Graduation project:

Comparative analysis of design recommendations for Quay Walls

Gemeentewerken
Gemeente Rotterdam

TU Delft

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Preface

Always I wanted to do something with water and ships. It was not possible for me to join the navy, neither was it possible to work on an international trade ship. But finally I found something to combine those two topics in the graduation project of my study Civil Engineering at Delft University of Technologies. It contains a study for quay wall design, in which water, soil and mooring of ships are the main topics.

The work in front of you contains the results of a comparative study for literature and design approaches for quay wall structures. The purpose of this study is to compare design guidelines, with special attention to safety and design approach.

I wanted to thank Prof. drs. ir. J.K. Vrijling, ir. J.G. de Gijt and ing. H.J. Everts for their help during the graduation process, collecting literature and be supportive during the calculations. I also want to thank my room mates at the Public Works of Rotterdam, which helped me during the calculations and were very nice company during my graduation period.

A special thanks also goes to my family which supported me when I was often very busy with my graduation project.

I had a lot of fun during writing this graduation report, and I also hope you will have when reading it!

Rotterdam, June 2006

Emiel Meijer

Summary

The design and construction of quay walls is a problem in which people are interested for ages. There are many methods and recommendations developed for this topic. They differ for each country and type of structure and are developed for local conditions. Some of these methods and recommendations are collected in a national code, which can be used for the design of maritime structures.

In Europe they try to normalize all national codes to obtain one European standard. Each of the European members can add their national parameters to this code. For the normalization of codes it is interesting to investigate which codes are available and which methods are used in the national codes.

In this analysis are the *difference between design guidelines for quay walls considered, with special attention to the safety analysis and design process, to find an overview of design results*. Therefore first a global analysis of contents is carried out. This results in 4 guidelines with a clear safety approach, also adapted to the latest design philosophies: CUR 166 and Handbook Quay Walls (both Dutch), EAU 2004 (German), Eurocode 7 (European Union). The last one is not used, because it is still under construction and includes mainly the safety approaches treated in the other 3 guidelines.

For CUR 166, Handbook Quay Walls and EAU 2004 comparative calculations are carried out in a beam on elastic foundation model. Two cases are considered which are very typical for quay walls in the Port of Rotterdam: a quay wall with 12 m retaining height and a quay wall with 30 m retaining height (this one includes a superstructure).

First, for all guidelines the characteristic parameters are determined: water levels, geotechnical properties and external loads. The geometrical aspects and material parameters are kept the same for the analysis.

The safety approach in the guidelines give the greatest difference in the design process. If a fault tree is present, the failure mechanism can be overviewed, which is very useful for design calculations. Mainly the application of safety factors on actions or action effects lead to different answers. The CUR 166 applies safety factors on soil strength parameters (actions), the Handbook Quay walls and EAU 2004 applies the safety factors on action effects (internal forces).

The EAU 2004 gives in all calculations the smallest bending moments and anchor forces for the application in a beam on elastic foundation program. This is mainly due to the higher strength of the soil properties in the EAU 2004 and due to the fact that the Blum schematization does not satisfy in the beam on elastic foundation program.

For the calculation of case 1 (retaining height 12 m) the Handbook Quay Walls gives higher bending moments than CUR 166, mainly due to the application of a special load combination with extreme scour. For the calculations of case 2 (retaining height 30 m) the CUR 166 gives higher bending moments than Handbook Quay Walls. This is mainly due to the application of the relieving platform in combination with safety factors on the soil strength parameters, which results in higher bending moments.

It can be concluded that the EAU 2004 is not useful for application in a beam on elastic foundation program. The CUR 166 and Handbook Quay Walls are very useful for a beam on elastic foundation method. Mainly the Handbook Quay Walls is very specialized on quay walls structures. It includes certain load combinations, descriptions for the calculation of a superstructure and applies partial safety factors on action effects. This makes the Handbook Quay Walls more useful for the design of quay wall structures than CUR 166.

Summary in Dutch - Samenvatting in het Nederlands

Het bouwen en ontwerpen van kademuren is een probleem waarin men al eeuwen in is geïnteresseerd. Er zijn vele methoden en richtlijnen ontwikkeld voor kademuurontwerp. Deze methoden en richtlijnen verschillen per land en type constructie, en zijn vaak ontwikkeld voor locale omstandigheden. Sommige methoden en richtlijnen zijn verzameld in een nationale norm.

In Europa probeert men de afzonderlijke nationale nomen van de lidstaten te combineren tot één Europese norm. Elke lidstaat kan zijn eigen nationale parameters toevoegen aan deze norm. Het is daarom interessant te onderzoeken welke richtlijnen er bestaan en welke ontwerpmethoden gebruikt worden.

In deze vergelijkende analyse zijn de *verschillen tussen de beschikbare ontwerprichtlijnen beschouwd, met nadruk op veiligheidsanalyse en ontwerpaanpak, om een overzicht te geven van ontwerpresultaten*. In eerste instantie is een globale analyse uitgevoerd naar de beschikbare richtlijnen. Dit resulteert in 4 richtlijnen met een duidelijke veiligheidsbeschouwing, aangepast aan de laatste ontwikkelingen: CUR 166 en Handboek Kademuren (beide Nederlands), EAU 2004 (Duits), Eurocode 7 (Europees). De Eurocode 7 is echter nog niet bruikbaar, omdat deze nog niet volledig ontwikkeld is en reeds de veiligheidsbeschouwing van de 3 overige richtlijnen bevat.

Voor de CUR 166, Handboek Kademuren en EAU 2004 zijn vergelijkende berekeningen uitgevoerd in een verenmodel. Twee cases zijn beschouwd met een typische constructie, zoals die in de Rotterdamse haven wordt gebouwd: een kademuur met 12 m kerende hoogte en een muur met 30 meter kerende hoogte (dit ontwerp bevat ook een bovenbouwconstructie).

Voor de ontwerpcases zijn karakteristieke parameters bepaald: waterstanden, geotechnische eigenschappen en uitwendige belastingen. De geometrische en materiaal eigenschappen zijn constant gehouden, omdat deze de basis vormen voor de vergelijkende analyse.

Het grootste verschil tussen de richtlijnen zit in de veiligheidsbenadering. Als een foutenboom aanwezig is, kan een overzicht van faalmechanismen worden gegeven, welke zeer bruikbaar is bij ontwerpberekeningen. Hoofdzakelijk het gebruik van veiligheidsfactoren op belasting of belastingseffecten leidt tot verschillende antwoorden. De CUR 166 past de veiligheid toe op de sterkte eigenschappen van de grond (belasting), terwijl het Handboek Kademuren en de EAU 2004 de veiligheid hoofdzakelijk toepassen op de snedenkrachten (belastingseffecten).

De EAU 2004 resulteert in alle situaties tot de kleinste momenten en ankerkrachten voor de toepassing in een verenmodel. Dit komt hoofdzakelijk door de sterkere grondeigenschappen in de EAU 2004 en doordat niet wordt voldaan aan de Blum schematisering.

Voor de berekening van case 1 geeft Handboek Kademuren grotere momenten dan CUR 166. Dit komt voornamelijk door de toepassing van een speciale belastingscombinatie met extreme ontgronding in het Handboek Kademuren. Voor de berekeningen van case 2 geeft het Handboek Kademuren juist de kleinste momenten ten opzicht van de CUR 166. Dit komt voornamelijk door de toepassing van een ontlastvloer in combinatie met veiligheidsfactoren op de grond sterkte eigenschappen voor de CUR 166.

Geconcludeerd kan worden dat de EAU 2004 niet goed toepasbaar is in een verenmodel en dat de CUR 166 en het Handboek Kademuren dat juist wel zijn. Hoofdzakelijk het Handboek Kademuren is gespecialiseerd voor het ontwerp van kademuren. Het omvat verschillende belastingcombinaties, beschrijving van het ontwerp van een bovenbouwconstructie en gebruikt veiligheidsfactoren op de belastingseffecten. Dit maakt het Handboek Kademuren meer geschikt voor het ontwerpen van een kademuur dan de CUR 166.

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1 Introduction

Quay walls are earth retaining structures, that separate the land from the water, for the mooring of ships. Large loads are working on the structure, which will only increase in the nearby future. This makes the design and construction of a quay wall interesting and complicated. Therefore several design guidelines are available to give recommendations for the design and construction of quay walls.

Nowadays, with the European standardization of codes, it is planned to collect all design guidelines to compose one European code: the Eurocode. Each country can add his national parameters to this code. Therefore it is worthwhile to investigate the national design guidelines. This comparative analysis has the purpose to show the effect of different design philosophies. First all important literature is collected and some important items in the literature are compared. After the general analysis some guidelines, that are useful for further research, will be carried out in a comparative calculation analysis. This will be done for two cases that are typical for the Port of Rotterdam, with a small and large retaining height.

In “2.Problem analysis” the problem will be defined and some simplifications are considered. In “3.Quay walls and design” the quay wall will be defined, together with the functions and type of walls. In “4.Available design recommendations”, a consideration is given about the available national and international literature and in “5.Probabilistic design philosophy” is explained what kind of probabilistic safety approaches there are. The characteristic values needed for a design are overviewed in “6.Determination characteristic values”. The chapters 7 ,8 and 9 describe the specific design philosophies of Handbook Quay Walls, CUR 166 and EAU 2004. In the last chapter “10.Conclusions and recommendations” the conclusions are drawn and recommendations for further design are described. In the Annexes background information is given and calculations are carried out.

2 Problem analysis

2.1 Situation sketch

The design and construction of quay walls is a "problem" in which people are interested for ages. There are many methods and recommendations developed for this topic. They differ for each country and type of structure and are developed for local conditions. Some of these methods and recommendations are collected in a national code, which can be used as guideline for designing structures.

In Europe they try to normalize all national codes to obtain one European standard. Each of the European members can add their own national parameters to this code. For the normalization of codes it is interesting to investigate which codes are available and which methods are used in the national codes.

2.2 Problem description

There are many international design guidelines for quay wall design for which is not known how they are related too each other, especially in relation with European standardization.

Therefore the **question** is:

"What are the differences between design guidelines for quay wall design, with special attention to safety approach and design method?"

2.3 Objective

"Consider the differences between design guidelines for quay walls, with special attention to the safety approach and design process, to find an overview of design results that is useful for future designs."

In general this means that a number of design guidelines for quay walls must be collected. The guidelines will be considered and those who are adapted to the latest semi-probabilistic design philosophy will be worked out in detail. For these guidelines it is the purpose to overview the difference in calculation results due to safety factors, especially those working on the soil parameters and internal forces. At the end it must be clear which guideline is most useful and which method of applying safety factors is most favorable, to give some recommendations for future quay wall design.

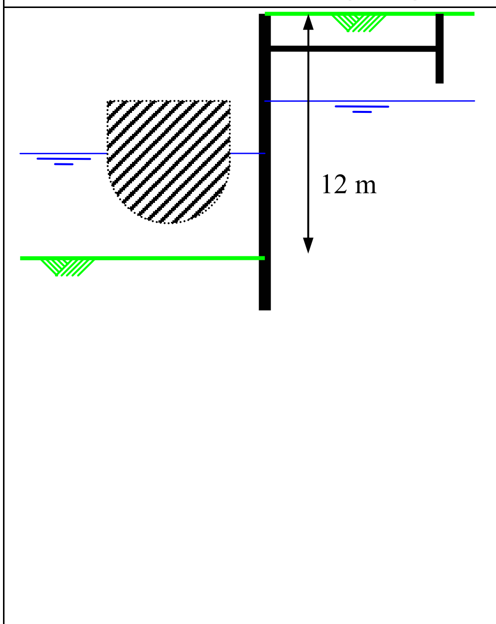
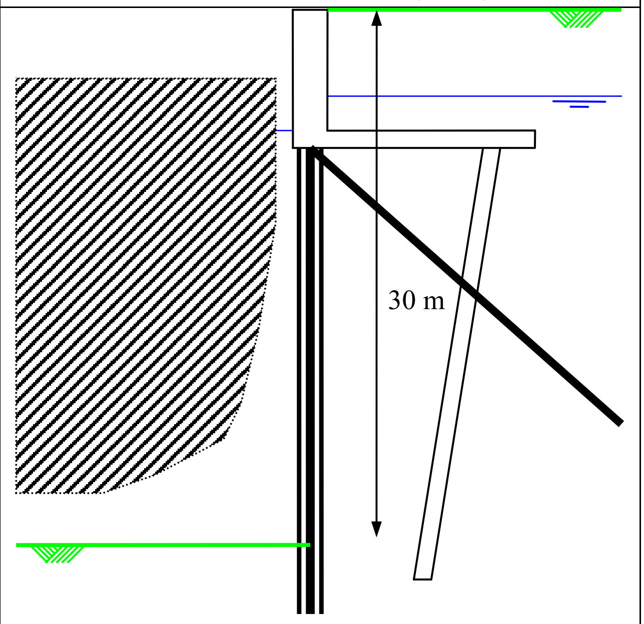
2.4 Design cases for comparative calculations

2.4.1 Design cases

The design guidelines which contain sufficiently information about safety will be worked out for certain cases to make the comparative calculations. Two cases are chosen, which can be seen as typical cases for quay walls in Rotterdam: a “small” quay wall with sheet pile wall, tie rod and anchor wall; and a “large” quay wall with combined sheet pile wall, anchor pile and superstructure. These cases will be worked out for conditions comparable with the Port of Rotterdam.

Figure 2-1: Yangtze Harbor in the Port of Rotterdam with top view on the construction area of the Euromax Quay Wall

Therefore the new quay wall under construction, the Euromax quay at the Maasvlakte, will be used as reference. The detailed conditions will be investigated in the chapter 6 and annex C about the characteristic design parameters.

Case 1: 12 m retaining height	Case 2: 30 m retaining height
	

2.4.2 Delimiting the comparative calculations

The cases used for the comparative calculations will be very theoretical with a lot simplifications. For the comparative analysis no real structure have to be designed, it is more important which design aspects are treated and how they influence the design results. Therefore the comparative analysis is delimited and is assumed that:

- No construction phases are taken into account
- Only the sheet pile calculation will be carried out and compared, no other failure mechanisms (the sheet pile calculation in this case is the calculation of the minimum toe level, maximum bending moment and anchor force)
- Displacements are not taken into account
- No cost calculations are made, no amount of steel will be calculated
- Soil conditions are based on 1 cone penetration diagram
- No changing of the angle of inclination is taken into account due to vertical loads on top of the wall
- Arching in the active earth pressure in all directions is not taken into account
- No drainage systems are applied in the cases (in a real quay wall a drainage can reduce the excess pore pressure behind the wall)
- All calculations will be done for one linear meter of quay wall structure
- All calculation will be done with a beam on elastic foundation computer program (or if possible with an associating model)

However, the aspects mentioned above can be very important for the structure, for example the amount of steel that will be used is important for the cost of a quay wall.

In the comparative analysis the toe levels of the wall can have a simple, partially fixed and fixed earth support. The figure below shows how this is treated.

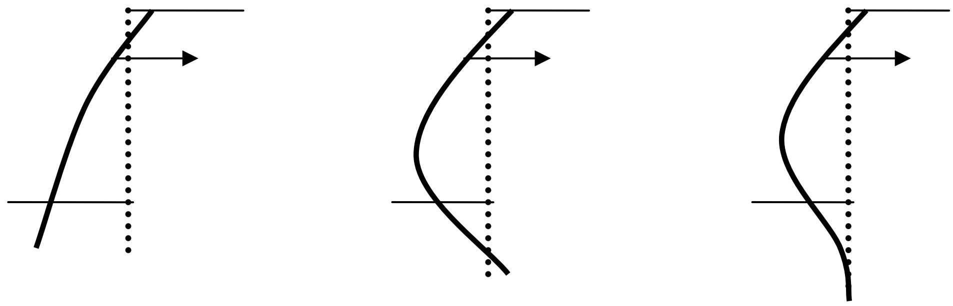


Figure 2-2: Simple earth support Partially fixed earth support Fixed earth support

2.4.3 Calculation model: beam on elastic foundation

The calculations will be carried out with a beam on elastic foundation model. A computer program, called “MSheet”, is available for a model with springs. This model is chosen because it is a well known model, that is used a lot at the moment for the design of sheet pile walls in the Port of Rotterdam. The computer program that is made for this model is very user-friendly and it is easy to apply safety factors in this model.

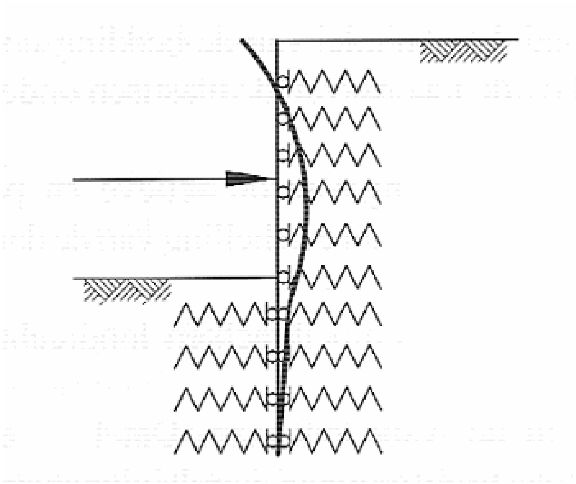


Figure 2-3: Beam on elastic foundation model, with springs that schematize the soil

The springs in this program schematize the soil stiffness (see figure 2-4). However, the real soil stiffness does not behave like a spring with one stiffness parameter. It depends on the amount of displacements and stresses that are working in the soil. In this model only the horizontal displacements are taken into account. An adaptation of the angle of inclination due to vertical displacements is therefore not included and must be done by hand. So the calculations are just an approximation of the reality.

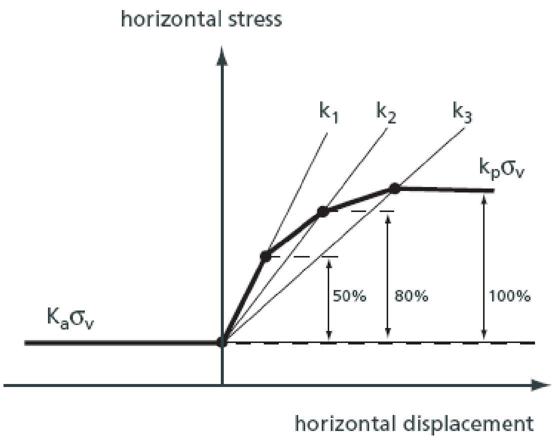


Figure 2-4: Schematization of soil stiffness for a beam on elastic foundation model

In this model there are two options of applying earth pressure coefficients. The earth pressure coefficients can be introduced in the program by hand. Then it is easy to change these coefficient. However, in this schematization no discontinuities are allowed. So no change in area loads and ground surface can be applied. This is only possible when the earth pressure coefficients are calculated from the c, φ and δ parameters (option two). Those parameters are calculated numerical by the program and can if necessary be adapted by hand for each spring, but that costs a lot of work.

3 Quay walls and design

3.1 Definition of a "Quay" and a "Quay wall"

The "quay" is that part of the harbor for mooring, loading and unloading ships, where bulk and cargo can be transported and/or stored

The "quay wall" is a retaining structure, separating the land from the water, for the mooring of ships.

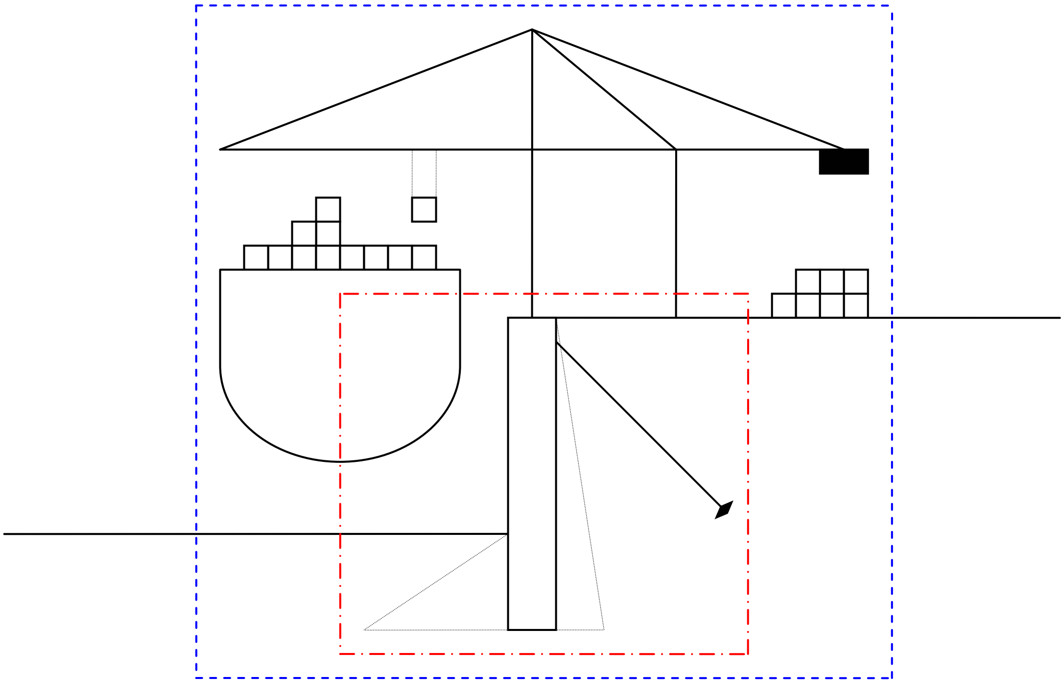


*Figure 3-1: Definition area of quay(wall): **Quay** area inside the dashed line; **Quay Wall** inside the dashed-dotted line*

3.2 Quay wall in worlds history

For ages people are trying to move over water, because they want to explore and conquer the world, but also from an economical point of view. Therefore traders and conquerors were sailing the oceans. Those people needed places to berth their ships, which later grow out to harbors. From these developments not only a large growth of prosperity is visible, but also a growth in knowledge about new technologies.

The oldest harbor known, is found in India near Lothal and probably dates form 4000 years ago. It is founded due to a large trade between countries in Asia. Also in the Mediterranean, harbors were formed for trade. Alexandria was the last three ages before Christ the main trade centre in this region. Also the construction of harbors developed in these ages. The Romans were the first who used a kind of concrete for the construction of quay walls.

In the Middle Ages the Vikings sailed the Western European waters with very fast ships. In this period there were two major problems: siltation of harbors and the poor equipment available in the harbor. In general there were no quay walls of stone and the cargo had to be transshipped by hand. Later, cranes became available to do this work, but with these cranes the next problem raised: a strong subsoil was needed. This played a very important role in the development of quay walls with vertical bearing capacity.

In the Netherlands, Amsterdam was the first place where quay walls were constructed. In the Golden Ages (1700 ac) this was the trade centre of the world. This also gave raise to the construction of quay walls in Middelburg, Dordrecht, Stavoren, Delft and Rotterdam.

In the nineteenth century the steam engine was presented and in the twentieth century there was a large development in the tonnage of ships. The consequences were larger ships with a larger draught. The draught of the ship has a lot of influence on the retaining height of the quay wall. Another consequence is the growing possibility of self-berthing of the ships and the extra scour due to propeller currents. Also the method of transshipment changed, which lead to higher loads at the quay and larger quay walls. All these developments lead to the development of a quay wall piled up by stones to a sophisticated design.

The developments in Rotterdam show the struggle for the search of a good solution for a quay wall with large retaining height, high loads and a weak subsoil. The draughts of the ships changed from 5 m in the 17th century up to 20 meters in the nearby future.

3.3 Main types of quay walls

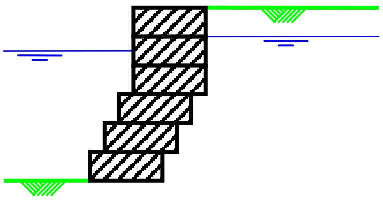
A quay wall is a soil retaining structure, which occurs in many shapes. All these structures have the same function:

- Mooring place for ships
- Soil retaining function
- Bearing capacity for crane loads, goods and storage
- Sometimes a water retaining function

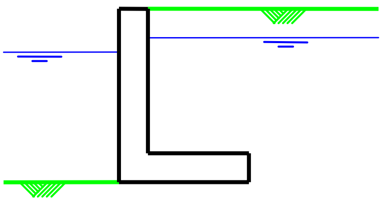
To fulfill all these functions, 4 main types of structures can be considered:

1. Gravity Walls

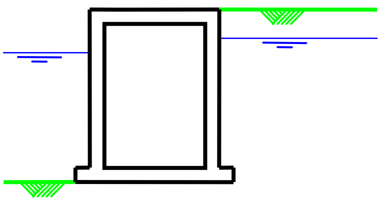
1.1 Block walls



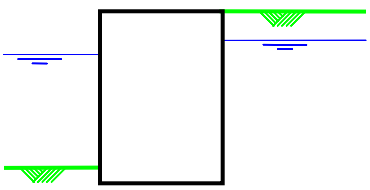
1.2 L-wall



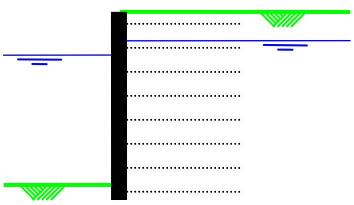
1.3 Caisson wall



1.4 Cellular wall

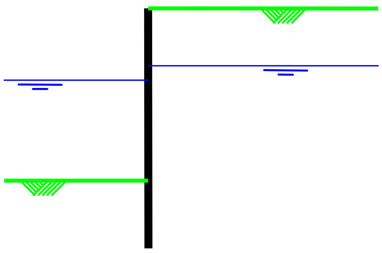


1.5 Reinforced earth structure

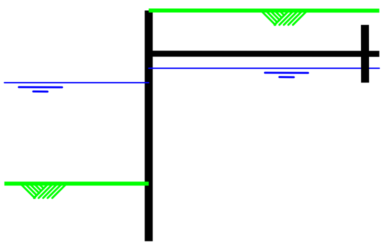


2a. Embedded Walls

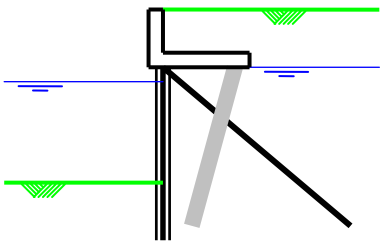
2a.1 Cantilever wall



2a.2 Anchored wall

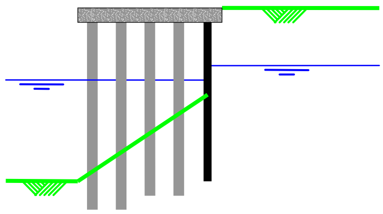


2b. Embedded Walls (with relieving platform)

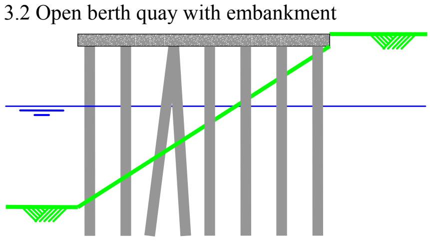


3. Open berth quay

3.1 Open berth quay with retaining wall



3.2 Open berth quay with embankment



3.4 Parameters of soil, material, loads and geometry

For the design of a quay wall there are some important parameters needed. These parameters should be summarized in a program of requirements:

3.4.1 Important parameters

- Geometrical and hydrographical parameters
- Water levels, waves, currents
- Ice loads
- Meteorological parameters
- Morphological parameters
- Nautical parameters
- Seismological parameters
- Geotechnical and geo-hydrological parameters
- Environmental parameters

3.4.2 Geotechnical parameters

For the determination of the geotechnical parameters test can be done to find the characteristic values. It is not always possible to find the parameters by testing. In that case some tables are available with conservative parameters, which can be used.
The calculation models require specific soil parameters. Which parameters are needed is indicated in table below.

