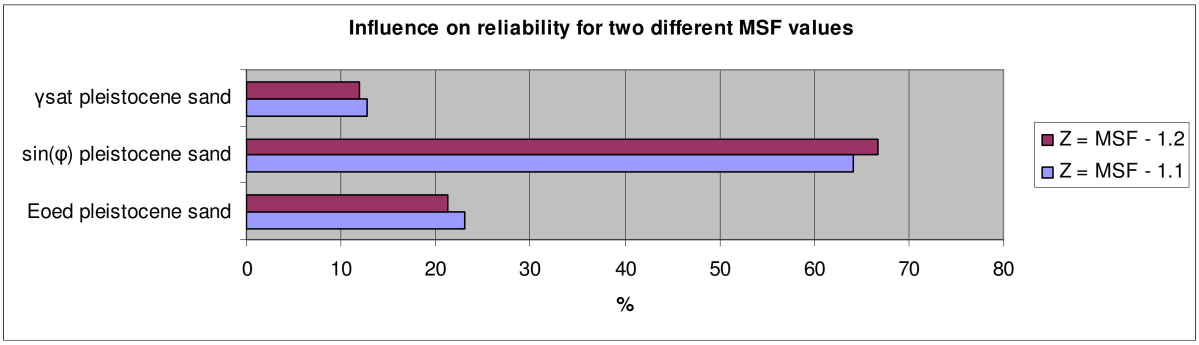


Figure 7-4 Influence in % on reliability for two different MSF values final calculation

7.5 Adaptations in target reliabilities (fault tree)

From all benchmark calculations it appeared that soil mechanical failure is a critical failure mode. With the design guidelines of CUR 166 the required reliability index (4.396) for this mechanism is even for an anchored sheet-pile (benchmark 1) by far not met (3.381, probably 3.681 when applying section 7.4). This gives a motive to redistribute the failure space over the analyzed mechanisms, in other words to adapt the target reliabilities.

The required reliability index is relatively high as soil mechanical failure is a mechanism that has a higher probability of occurrence than for instance anchor failure. Basically, these types of structures are most sensitive to soil mechanical failure. This can be solved when asking a higher target reliability index for the anchor. This higher reliability can be reached by applying an additional factor on the maximum anchor force.

This also has positive implications for the partial safety factors. The dominant parameter in soil mechanical failure is always the internal friction angle, which is expressed in extreme high partial safety factors. The higher the target reliability the higher the partial safety factor, and this in fact only influences the absolute dominant internal friction angle. For instance in case of soil mechanical failure of benchmark 2:

α_φ : 0.8
$\beta_{CUR\ 211}$: 4.396
CoV : 0.2
γ_φ : 2.82

If the target reliability index is reduced to for instance 3.500 the situation becomes:

α_{φ}	:	0.8
$\beta_{CUR\ 211}$	:	3.500
CoV	:	0.2
γ_{φ}	:	1.90

On the other hand, when the target β for anchor failure is made higher this would also influence the internal friction angle, but here more parameters are important (not one dominant parameter) which makes the increase in safety factors to be more spread out over different parameters. For instance some additional factor on the anchor parameters can be placed to reach the required higher target reliability.

However, strengthening of the anchor also influences the deformations and bending moment in the wall. A weaker anchor would imply larger deformations, but more mobilization of the passive soil wedge and therewith smaller bending moments in the wall. These effects should be considered when adapting the failure space.

These changes in target reliabilities and therewith change in failure tree would provide the partial safety factors for the different mechanisms to be more balanced.

Another topic related to the division of failure space is the correlation between failure mechanisms. Both from benchmark 1 and 2 it can be seen that there is a correlation between mechanisms wall failure in bending and failure of the passive soil wedge. Both mechanisms are mainly induced by the failure of the lower soil layer (in the probabilistic calculation through reduction of the internal angle of friction). This correlation also manifests itself through the partial safety factors. Both the stiffness parameters of the soil and the internal angle of friction need a design value lower than the characteristic value. The magnitude of the factors differs. This is due to the fact that for soil mechanical failure in fact only the internal angle of friction via failure of the passive wedge is causing the failure of the structure, whereas for wall failure in bending (or combined with axial force) also other mechanisms play a role. This is mainly the failure of soil elements in the higher soil layers which cause the load on the wall to increase and therewith the bending moment in the wall.