Calculations models	**Parameters**
Classical model (Blum)	γ, φ' and c' or c_u, δ
Beam on elastic foundation model	γ, φ' and c' or c_u, δ, k_h
Finite element method (FEM):	
- Mohr-Coulomb model	γ, φ' and c' or c_u, δ, E, ν, ψ
- Hardening Soil model	γ, φ' and c' or c_u, δ, ψ, E_{ur}, ν_{ur}, E_{oed}, p^{ref}, K_0, R_f
- Consolidation	k_x, k_y
- Interface between soil and structure	R_{inter}
Groundwater flow:	Homogeneous soil: k
	Layered soil: k_x, k_y
Critical gradient, by Terzaghi and Peck	γ
Anchors	γ, φ' and c' or c_u, δ
Sliding planes, Kranz	γ, φ' and c'

Table 3-1: Calculation models for quay wall calculations with the required soil parameters (according to CUR 166)

3.4.3 Loads (actions)

The loads that work on the quay wall structure are:

Permanent loads:	Variable loads:	Special loads:
• Deadweight structure • Water and earth pressure under frequent conditions • Shear force between soil and superstructure	• Earth pressure due to extra vertical loads • Water pressure • Water pressure due to groundwater flow • Ship operations • Berthing forces • Mooring forces • Load and unload • Storage • Crain loads • Traffic loads • Environmental loads • Wave loads • Ice loads • Temperature variations	• Extreme water levels • Storage in an emergency situation • Falling loads • Collapse • Earthquake • Extreme excavation

3.4.4 Design calculations

For a safe structure, failure and collapse must be excluded during the life time of the structure. To prevent from failure limit states indicate the limit between failure and non-failure. Most recommendations contain two main limit states, an ultimate limit state for extreme situations and a serviceability limit state for deformations. Which calculations must be done depends on the recommendation, type of structure and the local situation in which the structure is applied. The failure mechanisms can be summarized in an fault tree.

In the ultimate limit states the safety will be reached with safety factors on soil parameters, loads and resistance. The safety factors can be obtained due to many years of experience. Nowadays, with a probabilistic consideration of the fault tree these safety factors are obtained for several failure mechanisms. For the serviceability limit state it is important to know that the deformations are limited.

In general there are 4 main calculations are needed to treat the failure mechanisms of a quay wall:
- Failure of the sheet pile wall
- To much groundwater flow
- Not enough soil stability
- Failure of the support point

For a quay wall with relieving platform some additional checks can be done:
- Foundation calculations
- Calculation for the superstructure

The models which are used differ in each recommendation, which makes it possible that safety factors differ. So, per recommendation must be tested which failure mechanism is used and how this is implemented in a model. The safety approaches play an important role in the recommendations available for quay wall design. The safety approaches per recommendation will be investigated in the next chapter 4 and the probabilistic design method of a quay wall is described in chapter 5.

4 Design guidelines for quay walls

4.1 Available guidelines

In a lot of countries literature is available for the design of quay wall structures. The PIANC started some investigations for literature about maritime structures. At the moment this study is just in an early stage and will be carried out by “MarCom-Workinggroup 50”. The purpose of this working group is to make a general code for maritime structures, because there are a lot of different codes that describe a part of this topic, but not all. Besides the literature that is under investigation by the PIANC, also some additional literature from the Netherlands is used for this comparative study. Guidelines with descriptions of quay wall design:

From the Netherlands:
- Handbook Quay Walls, CUR 211
- Handbook Sheet pile structures, CUR 166

For European purpose:
- Eurocode 7: Geotechnical Design, Part 1: General Rules
- Calibration Study National Annex for Eurocode 7
  (in Dutch: Calibratiestudie voor opstellen Nationale Bijlage voor Eurocode 7)

Other European literature:
- Germany: Guidelines for the Committee for Waterfront Structures, Harbors and Waterways, EAU 1996/2004
- United Kingdom: Code of Practice for Maritime Structures, BS 6349 Parts 1 & 2
- Spain: Actions in the design of Maritime and Harbor Works, ROM 0.2-90

International literature:
- Japan: Technical Standards for Ports & Harbor Facilities in Japan
- USA:
  - Military Handbook for Seawalls, Bulkheads, Quay Walls - 1025/4
  - Engineer Manual 1110-2-2504, Design of Sheet Pile Walls

Some of these guidelines are written as a code by a national normalization institute. Other literature is made by an institute supported by the government, which gives additional recommendation on codes. In some cases the protection of the coast is a matter of national security, which must be protected by the army. Those guideline are usually made by an institute supported by the army.

A more detailed description of the guidelines is given in the next paragraphs and in Annex B is a summary given about the topics that will be treated in the design guidelines.

4.2 Handbook Quay Walls, CUR 211 [1]

4.2.1 General

This guideline is written by CUR, the Civil Engineering Centre for Research and Legislation, in cooperation with the Port Authority and Public Works of Rotterdam. The purpose is to give an overview of the knowledge and experience on the field of designing quay wall structures in the Harbor of Rotterdam. It is specially made for the design of quay walls with a superstructure and sheet pile wall with bearing capacity in very soft soil.

4.2.2 Safety approach

This handbook uses a semi-probabilistic safety approach and is adapted to the latest European safety recommendations. This approach is based on a fault tree specially made for a quay wall with relieving platform and combined sheet pile wall. It is an addition on the fault tree made in CUR 166 for sheet pile walls. The **safety factors are based on failure of the sheet pile profile**, one of the failure mechanisms in the fault tree. This is probably because the guideline is made for walls with axial loads working on top. These axial loads need vertical bearing capacity, which leads to a fixed earth support. Therefore the failure mechanism of the profile is more important than the failure mechanism due to insufficient horizontal passive earth pressure resistance. The safety factors made for the failure of the profile are also applicable for the other failure mechanisms in the fault tree, besides the failure of the anchor system. Some additional safety factors have to be applied on the failure mechanism for the anchoring element. It looks like that this approach use a new method of designing, applying representative soil parameters and safety factors on the load effects in stead of the loads. The safety level, based on the main failure mechanism of the quay wall, has a β-value of 3,4. The calculations are based on a wall with a **fixed earth support**.

4.3 Handbook Sheet pile structures, CUR 166 [2,3]

4.3.1 General

This handbook is also written by the CUR, because in the Netherlands there was no special code for sheet pile structures. This guideline treats all types of sheet pile structure, especially the steel sheet pile wall. It can be used for the design of a building pit as well as for the design of a quay wall. The handbook gives also a lot of constructional information about driving piles and other construction works. There is also much information about calculation models and methods.

4.3.2 Safety approach

The handbook contains a semi-probabilistic approach, which is adapted to the latest European recommendations. The approach is based an a fault tree special written for sheet pile walls. There are different sheet pile walls which result in different safety levels. Each safety levels has its own safety factors. This also holds for the failure mechanisms in the fault tree. The **safety factors** are based on the failure mechanism of **insufficient passive earth pressure resistance**. The quay wall structure belongs to safety level III and has a β-value of 4,2 for the main failure mechanism of the failure of the sheet pile wall. A special step-by-step plan is given to apply the right safety approach. The calculations are based on a wall with **simple earth support** and safety factors applied on the soil strength parameters (tan(φ), c’ and δ).

4.4 Eurocode 7: Geotechnical Design, EN NVN 1997-1 [4]

4.4.1 General

The Eurocode is established to make one European code for geotechnical designs. This code is in a provisional stage to give all members of the European Union time to adapt to this guideline and let all members develop there own additional parameters. In this stage research is done for the national parameters in the Netherlands.

The Eurocode is one of 9 Eurocodes and contains the design of geotechnical structures as: fills, de-watering, spread foundations, pile foundations, anchorages, retaining structures and embankments. All structures and design aspects are described in a legal way, with a lot of text.

4.4.2 Calibration study National Annex Eurocode 7 – Geotechnical Design

The National Annex in the Netherlands for additional safety factors is still under investigation and not yet finished. Fugro has already written a calibration study, with comparisons between NEN-norms, CUR-publications and the general safety factors given in Eurocode 7. The safety parameters given in the Fugro-study for retaining structures are mainly based on the safety factors of CUR 166. Application of safety factors on the soil strength parameters (tan(φ), c’ and δ).

4.4.3 Safety approach

The Eurocode has the purpose to normalize all European design guidelines to one single guideline. Therefore it consist design process that compares with design processes in Germany, the Netherlands and other European countries. In general checks must be carried out for the serviceability limit state and several failure mechanisms in the ultimate limit state:

- Static equilibrium
- Resistance for structural and ground limit states
- Uplift
- Resistance of failure by heave due to seepage

Failure mechanism, "Resistance for structural and ground limit states", contains 3 design methods. In these methods the safety factors can be applied on loads or load effects, soil parameters and resistance. These safety factors are different per type of structure and safety level. In general the factors are made for safety level 2. This can be changed for a higher and lower level by multiplying with a factors 1,1 for the higher level and 0,9 for a lower level. These level are not based on a legal description, they are only informative.

4.5 Code of Practice for Maritime Structures, BS 6349 Part 1 & 2 [6]

4.5.1 General

BS 6349 is written by the British Standard Institute, the national normalization institute of the United Kingdom. Only the first 2 parts are studied: part 1, "General Criteria" written in 2000 and part 2, "Design of quay walls, jetties and dolphins" written in 1988. The British Standards are not adapted to the latest European guidelines, because **they rather trust their own standards**, than the European codes who are just under investigation. In the first part a basic description is given of criteria that are important in the design of maritime structures. In the second part the design methods of sheet pile walls, gravity walls and jetties are given in a short way. The BS 6349 refers a lot to BS 5400 for steel and concrete bridges, BS 6031 for soil structures, BS 8004 for foundations and BS 8002 for retaining structures. Not much attention is paid to calculation models and methods.

4.5.2 Safety approach

No clear safety approach is given. Some safety factors are available for loads on a jetty. To apply loads there is a distinction given for loads in a normal situation, extreme situations and loads during construction. For calculating of anchor forces and deformations they refer to BS 8002 for retaining structures. Stability calculations of the structure are described in BS 6031 considering soil structures.

4.6 Recommendations of the Committee for Waterfront Structures, EAU 1996/ 2004 [7,8]

4.6.1 General

The EAU is written by a committee for waterfront structures in Germany. At the moment 3 versions are applied: 1990, 1996 and 2004. In version 1996 several waterfront structures are treated: bottom protections, sheet pile walls, anchors, pile foundations (considering relieving platforms), embankments and mooring piles. The quay wall structure is described as parts of these structures.

4.6.2 Safety approach

The safety approach of the latest version 2004 is adapted to the Eurocode, which has a semi-probabilistic approach. However, the approach still corresponds with the other EAU versions which are based on a lot of experience. The safety factors are applied for 3 load cases in the ultimate limit state. These load cases depend on safety levels, combinations of actions and load cases. For the ultimate limit state 3 main failure mechanisms are defined:

- Loss of support safety

- Failure of structures and components (safety factors on actions effects)
- Loss of overall stability

The factors for the serviceability limit state use representative parameters and must be applied to check deformations.

4.7 Actions in the Design of Maritime and Harbor Works, ROM 0.2-90 [9]

4.7.1 General

ROM 0.2-90 is a part of the ROM-program which treats maritime and harbor works and has the purpose to make guidelines. It is written for the general direction of harbors and coasts in Spain. ROM 0.2-90 belongs to part "ROM 0.-" which treats general considerations about loads on structures. Not treated in this comparative analysis are "ROM 0.5" for geotechnical recommendations, "ROM 2.-" for the design of mooring facilities and "ROM 4.-" for the design of superstructures. These parts could be interesting.
A lot of information is given about the combination of permanent, variable and special loads. This guideline is useful for the determination of normative load combinations.

4.7.2 Safety approach

This handbook is made to give an overview of loads that work on maritime and harbor structures. The loads are divided in permanent, variable and special loads. The safety approach is given in a simple way. Much attention is paid to **load combinations**. The variable load in the load combination is divided in a dominant value and not-dominant values. The not-dominant values should be reduced with a combination factor. Also some general safety factors are given for fundamental and special loads.

4.8 Technical Standards for Port and Harbor Facilities in Japan [10]

4.8.1 General

This guideline is written by the bureau of ports and harbors and the harbor research institute of Japan. The English versions of this handbook are reconsidered by the Overseas Coastal Area Institute of Japan. It contains a lot of technical concepts that are applied in Japan for harbor facilities. This is very important for Japan, because the harbor is of great importance for the economical development.
A lot of structures are treated, but all very simple: gravity structures, foundations, waterways, basins, pavements, breakwaters, locks, sheet pile walls, jetties, mooring piles, floating jetties, a marina and pipelines. The quay wall with relieving platform is not very well treated.

4.8.2 Safety approach

In this handbook some empirical safety factors are given for safety on variable design parameters. If the variability of the parameter is larger, the safety factor will be larger. The factors are made for standard conditions, but must be adapted to local circumstances. For a lot of structures the a **design process** is given.

4.9 Military Handbook Seawalls, Bulkheads, Quay walls MIL-HDBK-1025/4 [11]

4.9.1 General

In the United States military handbooks are available for designing structures. One of these handbooks is written by the NAVFAC, Naval Facilities Engineering Command, that uses its knowledge for the US Navy. The NAVFAC has written a lot of publications. For this study the publications for Waterfront Operational Facilities are important, especially the handbook for seawalls, bulkheads and quay walls which treats several types of quay wall structures: bulkheads, containing sheet pile walls; quay walls, containing gravity structures; and seaheads, with breakwaters and embankments.

4.9.2 Safety approach

There is no safety approach in this handbook, except some factors that can be applied on loads. It gives only a general approach of types of structure and why they would be chosen for a certain local situation. The main reason for the absence of a clear safety approach is that the handbook is written in 1988.

4.10 Engineer Manual 1110-2-2504, Design of Sheet Pile Walls [12]

4.10.1 General

Besides military handbooks in the United States, there are also engineering manuals written by the United States Army Corps of Engineers. These are also military guidelines. The manual which has the most agreement with quay walls with relieving platform is EM 1110-2-2504, which treats the design of sheet pile walls. The quay wall is not described in this manual, but it contains a lot of information about sheet pile walls.
A lot of attention is paid to investigation of the soil parameters, with associating soil pressure. An analysis of stability, forces in the cross-section and construction recommendations are given. This engineering manual for sheet pile walls refers to several other engineering manuals.

4.10.2 Safety approach

No very detailed safety approach is given, therefore it refers to other engineering manuals. Some assumptions are made for test results of strength parameters in the soil. It is mentioned that the design soil parameters must be decreased for the calculation of the embedment depth. Three load situations are given: normal loads, not-normal loads and extreme loads.

4.11 Literature of importance for the comparative analysis

As a result of the study for literature, calculations must be carried out to give a detailed investigation of the differences and comparisons between the guidelines. An overview of the contents of the guidelines is given in Annex B. However, not all literature is relevant:

- Eurocode 7: Geotechnical Design, Part 1: General Rules
  → *Has a semi-probabilistic approach, but is not yet finished*
- Calibration study National Annex for Eurocode 7
  → *One of many studies done for the national annex*
- Code of Practice for Maritime Structures, BS 6349 Parts 1 & 2
  → *Nearly no safety approach*
- Actions in the design of Maritime and Harbor Works, ROM 0.2-90
  → *Only for loads, no further safety approach*
- Technical Standards for Ports & Harbor Facilities in Japan
  → *No clear safety approach given*
- Military Handbook for Seawalls, Bulkheads, Quay walls – 1025/4
  → *No safety approach*
- Engineer Manual 1110-2-2504, Design of Sheet Pile Walls
  → *Little safety approach*

The literature that is relevant for further investigation is summarized below:

- Handbook Quay Walls, CUR 211
  → *Semi-probabilistic approach, specially made for quay walls*
- Handbook Sheet pile structures, CUR 166
  → *Semi-probabilistic approach, well known in the Netherlands*
- Waterfront structures, EAU 2004
  → *Well known guideline in Europe, semi-probabilistic approach*

With these guidelines the comparative design calculations will be made.

5 Probabilistic design philosophy for quay walls

The latest probabilistic design philosophy is described in CUR 190 [13]. An overview is given in the next paragraph.

5.1 General

For the design of a structure some safety demands must be considered. These aspects are usually included in design rules, which are drawn up from failure mechanisms and describe a limit state. A **failure mechanism** gives a description of the way a structure fails and can not fulfill his function anymore. This can be due to a permanent or temporary situation. If the structure **collapse**, this can be seen as permanent failure. The state in which a structure does not yet fails, is called a **limit state**. In general two limit states can be determined:

ULS Ultimate Limit State (during an extreme situation)
SLS Serviceability Limit State (during service time)

When the ULS is exceeded the structure will collapse and looses his function. In quay wall structures there are lots of failure mechanisms that can introduce such a collapse. Also deformations can introduce a collapse. The mechanisms depend on the type of structure and what is known about the safety of this structure.

If the SLS is exceeded the deformations are too large. The structure will not collapse, but can not fulfill his function any more during service time. This is also a limiting situation. The safety measures for preventing from this serviceability limit state are usually not as high as for the ultimate limit state.

In all limit states there are always two contrary factors working on the structure. The solicitation and the resistance. If the solicitation is higher than the resistance failure will occur. So the resistance must be larger than the solicitation.

R (resistance) > S (solicitation)

This can be described in a mathematical function, called the **reliability function (Z)**, where failure is described as $Z \leq 0$. Each failure mechanism can be expressed in such a reliability function.

Z = R (resistance) - S (solicitation) ≤ 0

The **probability of failure** due to a mechanism is defined as the probability that the solicitation is higher than the resistance:

$$P_f = P(Z \leq 0) = P(S \geq R)$$

The **reliability** is defined as the probability that failure will not occur:

$$P(Z>0) = 1 - P_f$$

A problem is that the solicitation and resistance are no fixed parameters. They are distributed over a certain range. The wind force on a structure for example, is not always the same. This is also the problem for soil parameters, which can be different for each location. If a certain safety against failure will be guaranteed, the probability that the solicitation exceeds the resistance must be check.

In general a principle is taken into account that the solicitation may exceed in 5% of the cases and that the resistance may underspend in 5% of the cases. These 5%-characteristic value of the mean is called the **representative value**. To reduce the probability of failure, a safety factor can be applied on this representative value. This will reduce the resistance and will increase the solicitation. This value is called the **design value**.

$$R_d \geq S_d \text{ , } R_d = R_{rep} / \gamma_R \text{ en } S_d = S_{rep} * \gamma_R \quad \text{(figure 5-1)}$$

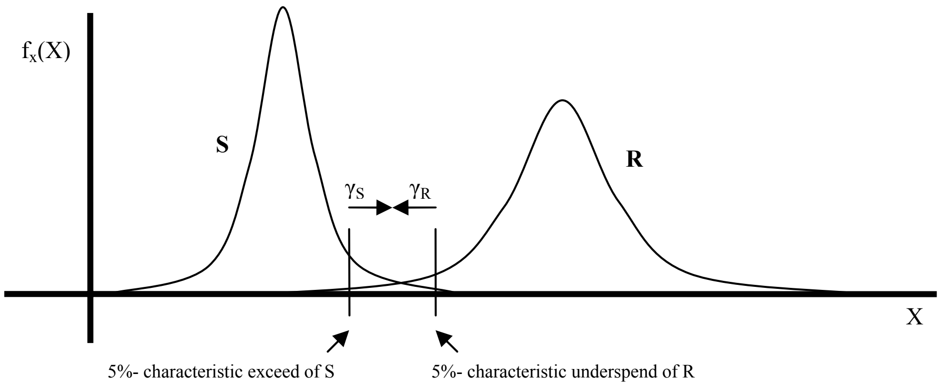


Figure 5-1: The 5%-characteristic values for R and S, with safety factors

There are three levels to determine the safety:

Level III: Calculation of the probability of failure with reliability functions of all solicitation and resistance parameters. The reliability is direct related to the probability of failure.

Level II: The same as Level III, but with a linearization of the reliability function (Z) for the design point. This methods reduces the reliability functions to a normal distribution for all resistance and solicitation parameters. Then also the reliability function (Z) will be a normal distribution and reliability index (β) can be found.

Level I: At this level probabilities of failure will be calculated. There must be enough space between the representative values of solicitation and resistance, gained by so called partial safety factors.

The Level I calculation can be connected to a reliability calculation, with a Level II calculation. The connection will be made in a point where the highest probability of failure will be found for resistance and solicitation. The distance to this point, which is a linearization of the reliability function (Z), is defined as the **reliability index $\beta_z(=\mu_z/\sigma_z)$**. Also an **influence coefficient $\alpha_{R/S}(=\sigma_{R/S}/\sigma_z)$** can be defined, which gives the distribution between the resistance/solicitation and the reliability function. From the design point the partial safety factors can be determined.

$$R_d \geq S_d \text{ , } R_d = R_{rep} / \gamma_R \text{ and } S_d = S_{rep} * \gamma_R$$

$R_{rep} = \mu_R + k_R*\sigma_R$, $k_R = -1{,}64$ (gives the 5%-underspend in a normal distribution)
$S_{rep} = \mu_S + k_S*\sigma_S$, $k_R = 1{,}64$ (gives the 5%-exceed in a normal distribution)

$$\gamma_R = R_{rep}/R_d = (\mu_R + k_R*\sigma_R) / (\mu_R + \alpha_R* \beta_z*\sigma_R)$$
$$\gamma_S = S_d/S_{rep} = (\mu_S + \alpha_S* \beta_z*\sigma_S) / (\mu_S + k_S*\sigma_S)$$

Also an important factor in determining the safety factors is the level (= in this case the height) of safety. Structures have no infinite life time, most of them will be constructed for a structural life time of 50 years. After this life time the structure may loose his function. A certain safety level must be guaranteed, which will be outlined in the β-value. For a higher β-value the admissible probability of failure is lower. So the admissible probability of failure for a single failure mechanism is smaller than the probability of failure for the total structure. A longer life time will result in a higher β-value, so a lower admissible probability of failure. As mentioned before, if the β-value is known the partial safety factors can be determined.

With these probabilistic methods, the probabilities of failure can be determined for different failure mechanisms. However, a structure can have more than one mechanism. These can be outlined in a fault tree. In this tree one main mechanism is described, with several sub-mechanisms. For each sub-mechanism a reliability index can be determined. If this is known for all sub-mechanisms, the probability of failure for the main mechanism can be calculated.

For each of the mechanisms the analysis of partial safety factors can be carried out. This is a lot of work and gives difficult designs. Sometimes it is chosen to determine the factors for one mechanism, which are also applicable and satisfy the other mechanisms.

5.2 Safety for quay wall structures

5.2.1 General

The quay wall is a structure that separates the water from the soil. This is also the most important safety problem in such a structure. Important aspects in the design of a quay wall are the retaining of soil and water so that the wall will not collapse. Especially the soil makes the design difficult, because the soil works both as solicitation and resistance, and the soil properties are distributed over a large range. The guidelines use different approaches to apply the safety on soil. The influence of safety factors on the soil will be overviewed in the Annex F.

A quay wall is a large structure, which cost a lot of money. For economic reasons it is important to optimize the design. With the probabilistic approach described before, the structure will be optimized for sufficient safety, but with not more material than necessary. A lot of experience with quay wall construction is available in older design guidelines. However, in most new guidelines it is tried to apply the probabilistic approach in combination with experience from previous guidelines.

In the two Dutch guidelines, Handbook Quay Walls and CUR 166, a fault tree is used to present the failure mechanisms for the limit states. In these trees the failure mechanisms are treated which can occur in a specific structure. If a tree is available for more than 1 type of structure the failure mechanisms are more general. This is the case for CUR 166 (figure 5-2), which is originally made for a sheet pile structure. The safety factors for the sheet pile calculation of the CUR 166 are based on the failure mechanism "Passive earth pressure insufficient", made for a design with simply supported sheet pile wall. This sheet pile structure can be applied in a building pit as well as in a quay walls. The quay walls in CUR 166 are based on a safety level III, which is the highest safety level in CUR 166 with a β-value of 4,2.

Failure mechanism	Reliability index (β)		Admissible probability of failure	
	Handbook Quay Walls	**CUR 166**	**Handbook Quay Walls**	**CUR 166**
Quay Walls structure fails	**3,4**		**$3{,}37 \cdot 10^{-4}$**	
Sheet pile structure fails		**4,2**		**$1{,}34 \cdot 10^{-5}$**
Failure of sheet pile	**3,707**	**4,39**	**$1{,}05 \cdot 10^{-4}$**	**$5{,}57 \cdot 10^{-5}$**
Failure of sheet pile profile	3,872	4,48	$5{,}39 \cdot 10^{-5}$	$3{,}71 \cdot 10^{-6}$
Passive earth pressure insufficient	4,396	4,48	$5{,}39 \cdot 10^{-6}$	$3{,}71 \cdot 10^{-6}$
Failure of tensile element/ support	**3,828**	**4,44**	**$6{,}47 \cdot 10^{-5}$**	**$4{,}45 \cdot 10^{-6}$**
Insufficient total stability	**4,247**	**4,48**	**$1{,}08 \cdot 10^{-5}$**	**$3{,}71 \cdot 10^{-6}$**
Groundwater flow too large	**4,247**	**4,48**	**$1{,}08 \cdot 10^{-5}$**	**$3{,}71 \cdot 10^{-6}$**

Table 5-1: Reliability indices and probabilities of failure for failure mechanisms from CUR 166 and Handbook Quay Walls

The Handbook Quay Walls uses a similar approach as CUR 166. The fault tree for Handbook Quay Walls (figure 5-3) is based on a quay wall structure with combined wall, superstructure and tension pile. This structure is much more specific than the structures used in CUR 166.

The superstructure distributes vertical loads to the top of the combined wall. Sufficiently vertical bearing capacity must be available in the soil. The safety factors for sheet pile calculation in Handbook Quay Walls are therefore based on the failure mechanism of "Failure of sheet pile profile" and not on the "Passive earth pressure insufficient", because this usually will be satisfied due to the large toe level for the vertical bearing capacity. The quay walls in Handbook Quay Walls are based on a safety level 2, with a β-value of 3,4.

The German guideline EAU 2004, doesn't give a fault tree, but splits the ultimate limit state in three sub-limit states. However, it is tried to overview the limit states in a fault tree (figure 5-4). Per structural element it is describes to which limit state it belongs and which safety factor there must be used. Because the EAU 2004 can be used for several types of structures, some reduction can be applied to fit the safety factors for a certain type of structure. No description is given for which failure mechanism the safety factors are made and on what kind of safety level they are based.

Most of the safety factors are also based on a certain life time of the structure (in most cases 50 years), a safety level that must be maintained and depends on the limit state.