Anchor failure, however, is caused by an entirely different mechanism. Mainly due to the failure of the layer where the anchor is fixed and the increasing stiffness in the lower soil layer. Part of the increase in anchor force is induced by the same mechanism as for wall failure and soil mechanical failure, i.e. reduction of the passive soil resistance. This reduction gives the wall possibilities to move which is counteracted by the anchor.

It is possible to define correlations between the mechanisms with the formula: $\rho_{Z1,Z2} = \Sigma \alpha_{i,1} \alpha_{i,2}$. These calculations have not been made in this report due to the limited time. It is however clear that the correlation between soil mechanical failure and wall failure is strong and their correlation with anchor failure weak. In the failure tree it is not clear what correlations are assumed and therefore it is not possible to define the differences and adaptations that should be made. However, it is clear that it is conservative to assume no correlation or a small correlation between soil mechanical failure and wall failure.

7.6 Model uncertainty

An important issue is the uncertainty in the used models. There is some uncertainty in the PLAXIS model and the calculations in this model. Furthermore there is uncertainty in the applied probabilistic method FORM. Both uncertainties count in the uncertainty of the final output α_i and β and some additional sources of uncertainty can be listed as well.

7.6.1 Uncertainty in PLAXIS

PLAXIS requires tolerance settings. The stricter these settings the more calculation time required and the more aborted calculations, because convergence is more difficult to find. During the calculations the most optimal settings were found. It should be noted that it is not recommended to make changes in the default calculation settings during daily use of PLAXIS.

An important setting is the ‘tolerated error’, which is part of the load advancement procedure that is used for this type of calculations. The standard setting is 0.01, but during the calculations it was found out that faster and more often convergence was found when this tolerated error is 0.03. This implies a higher uncertainty in results, but it is more pragmatic.

Furthermore it was necessary to deselect arc-length control. The difference between load-control and arc-length control criterion is shown in Figure 7-5. the probabilistic calculations are searching for a design point where failure ‘just’ occurs. When arc-length control is selected often the soil body collapses according to this criterion (i.e. the calculation is aborted), whereas the load control procedure combined with additional load steps gives opportunities to finish the calculation. When the soil body collapses during the calculation the calculation is stopped and the results cannot be used for probabilistic analysis. This gave many problems during the start-up period of this thesis until the standard arc-length control was deselected. Furthermore deformations are more correctly calculated when using load control.

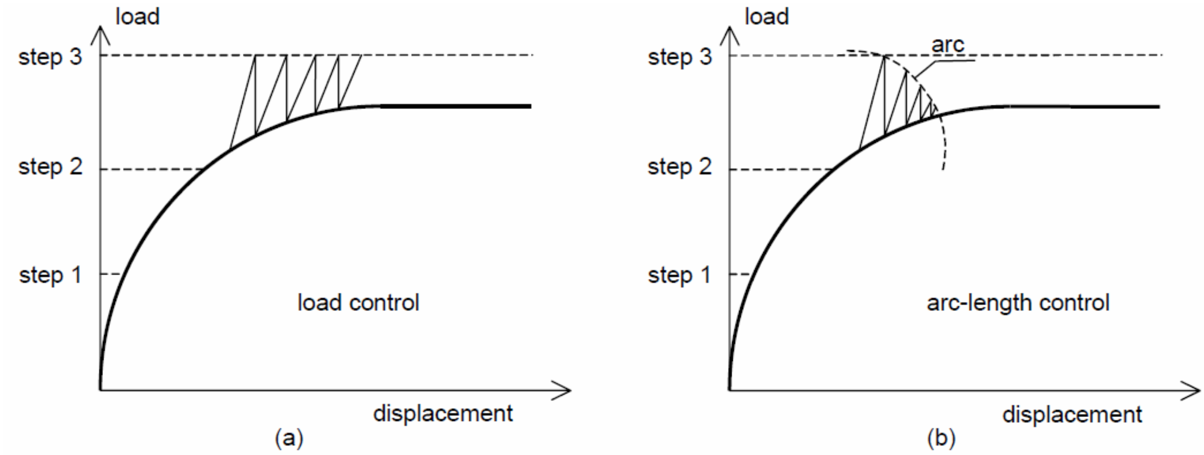


Figure 7-5 load-control and arc-length control (Brinkgreve & Broere, 2008)