In general a fault tree for quay wall structures contains 4 main failure mechanisms:

- Failure of sheet pile wall due to a yielding profile or insufficient passive earth pressure
- Failure of ground due to groundwater flow
- Failure of ground due to insufficient total stability of the structure in the ground
- Failure of the tensile/anchor element

For specific structures additional elements must be checked. This is the case for the quay wall with superstructure.

5.2.2 Fault tree for CUR 166, Sheet pile structures

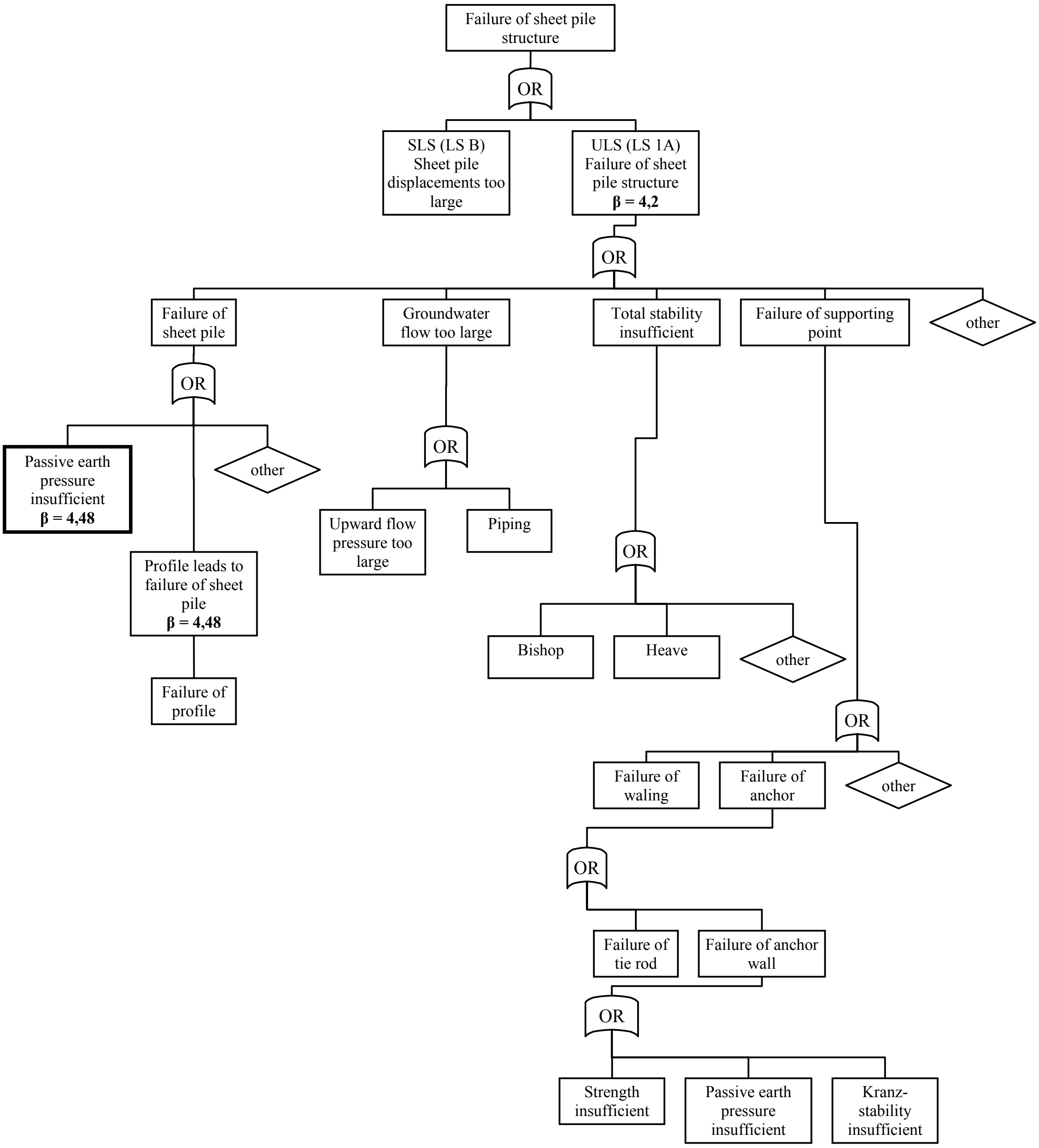


Figure 5-2: Fault tree for CUR 166, Sheet pile structures; the safety factors for the sheet pile calculation are based on the failure mechanism "Passive earth pressure insufficient"

5.2.3 Fault tree for CUR 211, Handbook Quay Walls

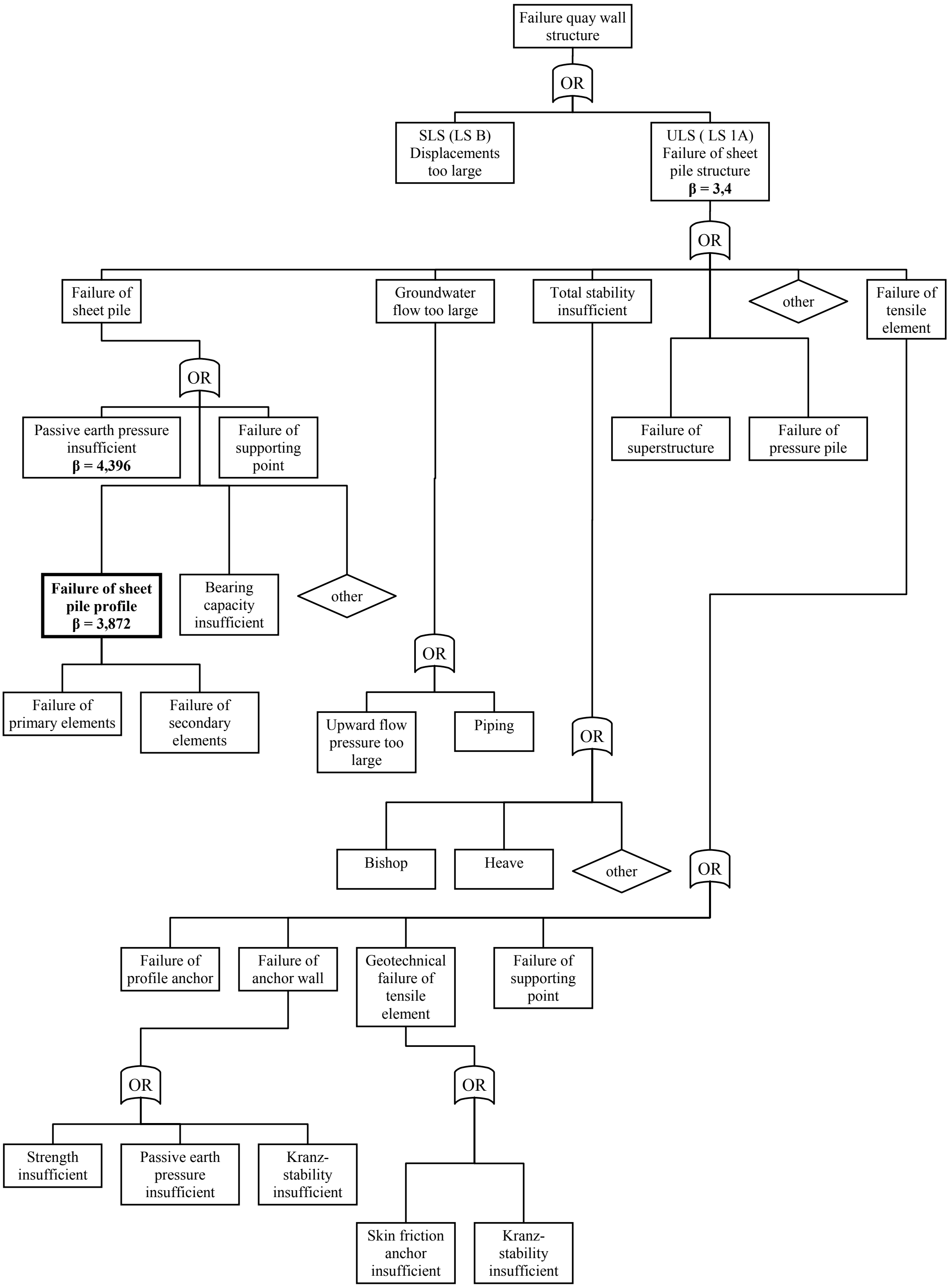


Figure 5-3: Fault tree for CUR 211, Handbook Quay Walls; the safety factors are based on the failure mechanism "Failure of sheet pile profile"

5.2.4 Fault tree for EAU 2004, Waterfront structures

From the comparative analysis these failure mechanism are found for the EAU 2004.

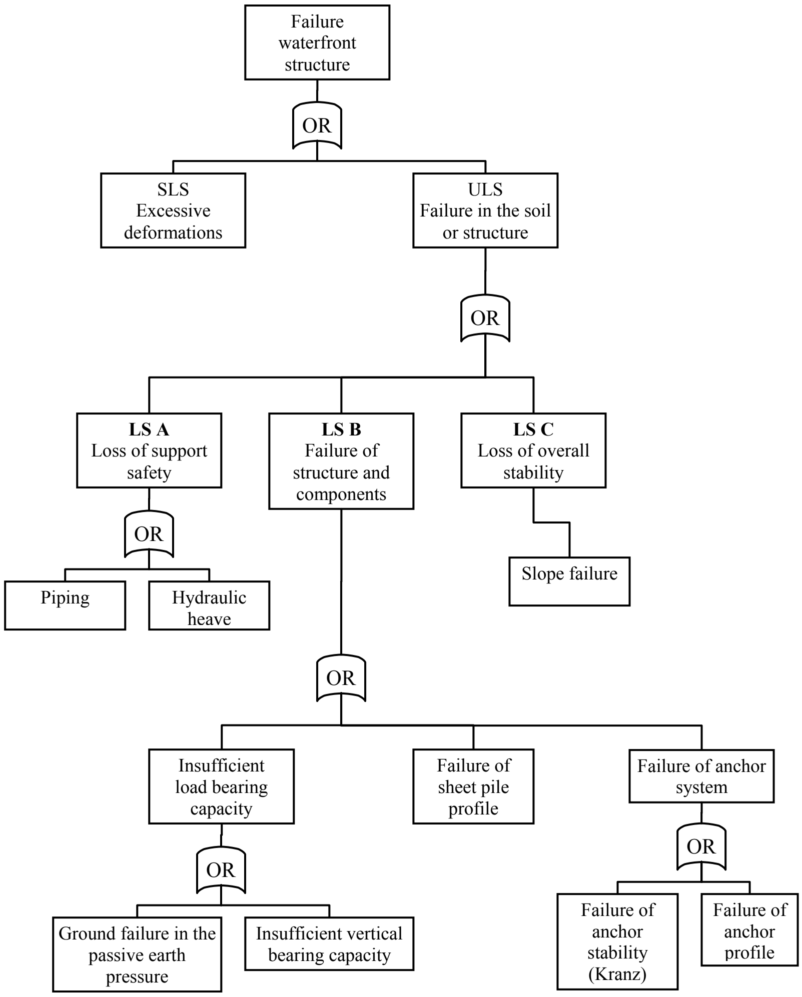


Figure 5-4: Fault tree for EAU 2004, Waterfront structures; no information given about the failure mechanism of the safety factors

6 Determination of characteristic values

6.1 General

As mentioned before, three guidelines will be used for designing the quay wall structure. The first step in the design process of each guideline is to determine the characteristic design parameters. In the Netherlands representative values are the same as a 5%-exceed/underspend characteristic value, but this is not always the case in other international guidelines. For example, the characteristic values of the soil parameters in the EAU are based on cautious mean values.
Usually some characteristic values for a design are given in the terms of reference [20], some other values must be interpreted from laboratory investigations and there are also parameters determined from plausible assumptions.
For a good comparative study all calculations should be done under the same conditions, this means on the same design basis. In this chapter characteristic/representative values will be discussed and in Annex C the characteristic parameters are calculated for a reference project: the Euromax quay wall in the Yangtze Harbor (Rotterdam, the Netherlands). These values will be used in the calculations carried out in Annex D and E.

6.2 Geometry parameters

6.2.1 Design depth of the bottom

For designing a quay wall the design depth and the contract depth are important. For the calculation only the design depth is considered. In the contract depth also dredging works, a bottom protection and scouring due to propeller currents are included. This is not described in CUR 166, so not possible to compare.

6.3 Water levels

The three design guidelines use each another approach of estimating the normative water level situations.

6.3.1 General data

For usage of the guidelines, some definitions of water levels must be overviewed. Most of these definitions are described in Handbook Quay Walls.

- <u>MLW</u>: Mean water level of all low waters.

- <u>MHW</u>: Mean water level of all high waters.

- <u>MLWS:</u> Mean level of the occurring minimum water level twice a day of the tide, when sun and moon are in phase. This water level is not given, but is the level between MLW and LLWS. It is **assumed** that the MLWS is the **average of the MLW and the LLWS**.

- <u>LLWS</u> (≈ OLW): The mean value of the lowest low waters spring measured over a period of 5 years. For harbors in an estuary a corresponding value is given where also the influence of the river is taken into account. The so called corresponding low river water level. This is the case for the Europa Harbor which is reference for the Yangtze Harbor.

- <u>LLW</u>: This is the lowest low water that is measured.

- <u>Characteristic 5%</u>-underspend water level: This is the water level that will underspend for 5% of all mean low waters that are measured.

6.3.2 Handbook Quay Walls

In Handbook Quay Walls a special probabilistic water level analysis is described. This analysis is based on the probability distribution function of high and low waters. From this analysis mean values and standard deviations can be determined. From the free water analysis the groundwater level can be estimated. A time shift of 2 hours can be assumed, which is done in an example for the Maasvlakte given in Handbook Quay Walls.

The probability distribution function for high and low free water must be approximated from measurements done over a certain period (in this case 4 years).

For high water $F = 1 - \exp\left(-\exp\left(\frac{x-a}{b}\right)\right)$

For low water $F = \exp\left(-\exp\left(\frac{x-a}{b}\right)\right)$

From the approximation a mean and standard deviation can be calculated with a and b values.

Mean $\mu = a - 0{,}5572 \cdot b$

Standard deviation $\sigma = \frac{b \cdot \pi}{\sqrt{6}}$

The low free water must be recalculated to a period of 50 years in stead of the period of the measurements (4 years).

$$\mu_{50} = a_{50} - 0{,}5572 \cdot b$$

$$a_{50} = a - \Delta a$$

$$\Delta a = b \cdot \ln(50 / t_{measured})$$

The design free water levels will be:

$$h_{50,LW} = \mu_{50,LW} - \gamma_{sf} \cdot \sigma_{LW}$$

$$h_{HW} = \mu_{HW} + \gamma_{sf} \cdot \sigma_{HW}$$

The high groundwater level can be derived from the free water levels. As mentioned before a shift of 2 hours can be used. On the mean and standard deviation of the groundwater a partial safety factor should be applied.

$$h_{g,HW} = \mu_{HW} + \gamma_{sf} \cdot \sigma_{g,HW}$$

6.3.3 CUR 166

The CUR 166 refers to a water level analysis with measurements over a long time. For the determination of design water levels 2 situations can be considered for locations with tidal difference:

The situation without drainage looks like situation 3a of the EAU 2004, but for the CUR 166 the LLWS is used in stead of the MLWS as used in the EAU.

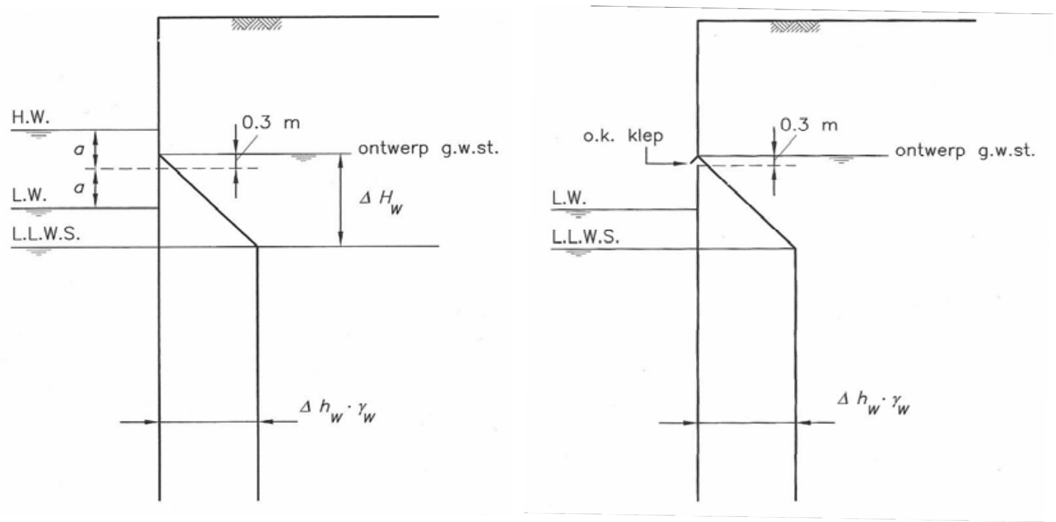


Figure 6-1: Hydrostatic pressure according to CUR 166 for situation (left, 1) without drainage and with (right, 2) drainage system

6.3.4 EAU 2004

For the EAU 2004 (like CUR 166) normative water level situations are described under tidal conditions. These cases correspond with loading cases determined for EAU 2004. The situation for the Europa Harbor looks like situation 3, described in the EAU 2004. This situation describes tidal conditions, divided into four sub-conditions.

tidal area

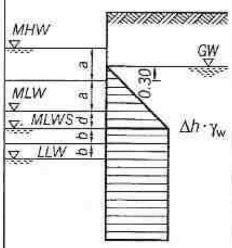
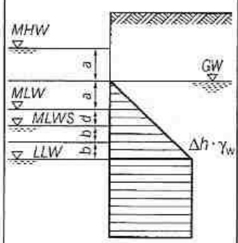
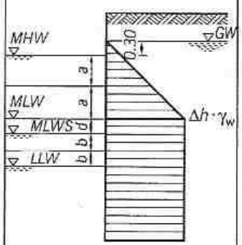
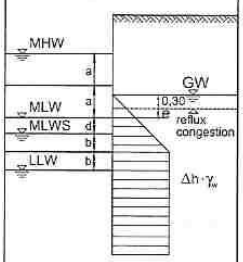
Situation	Figure	Load cases as per R 18		
		1	2	3
3a Major water level fluctuations without drainage – normal case		$\Delta h = a + 0.30$ m $a = \dfrac{\text{MHW} - \text{MLW}}{2}$ $d = \text{MLW} - \text{MLWS}$	–	–
3b Major water level fluctuations without drainage – limit case extreme low water level		–	–	$\Delta h = a + 2b + d$ $a = \dfrac{\text{MHW} - \text{MLW}}{2}$ $b = \dfrac{\text{MHWS} - \text{LLW}}{2}$ $d = \text{MLW} - \text{MLWS}$
3c Major water level fluctuations without drainage limit case falling high water		–	–	$\Delta h = 0.30\ \text{m} + 2a$
3d Major water level fluctuations with drainage		$\Delta h = 1.00\ \text{m} + e$ for outer water level in MLWS	$\Delta h =$ 0.30 m $+ b + d$ $+ e$	–

Figure 6-2: Hydrostatic pressure according to EAU 2004, for several cases with and without drainage

6.3.5 Waves

The quay in the Yangtze Harbor has an east-west direction and is protected against waves coming from sea. The only waves that can reach the quay are developed by wind in the harbor itself and stern waves, due to passing ships. The wind waves are very low and short, because the distance to develop the waves is very short. The waves due to passing ships will also be low, because the speeds of the vessels are very low and therefore the stern waves will be low. This are the reasons why waves will not be taking into account and are **neglected**. This is also done in the terms of reference written for the Euromax quay.

6.3.6 Currents

Currents due to passing ships will be **neglected**. The passing ships have a very low speed and the quay is high enough to withstand stern waves. The currents from propellers will also be neglected. The reason is that a design depth is assumed. In a detailed calculation it would probably be necessary to apply a bottom protection.

6.4 Soil parameters

For the comparative calculations the same soil investigations should be done. In all guidelines a general description is given how the properties must be determined.

Handbook Quay Walls, CUR 211	Sheet pile structures, CUR 166	Waterfront structures, EAU 2004
Gives only a description how to investigate the soil and which tests can be done for a certain case.	Gives a general description how to find the properties from cone penetration tests and boreholes, and some representative values are available in table 3.1 of CUR 166.	Gives a general description how to find the properties form cone penetration tests and boreholes, and some characteristic values (cautious mean empirical values) are available in table 9-1 of EAU 2004.
Refers to CUR 166 and several NEN codes	*Refers to NEN 6740 and other NEN codes*	*Refers to several DIN codes*

Table 6-1: Soil properties described in the guidelines

6.4.1 CUR 166 and Handbook Quay Walls soil properties

Soil properties which can be derived from a cone penetration diagram are overviewed below.

Grondsoort			Representatieve waarde van de grondeigenschappen van het laaggemiddelde						
Hoofd-naam	Bijmengsel	Consis-tentie 1)	γ 2) kN/m³	γ_{sat} kN/m³	q_c 3) 6) MPa	E_{100} 6) MPa	φ' °	c' kPa	c_u kPa
grind	zwak siltig	los	17	19	15	37	32,5	-	-
		matig	18	20	25	75	35,0	-	-
		vast	19 20	21 22	30	90 105	37,5 40,0	-	-
	sterk siltig	los	18	20	10	30	30,0	-	-
		matig	19	21	15	45	32,5	-	-
		vast	20 21	22 22,5	25	75 110	35,0 40,0	-	-
zand	Schoon	los	17	19	5	15	30,0	-	-
		matig	18	20	15	45	32,5	-	-
		vast	19 20	21 22	25	75 110	35,0 40,0	-	-
	zwak siltig kleiig		18 19	20 21	12	35 50	27,0 32,5	-	-
	sterk siltig kleiig		18 19	20 21	8	15 30	25,0 30,0		
leem 4)	zwak zandig	slap	19	19	1	2	27,5 30,0	0	50
		matig	20	20	2	3	27,5 32,5	1	100
		vast	21 23	21 22	3	5 7	27,5 35,0	2,5 3,8	200 300
	sterk zandig	-	19 20	19 20	2	3 5	27,5 35,0	0 1	50 100
klei	schoon	slap	14	14	0,5	1	17,5	0	25
		matig	17	17	1,0	2	17,5	5	50
		vast	19 20	19 20	2,0	4 10	17,5 25,0	13 15	100 200
	zwak zandig	slap	15	15	0,7	1,5	22,5	0	40
		matig	18	18	1,5	3	22,5	5	80
		vast	20 21	20 21	2,5	5 10	22,5 27,5	13 15	120 170
	sterk zandig	-	18 20	18 20	1,0	2 5	27,5 32,5	0 1	0 10
	organisch	slap	13	13	0,2	0,5	15,0	0 1	10
		matig	15 16	15 16	0,5	1,0 2,0	15,0	0 1	25 30
veen	niet voorbelast	slap	10 12	10 12	0,1	0,2 0,5	15,0	1 2,5	10 20
	matig voorbelast	matig	12 13	12 13	0,2	0,5 1,0	15,0	2,5 5,0	20 30
variatiecoëfficiënt			0,05		-	0,25	0,10	0,20	

De tabel geeft van de desbetreffende grondsoort de lage, respectievelijk de hoge representatieve waarde van gemiddelden. Binnen een gebied, gedefinieerd door de rij van het bijmengsel en de kolom van de parameter (een cel), geldt:
als een verhoging van de waarde van een van de grondeigenschappen tot een ongunstiger situatie leidt dan de toepassing van de in de tabel gepresenteerde lagere representatieve waarde, moet de rechter waarde op dezelfde regel zijn gebruikt. Is er rechts geen waarde vermeld dan moet de waarde van de regel eronder worden toegepast. Dit is bijvoorbeeld het geval bij negatieve kleef op een paal waar een hogere waarde van φ', c' en c_u ook een hogere waarde van de negatieve kleef oplevert.

1) los : $0 < R_n < 0{,}33$
matig : $0{,}33 \leq R_n \leq 0{,}67$
vast : $0{,}67 < R_n < 1{,}00$
2) De γ-waarden zijn van toepassing bij een natuurlijk vochtgehalte.
3) De hier gegeven q_c-waarden (conusweerstand) moeten dienen als ingang in de tabel en mogen niet in de berekeningen worden gebruikt.
4) De waarden hebben betrekking op verzadigd leem.
6) Voor grind, zand en in beperkte mate ook voor leem en sterk zandige klei zijn q_c, E_{100} en φ' genormeerd voor een effectieve verticale grondspanning σ'_v van 100 kPa. Om voor de in het terrein gemeten waarden van q_c een juiste ingang in de tabel te verkrijgen, moeten deze waarden zijn geconverteerd naar het niveau van de effectieve verticale grondspanning σ'_v van 100 kPa. In dat kader moet de formule $q_{c;tabel} = q_{c;terrein} \times C_{qc}$ worden gebruikt, waarbij C_{qc} moet zijn ontleend aan $C_{qc} = (100/\sigma'_v)^{0,67}$. Voor de hoek van inwendige wrijving φ' en de cohesie c' geldt dat deze afhankelijk zijn van de consistentie van de grond. Dit betekent dat deze conversie ook nodig is voor φ' en c'.

VOORBEELD In schoon zand op een diepte van 5 m onder water is gemeten $q_{c;terrein} = 9$ MPa en $\sigma'_v = 50$ kPa. Uit de formule voor C_{qc} volgt dan $C_{qc} = 2^{0,67} = 1{,}6$. Volgens de formule voor $q_{c;tabel}$ geldt dan in dit voorbeeld $q_{c;tabel} = 9 \times 1{,}6 = 14{,}4$ MPa. Dit betekent dat $E = 45$ MPa en $\varphi' = 32{,}5°$.
De elasticiteitsmodulus bij belastingsherhalingen $E_{werkelijk}$ mag zijn bepaald uit: $E_{werkelijk} = 3 \times E'_{werkelijk}$

Table 6-2: Unfavorable representative soil properties according to CUR 166 and Handbook Quay Walls

6.4.2 Modulus of sub-grade reaction for CUR 166 and Handbook Quay Walls

If a beam on elastic foundation model is used for calculations, the stiffness of the soil is taken into account by the modulus of sub-grade reaction. This parameter is the schematized soil stiffness. For CUR 166 high and low values of the modulus of sub-grade reaction are given, determined from experience of bottom conditions in the Netherlands. In Handbook Quay Walls, the mean CUR-values of the high and low modulus of sub-grade reactions are used. In the EAU 2004 only the E-value is given for the compressibility, because they use the Blum method, which does not take the soil stiffness into account.