The arc-length control option does not influence the uncertainty of the model. Therefore with respect to PLAXIS the uncertainty is determined by the setting ‘tolerated error’. The model uncertainty is 3%.

7.6.2 Uncertainty in FORM procedure

Prob2B requires the settings for convergence criteria as discussed in section 4.9. This implies there is certain accuracy in β and α_i. The calculations are eventually done with convergence criteria of 0.01 or 0.05, which implies a 1-5 % uncertainty.

However, there is still some space left between the found design point and Z=0. The convergence criteria do not directly prescribe a certain minimum distance between the found design point and Z=0. Instead it is prescribed that β may not change too much in the next step and the obtained Z-value over the change dZ/du may not be too large. This results in final calculations with Z-values larger or smaller than zero, but it is true that there is a (local) minimum close to Z = 0. The final Z-values of all calculations are presented and analyzed in Table 7-10 ('O' implies 'order of magnitude').

Anchored sheet-pile (21 m)	Z_{final}	$O(Z_{start})$	$O(Z_{final})$	$O(Z_{final})/O(Z_{start})$	**Unit**
Anchor failure[13]	-0.11	10^{2}	10^{-1}	$\mathbf{10^{-3}}$	[kN]
Wall failure[13]	-23.10	10^{3}	10^{1}	$\mathbf{10^{-2}}$	[kNm]
Soil mechanical failure[13]	0.047	10^{0}	10^{-2}	$\mathbf{10^{-2}}$	[-]
Deformations	0.0005	10^{-1}	10^{-4}	$\mathbf{10^{-3}}$	[m]
Anchored sheet-pile (23 m)					
Anchor failure	4.68	10^{2}	10^{0}	$\mathbf{10^{-2}}$	[kN]
Wall failure[13]	194.60	10^{3}	10^{2}	$\mathbf{10^{-1}}$	[kNm]
Soil mechanical failure	0.006	10^{0}	10^{-3}	$\mathbf{10^{-3}}$	[-]
Deformations	-0.0006	10^{-1}	10^{-4}	$\mathbf{10^{-3}}$	[m]
Quay wall with relieving floor					
Anchor 1 failure[13]	13.05	10^{2}	10^{1}	$\mathbf{10^{-1}}$	[kN]
Anchor 2 failure	33.27	10^{2}	10^{1}	$\mathbf{10^{-1}}$	[kN]
Wall failure	8110	10^{5}	10^{3}	$\mathbf{10^{-2}}$	[kNm]
Soil mechanical failure	0.007	10^{0}	10^{-3}	$\mathbf{10^{-3}}$	[-]
Deformations	-0.0002	10^{-2}	10^{-4}	$\mathbf{10^{-2}}$	[m]

Table 7-10 Z- values and deviations for all probabilistic calculations

The table shows the order of magnitude of start (first calculation) and final value (design point) of Z. The order of magnitude of the final Z-value is divided by the order of magnitude of the start value, which gives the order of magnitude of the relative deviation from the point Z with respect to Z = 0. It is shown that all calculations fall in between the ~10% deviation border (when checking the absolute values the maximum deviation is for wall failure of the quay wall with relieving floor, 13%). Two of the calculations with a deviation of ~10% are calculated with less strict convergence criteria (0.05 instead of 0.01), i.e. wall failure calculation of the elongated anchored sheet-pile and anchor 1 failure calculation of the quay wall with relieving floor. The only other with a deviation of ~10% is anchor 2 failure. All other parameters have a lower than 10% relative deviation of Z = 0, in the order of magnitude of 1% or 0.01%.