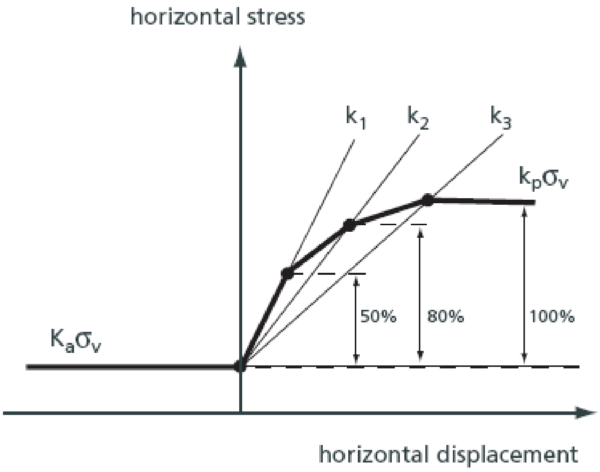


Figure 6-3: Definition of the modulus of sub-grade reaction according to CUR 166

	secans-waarde k_h (kN/m³)					
	$p_0 < p_h < 0{,}5p_{ea;h;p;rep}$ [1]) 1 [2)]	2 [3)]	$0{,}5p_{ea;h;p;rep} \leq p_h \leq 0{,}8p_{ea;h;p;rep}$ 1 [2)]	2 [3)]	$0{,}8p_{ea;h;p;rep} < p_h \leq p_{ea;h;p;rep}$ 1 [2)]	2 [3)]
zand q_c (MPa)						
los 5	12000	27000	6000	13500	3000	6750
matig 15	20000	45000	10000	22500	5000	11250
vast 25	40000	90000	20000	45000	10000	22500
klei c_u (kPa)						
slap 25	2000	4500	800	1800	500	1125
matig 50	4000	9000	2000	4500	800	1800
vast 200	6000	13500	4000	9000	2000	4500
veen c_u (kPa)						
slap 10	1000	2250	500	1125	250	560
matig 30	2000	4500	800	1800	500	1125

Table 6-3: Soil stiffness properties (modulus of sub-grade reaction) according to CUR 166

6.4.3 EAU 2004 Soil properties

Soil properties which can be derived from a cone penetration diagram are overviewed below:

No.	1	2	3	4	5		6		7	8	9	10
1	Soil type	Soil group as per DIN 18 196[1)]	Pene-tration resistance q_c	Strength resp. con-sistency in initial state	Weight density, submerged weight density γ_k	γ'_k	Compressibility[2)] Initial loading[3)] $E_S = \nu_e \sigma_{at}(\sigma/\sigma_{at})_e^w$ ν_e	w_e	Shear parameters of the drained soil φ'_k	c'_k	Shear para-meters of the un-drained soil $c_{u,k}$	Perme-ability factor k_k
2			MN/m²		kN/m³	kN/m³			degrees	kN/m²	kN/m²	m/s
3	Gravel, uniform	GE U[4)] < 6	< 7,5 7,5–15 > 15	low medium high	16,0 17,0 18,0	8,5 9,5 10,5	400 900	0,6 0,4	30,0–32,5 32,5–37,5 35,0–40,0			$2 \cdot 10^{-1}$ to $1 \cdot 10^{-2}$
4	Gravel, non-uniform or intermittent	GW, GI 6 ≤ U[4)] ≤ 15	< 7,5 7,5–15 > 15	low medium high	16,5 18,0 19,5	9,0 10,5 12,0	400 1100	0,7 0,5	30,0–32,5 32,5–37,5 35,0–40,0			$1 \cdot 10^{-2}$ to $1 \cdot 10^{-6}$
5	Gravel, non-uniform or intermittent	GW, GI U[4)] > 15	< 7,5 7,5–15 > 15	low medium high	17,0 19,0 21,0	9,5 11,5 13,5	400 1200	0,7 0,5	30,0–32,5 32,5–37,5 35,0–40,0			$1 \cdot 10^{-2}$ to $1 \cdot 10^{-6}$
6	Sandy gravel, d < 0,006 mm < 15 %	GU, GT	< 7,5 7,5–15 > 15	low medium high	17,0 19,0 21,0	9,5 11,5 13,5	400 800 1200	0,7 0,6 0,5	30,0–32,5 32,5–37,5 35,0–40,0			$1 \cdot 10^{-5}$ to $1 \cdot 10^{-6}$
7	Gravel-sand-fine grain d < 0,06 mm > 15 %	$G\bar{U}$, $G\bar{T}$	< 7,5 7,5–15 > 15	low medium high	16,5 18,0 19,5	9,0 10,5 12,0	150 275 400	0,9 0,8 0,7	30,0–32,5 32,5–37,5 35,0–40,0			$1 \cdot 10^{-7}$ to $1 \cdot 10^{-11}$
8	Sand, uniform coarse sand	SE U[4)] < 6	< 7,5 7,5–15 > 15	low medium high	16,0 17,0 18,0	8,5 9,5 10,5	250 475 700	0,75 0,60 0,55	30,0–32,5 32,5–37,5 35,0–40,0			$5 \cdot 10^{-3}$ to $1 \cdot 10^{-4}$
9	Sand, uniform, fine sand	SE U[4)] < 6	< 7,5 7,5–15 > 15	low medium high	16,0 17,0 18,0	8,5 9,5 10,5	150 225 300	0,75 0,65 0,60	30,0–32,5 32,5–37,5 35,0–40,0			$1 \cdot 10^{-4}$ to $2 \cdot 10^{-5}$
10	Sand, non uniform or intermittent	SW, SI 6 ≤ U[4)] ≤ 15	< 7,5 7,5–15 > 15	low medium high	16,5 18,0 19,5	9,0 10,5 12,0	200 400 600	0,70 0,60 0,55	30,0–32,5 32,5–37,5 35,0–40,0			$5 \cdot 10^{-4}$ to $2 \cdot 10^{-5}$
11	Sand, non uniform or intermittent	SW, SI U[4)] > 15	< 7,5 7,5–15 > 15	low medium high	17,0 19,0 21,0	9,5 11,5 13,5	200 400 600	0,70 0,60 0,55	30,0–32,5 32,5–37,5 35,0–40,0			$1 \cdot 10^{-4}$ to $1 \cdot 10^{-5}$
12	Sand, d < 0,06 mm < 15 %	SU, ST	< 7,5 7,5–15 > 15	low medium high	16,0 17,0 18,0	8,5 9,5 10,5	150 350 500	0,80 0,70 0,65	30,0–32,5 32,5–37,5 35,0–40,0			$2 \cdot 10^{-5}$ to $5 \cdot 10^{-7}$
13	Sand, d < 0,06 mm > 15 %	$S\bar{U}$, $S\bar{T}$	< 7,5 7,5–15 > 15	low medium high	16,5 18,0 19,5	9,0 10,5 12,0	50 250	0,9 0,75	30,0–32,5 32,5–37,5 35,0–40,0			$2 \cdot 10^{-6}$ to $1 \cdot 10^{-9}$
14	Silt, anorganic cohesive soils with low plasticity (w_L < 35 %)	UL		safe stift semi-fine	17,5 18,5 19,5	9,0 10,0 11,0	40 110	0,80 0,60	27,5–32,5	0 2–5 5–10	5–60 20–150 50–300	$1 \cdot 10^{-5}$ to $1 \cdot 10^{-7}$
15	Silt, anorganic cohesive soils with medium plasticity (50 % > w_L > 35 %)	UM		safe stift semi-fine	16,5 18,0 19,5	8,5 9,5 10,5	30 70	0,90 0,70	25,0–30,0	0 5–10 10–15	5–60 20–150 50–300	$2 \cdot 10^{-6}$ to $1 \cdot 10^{-9}$
16	Clay, anorganic cohesive soils with low plasticity (w_L < 35 %)	TL		safe stift semi-fine	19,0 20,0 21,0	9,0 10,0 11,0	20 50	1,0 0,90	25,0–30,0	0 5–10 10–15	5–60 20–150 50–300	$1 \cdot 10^{-7}$ to $2 \cdot 10^{-9}$
17	Clay, anorganic cohesive soils with medium plasticity (50 % > w_L > 35 %)	TM		safe stift semi-fine	18,5 19,5 20,5	8,5 9,5 10,5	10 30	1,0 0,95	22,5–27,5	5–10 10–15 15–20	5–60 20–150 50–300	$5 \cdot 10^{-8}$ to $1 \cdot 10^{-10}$
18	Clay, anorganic cohesive soils with high plasticity (w_L > 50 %)	TA		safe stift semi-fine	17,5 18,5 19,5	7,5 8,5 9,5	6 20	1,0 1,0	20,0–25,0	5–15 10–20 15–25	5–60 20–150 50–300	$1 \cdot 10^{-9}$ to $1 \cdot 10^{-11}$
19	Silt or clay, organic	OU und OT		very soft soft stift	14,0 15,5 17,0	4,0 5,5 7,0	5 20	1,00 0,85	17,5–22,5	0 2–5 5–10	2 – < 15 5–60 20–150	$1 \cdot 10^{-9}$ to $1 \cdot 10^{-11}$
20	Peat[5)]	HN, HZ		very soft soft stift semi firm	10,5 11,0 12,0 13,0	0,5 1,0 2,0 3,0	[5)]	[5)]	[5)]	[5)]	[5)]	$1 \cdot 10^{-5}$ to $1 \cdot 10^{-8}$
21	Mud[6)], Faulschlamm	F		very soft soft	12,5 16,0	2,5 6,0	4 15	1,0 0,9	[6)]	0	< 6 6–60	$1 \cdot 10^{-7}$ $1 \cdot 10^{-9}$

Table 6-4: Cautious mean (characteristic) values of the soil properties according to EAU 2004

6.5 Wall parameters

The properties of the sheet pile wall are mainly kept the same for a good comparative analysis. The change of profiles can be investigated for specific circumstances. There are a few profiles from which sheet pile wall parameters can be chosen: Hoesch, Arcelor and Larsen. Tables are available with dimensions, the moment of inertia and the section modulus. Sometimes these moments must be reduced due to the type of profile and local circumstances.

6.5.1 Sheet pile classes according Eurocode 3

In Eurocode 3 sheet pile classes are defined, for U-shaped and Z-shaped profiles, depending on the limiting yielding stress, the flange thickness and the flange width.

Class	Z-shaped profile	U-shaped profile
1	Same as for class 2 with control of rotation	Same as for class 2 with control of rotation
2	$(b/t_f) / \varepsilon \leq 45$	$(b/t_f) / \varepsilon \leq 37$
3	$(b/t_f) / \varepsilon \leq 66$	$(b/t_f) / \varepsilon \leq 49$
4	All profiles not sufficient for classes 1, 2 and 3	
b = flange width, t_f = flange thickness, $\varepsilon = \sqrt{235/f_y}$		

Table 6-5: Sheet pile classes according CUR 166 (and Eurocode 3)

6.5.2 Corrosion of the sheet pile wall

Corrosion can reduce the cross-section of the wall and therefore reduce the strength of the sheet piles. Per guideline general values are given for the loss of material per year (corrosion speed) of the sheet pile wall. This differs per (water level)zone. The quay wall is surrounded by salt water, which increases the corrosion speed.

The highest corrosion speeds takes place in the splash zone. The highest bending stresses occur around the field moment or at the place of the anchor support, so this are the places where a high corrosion speed occurs. The cross-section of the wall must be increased with a certain factor to take into account the aspect of corrosion.

CUR 166

In this guideline values are given from Eurocode, Handbook Quay Walls and ROBK. The last one is given below.

Zone [mm/year]	Fresh water	Salt water
Atmospheric zone	0,012	0,050
Splash zone	0,012	0,120
Underwater zone	0,012	0,026
In bottom zone	0,012	0,014

Table 6-6: Corrosion of sheet pile wall in mm/year, per exposed side, according to CUR 166 (based on ROBK)

The highest corrosion speed occurs in the splash zone for salt water. For the cases of the comparative analysis this will be a corrosion speed of 0,12 mm year, which results in **6 mm corrosion over 50 years**.

<u>Handbook Quay Walls</u>

This handbook also refers to EAU 1996 and BS 6349. The values determined for Handbook Quay Walls are combined from the EAU and British Standard.

Zone [mm/year]	from … to …	
Atmospheric zone	0,050	0,100
Splash zone	0,150	0,400
Tidal zone	0,100	0,250
Low water zone	0,100	0,250
Under water zone	0,050	0,200
Bottom zone	0,020	0,050

Table 6-7: Corrosion of sheet pile wall in mm/year for tidal zones, according Handbook Quay Walls

Highest corrosion speed in splash zone. For the cases of the comparative analysis this will be a maximum corrosion speed of 0,4 mm year, which results in maximum **20 mm corrosion over 50 years**.

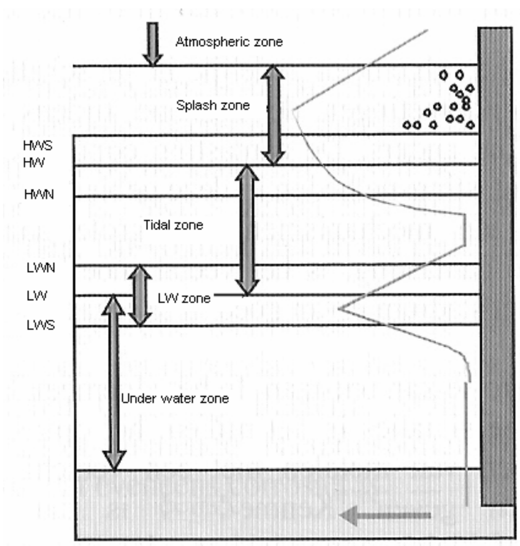


Figure 6-4: Corrosion zones, the arrow gives the height of the corrosion speed (Handbook Quay Walls)

<u>EAU 2004</u>

The values given in EAU 2004 are based on measurements done in the North Sea and Baltic Sea. A graph is presented for the measurements results.

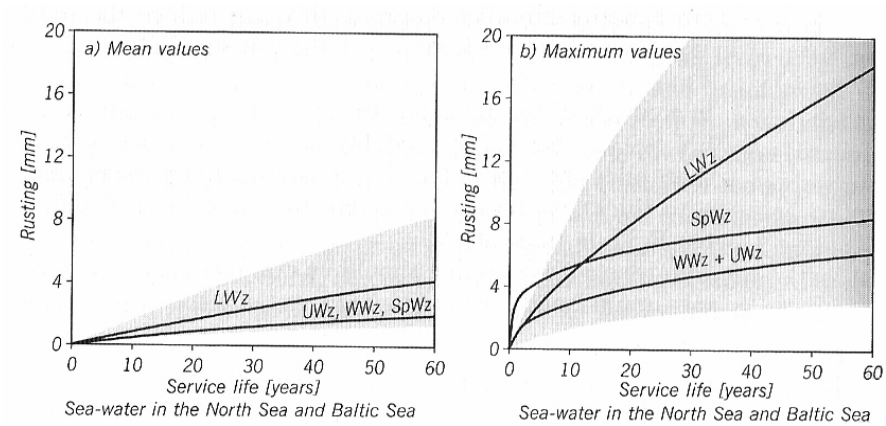


Figure 6-5: Maximum and mean values for several corrosion zones in the Baltic and North Sea

Zone, North Sea and Baltic Sea [mm/year]	Mean	Maximum
Atmospheric zone	0,01	0,01
Splash zone	0,03	0,15
Intertidal zone	0,03	0,11
Low water zone	0,07	0,30
Permanent immersion zone	0,03	0,11
Bottom zone	0,01	0,01

Table 6-8: Corrosion of sheet pile wall in mm/year for the North Sea, according EAU 2004

Highest corrosion speeds occurs in low water zone. For the cases of the comparative analysis this will be a maximum corrosion speed of 0,3 mm year, which results in maximum **15 mm over 50 years**.

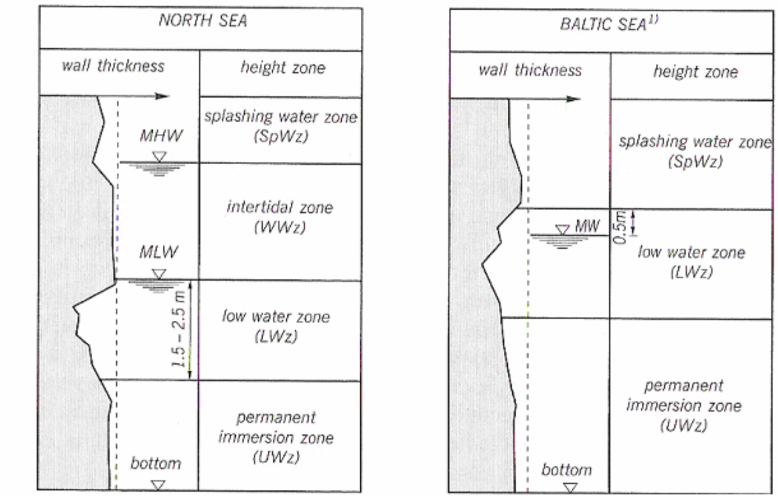


Figure 6-6: Corrosion zones according EAU 2004

In general

Handbook Quay Walls gives the highest corrosion speed of **20 mm** over 50 years in the splash zone. The EAU 2004 gives a maximum value in the **same range**, but at another place of the wall (low water zone). The CUR 166 gives the lowest corrosion over 50 years, somewhere around **6 mm**, but it is not clear if this is a mean or maximum value. ***From research done by the Public Works of Rotterdam it is clear that even the Handbook Quay Walls gives corrosion speeds that are too low.***

6.5.3 Oblique bending, according CUR 166

The CUR 166 takes oblique into account. The Handbook Quay Walls refers to the CUR 166 for this topic. Due to oblique bending of the sheet pile wall, the section modulus (bending) and the moment of inertia (deflection) will be reduced.

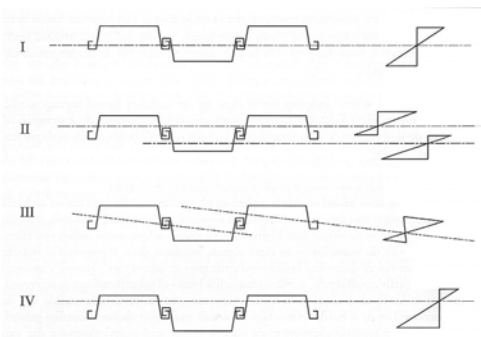


Figure 6-7: Examples of oblique bending, due to interlocks in the neutral line (CUR 166)

This is only necessary when an U-profile will be applied, because oblique bending occurs when the interlocks are in the neutral line. For other profiles and triple combined U-profiles, the reduction factor is 1 (no reduction).

$$W_{corr} = \beta_B \cdot W_{el}$$
$$I_{corr} = \beta_D \cdot I_{el}$$
, β-factors for the reduction due to oblique bending

The maximum reduction factor for oblique bending in a sheet pile wall with anchor is ≥ 0,55. This factor can be increased due to the influence of the soil and the installation given by 6 factors.

$$\beta_B = \beta_{B;0} + \sum_{i=1}^{6} \Delta\beta_{B;i} \leq 0$$

$$\beta_D = \beta_{D;0} + \sum_{i=1}^{6} \Delta\beta_{D;i} \leq 0$$

For a single sheet pile profile the **0,55-factor** can be increased, for example with:

- **0,10** moderate strength of the cohesive and non-cohesive layers
- **0,05** resistance in the direction perpendicular due to the anchor which is installed
- **0,05** resistance in longitudinal direction due to the concrete beam
- **0,05** resistance in vertical direction due to the concrete beam on top of the wall
- **0,10** installation of the sheet pile without special measures
- **0,10** existence of a sand layer (almost 5 m thick) above groundwater level

6.5.4 Sheet pile profiles

In general there are U-shaped and Z-shaped profiles for this case. A choice must made between these two types. A disadvantage of the U-shaped profiles is that they can have bending moments in the longitudinal direction, called oblique bending. This is not the case for Z-shaped profiles, but they have the disadvantage that the highest bending stresses are situated at the places of the interlocks and that the interlocks will be pushed open. This is not the case for U-shaped profiles, the interlocks will be push together when installed correctly.

A combined wall consists of two profiles, for example a tubular (primary) profile and an infill (secondary) profile. The tubular profile is for the vertical and horizontal bearing capacity and the infill profile is for the resistance against hydraulic failure and the bearing of soil pressure. The soil pressure on the infill piles can sometimes be reduced due to arching. The horizontal loads will immediately be transferred to the tubular piles, the only loads working on the infill piles in that case is the water pressure. This is due to the large difference in stiffness between the tubes and the infill piles.

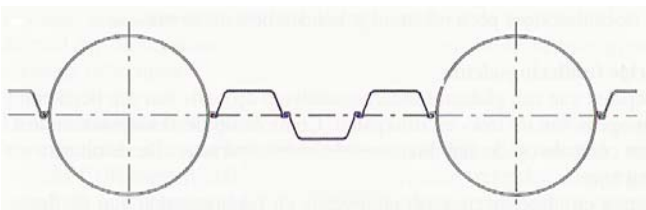


Figure 6-8: Profile with combined wall and triple U-shaped infill profile

The tube is in most cases longer than the infill pile, because the toe level of the infill pile is based on the hydraulic resistance. In the top part of the combined profile the stiffness can be delivered by both profiles and in the bottom part only by the tube.
Tubes can be delivered in the sizes that are needed, no real standards are described. This is not the case for the infill piles, which can be chosen from standard profiles like Hoesch, Arcelor (see Annex C) and Larsen. The advantage is that less tubular profiles are needed.

The section modulus of the tube is:

$$I_{combi} = \frac{I_{tube} + I_{n;pile}}{L}$$

, with D (diameter) of the tube and L (combined length) of the wall

$$I_{tube} = \frac{\pi}{64} \cdot (D^4_{outside} - D^4_{inside})$$

6.6 Loads working on the quay wall structure

Loads working on the structure are coming from permanent actions, as water and soil pressure. They can also be due to extern variable loads working horizontal and vertical at the quay wall. In the guidelines there is general data given about the variable loads. The data of Euromax, given in the terms of reference, will be discussed compared to the data from the guidelines in Annex C.

6.6.1 Terrain loads

Terrain loads are working on the quay platform, due to storage. In Handbook Quay Walls and the EAU 2004 some general values are recommended.

Handbook Quay Walls		**EAU 2004**	
(Representative weight: 17 % of the containers is not loaded)		(Representative weight)	
Maximum weight of a container	300 kN	Weight, 20 ft container	200 kN
1 layer container	15 kN/m2	Weight, 40 ft container	300 kN
2 layers containers	25 kN/m2	Light traffic (cars)	5 kN/m2
3 layers containers	30 kN/m2	General traffic (HGV's)	10 kN/m2
4 layers containers	40 kN/m2	General Cargo	20 kN/m2
5 layers containers	50 kN/m2	Container empty, stacked 4 high	15 kN/m2
		Container full, stacked 2 high	35 kN/m2
		Container full, stacked 4 high	55 kN/m2

Table 6-9: Terrain loads according Handbook Quay Walls and EAU 2004

6.6.2 Traffic load

The traffic load can be due to transport vehicles and other transport systems. They drive where there are no cranes or containers. Because a large terrain load is applied for the storage of containers, it is assumed that the **traffic loads are included in the terrain loads**.

6.6.3 Crane loads

For the two cases the cranes are used for transshipment of containers from ship to the shore. Loads coming from cranes can be very high. Therefore it's important to know what kind of foundation there is. For case 1 a mobile crane is chosen, because it's a quay for inland navigation vessels and the crane doesn't have to have a large reach. For case 2 a large container crane is chosen to load and unload the largest container vessels coming from sea.

Cranes are described in Handbook Quay Walls and EAU 2004. CUR 166 does not have a description of cranes and their loads. In the two other guidelines general values are given for cranes.

Rail distance [m]	Bearing capacity waterside [kN]	with an outreach [m]	Bearing capacity landside [kN]	with an back reach [m]	Dead weight [kN]	Maximum wheel load waterside [kN]	Maximum wheel load landside [kN]	Wheel distance [m]
15,24	410	36	410	13	5150	293	274	1,75
15,24	500	38	500	12	8100	474	433	1,20
20,00	500	43	500	16	9770	568	542	1,00
30,48	500	40	500	18	8970	408	609	1,24
35,00	670	52	670	25	12122	691	691	1,05
48,00	450	30	450	20	7350	420	383	1,50

Table 6-10: Crane load for container cranes, according Handbook Quay Walls

Crane loads EAU 2004	Rotating cranes	Container cranes and other transshipment gear
Bearing capacity [t]	7 – 50	10 - 80
Dead weight [t]	180 - 350	200 - 2000
Portal span [m]	6 – 19	9 - 45
Clear portal height [m]	5 – 7	5 - 13
Max. vertical corner load [kN]	800 - 3000	1200 - 8000
Max. vertical wheel surcharge load [kN/m]	250 - 600	250 - 700
Horizontal wheel loads		
Transverse to rail	up to approx. 10% of vertical load	
In direction of rail	up to approx. 15% of vertical load of the braked wheel	
Claw load [kN] (claw load over 10 m^2)	mobile cranes up to 2600	

Table 6-11: Crane loads, according EAU 2004

6.6.4 Mooring loads on the bollard

The force on the bollards depends on the movement of water due to berthing. The guidelines give a table to estimate the line pull force from the water displacement. In EAU 2004 and CUR 166 this displacement is just the water displacements of the ship and in Handbook Quay Walls it is done for DWT's of displacements. The largest load will occur when the line is perpendicular to the quay.
The table with line pull forces in the EAU 2004 contains characteristic loads. For the design of a bollard a load factor is given of 1,3. **It is doubtful of this safety factor is also needed for a sheet pile calculation.** Handbook Quay Walls refers to this value. CUR 166 also refers to the EAU 2004, but does not apply this safety factor and uses the factor from the table as design value. If the mooring and fender loads, for ships larger than 50.000 ton, are determined with wind loads the line pull load must be increased with 25%, according EAU 2004. This is not the case for the comparative analysis. In the CUR 166 a difference is made between sea vessels and inland navigation vessels.

Displacements [ton]/ Displacement · 10^1 [kN]/ DWT [ton]	Line pull force [kN]
< 2.000	100
< 10.000	300
< 20.000	600
< 50.000	800
< 100.000	1000
< 200.000	1500
> 200.000	2000

Table 6-12: Mooting force for water displacement, according to CUR 166, Handbook Quay Walls, EAU 2004

Ship Classes	Line pull force [kN]
Class I + II	150
Class III + IV	200
Class V + VI	250

Table 6-13: Mooring forces for several types of inland navigation vessels, only according to CUR 166

6.6.5 Fender loads

In the guidelines fender loads are described in general, but in EAU 2004 (and very general in CUR 166) this is done with the theory of kinetic energy absorption. The fender system depends on that amount of energy adopted by the fender. This energy is developed by a berthing ship and is a multiplication of the movement of water, the berthing velocity and some correction factors who enlarge or reduce the energy:

$E_d = ½ * G * v_s^2 * C_E * C_M * C_S * C_C$

G	mass of the ship
v_s	berthing velocity
C_M	Virtual mass coefficient
C_E	Eccentricity coefficient
C_S	Softness coefficient
C_C	Configuration coefficient

With the kinetic energy theory a reaction force can be determined for a certain fender system.

7 Design with CUR 211, Handbook Quay Walls

7.1 General

This chapter contains the design philosophy of Handbook Quay Walls [1], which is specially made for the designs of quay walls with combined sheet pile wall, superstructure and tension pile. The main aspects of the design are treated in this chapter. The calculations done with this recommendation are described in Annex D and E.

7.2 Design process

The design process contains the following aspects:

For the Ultimate Limit State:
1. Determination of representative values, design values and normative load combinations
2. Global calculation of the superstructure: weight density and redistribution of loads through the structure to the foundation elements
3. Calculation of the minimum toe level for the sheet pile wall
4. Calculation of sheet pile wall and anchor pile for the check of internal forces
5. Calculation of other foundation elements (for example pile foundation)
6. Detailed calculation of the superstructure
7. Determination of other failure mechanisms for the stability of the sheet pile structure
8. Constructional aspects

For the Serviceability Limit States and Ultimate Limit State
9. Determination of displacements

7.3 Determination of representative values, design values and normative load combinations

The determination of characteristic values is described in chapter 6 and Annex C. In Handbook Quay Walls these values must be transformed into design values.