This can be used to estimate the uncertainty in the reliability index. However it is difficult to translate this deviation from Z = 0 to a deviation in β as it is not clear whether β is linearly increasing with distance from Z = 0. If it is nevertheless assumed this is the case, the uncertainty of β due to the FORM method is maximal 10%. It is assumed that this deviation from Z = 0 does not affect the influence factors, because these are depending on dZ/du which are directly dependent on the convergence criterion for Z.

There is another uncertainty in β because there is a probable discrepancy between the calculated β and the actual β due to the assumption that linearization can be applied in the design point. This is inherent to the FORM method. This linearization however mostly affects the influence factors α_i as these factors are directly determined from the linear step dx_i and the subsequent change dZ. It is therefore assumed no additional uncertainty in reliability index is needed to be taken into account, but

[13] Convergence criteria 0.05

only for α_i. As no reference calculation with another method is made it is hard to estimate this uncertainty. 5% till 10% is possible as can be seen in the large differences between the partial safety factors based on α_i and β and the partial safety factor based on μ/X^*.

7.6.3 Other uncertainties

There can be thought of more uncertainties. Although these uncertainties are more difficult to quantify than the numerical uncertainties and FORM uncertainties, they are very important in the total uncertainty of the model output. The sources that are relevant are listed below.

- There is a discrepancy between the PLAXIS model and reality. It should be hold in mind that PLAXIS itself is an approach to describe reality and that the reliability indices and influence factors are always bound to certain models and that they can indicate reliability indices in the real circumstances. It can be said however that PLAXIS (or FEM in general) is the best way (in comparison to MSheet or Blum for instance) to do this type of calculations as it includes effects like arching and shear hardening of the soil. It is not possible to quantify this uncertainty, but awareness of the shortcomings of the model is important.
- Uncertainty in definition of coefficients of variation (they might be higher or lower). This problem can be faced by stating that the results are only applicable for the presented set of CoV's and values. The try-out with the reduced CoV's (section 7.2.1) showed that the results can be influenced considerably by this variable.
- Human errors are also possible in this type of modelling and calculations. PLAXIS requires the definitions of the model, the input of parameter values and the settings of the calculations. Often the model is based on limited information and abstractions to fit the input format. Furthermore, the input of en PLAXIS and Prob2B should be done manually which gives the risk of errors. Other people have checked the models in order to make the chance on errors as small as possible.

7.6.4 Summary

All elements that participate in model uncertainty are summarized below:

- Uncertainty in PLAXIS model 3%
- Uncertainty in FORM procedure
    - convergence criteria 1% - 5%
    - deviation from Z = 0 1% - 10%
    - Linearization 5% - 10%
- Uncertainty in modelling of reality
- Uncertainty in definition of CoV's
- Uncertainty due to possible human errors

If the latter three elements are estimated to count for an uncertainty of 5 %, the upper bound of the uncertainty in β is 23%, whereas the lower bound is 10 %. A realistic estimate is therefore 15% uncertainty in β. For the influence factors α_i the deviation form Z = 0 is not that relevant with respect to uncertainty. Therefore the upper bound is 18% and the lower bound 13 %. 15% is a realistic estimate for the uncertainty in α_i. It should be hold in mind that the influence factors are correlated to each other and therefore it is not necessary to take 15% for each parameter. Furthermore, the distribution of β is not uniform, but more Gaussian or log-normal. For instance, the probability that the deviation from Z =0 is 10% is smaller than the probability that this deviation is 1 %.

It is clear that this analysis is very quick and dirty. These figures are just presented to give an idea of the uncertainty in model results and to create awareness of all the nuances that should be made when analyzing the results. More research however on these topics should be done to get more insight into these uncertainties.

8 Conclusions and Recommendations

8.1 Introduction

This chapter contains the conclusions and recommendations following from these conclusions. The conclusions and recommendations are categorized according to the different chapters of this MSc thesis.

8.2 Conclusions

8.2.1 Problem analysis (research questions)

The main objective of this research is ‘Calibrating the partial safety factors from CUR 211’. This objective was subdivided in four research questions that can be answered after the analyses in the previous chapters.