The design philosophy is based on:
- Safety Class 2 (β = 3,4) (see chapter 5 for an explanation)
- Life time of the structure 50 years

The safety factors which must be applied on the representative values are described below.

Parameters	Partial safety factor γ_m	Parameters	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		
Values valid for all limit states * mean values are applied			

Table 7-1: Safety factors for unfavorable representative soil parameters

Parameters	Partial safety factor γ_{GE}
Bottom level	1,20
Groundwater level	2,00
Free water level	0,60
Value for ultimate limit states 1A and 1B	

Table 7-2: Safety factors for geometrical parameters, applied on the standard deviation of the schematized tidal motion

Parameters	Partial safety factor γ_f
Bending moment, normal force, shear force	1,30
Anchor force from sheet pile calculation	1,20
Epas maximum / Epas mobilized	1,30
Value for ultimate limit states 1A and 1B	

Table 7-3: Safety factors for internal forces from the sheet pile calculation with characteristic values

Sub-failure mechanisms anchoring or tension element	Extra safety factor S
Ground failure	1,00
Failure anchor - wall connection	1,50
Failure tie rod or profile tension element	1,30
Value for ultimate limit states 1A and 1B	

Table 7-4: Extra safety factors for anchoring element

Normative load combinations must be determined separately for the superstructure, combined wall, anchor pile and foundation piles. First a distinction can be made between the type of loads and how these loads are combined. Reduction factors should be applied on some loads.

Two **loading combinations** can be considered:

- Fundamental combination, fundamental situations during serviceability
- Special load combination, in extreme or special situations

For these two load combinations, three **types of loads** can be determined:

- Permanent loads
- Variable loads
- Special loads

Design values of loads in load combinations in the ultimate limit state					
Combination type	Permanent loads G_d		Variable loads Q_d		Special loads $F_{a,d}$
	Unfavorable	Favorable	Dominant variable	Other simultaneously occurring variable loads	
Fundamental	$\gamma_{f:g\ max} \times G_{rep\ max}$	$\gamma_{f:g\ min} \times G_{rep\ min}$	$\gamma_{f:q} \times Q_{1:rep}$	$\gamma_{f:q} \times \Psi_{0,j}\ Q_{j:rep}$	-
Special	$\gamma_{f:g\ max} \times G_{rep\ max}$	$\gamma_{f:g\ min} \times G_{rep\ min}$	$\gamma_{f:q} \times \Psi_{1,1}\ Q_{1:rep}$	$\gamma_{f:q} \times \Psi_{2,1}\ Q_{1:rep}$	$F_{a,rep}$

Table 7-5: Load combination philosophy

Depending on the limit state, load combination and type of load the representative values can be distinguished in three categories:

- Combination loads
- Momentaneous loads, a variable load that probably will occur in the load combination
- Quasi-permanent loads, a variable load present over a longer period

Table 7-6: Load factors for load combination

Type of load	Combination factor Ψ_0	Momentaneous factor Ψ_1	Quasi-permanent factor Ψ_2
Soil pressure	1,00 [1)]	1,00 [2)]	1,00 [3)]
Water pressure	1,00	1,00	1,00
Variable favorable loads	0,70	0,60	0,50
Meteorological loads *	0,70	0,30	0

[1)] based on 85% of the characteristic free water level
[2)] based on 80% of the characteristic free water level
[3)] based on 60% of the characteristic free water level
* Meteorological loads are loads due to waves, currents, air and water temperature, snow, ice and earthquakes

Examples of **fundamental load combinations**:

1. A permanent load and a dominant variable load due to maximum excess pore pressure at low water, combined with one meteorological load and two simultaneously occurring variable loads, e.g.: terrain loads, crane loads, traffic loads, bollard loads.
2. Permanent load and a dominant variable load due to a high groundwater level with maximum gradient, combined with one meteorological load and two simultaneously occurring variable loads, e.g.: terrain loads, crane loads, traffic loads, bollard loads.
3. A permanent load and a dominant variable load due to a low free water level with the same low groundwater level, combined with one meteorological load and two simultaneously occurring variable loads, e.g.: terrain loads, crane loads, traffic loads, bollard loads.

Examples of **special load combinations**:

1. A special load (not functioning drainage system) due to extreme low free water level with a high ground water level, combined with a permanent load and two simultaneously occurring variable loads.
2. A special load (not functioning drainage system) due to extreme high groundwater level with maximum gradient, combined with a permanent load and two simultaneously occurring variable loads.
3. A special load (extreme scour of 1 m) due to propeller currents and an excess pore pressures situation as dominant variable load (low free water level based on 85% of characteristic value), combined with a permanent load and two simultaneously occurring variable loads.
4. A special load (extreme terrain loads) due to emergency storage and an excess pore pressure situation as dominant variable load (low free water level based on 85% of characteristic values), combined with a permanent load and two simultaneously occurring variable loads.

7.4 Global calculation of the superstructure: weight and redistribution of loads through the structure

The superstructure has a relieving capacity that reduces the effective soil pressure behind the wall. It also redistributes the loads working on the superstructure to the foundation elements. The global calculation must be carried out to find the **representative axial load** working on the sheet pile wall. To find the axial load, the superstructure calculation should also be done with representative values. However, **if no superstructure is applied in a quay wall, this part of the design process can be skipped.**

The redistribution of loads through the superstructure depends on the stiffness of the structure. The stiffness of the structure can be found by applying test loads of 100 kN. From the displacements of the superstructure due to this load, the stiffness can be determined:

$$c_v = \frac{F_e}{\delta_v}$$

A higher stiffness will lead to a better redistribution of loads over the structure to the foundation elements.

The stiffness of the superstructure must be calculated for horizontal and vertical loads. The loads will be transformed to line loads working on one linear meter of the quay wall. The superstructure can be schematized to a static structure. With this schematization, the line loads and the weight density of the structure, the loads working on the foundation elements can be calculated.

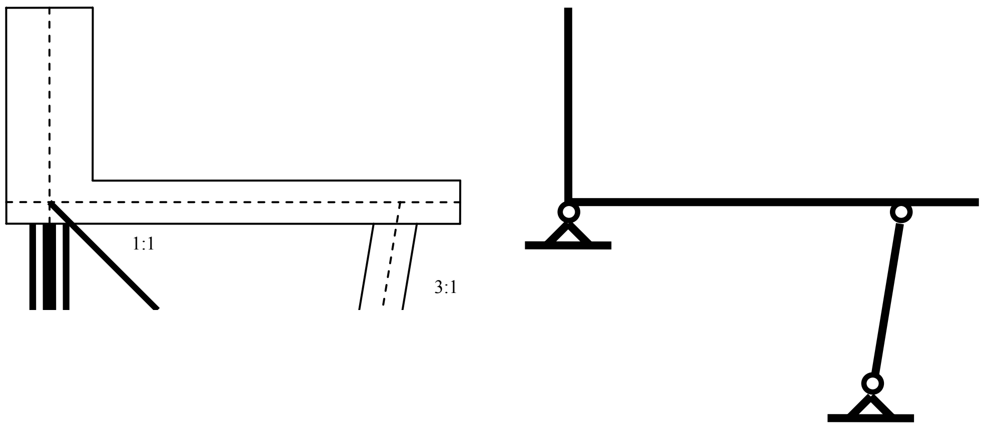


Figure 7-1: Schematization of the superstructure to a static system

7.5 Determination minimum toe level

The minimum toe level is in the case of a quay wall with superstructure mainly determined by the vertical bearing capacity. A safety factor of 1,3 between maximum mobilized passive earth pressure and the real mobilized passive earth pressure is in most case more than sufficient. However, if no vertical bearing capacity is necessary, the factor 1,3 becomes important. It means that less than 77% of the passive earth pressure should be mobilized.

$$\frac{E_{pas,\max}}{E_{pas,mob}} = 1{,}3 \Rightarrow E_{pas,mob} = \frac{E_{pas,\max}}{1{,}3} = \frac{100\%}{1{,}3} = 77\%$$

If the vertical bearing capacity is the limiting factor for the toe level, this will usually lead to a (partially) fixed wall in the soil. If no axial load on top of the wall is applied, the safety factor for the maximum passive earth pressure leads to a simply supported wall. However, for economical reasons it can be wise to increase the toe level to reduce the bending moments.

For a combined wall the minimum toe level of the main elements should be determined for the vertical bearing capacity. The infill element should have a minimum depth based on the stability of the soil at the harbor bottom. A check for piping will determine the length of these infill piles.

7.6 Calculation of redistribution of forces through the sheet pile structure

The internal forces should be checked after determining the most favorable toe level. The bending moments, shear force and anchor force can be calculated. If a superstructure is present, the axial load working on the sheet pile wall can have favorable and unfavorable effects, which can change the internal forces due to redistribution of loads.

- **Eccentricity of the axial load** on the wall, due to the supporting point of the superstructure on the combined wall. An oblique wall has also some influence on the eccentricity.

$$M_{nd} = N_d \cdot e_{ecc}$$
$$e_{ecc} = 0{,}5 \cdot D_{wall}$$

- **Second order moment** due to the axial load

$$M_{nd} = N_d \cdot \frac{n}{n-1} \cdot \delta$$
$$n = \frac{\pi^2 \cdot EI}{l_k^2}$$

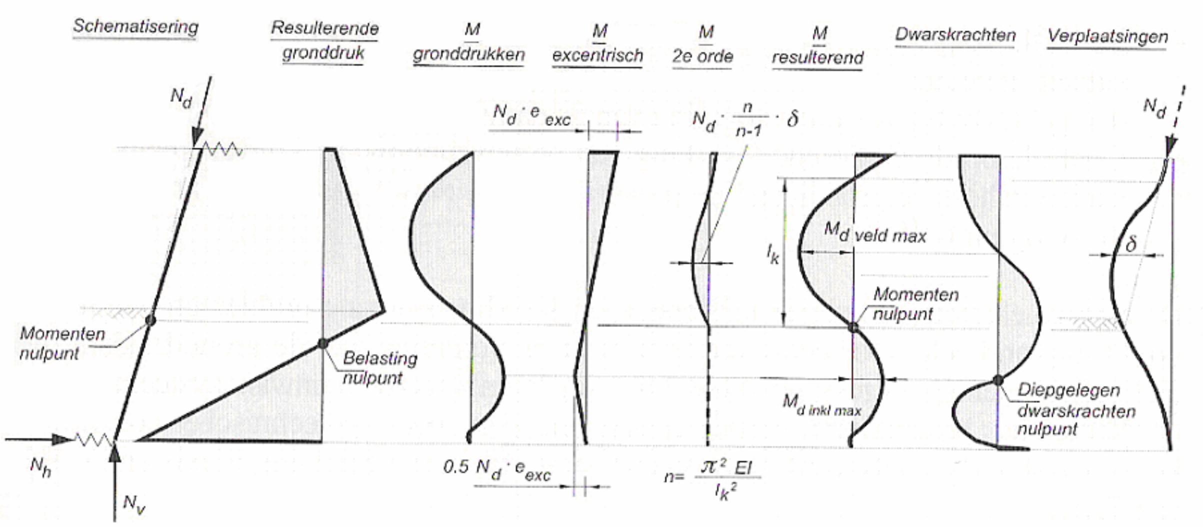


Figure 7-2: Structural aspects due to axial loads, form left to right: extra force at the toe, bending moment due to earth pressure, eccentricity and second order effects give a resulting moment distribution

The eccentricity causes a favorable bending moment at the top of the wall. The axial load which introduces this moment, is a summation of the loads coming from the superstructure and vertical load coming from the anchor pile. If the anchor is placed at an angle of 45˚, the horizontal and vertical anchor forces are equivalent with the shear force at the top of the wall. This way of introducing the anchor force and eccentricity needs an iterative calculation.

The vertical bearing capacity of the wall can cause a fixity in the soil, which can result in extra resistance at the toe of the wall. This force is a multiplication of the tangent angle of the internal friction of the soil present at the toe and the vertical resistance force due to axial loads working at the toe (see figure 7-3).

In the calculation also **arcing** can reduce the maximum moment, but this may only be done when it is **proved with a finite element program**.

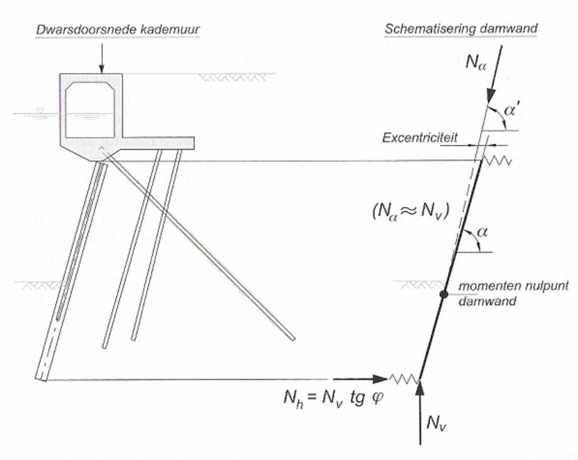


Figure 7-3: Plastic spring applicable for the fixation in soil

The axial load from the superstructure calculation must be a representative value for application on the sheet pile wall. The eccentricity and second order moment caused by this load will also be applied as a summation of representative values to the moment distribution. The maximum moment resulting from these calculations (including eccentricity and second order moments) should be increased with a safety factor 1,3.

$$M_{d,\max} = 1{,}3 \cdot M_{rep,\max}$$
$$M_{rep,\max} = M_{\max} + M_{ecc} + M_{2nd,order}$$

The normal force in the wall is a summation of the representative axial load from the superstructure and the representative anchor force. These values should be increased with a safety factor of 1,3.

$$N_{d,\max} = 1{,}3 \cdot (N_{axial,structure,rep} + F_{anchor,rep})$$

The maximum yielding stress may not be exceeded. This will be calculated with:

$$\sigma_{yielding} = \frac{M_{d,\max}}{W} \pm \frac{N_{d,\max}}{A}$$

In case of a combined wall, the secondary infill elements transfer most of their effective earth pressure loads to the main elements, due to horizontal arching. The only loads that work on the infill piles is the hydrostatic pressure.

The anchor force resulting from the sheet pile calculation should be increased with a factor 1,2. For the calculation of the anchor pile profile an extra safety factor of 1,2 should be applied and for the anchor connection to the wall an extra safety factor of 1,5 should be applied. For ground failure no extra safety factor have to be applied on the anchor force.

$$F_{d,anchor} = 1{,}2 \cdot F_{rep,acnhor}$$
$$F_{d,anchor,profile} = 1{,}2 \cdot F_{d,anchor} = 1{,}44 \cdot F_{rep,acnhor}$$
$$F_{d,anchor,connection} = 1{,}5 \cdot F_{d,anchor} = 1{,}8 \cdot F_{rep,acnhor}$$
$$F_{d,anchor,ground-failure} = 1{,}0 \cdot F_{d,anchor} = 1{,}2 \cdot F_{rep,acnhor}$$

A check of the stability of the anchor in the soil is done with the Kranz-method.

7.7 Calculation of foundation elements

A wall with superstructure has not only a foundation support at the combined wall, but also on other piles. The superstructure redistributes the loads between the foundation elements. There is referred to special CUR-reports to determine the vertical bearing capacity of pile foundations.

7.8 Detailed calculation of the superstructure

Based on the redistribution of the loads through the superstructure a definitive calculation of the structure can be made. If there are large deviations with the global calculation done before, a new calculations for the bearing capacity of the sheet pile should be made. Because a superstructure with more concrete can have influence on the axial load.

7.9 Check of the total stability

A deep circular sliding plane must be checked. The calculation will be done with **representative values for the soil parameters**, loads and resistance forces, deviating from NEN 6740. The relation between operating moment and stabilizing moment must be larger than a factor 1,3. A drained and undrained situation can be calculated.

$$M_{ad} \leq 1{,}3 \cdot M_{rd}$$

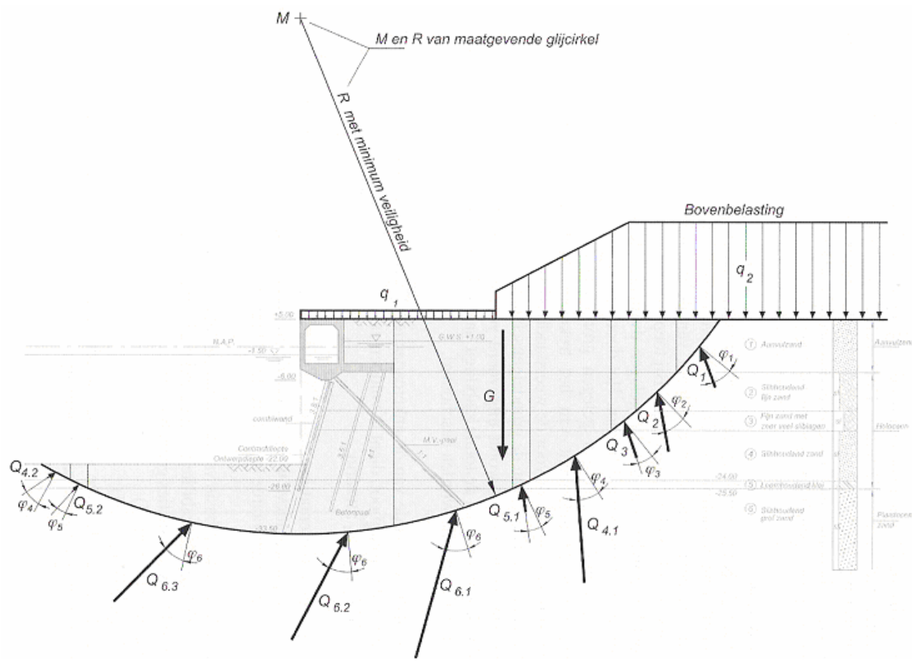


Figure 7-4: Total stability with Bishop for quay walls

7.10 Check of the Kranz-stability

The Kranz-stability is also a check for a deep failure plane, not for the total structure, but for the soil mode loaded by the tensile element. The failure plane starts at the shear force zero point at the wall, to the centre of gravity of the anchorage system. In case of an anchor wall, the failure plane goes to the under side of the wall.

The anchor force for Kranz-stability must be calculated with **representative soil properties**. The representative value of the anchor capacity due to equilibrium in the soil, must be larger than **1,5** times the maximum anchor forces from sheet pile calculation.

$$F_{kr;rep} = 1{,}5 \cdot F_{d,anchor} \, (1{,}5 \cdot 1{,}2 \cdot F_{rep,anchor})$$

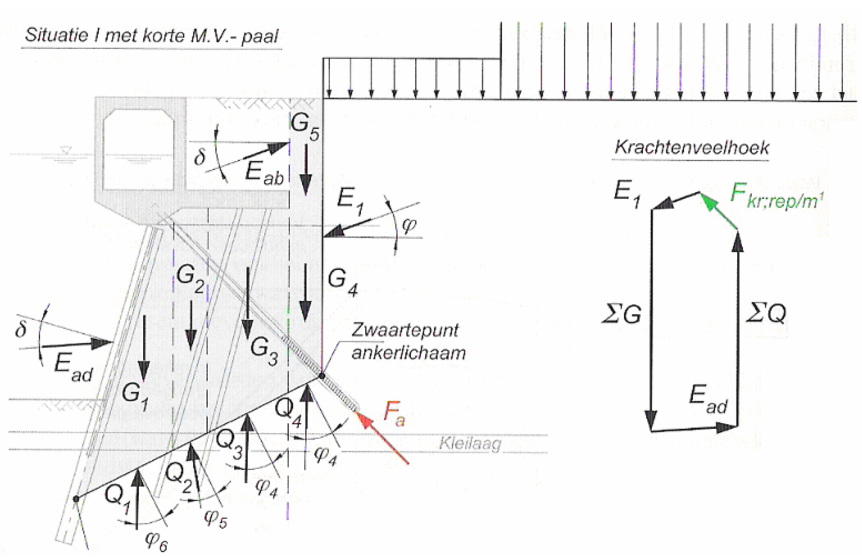


Figure 7-5: Kranz stability for a quay wall with superstructure

7.11 Check of heave

This failure mechanism (in Dutch = opbarsten) occurs when a high water pressure is present under an impermeable layer in the harbor bottom of the quay. Cracking of the bottom can be the result. An equilibrium calculation must be made with **representative soil parameters**. Safety factors should be applied on the resulting hydrostatic pressure under the impermeable layer and the weight of the layers on top. A distinction is made between construction and serviceability phase. For the serviceability phase yields:

$$W_d \le G_d \Rightarrow \gamma_{fW} \cdot W_{rep} = \gamma_{fG} \cdot G_{rep}$$

$$\gamma_{fW} = 1{,}50$$

$$\gamma_{fG} = 0{,}90$$

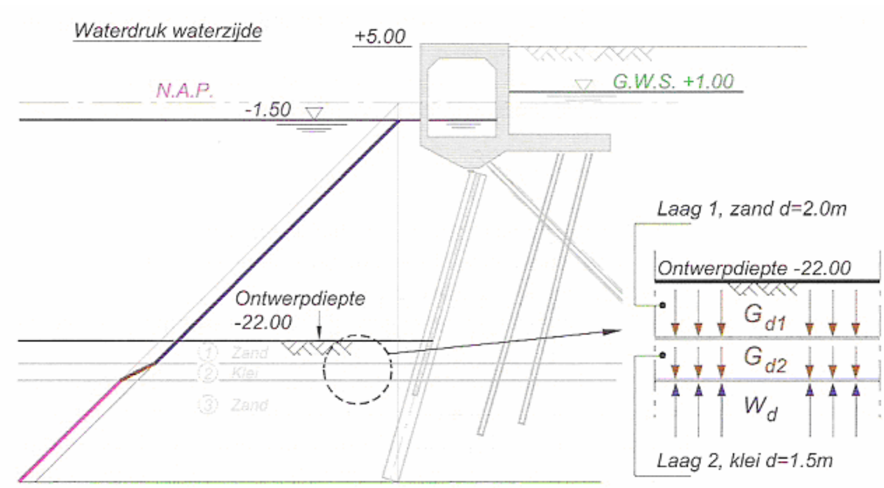


Figure 7-6: Heave due to high hydrostatic pressure under an impermeable layer

7.12 Check of hydraulic heave and piping

Piping

This check is important for the determination of the infill sheet pile. However, if no infill piles are used, this check must be made for the total sheet pile wall. Piping is based on internal erosion in the passive soil pressure area between harbor bottom and toe of the wall. If in this area the groundwater flow pressure in upward direction is larger than the weight of the vertical effective soil pressure, soil particles are washed out. The critical current is exceeded. Due to piping, canals appear in the soil which has effect on the stability the structure.

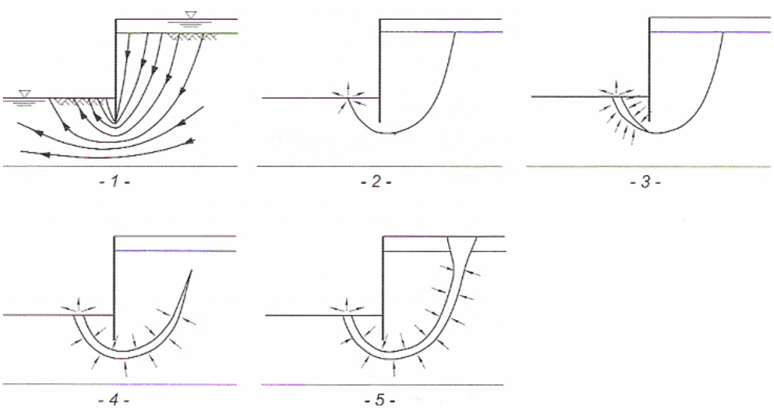


Figure 7-7: Piping mechanism, ***piping occurs for the shortest distance between two water levels, not as in this picture!***

The critical gradient is given by (m depends on a flow net analysis): $i_{crit} = \dfrac{m \cdot H_{crit}}{D} \leq 0{,}5$

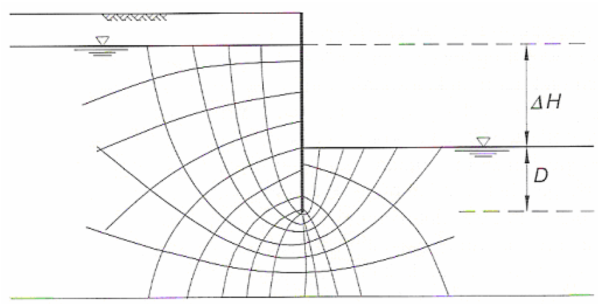


Figure 7-8: Defining the critical gradient by Terzaghi for piping

The critical gradient must be checked for two situations. The toe level of the (infill) sheet pile will be reduced with 0,5 m. Important is that no bottom protection is applied.

1. Unfavorable low free water based on 5%-underspend, a high normal groundwater level with a extreme scour of 2 m.
2. Unfavorable low free water based on 5%-underspend over the reference period and a unfavorable high groundwater level based on a not working drainage system.

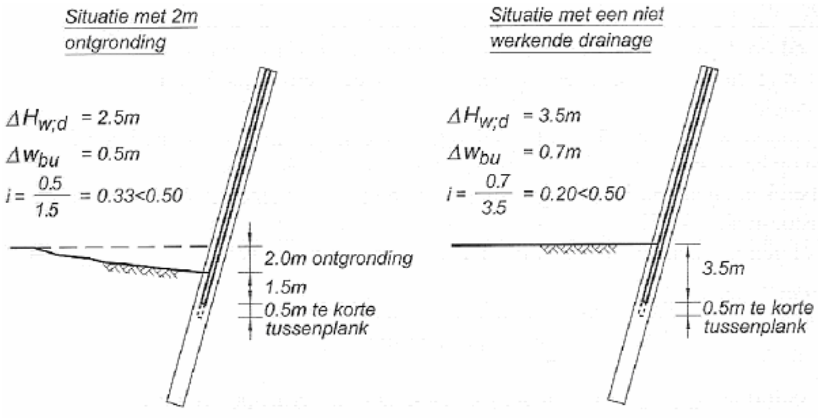


Figure 7-9: Two situations for piping

Hydraulic heave (in Dutch: hydraulische grondbreuk)
Also hydraulic heave must be checked. In this case it is not cracking of an impermeable layer, but **instability of a permeable layer due to flow forces**. For hydraulic heave also a check is necessary for the vertical equilibrium between the soil at the passive side. The design value of the flow pressure must be in equilibrium with the effective weight of the soil in a certain area. This area has a width of 50% of the toe level of the (infill) sheet pile. Safety factors are applied on the flow pressure and the effective weight of the soil.