1. How can the reliability index of a quay wall be determined?

The reliability index (β) of a quay wall can be determined by first defining the most important failure modes. For each failure mechanism a Limit State Function (LSF) must be defined. By applying the First Order Reliability Method (FORM), design values and influence coefficients for the different stochastic parameters can be found. It should be noticed that these design values are approximations as linearization in the design point is applied. By combining the probability that the design values of the different parameters are present the probability of failure and therewith the reliability is obtained. Combining the probabilities of failure for different mechanisms (including their correlations) gives the overall probability of failure of the quay wall and therewith its reliability index.

2. *What are the reliability indices and parameter influence factors of the two benchmark quay walls?*

The reliability index of the quay walls is subdivided into indices for four mechanisms. Three mechanisms in Ultimate Limit State and one in Serviceability Limit State. For the original first benchmark (anchored sheet-pile wall), the reliability indices for wall failure and anchor failure are close to the prescribed reliability index for these mechanisms in CUR 211. For soil mechanical failure the reliability index was much too low (3.0 instead of 4.4), which is partly caused by the fact that the sheet-pile is too short. For deformations the reliability target is met, there is still some safety margin left. The elongated anchored sheet-pile wall shows the same trend. However, still the reliability index for soil mechanical failure is too low (3.4).

The second benchmark (quay wall with relieving floor) shows a different image of the reliability indices per mechanism. The upper anchor of the structure has a higher reliability index than prescribed, while the lower anchor has a lower reliability index. Furthermore the index for wall failure in bending is too low (2.6 instead of 3.9), even as the index for soil mechanical failure (2.78 instead of 4.4). For deformations the target is met.

For soil mechanical failure the reliability index is slightly underestimated, because the φ-C reduction is due to pratical reasons not pushed through until a safety factor value of MSF = 1.0, but MSF = 1.1. A reference calculation showed that the underestimation of β is about 0.3.

The estimated uncertainty in reliability index β is around 15%. This includes the uncertainty in the FORM procedure, PLAXIS, the definition of the coefficients of variation and human errors. For a reliability index of 3.8 this implies that the range in β is 3.27 – 4.37. This range is not uniformly distributed, but it is more like Gaussian curve, which implies that the probability of occurance of the lower band is relative small. With the brief analysis of model uncertanity it is not possible to define a standard deviation for β.

3. Which partial safety factors can be derived from these reliability indices and influence factors?

When generalizing the results of the three benchmark calculations, Table 8-1 of partial safety factors ($\gamma_{k,i} = \dfrac{1-1.64V_i}{1-\alpha_i \beta V_i}$) is obtained.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (generalized)		
			Anchor	**Wall**	**Soil**
$EI_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ
$EA_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ
$w_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ
$W_{sheet\text{-}pile}$	1.00	1.20	μ	μ	μ
D_a	1.00	1.10	μ	μ	μ
$f_{y,steel}$	1.00	1.00	1.10	μ	μ
EA_a	1.00	1.20	1.05	μ	μ
E_{50}	1.00	μ	0.30	1.75	2.50
E_{oed}	1.00	μ	0.30	1.75	2.50
E_{ur}	1.00	μ	0.30	1.75	2.50
φ	1.20	1.00	1.80	2.40	3.10
C	1.10	1.00	μ	μ	μ
γ	1.00	μ	1.05	0.85	0.80
Water level difference (Δ)	0.30	0.80	0.15	0.10	0.10
Surcharge load	1.00	1.20	1.05	1.05	1.05
Retaining height (Δ)	0.35	0.40	0.15	0.15	0.15

Table 8-1 Generalized partial safety factors over characteristic value (μ = mean value)

With inclusion of spatial correlation in the formula for the partial safety factor (i.e. $V(\bar{r}) = 0.5V(r_i)$) the factors of Table 8-2 are obtained.

Parameter	CUR 166 (class III)	CUR 211 (class II)	Calculations (generalized)		
			Anchor	**Wall**	**Soil**
$EI_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ
$EA_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ
$w_{sheet\text{-}pile}$	1.00	1.00	μ	μ	μ
$W_{sheet\text{-}pile}$	1.00	1.20	μ	μ	μ
D_a	1.00	1.10	μ	μ	μ
$f_{y,steel}$	1.00	1.00	1.20	μ	μ
EA_a	1.00	1.20	1.05	μ	μ
E_{50}	1.00	μ	0.40	0.95	1.00
E_{oed}	1.00	μ	0.40	0.95	1.00
E_{ur}	1.00	μ	0.40	0.95	1.00
φ	1.20	1.00	1.15	1.25	1.35
C	1.10	1.00	μ	μ	μ
γ	1.00	μ	1.05	0.85	0.80
Water level difference (Δ)	0.30	0.80	0.15	0.10	0.10
Surcharge load	1.00	1.20	1.05	1.05	1.05
Retaining height (Δ)	0.35	0.40	0.15	0.15	0.15

Table 8-2 Generalized partial safety factors over characteristic value with inclusion of spatial correlation (μ = mean value)

4. How can the reliability indices and influence factors be used to optimize the target reliabilities per failure mode and to describe the dependencies between the mechanisms?

From the benchmark calculations it appeared that soil mechanical failure is the most critical mechanism. It occurs with highest failure probability for the given designs and therefore it is not convenient to give this mechanism (failure of the passive soil wedge) a higher target reliability than anchor or wall failure. It would be more economical to strengthen the anchor and make this target reliability higher, than to work with extreme high partial safety factors on the internal angle of friction to prevent the passive soil wedge from failing.

Furthermore for the given designs there is a strong correlation between soil mechanical failure and wall failure in bending, as both mechanisms are mainly caused by the failure of the passive soil wedge. A weak correlation can be found between these two mechanisms and anchor failure. The correlations manifest themselves also through the partial safety factors. For soil mechanical failure and wall failure in bending the stiffness parameters of the soil get a partial factor > 1, whereas for anchor failure this factor is < 1. The same can be seen at the values of the partial safety factor for the internal angle of friction. This value is for soil mechanical failure and wall failure in the same order of magnitude.

8.2.2 Method

Furthermore, there are conclusions about the applicability of PLAXIS-FORM in Prob2B for this type of calculations:

- It is important to be aware of the fact that PLAXIS-FORM is a model of reality with a specific goal: describing the probability of occurance of certain failure modes of quay walls. This model is not equal to reality and therefore all conclusions with respect to PLAXIS or Prob2B are refering to the model and not to reality.
- Prob2B works fine in loading and steering PLAXIS models. It offers several probabilistic methods, it can be coupled to other models like MatLab and Excel and the interface is user friendly. It requires some effort to figure out the meaning and implications of all settings, but the tool in itself works fine. Moreover, the hardening soil model is available which makes probabilistic calculations for sheet-piles and quay walls with relieving floor more convenient. The main drawback is that not all variables can be included in the analysis. The loads, water levels and geometrical levels are difficult to implement in Prob2B. Method Blum offers insight into the importance of the variation in the governing values of these parameters, but a comparison with PLAXIS is not possible. Fortunately, it turned out that these parameters have only small influence on the reliability and therefore this shortcoming in Prob2B does not lead to large problems.
- Especially for quay walls with relieving floor the use of PLAXIS has advantages. In PLAXIS effects like arching, 2D geometry and the additions of more soil stiffness parameters to describe the complex interaction between soil and structure can be included. These aspects are not modelled by methods like Blum and elastically supported beam theory.
- A drawback of PLAXIS, especially when structures are in a state close to failure is the relative long calculation time. This makes it with a normal 64-bit computer difficult (in terms of time) to perform Monte Carlo analyses or other fully probabilistic calculations. Level III calculations would give more accurate results than FORM can give with its linearization in the design point.
- The determination of the influence coefficients α_i is questionable, because in the design point the plastic deformation of the soil elements is a non-linear process. FORM however assumes a linear process in its determination of influence coefficients. For a linear Limit State, the following should hold:

$$\frac{\mu_i}{X_i^*} = \frac{1}{1-\alpha_i \beta V_i}$$

However, due to non-linearities in the failure mechanisms the left hand side often differs significantly from the right hand side of the equation. As the influence factors α_i are used to define the partial safety factors it is implicitly assumed that this difference is small, but that is not always the case as shown in the tables with comparisons between the partial safety factors of the left and right hand side of the equation.