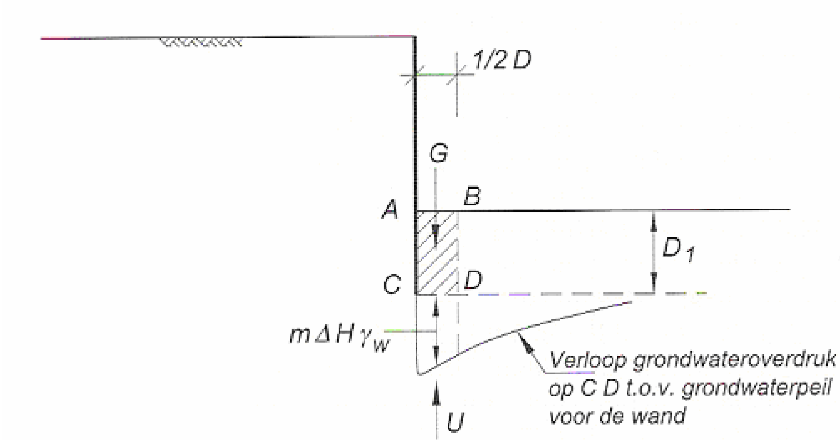


Figure 7-10 Hydraulic heave calculations with the Terzaghi method

7.13 Calculation of displacements

These calculations are based on finite element methods and must mainly be carried out for the serviceability limit state. For the application of arching in the ultimate limit state it is important to check the displacements with a finite element program. In the Ultimate Limit state it is also important that no large displacements occur that will lead to collapse of the structure or structural elements.

7.14 Calculation of the railway

Calculations must be made for loads working on the superstructure due to the railway. With these calculations reinforcement can be designed.

7.15 Calculation of details

Several details must be designed, especially the supporting points of the foundation elements and other points introducing forces into the superstructure.

8 Design with CUR 166, Sheet pile structures

This chapter contains the design philosophy according CUR 166 [3], which is originally made for the design of several types of sheet pile wall structures. For safety considerations a fault tree is given, which treats the failure mechanisms (see chapter 5). To handle these failure mechanisms, a step-by-step plan is established. In this chapter the main aspects of the design of a sheet pile structure as quay wall are described. The calculations done for this recommendation are carried out in Annex D and E.

8.1 Design process

The steps-by-step plan contains the following aspects and is treated in the next paragraphs.

Ultimate Limit State:
1. Determine normative conditions
2. Determine representative values of the parameters
3. Determine design values of the parameters
4. Choose a design scheme
5. Calculate the minimum toe level (simple support in the soil for the sheet pile wall)
6. Calculations for dimensioning of the wall (find the normative load situation by changing the modulus of sub-grade reaction and the groundwater level)
7. Check of the moment
8. Check of the shear force and the normal force
9. Check of the anchor force

Ultimate and Serviceability Limit State:

10. Check of displacements

Ultimate Limit State:

11. Check of other failure mechanisms
12. Check of construction aspects

13. Verify all choices

8.2 Load combinations

The load combinations given in CUR 166 treat difference in soil stiffness and water levels. No load combinations are given for the application of external loads. In CUR 166 is recommended that sufficient attention has to be paid to normative load combinations in the construction phase and that for the design of a lock combinations of bollard loads and water levels must be calculated.

8.3 Step 1,2,3: Determine normative conditions, representative and design values

The determination of characteristic values is described in chapter 6 and Annex C. In CUR 166 these values must be transformed in design values.

The design philosophy is based on:
- Safety Class III (β = 4,2) (see chapter 5)
- Life time of 50 years

The safety factors for the sheet pile calculations are given in the table below.

Table 8-1: Safety factors for sheet pile calculation according CUR 166

Parameters	γ and Δ for X_{rep} [2) 3)]						Design values
	Class I		Class II		Class III		
	γ	Δ	γ	Δ	γ	Δ	
Cohesion c'	1,00	-	1,00	-	1,10	-	X_{rep}/γ (c'/γ)
Angle of internal friction tan(φ')	1,05	-	1,15	-	1,20	-	X_{rep}/γ (c'/γ)
Retaining height (m) [1)]	1,60	0,20	2,20	0,30	2,60	0,35	max($\mu+\gamma\sigma$; $\mu+\Delta$)
Groundwater level (high side)	1,30	0,15	1,70	0,20	2,10	0,25	max($\mu+\gamma\sigma$; $\mu+\Delta$) or min($\mu-\gamma\sigma$; $\mu-\Delta$) [3)]
Groundwater level (low side)	0,66	0,05	0,87	0,05	1,50	0,05	max($\mu+\gamma\sigma$; $\mu+\Delta$) [4)]
Live loads:							
permanent	1,0		1,0		1,0		
variable	1,0		1,0		1,25		

[1)] The design value of the retaining height is taken into account by lowering the bottom at the passive side, the active side will remain the same
[2)] For another reference period than 50 years the safety factors must be adapted
[3)] The min and max water level depends on the case described in step 6, but is not applicable when there is a free water level
[4)] For the design value of the water level at the high side, sometimes the ground surface level can be assumed, because a higher physical value is not possible. This is not valid for water heads in aquifers.

These safety values should be applied on the representative values in the sheet pile calculation.

8.4 Step 4: Choose a design scheme

Two design schemes are available:
B. Design values in all calculation phases
C. Design values in the phases that will be checked, representative values in the preceding phases

Method B is recommended for a more optimized profile. When using this scheme, for the preceding phases (with representative values) a **safety factor 1,2 on moments and forces** must be applied. Otherwise the distance between the design value and representative value could be too small.

8.5 Step 5: Calculation of the toe level

The minimum toe level is reached (using **design values** for soil strength parameters), when almost 100% of the passive earth pressure is mobilized. For a sheet pile wall with anchoring this leads to a wall with a simple support.

Also other aspects can determine the minimum toe level: for example the vertical bearing capacity and closure of an aquifer to prevent from groundwater flow.

8.6 Step 6: Calculations for dimensioning of the wall

The minimum toe level can be increased to reduce the field moment. This is economically interesting. Five dimensioning calculations are necessary, to find the normative situation for elasticity of the soil and water levels.

Calculation	Limit State	Modulus of sub-grade reaction	Design value groundwater level
6.1	ULS	Low	High groundwater level
6.2	ULS	High	High groundwater level
6.3	*ULS*	*Low*	*Low groundwater level*
6.4	*ULS*	*High*	*Low groundwater level*
6.5	SLS	Low	-

Table 8-2: Calculations for normative situation, with difference in modulus of sub-grade reaction and soil strength parameters

For a free water level at the passive side, step 6.3 and 6.4 are not normative, because the lowest low water spring is normative for this situation. These two steps will not be taken into account. As mentioned in step 4, the moments, and forces in step 6.5 will be increased with a safety factor 1,2.

8.7 Step 7: Check of the bending moment

8.7.1 General

After calculations for the toe level and internal forces in the wall, these internal forces should be checked.

$$M_{s;d} \leq M_{r;d}$$

The highest moment of step 6, $M_{s;d}$, must be determined for the normative situation. For concrete a check of the crack width is important, for steel the yielding moment with the yielding stress, f_y, must be checked, including the factor for oblique bending (described in chapter 6 and Annex C). Oblique bending, β_B, gives a reduction on the section modulus, W. The safety factor, $\gamma_{m,st}$, for steel is 1,0.

For steel sheet pile walls that may only have yielding stress in the ultimate fiber an elastic check should be carried out.

$$M_{r;d} = \frac{\beta_B \cdot W_{el/pl} \cdot f_y}{\gamma_{m,st}}$$

For U-shaped profiles with high hydrostatic water pressure the yielding limit should be reduced. This is for water level differences of more than 20 m. This is not the case in a tidal area with a free water level. This design aspect can be neglected for quay walls and is probably only interesting for building pits.

8.7.2 Combined wall

The combined wall consists of primary and secondary elements. These to elements are not present over the total wall height. For the high part of the wall the stiffness will be determined by both elements, while the stiffness of the lower part is only due to the primary element.

$$I_{combi} = \frac{I_{tube} + I_{n;pile}}{L}$$

$$I_{tube} = \frac{\pi}{64} \cdot (D^4_{outside} - D^4_{inside})$$

For a strength calculation of the sheet pile wall it is allowed to neglect the secondary elements. The bending moment will only be taken by the tubes. It is also possible to use the strength of the total wall, depending on the sheet pile wall supplier.

$$M_{s;d} \leq \frac{M_{r;combi;d}}{L}$$

$$M_{r;tube;d} = \frac{W_{combi} \cdot f_y}{\gamma_{m;st}}$$

The main function of the secondary elements is the impermeability of the wall. These elements have a relative low stiffness, which will lead to arching, due to a lot of displacements in these elements. The effect of arching is a smaller effective earth pressure on the secondary elements, but higher earth pressures on the primary elements.

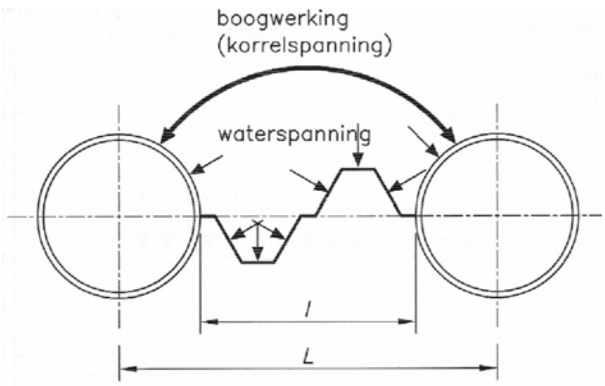


Figure 8-1: Arching due to difference in stiffness of tubes and infill piles

8.8 Step 8: Check of the shear and normal force

8.8.1 General

Control of the normal force is important if the anchor makes an angle with the wall or if combined walls with vertical loads are applied.

When a normal force is acting the buckling length of the wall must be checked, when the force is larger than 4% of the buckling force. The effect of the normal force on the plastic moment can be neglected if the normal force is less than 10% of the plastic capacity for Class 3 of the Eurocode sheet pile classification. For other U-profiles, the normal forces must be less than 25% of the plastic capacity.

The design value of the section moment must be reduced if the shear force is larger than 50% of the plastic resistance. The shear force can be normative, this often occurs in the zone with a lot of corrosion.

8.8.2 Vertical loads

For the consideration of vertical bearing capacity is a distinction made between only vertically loaded piles and piles loaded vertically and horizontally loaded. In the first case the bearing capacity is formulated by the toe resistance and the shaft resistance of the pile. The second case pointed the fact that the angle of inclination can change direction due to vertical loading.

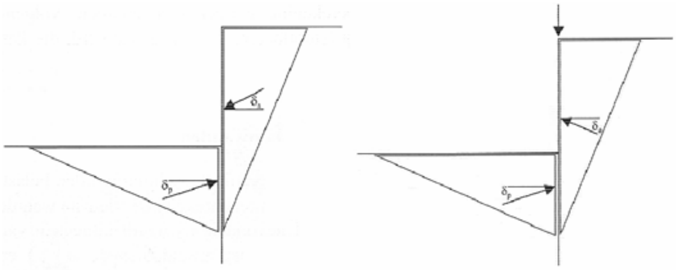


Figure 8-2: Changing the angle of inclination at the active side due to a vertical load on top of the wall

For external vertical loading larger than 12,5 kN/m^2, which looks the case for a superstructure, an interaction calculation is needed. For a final calculation it is recommended to do this with a finite element program.

When calculating with a vertical load, the eccentricity must be considered. The so called second order moment must be included in the sheet pile calculation and depends on the axial load (N), a factor n with the bugling length (l_b, distance to moment zero-point) and the displacements of the wall (δ).

$$M_{2de} = N \cdot \frac{n}{n - 1_b} \cdot \delta$$

$$n = \frac{\pi^2 \cdot EI}{l_b}$$

In a finite element program with "updated mesh analysis", is the second order moment automatically included in the sheet pile calculation.

8.9 Step 9: Check of anchor force

8.9.1 Step 9.1

Step 9.1 is only necessary when the stiffness of the anchor is uncertain. A sensitivity calculation must be done. First the calculation with the highest anchor force (step 6) must be determined. Use these parameters to calculate the 5%-upper value of the anchor stiffness.

8.9.2 Step 9.2

Find the highest anchor force from step 6 and 9.1, $F_{A;max}$.

8.9.3 Step 9.3

Use the maximum anchor force from step 9.2:

Design value anchor force for waling, anchor wall, bearing capacity of the soil: $F_{s;A;d} = 1{,}1 \cdot F_{A;\max}$

Design value anchor force for anchor cross-section: $F_{s;A;st;d} = 1{,}25 \cdot F_{A;\max}$

8.9.4 Step 9.4

Check the design value of the anchor force with the design strength of the soil, over a deep failure plane, with a Kranz verification, step 11.1.

Check the stability of the anchor wall. The strength of the anchor wall is the difference between passive and active mobilized earth pressure due to the anchor. The **soil parameters** are based on the **design parameters for step 3**. Terrain loads have an unfavorable effect on the active earth pressure and favorable effects on the passive earth pressure. This passive terrain load part must be neglected for the calculation.

$$F_{s;A;d}\,(step 9.3) \leq F_{r;A;p;d}$$

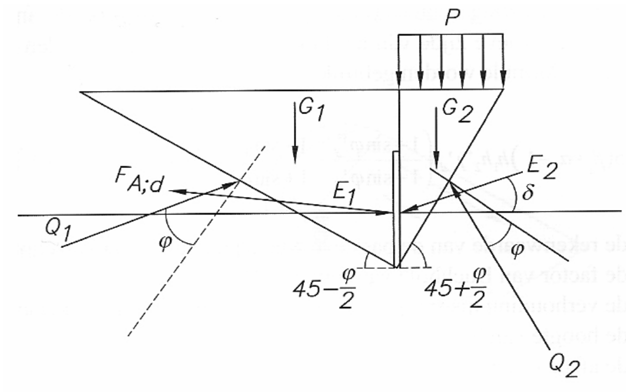


Figure 8-3: Stability of the anchor wall, with no terrain load at the passive side

The strength of the tie rod is the minimum strength of crack force and yielding force.

$$F_{s;A;st;d} \leq \min\left(\frac{F_{r,crack,rep}}{1{,}4}, \frac{F_{r,yieldin,rep}}{1{,}0}\right)$$

8.10 Step 10: Check of displacements

This is important if no large displacements are allowed, for example when there is a railway on the quay. These calculations must be carried out in **Serviceability Limit State**, with representative soil strength parameters (**step 6.5**) and low modulus of sub-grade reaction.

The displacements must also be check for the **Ultimate Limit State**. This can be calculated corresponding with **step 6.1**, with design soil strength parameters and low modulus of sub-grade reaction.

8.11 Step 11: Check of other failure mechanisms

8.11.1 Step 11.1: Deep straight slip plane for an anchored sheet pile wall

The anchor force determined for step 9.2 must be 1,5 times smaller than the anchor force due to Kranz, calculated with **representative soil parameters**.

$$1{,}5 \cdot F_{A;\max} \leq F_{r;kr;rep}$$

A fictive L is necessary, which is the length along the tie rod, between sheet pile wall and the resulting point of shear force between the anchor element and the soil around.

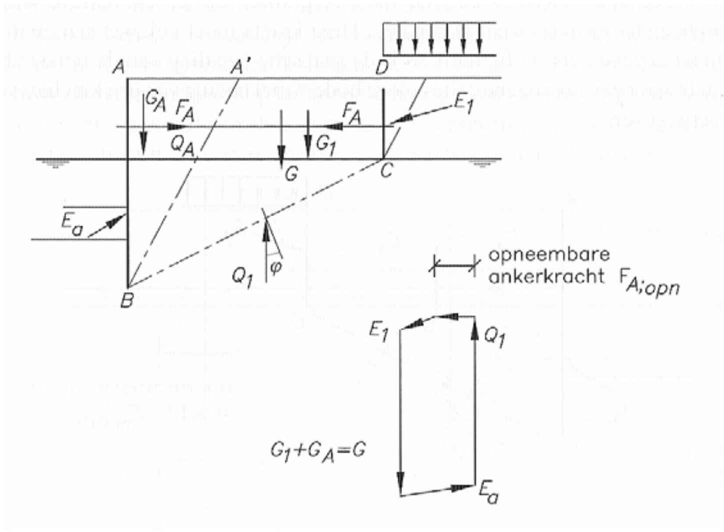


Figure 8-4: Kranz-model for an anchor system with anchor wall

8.11.2 Step 11.1: Deep straight slip plane for a tension pile

For the design of a tension pile, it is recommended to only use the length behind the active wedge for the bearing capacity.

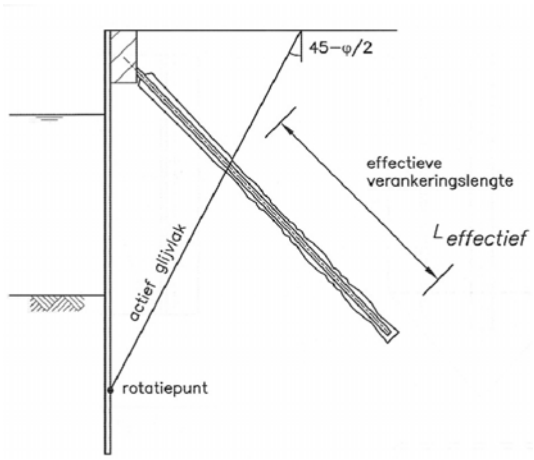


Figure 8-5: Effective anchor length for bearing capacity of tension pile

Special attention is paid to the MV-pile (Müller Verfahren), which is as described a good solution for quay wall structures. The calculation of this element is based on grout pressure: friction between soil and shaft of the grout column. The maximum tensile strength is a summation of the maximum shear stress between grout and sand in the soil. For this calculation a material safety factor of 1,4 must be applied.

The strength can also be based on a test loading, which will be done on an already driven pile. The last method for checking the strength of the MV-pile is based on the cone penetration test. The maximum shear force for MV-piles is 1,4% of the cone resistance in the Pleistocene sand, with a maximum of 250 kN/m^2. For an “arrow” type of pile point the percentage of the cone resistance should be enlarged to 1,6%, but still with a maximum of 250 kN/m^2. A summation of the maximum shear stress of the shaft over the effective length will lead to the bearing capacity.

When the MV-piles are placed at a centre to centre distance of less than 7·D (= effective diameter) the piles will influence each other.

The stability of the tensile element can be calculated by the method of Kranz for the lower failure plane.

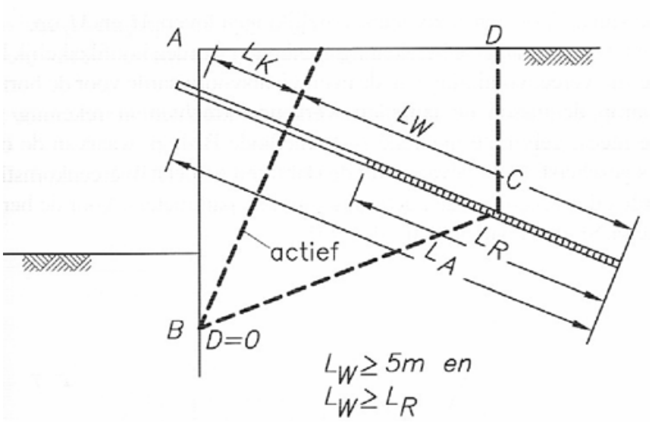


Figure 8-6: Kranz verification for a tension pile

8.11.3 Step 11.2: Crack of the bottom due to vertical earth pressure behind the wall

This is a check of the bearing capacity of the soil at the lower side of the wall (in Dutch = grondbreuk). It must be checked with **representative values**. A difference of 1,7 must be maintained between the weight of the soil at the passive quay side and the weight of the soil at the active side of the quay. This is important for walls with a small driving depth.

$\gamma_{\text{crack of the ground}} = 1{,}7$

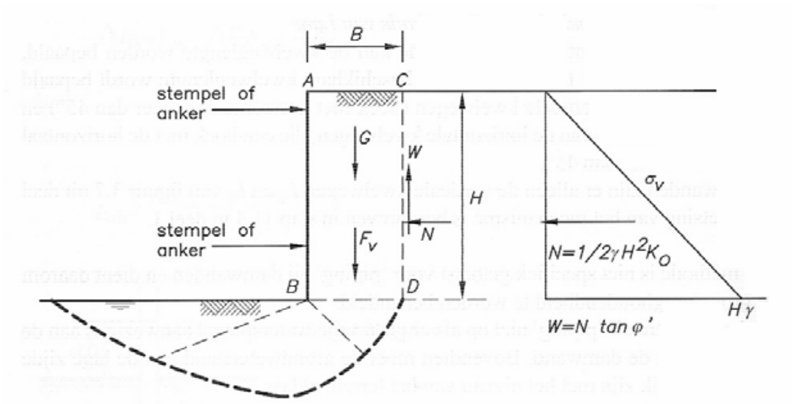


Figure 8-7: Crack of the bottom due to vertical earth pressure behind the wall

8.11.4 Step 11.3: Loss of total stability

This mechanism is calculated in the CUR 166, with the Bishop method. There is referred to NEN 6740 for this calculation. The **partial safety factors of NEN 6740** are used in these calculations. Because this mechanism is originally used for a safety class II, for safety class III the safety factor is increased to 1,1.

$\gamma_{\text{total stability}} = 1{,}1$

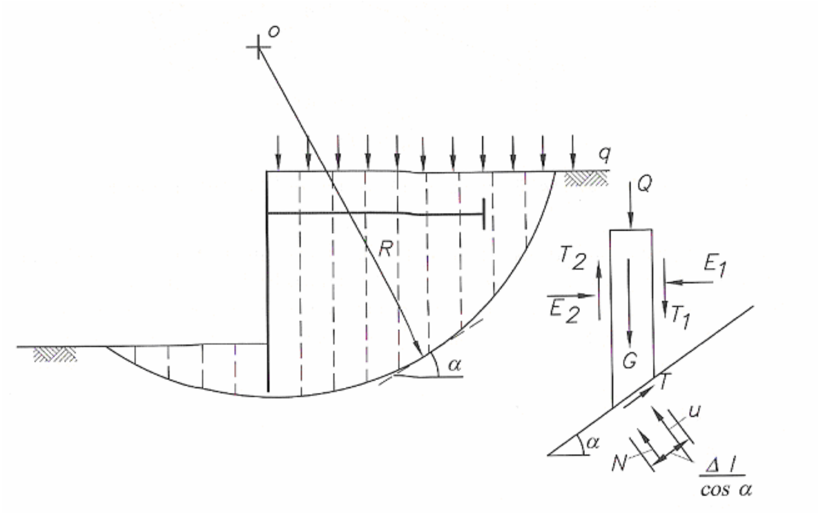


Figure 8-8: Bishop method for verifying total stability

The critical sliding plane will usually slide just under the toe of the wall due to the stiffness of the wall. This is not the case for a tie rod with very low stiffness. The CUR 166 argues that the sliding plane can slide through the line of the tie rods. However, this is not the case for anchor piles which are stiff enough to move with the total sliding plane.

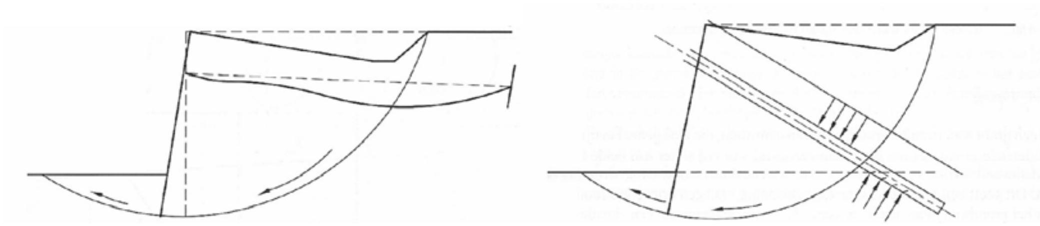


Figure 8-9: Loss of total stability with a sliding plane through the line of the anchor: steel cable (left) and anchor pile (right)

8.11.5 Step 11.4: Piping

The starting point of piping is hydraulic heave (Dutch = hydraulische grondbreuk).

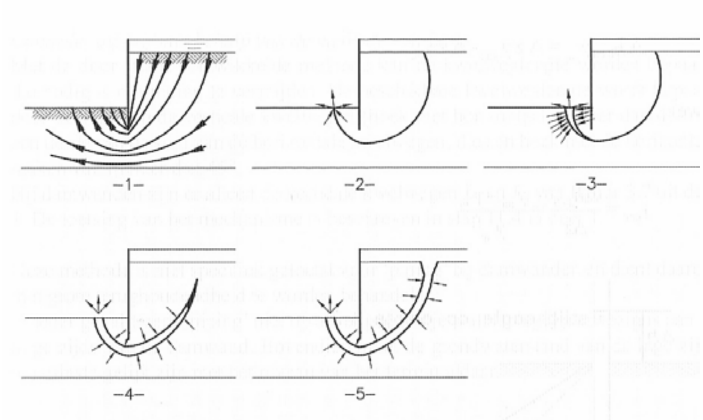


Figure 8-10: Piping mechanism, according CUR 166

A detailed (discharge gradient) and a global method (Lane) are used to check the mechanism of piping. The discharge gradient must be checked with a groundwater flow calculation with **representative soil values**. The discharge gradient must be less than 0,5.

$$i_{crit,rep} < 0{,}5$$

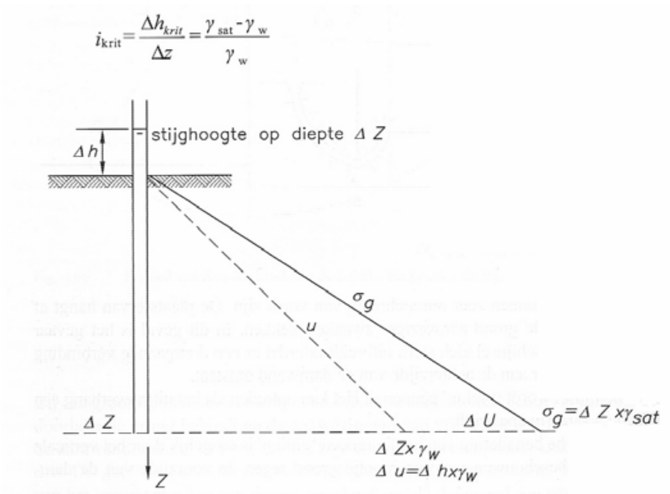


Figure 8-11: Critical gradient for piping mechanism

The Lane method depends on the length of the sheet pile wall and a seepage factor for certain types of soil. For this method a safety factor of 2, γ_{piping}, is given.

$$L_1 + L_2 \geq \gamma_{piping} \cdot C_L \cdot \Delta H$$

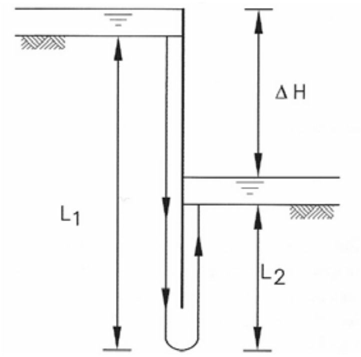


Figure 8-12: Piping mechanism by Lane

8.11.6 Step 11.5: Vertical bearing capacity

As mentioned in step 9.3 the anchor force must be increased with a factor 1.1 for the vertical bearing capacity of the soil. This is only important if the anchor makes an angle with the plane of the wall.

$$F_{s;A;d} = 1{,}1 \cdot F_{A;\max}$$

Vertical loads working on the sheet pile wall change the direction of the friction angle of the wall, which leads to another distribution of the forces on the wall.

8.12 Step 12: Construction aspects

Main aspects in the construction of a sheet pile wall are the driving of the piles and the monitoring during driving. These aspects will not be carried out for the comparative analysis, but largely explained in the CUR 166.

8.13 Step 13: Verifying choices

The design of a sheet pile wall is an iterative process, some choices must be reconsidered.

9 Design with EAU 2004, Waterfront structures

This chapter contains the design philosophy of EAU 2004 [8], which is based on experience gained over many years and based on the new probabilistic safety approach. The main design aspects for the design of a quay wall are treated in this chapter. The calculations for this recommendation are carried out in Annex D and E.

9.1 Safety approach for EAU 2004

The following failure mechanisms are treated in the EAU 2004 for sheet pile walls:

Limit State 1A: Limit state of loss of support safety
- Stability of the bottom against hydraulic heave
- Piping (foundation failure due to erosion)

Limit State 1B: Limit state of failure of structures and components
- Load bearing capacity
    - Ground failure in the passive earth pressure area due to horizontal action effects
    - Axial sinking in the soil due to vertical action effects
- Verification of sheet pile profile for stresses and strains
- Verification of anchor system

Limit State 1C: Limit state of loss of overall stability
- Verification of slope failure

Limit State 2: Limit state of serviceability
- Excessive deformations that make the structure unusable but do not bring about collapse

For the sheet pile calculation limit state 1B is most important. For this limit state the EAU 2004, in general, recommends:

The following procedure is useful for analyzing the stability for limit state LS 1B:
a) *Firstly, apply the characteristic actions to the chosen structural system and hence determine the characteristic action effects (**e.g. internal forces**).*
b) *Secondly, convert the characteristic action effects with the partial safety factors for actions into design values for action effects, the characteristic resistance to design values for resistance.*

Action or action effects **LS 1B: LS of failure of structures and components**		**LC1**	**LC2**	**LC3**
General permanent actions	γ_G	1,35	1,20	1,00
Hydrostatic pressure in certain boundary conditions	$\gamma_{G,red}$	1,20	1,10	1,00
Permanent actions due to steady-state earth pressure	γ_{E0g}	1,20	1,10	1,00
Unfavorable variable actions	γ_Q	1,50	1,30	1,00

Table 9-1: Partial safety factors for actions and action effects for LS 1B

Resistance LS 1B: LS of failure of structures and components		LC1	LC2	LC3
Soil resistance				
Earth resistance	γ_{Ep}	1,40	1,30	1,20
Earth resistance for determining the bending moment	$\gamma_{Ep,red}$	1,20	1,15	1,10
Ground failure resistance	γ_{Gr}	1,40	1,30	1,20
Sliding resistance	γ_{Gl}	1,10	1,10	1,10
Pile resistance				
Pile compression resistance	γ_{Pc}	1,20	1,20	1,20
Pile tension resistance for test load	γ_{Pt}	1,30	1,30	1,30
Pile resistance in tension and compression based on empirical values	γ_{P}	1,40	1,40	1,40
Grouted anchor resistance				
Resistance of steel tension member	γ_{M}	1,15	1,15	1,15
Pull-out resistance of grout	γ_{A}	1,10	1,10	1,10
Resistance of flexible reinforcing elements				
Material resistance of reinforcement	γ_{B}	1,40	1,30	1,20

Table 9-2: Partial safety factors for resistance for LS 1B

c) *Finally, compare the design values of action effects with the design resistance and show that the limit state equation is complied with for the failure mechanism under investigation.*

It is clear that on the characteristic internal forces safety factors must be applied for permanent action effects and for the effects due to variable actions (for example terrain loads). For the calculation of the minimum toe level it is more difficult to apply safety factors. Especially the application of safety factors on action effects. These will be applied on the equivalent force according to the Blum schematization.

9.1.1 Reduction of safety factor for the hydrostatic pressure

The partial safety factor for hydrostatic actions can be reduced if at least one of the 3 following conditions is satisfied:

1. Verified measurements must be available regarding the dependencies between ground and free water levels, so that these levels guarantee the hydrostatic pressure in calculations for LC1 and LC3.
2. Numerical models of bandwidth and frequency of occurrence of the true water levels lie on the safe side.
3. The geometrical boundary conditions present, limit the water level to a maximum value. Drainage systems do not represent a clear geometrical limit.

9.1.2 Reduction of safety factor for passive earth pressure

For the calculation of bending moments it is possible to reduce the safety factor on the passive earth pressure. The following cases can be distinguished:

1. The reduced partial safety factor can be used if below the calculation bottom (see figure 9-1) the non-cohesive soil exhibit at least a mean strength and cohesive soil exhibit a stiff state. Redistribution of active earth pressure is carried out down to the calculation bottom.
2. Below a level lower than the calculation bottom, soils are present with at least a medium strength, or rather stiff consistency. The reduced safety factor may only be used below this low level, called separating plane. The soft soils between calculation bottom and separating plane may only be applied as surcharge loads on the separating plane. Redistribution of active earth pressure is carried out down to the separating plane.

3. If no reduced partial safety factor is used the active earth pressure redistribution must be continued down to the level of the calculation bottom.
4. If below the calculation bottom only soils of lower strength or consistency are available than the bending moments must be calculated without reduced partial safety factors. The redistribution of active earth pressure is carried out down as far as the calculation bottom.

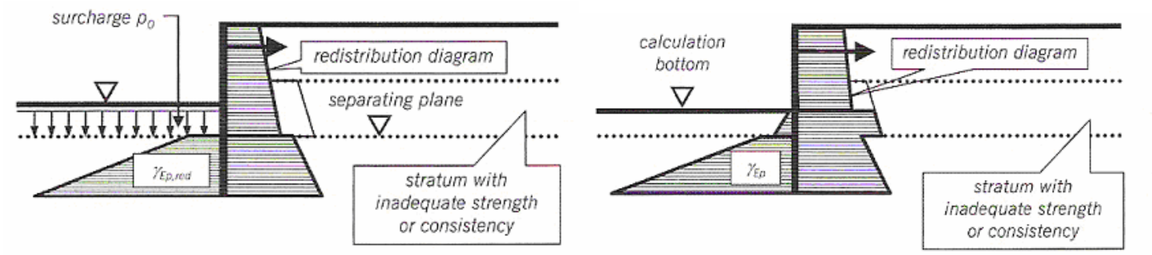


Figure 9-1: Loading diagrams for calculation of the bending moments with (left) and without (right) reduced partial safety factor; a difference in redistribution of active earth pressure is visible

9.2 Loading cases, safety levels and combinations of actions

The EAU 2004 three load cases are determined depending on a safety class and combination of actions.

Combinations of actions:
- CA 1, standard combination: Permanent actions and regularly variable actions during functional lifetime.
- CA 2, rare combinations: Seldom or one-off planned actions apart from the actions of the standard combinations.
- CA 3, exceptional combinations: an exceptional action that may occur at the same time and in addition to the actions of the standard combination

Safety classes
- SC 1, safety class 1: conditions related to the functional lifetime
- SC 2, safety class 2: temporary conditions during the construction or repair of the structure and temporary conditions during building measures adjacent to the structure
- SC 3, safety class 3: conditions occurring once or probably never during the functional life of the structure

Loading cases for Limit State 1
- LC 1, loading case 1 (= CA 1 + SC 1): Permanent design situation
  - Standard combination with conditions related to the functional lifetime
- LC 2, loading case 2 (= CA 2 + SC 1 or CA 1 + SC 2): Temporary design situation
  - Rare combination with conditions related to the functional lifetime
  - Standard combination with temporary conditions during construction or repair
- LC 3, loading case 3 (= CA 3 + SC 2 or CA 2 + SC 3): Exceptional design situation
  - Exceptional combination with temporary conditions during construction or repair
  - Rare combination with conditions occurring once or probably never during lifetime of the structure

Water level situation	Loading case	Load conditions
3a	LC 1	Permanent design situation: ♦ Normal water level fluctuations, no drainage ♦ Active earth pressure ♦ Earth pressure due to crane, live loads and surcharges from dead loads
3b	LC 3	Exceptional design situation: ♦ Extreme low water level, no drainage ♦ Active earth pressure ♦ Earth pressure due to crane, live loads and surcharges from dead loads ♦ Line pull loads (bollard) and fender loads ♦ Unusual scour
3c	LC 3	Exceptional design situation: ♦ Falling high water, no drainage ♦ Active earth pressure ♦ Earth pressure due to crane, live loads and surcharges from dead loads ♦ Line pull loads (bollard) and fender loads ♦ Unusual scour
3d	LC 1	Permanent design situation: ♦ Normal water level fluctuations, with drainage ♦ Active earth pressure ♦ Earth pressure due to crane, live loads and surcharges from dead loads
	LC 2	Temporary design situation: ♦ Normal water level fluctuations, with drainage ♦ Active earth pressure ♦ Earth pressure due to crane, live loads and surcharges from dead loads ♦ Line pull loads (bollard) and fender loads ♦ Restricted scour

Table 9-3: Loading cases interesting for the sheet pile calculation

When comparing these loading cases with the load combinations in Handbook Quay Walls, the LC 1 could be compared with the fundamental load combination (Handbook Quay Walls). The LC 3 gives combinations which occur in special situations, this could be compared with the special load combinations (Handbook Quay Walls).

9.3 Active earth pressure redistribution (in German: "Umlagerung")

The active earth pressure can be redistributed due to deformations in the wall and the forming of a vault. The active earth pressure can be redistributed over a height H_e. Three diagrams for a "trenching in front of the wall method" are determined depending on the relation between anchor support - distance between top and the calculation bottom. There are also 3 diagrams available for a "backfilling behind the wall method".

The diagrams of cases 1 to 3 are **only valid for the condition that the earth pressure can redistribute to the stiffer areas as a result of adequate wall deformation**. A horizontal earth pressure vault therefore occurs between the anchor and the soil support. The diagrams 1 to 3 may not be used if:

1. A sheet pile wall is backfilled to a large extent between bottom of watercourse and anchor, and sub sequent excavations in front of the wall are not so deep that additional deflections takes place.
2. Cohesive soils behind the sheet pile wall which are not yet sufficiently consolidated
3. A wall with increasing rigidity does not exhibit the wall deflections necessary for forming a vault, e.g. in reinforced concrete diaphragm walls.

If loading diagram 1 to 3 (of the trenching in front of the wall method) are not valid, then use loading diagrams 4 to 6. If the anchor support is to low, the earth pressure diagrams must be determined separately.

$$e_m = e_{ahm,k} = \frac{E_{ah,k}}{H_E}$$

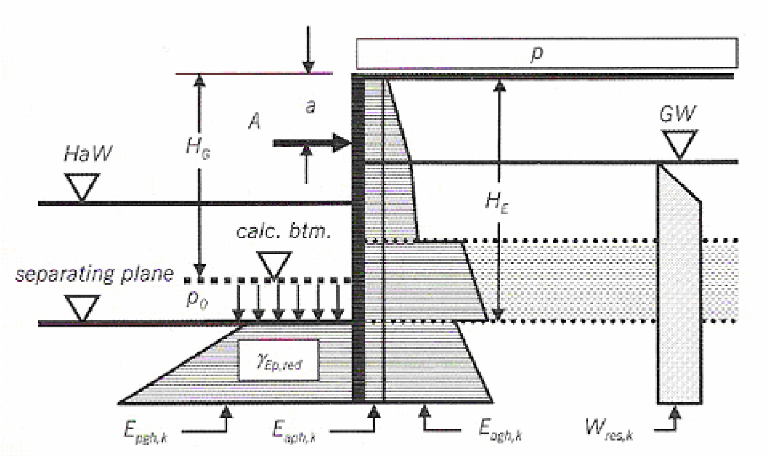


Figure 9-2: Load diagram for the redistribution of active earth pressure over a height H_E

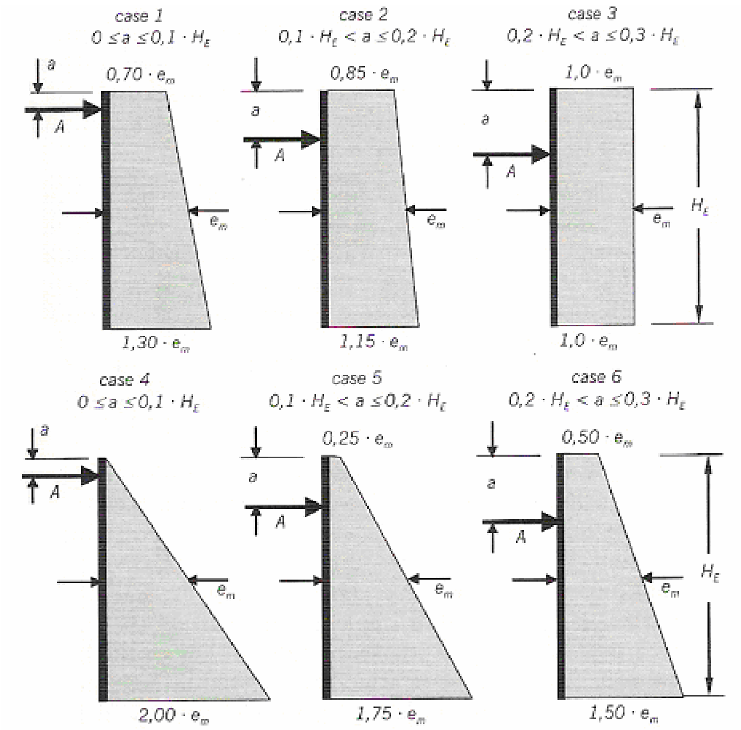


Figure 9-3: Earth pressure redistribution, (cases 1,2,3) trenching method in front of the wall, (cases 4,5,6) backfill behind the wall

9.4 Determining the toe level, ULS 1B

9.4.1 Minimum (partial fixed) toe level

The minimum toe level in EAU 2004 is determined with the Blum method. In this method the equivalent force "C" at the toe of the wall (according to Blum) will be increased with safety factors for action effects for permanent and variable loads.

$$C_{h,d} = \sum \left(C_{Gh,k} \cdot \gamma_G + (C_{Gh,W,k} \cdot \gamma_{G,red}) + C_{Qh,k} \cdot \gamma_Q \right)$$

The resistance will be delivered by the extra length (Δt_1) under the equivalent force "C". A safety factor will be applied on the force determined by this extra length (see figure 9-4). The necessary extra length can be calculated in this way.

$$e_{phC,k} = \sigma_{z,C} \cdot K_{phg,C} (+c'_k \cdot K_{phg,C})$$
$$E_{phC,k} = \Delta t_1 \cdot e_{phC,k}$$
$$E_{phC,d} = \frac{E_{phC,k}}{\gamma_{Ep}}$$
$$C_{h,d} \leq E_{phC,d}$$
$$\Delta t_1 \geq \frac{C_{h,d} \cdot \gamma_{Ep}}{e_{phC,k}}$$

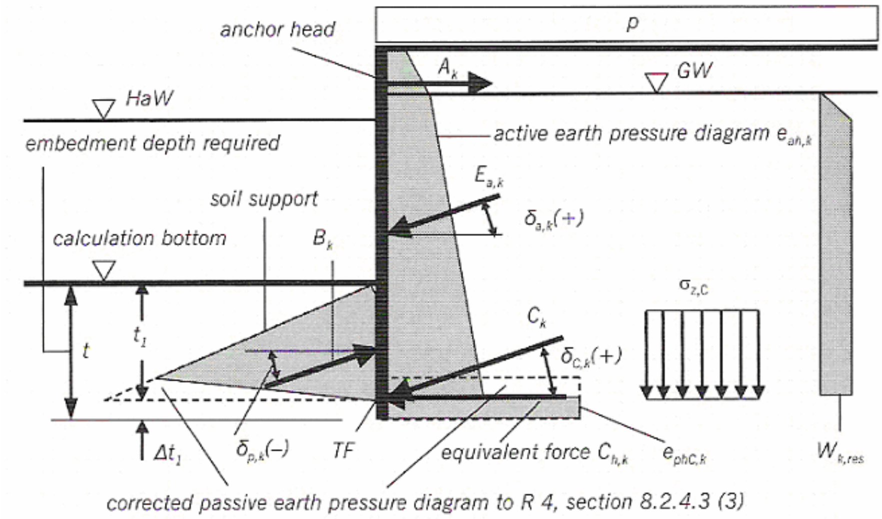


Figure 9-4: Loading diagram for the calculation of the minimum toe level

The EAU 2004 recommends **at least a partial fixity** in the soil. However, a wall with simple support as well as fixed support in the soil can be calculated. A simply supported wall has no extra length (Δt_1) . This approach is based on the tangent angle of the line of bending. The ε is the end tangent angle of the line of bending for the theoretical toe (t_1 in figure 9-4) and the end tangent max ε for a simple earth support.

$$t = t_1 + \Delta t_1$$
$$\Delta t_{MIN} = \frac{\frac{\tau_{1-0}}{100} \cdot t_{1-0}}{10}$$
$$\tau_{1-0} = 100 \cdot (1 - \varepsilon / \max \varepsilon)[\%]$$

Also technical, operational and economical requirements are important factors for the minimum toe level in addition to the structural calculations. This results usually in a partial fixity of the sheet pile wall.

<u>Application in a beam on elastic foundation model</u>

For a beam on elastic foundation method it is only possible to calculate the minimum toe level corresponding with a Blum schematization. Blum requires fully developed active and passive earth pressure. For larger toe levels, the springs in the beam on elastic foundation method can also be in an elastic state.

9.4.2 Staggered toe level for steel sheet piling

In large quay walls it often occurs the staggered toe levels are used along the wall for economic reasons. The EAU 2004 uses the Blum method to calculate the staggered toe levels. An usual length for staggering, as described in the EAU 2004, is 1,0 m. For a staggering of 1 m in practice the longer piles don't have to be checked. For larger staggering the load bearing capacity of the longer piles have to be checked with respect to multiple stresses due to bending moments combined with longitudinal and shear forces.

Sheet pile walls fixed in the soil

For walls fully fixed in the soil the total staggering (1m) can be applied to save steel. The longer piles are driven to the theoretical toe level calculated for a fixed wall, the shorter piles stop at a higher level s (≤ 1m). For walls partially fixed in the soil the saving of steel depends on the degree of fixity. The longer piles must be driven below the theoretical toe level for a partially fixed wall by a certain factor of the stagger dimension s_U, the shorter piles stop at a higher level s(≤ 1m).

$$s_U = \frac{(100 \cdot \tau_{1-0}) \cdot s}{(2 \cdot 100)}$$

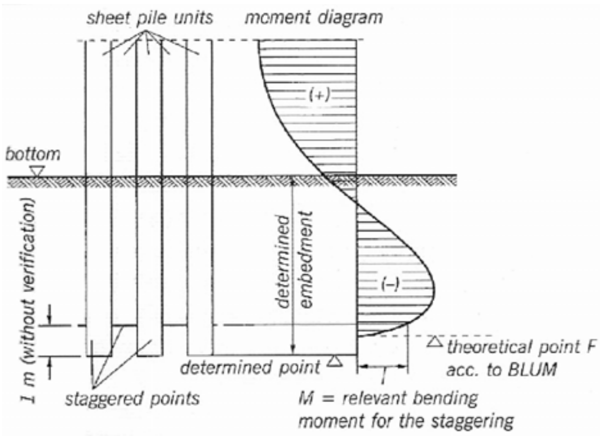


Figure 9-5: Staggered toe level for a wall fixed in the soil

Sheet pile walls free supported in the soil

For staggering with a wall simply supported in the soil the longer sheet piles must be driven below the theoretical toe of the wall:

$$s_U = \frac{s}{2}$$

If the stagger dimension is larger than 1 m, the load bearing capacity of the piles must verified.

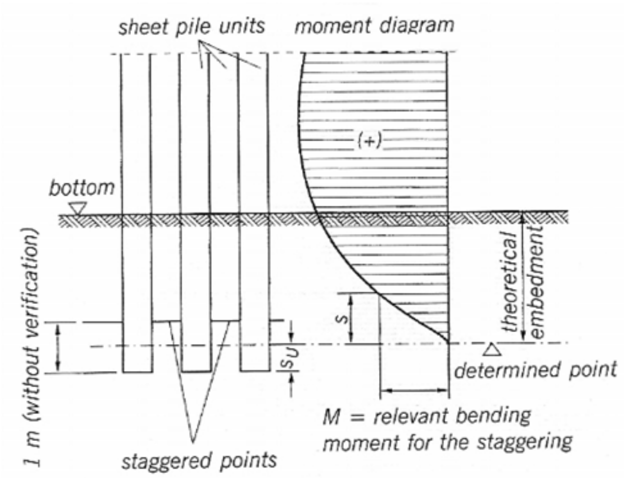


Figure 9-6: Staggered toe level free supported in the soil

Combined sheet piling

Sheet piling composed of primary and secondary piles must take into account the water pressure difference present for flows around the wall for safety against hydraulic heave in front of the shorter secondary piles.

Application in a beam on elastic foundation model

The staggering length for a combined wall is in most cases larger than 1 m. Extra verifications have to be carried out. In a beam on elastic foundation model the staggering length is included in the stiffness of the wall. Because the difference in stiffness between primary and secondary piles is very large, the minimum toe level will be determined for the primary piles and the staggering can be neglected.

9.5 Check for moments and anchor forces, ULS 1B

The calculation of the bending moment can be done with a reduced safety factor for the passive earth pressure resistance. This depends on the soil conditions as explained before (paragraph 9.1.2). The safety factors for actions must be applied on the effects of the permanent actions (due to active earth and hydrostatic pressure), added with the contribution of the moment due to the variable active earth pressure (active earth pressure due to for example terrain loads).

$$M_d = \sum \left(M_{G,k} \cdot \gamma_G (+M_{G,W,k} \cdot \gamma_{G,red}) + M_{Q,k} \cdot \gamma_Q \right)$$

The anchor force should be calculated in the same way as the bending moment. However, the reduced safety factor on the passive earth pressure is not applicable for anchor forces.

$$A_d = \sum \left(A_{G,k} \cdot \gamma_G + A_{G,W,k} \cdot \gamma_{G,red} + A_{Q,k} \cdot \gamma_Q \right)$$

9.6 Vertical wall analysis, ULS 1B

The characteristic vertical action effect must be at least equal to the upward vertical component of characteristic soil support that is mobilized. The characteristic upward overall forces of the soil support reaction includes the layers down to the theoretical toe (= minimum toe level), including a correction term. The angle of the active and passive earth pressure due to the angle of inclination must be realistic, because of the great importance of horizontal and vertical pressure on the wall. For the vertical direction the following equilibrium conditions must be satisfied:

- Vertical equilibrium
- Vertical load bearing capacity

Angle of inclination

For the angle of inclination the values are limited:

- Active earth pressure, straight slip planes

  $$-\tfrac{2}{3} \cdot \varphi'_k \le \delta_{a,k} \le +\tfrac{2}{3} \cdot \varphi'_k$$

- Passive earth pressure for angle of internal friction $\varphi'_k \le 35^\circ$, straight slip planes

  $$-\tfrac{2}{3} \cdot \varphi'_k \le \delta_{p,k} \le +\tfrac{2}{3} \cdot \varphi'_k$$

- Passive earth pressure for angle of internal friction, curved slip planes

  $$-\varphi'_k \le \delta_{p,k} \le +\varphi'_k$$

Influence of vertical forces on the angle of inclination

The downward vertical forces on the wall cause relative displacements. If at the same time another relative displacement occurs, different from normal deformations of sliding bodies for active and passive earth pressure, a different angle of inclination is necessary. This must be taken into account for the analysis of the vertical equilibrium and the vertical load bearing capacity:

- Reduction of the angle at the lower passive side leads to increase of the mobilized passive soil support B_k.
- The change in the direction of actions has no influence on the theoretical value C_{BLUM}.
- Reduction of angle of inclination leads to increase of the active earth pressure force.
- With large vertical loads on the wall a negative active earth pressure angle can be used.

Upward vertical forces in the wall due to for example anchor forces give change to the relative displacements. Then a reduction of active earth pressure can be applied with a limited positive angle of inclination. It is also necessary to vary the angle of inclination due the passive earth pressure to achieve vertical equilibrium.

For the vertical equilibrium the actions working on the wall and causing vertical loads (V_k), must be at least as large as the upward force caused by the characteristic soil support ($B_{v,k}$) accommodated by the passive earth pressure.

$$V_k = \sum V_k \ge B_{v,k}$$

9.6.1 Vertical load bearing capacity: Axial sinking

The wall must be driven deep enough to prevent from the axial sinking.

$$V_d = \sum V_{d,i} \ge R_{1,d}$$

The vertical forces working on the wall should be applied with safety factors:

- Actions at the top of the wall

  $$V_{F,d} = \sum (V_{F,G,k} \cdot \gamma_G + V_{F,Q,k} \cdot \gamma_Q)$$

- Vertical anchor force component

  $$V_{Av,d} = \sum (V_{Av,G,k} \cdot \gamma_G + V_{Av,Q,k} \cdot \gamma_Q)$$

- Active earth pressure down to the depth of the theoretical toe

  $$V_{Eav,d} = \sum (V_{Eav,n,G,k} \cdot \gamma_G + V_{Eav,n,Q,k} \cdot \gamma_Q)$$

- Vertical component as a result of equivalent force C

  $$V_{Cv,d} = \sum (V_{Cv,G,k} \cdot \gamma_G + V_{Cv,Q,k} \cdot \gamma_Q)$$

Enough toe and skin resistance must be accommodated to resist against axial sinking.

$$R_{1,d} = \frac{\sum R_{1k,i}}{\gamma_P}$$

9.7 Verification of stability of the anchor system, ULS 1B

9.7.1 Verification of horizontal bearing capacity for anchor walls

A distinction is made between anchor walls loaded in the centre of the wall and anchor walls who are loaded eccentric. The wall loaded eccentric gives usually a longer wall length, because it can be corresponds with an sheet pile wall with single anchor, which is partially fixed in the soil.

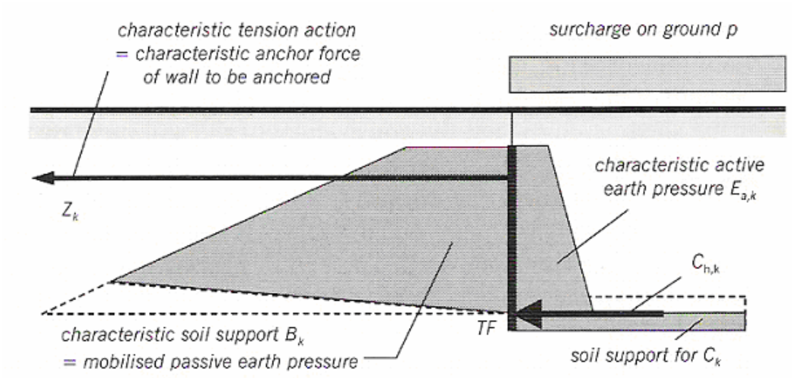


Figure 9-7: Stability of anchor wall eccentrically loaded

9.7.2 Verification of steel anchor cable/tie rod

Steel cable anchors
This type of anchor is only used for static loads. The strength of the cable must be calculated with the failure force of the anchor reduced by a partial safety factor.

$$N_d = \frac{N_{failure}}{\gamma}$$

Tie rod
A distinction is made between static and dynamic loads. For the static calculation the design value for the anchor force of permanent and variable loads must be higher than the resistance of the anchor, determined by minimum strength of shaft and core of the tie rod.