- Besides the non-linearities also the correlations between the parameters may influence the influence coefficients. They can disturb the view on the important parameters through the fact that another parameter gets an increased α_i only because it is correlated with the actual important parameter.
- The process of φ-C reduction is not completely stable, which makes it difficult to find convergence in a FORM calculation for soil mechanical failure.
- Prob2B-PLAXIS does not always work. Sometimes it is difficult to find convergence for stricter criteria. The method is therefore not very robust. It is unknown how many time is needed to do a calculation.
- Different reliability indices can be found for the same mechanism. This is due to the fact that Prob2B sometimes finds two different 'design points' close to each other, one of them is actually not the design point. This makes the search for Z = 0 (the real design point) complicated.
- When two failure mechanisms, one of which is soil mechanical failure, are close to each other, problems might occur with convergence in Prob2B. Prob2B cannot handle a soil body collapse during the probabilistic calculations. This is for instance the case in the original benchmark 1 where wall failure in bending interferes with soil mechanical failure through (partly) failure of the passive wedge.

8.2.3 Starting Points

Also conclusions about the starting points can be drawn, they are listed below:

- The definition of the coefficients of variation is important, because the partial safety factors are to a large extent formed by the coefficient of variation of a parameter (directly, but also via the influence coefficient α_i). Especially the inclusion of spatial correlation is important, as it can reduce the coefficients of variation for the soil parameters with 50%, which has large impact on the safety factor.
- The target reliability for passive soil wedge failure in the failure tree of CUR 211 is too high as explained before.
- The coefficients of variation for sheet-pile or combi-wall parameters are hardly relevant in the probabilistic calculation and therefore have little influence on the reliability. The anchor parameters are relative important compared to the wall parameters and therefore should get more attention.

8.2.4 Results

In addition to the results in form of partial safety factors and reliability indices as presented in section 8.2.1, more conclusions can be drawn about the results:

- The design and control method of CUR 166 chapter 4 with Finite Element Method is applicable for anchored sheet-piles like benchmark 1. This is only the case when some changes are made in the target reliabilities for the different mechanisms. The required reliability index for failure of the passive soil wedge should be reduced and the required reliability index for anchor failure can be raised.

- The application of generalized parameters (Table 8-1 and Table 8-2) leads to a suboptimal design, because no distinction is made between layers. For instance, for soil mechanical failure only factors need to be placed on the lower sand layer. This also counts to a large extent for wall failure in bending. When placing factors one each layer, a heavier and longer wall is chosen and a too high reliability index is obtained.
- Soil mechanical failure and wall failure in bending occur in a much different way than anchor failure. Therefore it is necessary to define low and high partial safety factors for stiffness parameters of the soil (< 1 for anchor failure and >1 for the other two mechanisms). The same holds for the internal angle of friction. Here, there can be defined two 'high' values. For anchor failure does not need an increased factor, but for soil mechanical failure and wall failure this is needed. Due to the correlations also different values need to be defined for the specific soil weight.
- The calculated reliability indices indicate the minimum reliability of the structure over its design lifetime, as it deals with a governing load situation (retaining height, water levels and loads).

8.3 Recommendations

Based on the conclusions a list of recommendations is presented here.

8.3.1 Method

- Prob2B is suited to perform probabilistic calculations in PLAXIS and therefore it is worthwhile to use it for future studies.
- Standardize the procedure of probabilistic calculations design of sheet-piles and quay walls, as it gives options to get insight into the importance of each parameter and the reliability that a certain design has. When a designer has some experience with Prob2B it is tractable to use it in sheet-pile and quay wall design as well.
- Use PLAXIS to model quay walls with relieving floor (structures that have clear 2-dimensional elements).
- More sophisticated computers would make it possible to perform level III analysis in PLAXIS.
- Plastic deformation and failure of a sheet-pile or combi-wall is non-linear and therefore non-linear probabilistic calculations should be applied as well. SORM instead of FORM can be tried or other higher order methods. This would reduce the difference between $\frac{\mu_i}{X_i^*}$ and $\frac{1}{1-\alpha_i \beta V_i}$.

8.3.2 Starting points

- More research should be done to check the influence of the correlations between parameters on their partial safety factor.
- Different methods should be researched to define the Limit State Function for soil mechanical failure.
- The discussion on the coefficients of variation should be conducted and more foundation of the chosen coefficients should be found. Especially the spatial correlation is an important topic as it influences the partial safety factors to a large extent. It is necessary to standardize the coefficients of variation and make clear how the coefficients are derived (this is not the case in NEN 6740, which makes it difficult to understand and interpret these values).
- Change the target reliability of failure of the passive soil wedge in the fault tree of CUR 211 into a lower value and raise the target reliability for anchor failure. More research needs to be done in order to define these target values. Furthermore, more research needs to be done to

find indicators for correlations between failure mechanisms in order to improve the fault tree and make a more optimal design possible.

8.3.3 Results

- Change the partial safety factors of CUR 211 according to Table 8-1. When more research is done with respect to spatial correlation and some additional calculations are done the values of Table 8-2 might be applied with some additional factors on the anchor and probably wall parameters.
- Be careful with application of the new geometrical and load factors, because their derivation method differs from the partial safety factors of the other parameters.
- Rewrite CUR 166 chapter 4 in order to be applicable for quay walls with relieving floor, i.e. change the FEM design procedure for CUR 211. A suggestion for this procedure (based on calculation scheme B) based on the results with inclusion of spatial correlation (i.e.Table 8-2) :

  Use characteristic values for the soil strength (C and φ). Sheet-pile parameters and cohesion can have their mean value. For the anchor diameter also the mean value can be used, whereas the axial stiffness and yield stress of the anchor should have their design value. The partial safety factors (according to Table 8-2*) can be used to define design values for the retaining height, water levels, external loads and soil stiffness parameters. Note that two different calculations need to be performed with two different stiffness parameter values and two different specific soil weight values (saturated or unsaturated):*

  1) $E_{d,high} = \frac{E_{char}}{0.4}$ *and* $\gamma_{d,low} = \frac{\gamma_{char}}{1.05}$

  and

  2) $E_{d,low} = \frac{E_{char}}{1.0}$ *and* $\gamma_{d,high} = \frac{\gamma_{char}}{0.8}$

  The φ-C reduction should be continued till a value of:

  - *1.15 (safety class II) for calculation 1*
  - *1.35 (safety class II) for calculation 2*

  From these reductions in soil strength the anchor forces, bending moments and deformations need to be checked. Furthermore the quay wall should not have been failed soil mechanically.

- Make calculations of more quay walls in order to get better ground for the conclusions. It is preferable to work with quay walls that are designed according to CUR 166, CUR 211 or EC7, because a comparison should be made. Also different characteristic soil configurations should be analyzed. The current benchmark calculations are specifically respresenting the situation in the port of Rotterdam.
- It would be worthwhile to define different partial safety factors for different soil layers as most often parameters of one layer are dominant. This would deliver a more optimal design.
- Even more convenient would be to make probabilistic calculations (or sensitivity analyses) for each quay wall design that is made to define the most important layers and parameters. Based on these analyses the partial safety factors can be placed on the correct parameters and a more optimal design can be made. This is already possible with the sensitivity analysis in PLAXIS or in a more sophisticated way with Prob2B.

- No special partial safety factors for SLS are needed, but for each calculation the deformations at the relevant spots in the wall or superstructure should be checked.
- Check the current table of partial safety factors by making validation probabilistic calculations and checking the reached reliability index.
- The model accuracy should be investigated in more detail. The current estimate of 15% for β and 15% for α_i are rough values.

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