9.7.3 Verification of stability of the deep failure plane for anchor walls and tension piles

The verification of the lower failure plane for the anchoring is done with the Kranz method. For this method a distinction is made between permanent and variable loads. First the permanent loads must be checked and then the permanent and variable loads together. The possible characteristic anchor force must be decreased with partial safety factor for passive earth pressure.

$$A_{G,k} \cdot \gamma_G (+A_{Q,k} \cdot \gamma_Q) \leq \frac{A_{poss,k}}{\gamma_{E_p}}$$

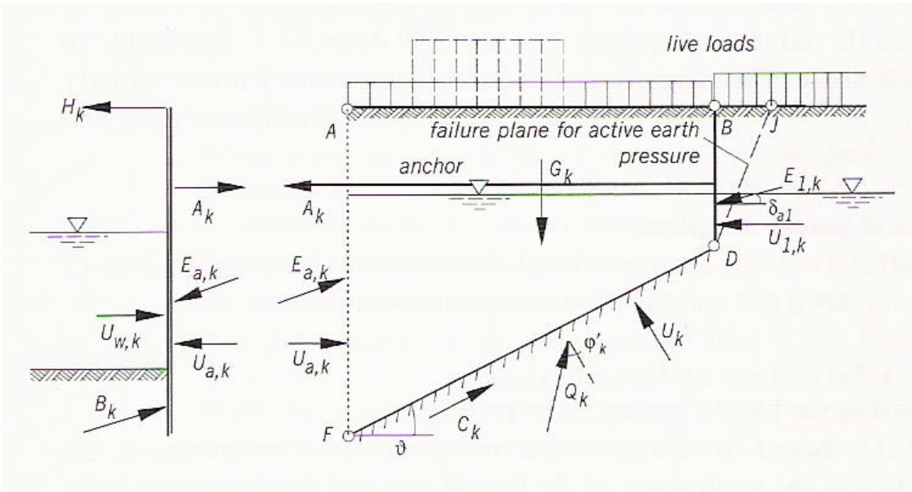


Figure 9-8: Verification of anchor stability with Kranz-method

As alternative for anchor walls, retaining walls can be secured with anchors transferring the tensile forces into the soil by skin friction. A similar verification for stability of the deep failure plane is possible by considering a vertical section through the soil as an equivalent anchor wall, in the centre of the force-transfer-length (l_r) of the anchor.

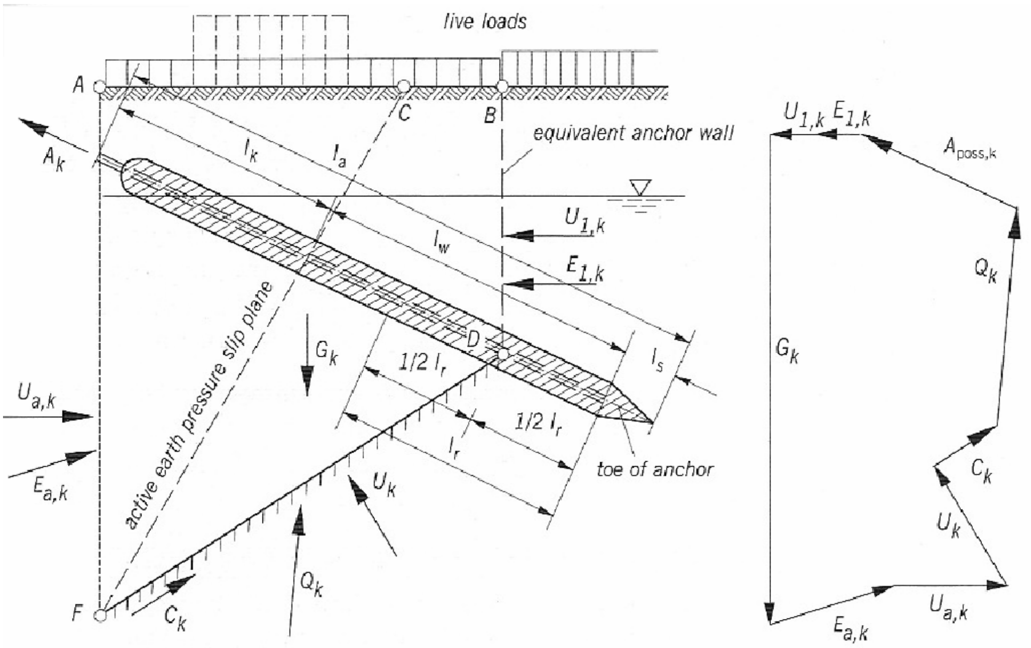


Figure 9-9: Failure mechanism for anchor pile (corresponds with a Kranz verification)

9.8 Stability of the bottom against hydraulic heave, ULS 1A

9.8.1 Hydraulic heave

In the EAU 2004 this is described as failure in a body of earth (permeable soil) in front of a structure loaded by the upward flow. The flow force depends on the water level gradient over a certain layer. Two methods are available to check this mechanism:

- Terzaghi-Peck method: The flow force (S'_k) in a certain body of earth must be smaller than the value of the weight of the body (G'_k) under uplift

$$S'_k \cdot \gamma_H \leq G'_k \cdot \gamma_{G,stb}$$

$$S'_k = \frac{\gamma_w (h_1 + h_r)}{2} \cdot \frac{t}{2}$$

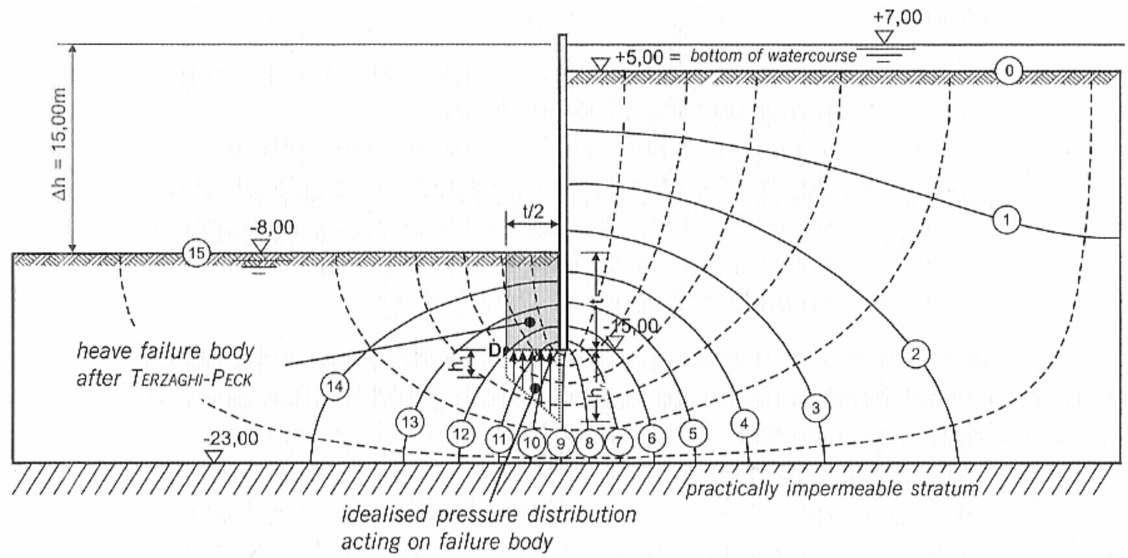


Figure 9-10: Hydraulic heave by Terzaghi-Peck method

- Baumgart-Davidenkoff method: The hydraulic gradient times the density of the water in a canal just next to the wall must be smaller than the density of the submerged soil weight.

$$(\gamma_w \cdot i) \cdot \gamma_H \leq \gamma' \cdot \gamma_{G,stb}$$

$$i = h_r / t$$

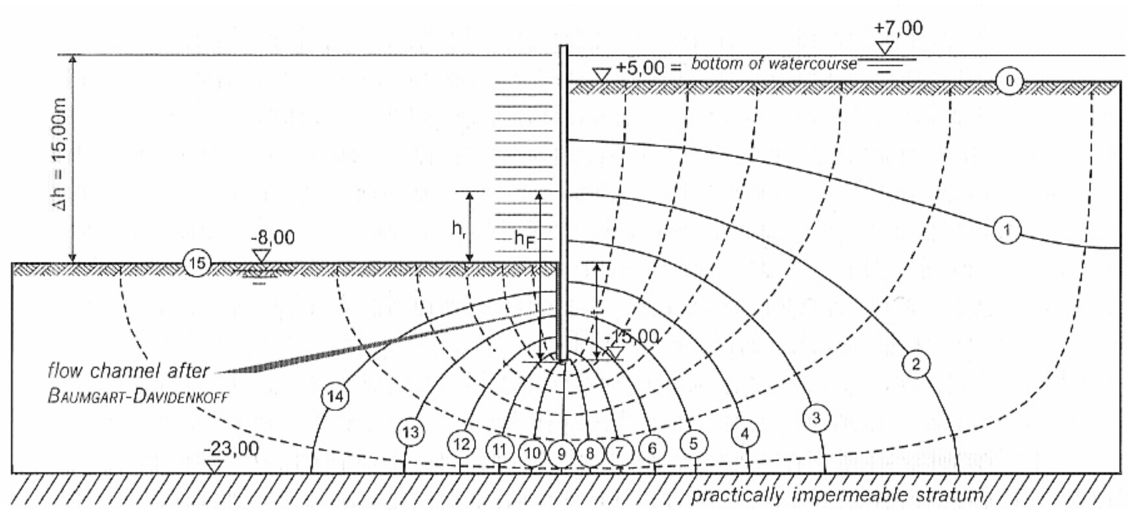


Figure 9-11: Hydraulic heave by Baumgarten-Davidenkoff method

9.8.2 Piping

No limit state is given for piping, but it is also a problem with the stability of the soil due to hydraulic flows, as described for hydraulic heave. It is mentioned that hardly any piping will occur in soils with impermeable layers.

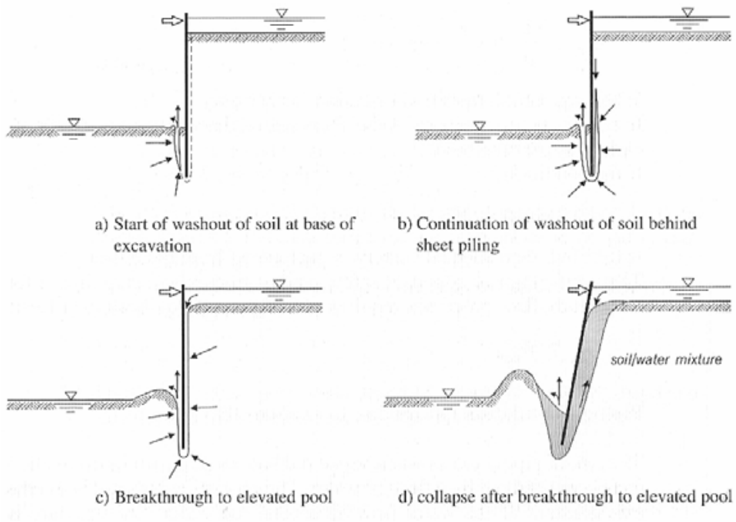


Figure 9-12: The mechanism of piping

9.9 Verification ULS 1C

Slope failure is the failure mechanism that is treated in limit state 1C. This is not the case for the sheet pile wall which will be used for the comparative analysis.

It must be argued that the stability of the sheet pile structure with passive and active earth pressures must be calculated in limit state 1B according EAU 2004. In older versions of the EAU this mechanism was treated for limit state 1C.

10 Conclusions and recommendations

From the comparative analysis, described in the chapters before, it can be concluded that there are 4 main points in this study, from which conclusions can be drawn:

- Available recommendations for quay wall design
- Determination of characteristic values
- Design philosophy
- Design results after application of the design philosophy

10.1 Available design guidelines for quay walls

There are 9 design guidelines collected that describe the design of quay wall structures or a part of it. Only 4 of these guidelines, which are all of European origin, are adapted to the latest semi-probabilistic design philosophy. Three of these guidelines can be used for further research: Handbook Quay Walls, CUR 166 and EAU 2004. The Eurocode is adapted to the latest probabilistic design approach, but the collection of national parameters is not yet finished. Therefore the Eurocode is not used for further research. It is remarkable that only literature that is written in English is of international importance.

R 1. If a guidelines should be of international importance, it must be written in English

10.2 Determination of characteristic values

Characteristic values for a quay wall design have to be determined for geometry properties, water levels, soil properties, material properties and external loads. In this study the geometrical properties are kept the same. In the next paragraphs the determination of the characteristic values has been compared according to the guidelines Handbook Quay Walls, CUR 166 and EAU 2004.

10.2.1 Water levels

The Handbook Quay Walls gives the largest water level difference for the design of a quay wall in the Port of Rotterdam. This is due to the fact that the free water level is based on a reference period of 50 years and that the high groundwater level is derived from tidal fluctuations in the free water. The CUR 166 gives the lowest water level difference. The water level situation in CUR 166 can be compared with water level situation 3a of the EAU 2004.

10.2.2 Soil properties

The soil properties, given for national unfavorable circumstances, from tables in the CUR 166 and EAU 2004 do not differ a lot. The unit weight is almost similar for the investigated soil layers. A remarkable difference occurs for the shear strength, which shows higher values for clay layers in the CUR 166. Most remarkable are the higher angles of internal friction in layers determined with the EAU 2004, especially the clay layers.

R 2. Use national guidelines for the determination of soil properties in a certain country

For the application of a beam on elastic foundation program the modulus of sub-grade reaction is necessary. The CUR 166 gives a table with a high and low modulus of sub-grade reaction and the Handbook Quay Walls takes the mean of these values. The low value of CUR 166 is in most cases normative. The EAU 2004 does not describe the modulus of sub-grade reaction, which makes the design approach of the EAU 2004 difficult for application in a beam on elastic foundation program.

R 3. Use CUR 166 for a detailed description of applying the modulus of sub-grade reaction in a beam on elastic foundation program

10.2.3 Wall and anchorage properties

The wall and anchorage properties are kept the same for the comparative analysis. They are chosen rather strong, so that they will satisfy for most loading conditions in the comparative analysis.

Oblique bending
Oblique bending is described in detail in CUR 166. This will not lead to large reductions of the section modulus for the cases in the comparative analysis, especially not when combined walls, Z-shaped profiles or triple U-shaped profiles are used.

R 4. Use CUR 166 for a detailed description of oblique bending

Corrosion
Handbook Quay Walls gives somewhat higher corrosion speed than the EAU 2004. The CUR 166 gives values that are 3 times lower than Handbook Quay Walls. From research of the Public Works of Rotterdam it is known that the corrosion speeds in reality are much higher, so that also the corrosion speed of Handbook Quay Walls is not sufficient.

R 5. Use even higher corrosion speeds than the corrosion speeds that are available in the Handbook Quay Walls

10.2.4 External loads

For external loads, as terrain, crane, bollard and fender loads, tables with characteristic values are available in the design guidelines. The terrain and crane loads on top of quay walls are most up to date in Handbook Quay Walls. EAU 2004 gives the best description of fender loads and CUR 166 gives most information about bollard loads.

10.3 Design philosophy, fault trees, failure mechanisms and safety factors

In the semi-probabilistic design philosophies the application of partial safety factors on actions, action effects and resistance are described. These safety factors are mainly based on safety levels, fault trees and failure mechanisms.

10.3.1 Safety levels

In CUR 166 and Handbook Quay Walls is described on which safety levels the calculation of the structure is based, together with the associating probabilities of failure. The permitted probability of failure for a quay wall in CUR 166 is lower than the probability of failure in the Handbook Quay Walls. In EAU 2004 this is not described. Therefore it is difficult to compare this design approach with other approaches.

R 6. For a clear design approach safety levels with associating probabilities of failure must be available

10.3.2 Fault trees and failure mechanisms

Fault trees, available in Handbook Quay Walls and CUR 166, give a clear overview of the failure mechanisms that must be treated. This is missing in the EAU 2004. It is therefore difficult to see which mechanisms must be treated in EAU 2004 and to which limit states the mechanisms belong.

R 7. It is important that fault trees are available in a design guideline to make clear which failure mechanisms must be treated

Fault trees can also be useful for the new Eurocode 7, especially for the addition of national parameters. With a fault tree it is possible to compare national parameters and show on which failure mechanisms the partial safety factors are based.

R 8. For Eurocode 7 it is recommended to include fault trees to get a clear addition of national parameters

10.3.3 Partial safety factors

The partial safety factors given in the guidelines are factors on actions, action effects and resistance. Partial safety factors can also be applied on the retaining height or harbor bottom, which in general will lead to a lowering of the harbor bottom. This gives an increase of actions or a decrease of resistance.

Safety factors on:	Actions	Action effects	Resistance
Handbook Quay Walls		×	×[1]
CUR 166	×		×
EAU 2004		×	×

[1] only limits the passive resistance for the calculation of the minimum embedment depth

Table 10-1: Application of safety factors on actions, action effects and resistance

The CUR 166 uses partial safety factors on actions and resistance for the calculation of the minimum toe level as well as for the internal and anchor forces. These safety factors are applied on the soil strength parameters and realize a decrease of the soil strength.

The Handbook Quay Walls does not use safety factors for the calculation of the minimum toe level, it only limits the mobilized passive earth pressure. For the calculation of internal and anchor forces the partial safety factors are applied on action effects. These internal and anchor forces will be increased with a certain factor. For the anchor force extra safety factors are available. Applying the safety factors only on action effects keeps the position of the maximum moment at the same height on the wall, as in the characteristic situation.

R 9. Application of safety factors on action effects keeps the maximum bending moment, calculated for design and characteristic values, at the same height on the wall

The EAU 2004 applies safety factors on resistance and action effects. The resistance is represented by the passive earth pressure. It is difficult to apply safety factors on the earth pressure coefficients in a beam on elastic foundation program. For the calculation of the minimum (partially-)fixed toe level the action effects are represented by the equivalent force according to the Blum schematization. For the calculation of internal and anchor forces the action effects are represented by the internal and anchor forces. For the action effects also permanent and variable contributions are distinguished.
The reduction of moments due to the forming of a vault is different for the EAU 2004 version in comparison to the older versions. In the EAU 2004 redistribution of active earth pressure can take place dependent on the construction method.
Under certain soil conditions the partial safety factors can be reduced. These reductions make it difficult to distinguish the semi-probabilistic approach from the experience. The application of partial safety factors looks like a black box in this way.

R 10. Prevent the application of safety factors to be a black box model by making a clear distinction between probabilistic design and design from experience

10.4 Comparative design calculations for case 1 and 2

These comparative calculations are carried out with the design methods of the three semi-probabilistic guidelines: Handbook Quay Walls, CUR 166 and EAU 2004. Two cases are considered: sheet pile wall with retaining height of 12 m (case 1) and combined sheet pile wall with retaining height of 30 m, including a superstructure (case 2).

10.4.1 In general about the calculations

The calculation results depend on a lot more aspects than only the different safety factors. Especially load combinations, water level difference and difference in soil properties between Dutch and German recommendations have major influence. It must be argued that many of these safety factors, water levels and soil properties belong to each other in a specific safety approach and that they are not applicable in other safety approaches.

R 11. Do not use safety factors from different safety approaches, because this can cause designs with too large or too little safety

The results calculated for cases 1 and 2 can only be compared to each other. It is not investigated how the system reacts in reality.

R 12. Extra research must be done to compare the comparative calculation results to the reality

10.4.2 Minimum toe level

The CUR 166 and Handbook Quay Walls give similar minimum toe levels, not more than 5% difference between for cases 1 (12 m retaining height). The EAU 2004 approach does not describe a method of calculating minimum toe levels.

The embedment depth for the minimum toe levels is very small for a quay wall structure, in comparison to walls in a building pit. This is due to the small water level difference for quay walls.

R 13. For sheet pile walls in building pits further researches should be done to show the effect of larger water level differences

10.4.3 Maximum bending moment

The EAU 2004 gives in both cases 1 and 2 the smallest maximum bending moments. However, it does not satisfy the Blum schematization, because the modulus of sub-grade reaction from CUR 166 is used. It is not possible to make a Blum schematization for embedment depths larger than the minimum depth. The displacements are too small for fully developed active and passive earth pressure conditions, which is a basic condition for a Blum schematization.

R 14. Do not use the modulus of sub-grade reaction defined in CUR 166 in combination with safety factors from EAU 2004

For a quay wall with a retaining height of 12 m the CUR 166 gives 20 % smaller maximum bending moments than Handbook Quay Walls. The special load situation with extreme scour and larger water level difference for Handbook Quay Walls causes this difference.
For a quay wall with a retaining height of 30 m in combination with a superstructure (case 2) the Handbook Quay Walls gives 20 % smaller bending moments than CUR 166. The relieving platform causes a large increase of moments for safety factors on soil strength parameters in comparison to safety factors on action effects.

R 15. For quay walls with superstructures the Handbook Quay Walls gives more economic maximum bending moments than CUR 166

The second order moment gives a small increase of the moments (± 5%). The application of an eccentricity, by placing the axial load not in the centre of the profile, causes major reductions in bending moment. However, an eccentricity is not taken into account in this analysis.

R 16. Apply an eccentricity, in case of an axial load on top of the wall, to obtain large reductions of the maximum bending moment

10.4.4 Anchor forces for anchor profile

The EAU 2004 gives in both cases 1 and 2 the smallest anchor forces, but does not satisfy the Blum schematization, as already is explained for the maximum bending moment.

The Handbook Quay Walls has anchor forces similar (maximum difference 5%) to the anchor forces of CUR 166 for the calculation of the anchor profile. This holds for anchor forces calculated for case 1, retaining height 12 m, and for case 2, retaining height 30 m including superstructure.

R 17. Handbook Quay Walls and CUR 166 give similar anchor forces

10.5 Which guideline is useful for designing quay walls

10.5.1 Most useful design guideline per design aspect

An overview is given of the design guidelines that are most useful for the design of a quay wall with sheet pile wall in the Port of Rotterdam, calculated in a beam on elastic foundation program:

- Free and groundwater level → **Handbook Quay Walls**
  ⇒ Gives the highest water level difference and uses clear safety factors for ground and free water

- Soil properties → **CUR 166** and **Handbook Quay Walls**
  ⇒ Give the best overview of the determination of soil properties that are only dependent on the cone penetration test

- Wall and anchor parameters
  - Eccentricity/2nd order effect → **Handbook Quay Walls**
    ⇒ Describes a detailed calculation method
  - Oblique bending → **CUR 166**
    ⇒ Gives the most detailed description
  - Corrosion → **NON**
    ⇒ Handbook Quay Walls gives the largest values which are even too low

- External loads
  - Terrain loads → **Handbook Quay Walls**
    ⇒ Gives the most detailed data of loads for goods and containers
  - Crane loads → **Handbook Quay Walls**
    ⇒ Gives specific values per type of crane
  - Bollard loads → **CUR 166**
    ⇒ Gives an overview for sea and inland navigation vessels
  - Fender loads → **EAU 2004**
    ⇒ Descriptions given about the amount of energy that must be absorbed

- Minimum toe level → **Handbook Quay Walls** or **CUR 166**
  ⇒ EAU 2004 does not describe a clear method for the calculation of the minimum toe level

- Internal forces and anchor force → **Handbook Quay Walls** and **CUR 166**
  ⇒ Handbook Quay Walls includes special load combinations
  ⇒ Both guidelines are easy to apply in a beam on elastic foundation program
  ⇒ Handbook Quay Walls gives the best result for quay walls with superstructure
  ⇒ CUR 166 gives the best description of the normative load situation with modulus of sub-grade reaction

- Superstructure → **Handbook Quay Walls**
  ⇒ Describes the schematization of the superstructure and calculation process in detail

- Other failure mechanisms → **Handbook Quay Walls** or **CUR 166**
  ⇒ In the EAU 2004 it is not very clear which mechanisms must be treated due to a missing fault tree

10.5.2 Most useful design guideline

For good design results the safety factors of only 1 design philosophy should be chosen to prevent for using safety factors from different philosophies. It is **Handbook Quay Walls** that is argued in most design aspects mentioned above. Therefore this design guideline will be recommended and can probably be seen as the **"most useful" design guideline**. It is easy to apply in a beam on elastic foundation program; it deals with fundamental and special load combinations; it gives a clear description of the calculation of a superstructure; it only applies safety factors on action effects; and a well defined explanation of the design philosophy is available.

10.6 Further research

The comparative analysis is done during several months. In this time new recommendation could be available. Sometimes only parts of recommendation series are overviewed. Extra research should be carried out for this extra or new literature.

The design calculations are done under several simplifications which make the calculations easier. Aspects as construction, costs, arching, effect of other failure mechanisms than the failure of the sheet pile and the changing of the angle of inclination are of major importance. Also the application of other earth pressure schematization models than the beam on elastic foundation model can give different results. It could be interesting to see results in a finite element method, because in this method the second order effect, eccentricity and changing of angle of inclination due to axial loads on the wall are included. For the EAU 2004 another calculation model (GGU Retain) is available. This model is based on Blum and it is very interesting to overview the calculation results in this model in comparison with the results from the calculations done in a beam on elastic foundation program.

A Reference list

A.1 Literature

[1] CUR-publication 211, *Handboek Kademuren* | ISBN 90 3760 282 7 | CUR, Gouda, the Netherlands, 1st issue, 2003
[2] CUR-publication 166, *Damwandconstructies* | ISBN 90 376 0031 X | CUR, Gouda, the Netherlands, 3rd issue, 1997
[3] CUR-publication 166, *Damwandconstructies* | ISBN 90 376 063 8 | CUR, Gouda, the Netherlands, 4th issue, 2005
[4] EN 1997-1:2004, *Eurocode 7:Geotechnical Design – Part 1: General Rules* | CEN, Brussel, Belgium, 2004
[5] Seters, A.J. van, *Calibratiestudie Opstellen Nationale Bijlage Eurocode 7 – Geotechnical Design* | Fugro Ingenieursbureau B.V., Leidschendam, the Netherlands, 2005
[6] BS 6349 Part 1 & 2, *Code of Practice for Maritime Structures* | ISBN 0 580 33169 5 | British Standards, United Kingdom, 1999 – 2003
[7] EAU 1996, *Recommendations of the Committee for Waterfront Structures, Harbors and Waterways* | ISBN 3 433 01790 5 | Ernst & Sohn, Berlin, Germany, 7th edition, 2000
[8] EAU 2004, *Recommendations of the Committee for Waterfront Structures, Harbors and Waterways* | ISBN 3 433 01666 6 | Ernst & Sohn, Berlin, Germany, 8th edition, 2006
[9] ROM 0.2-90, *Actions in the Design of Maritime and Harbor Works* | Dirección de Puertos y Costas, Madrid, Spain, 1990
[10] OCDI, *Technical Standards for Port and Harbor Facilities in Japan* | The Overseas Coastal Area Development Institute of Japan, Tokyo, Japan, 1991
[11] MIL-HDBK-1025/4, *Military Handbook for Seaheads, Bulkheads, Quays* | Naval Facilities Engineering Command (NavFac), USA, 1988
[12] Engineer Manual 1110-2-2504, *Design of Sheet pile Walls* | U.S. Army Corps of Engineers, Washington, USA, 1994
[13] CUR-publication 190, *Probability In Civil Engineering Part 1: The Theory of Probabilistic Design*, course CT4130 | ISBN 90 376 0102 2 | CUR, Gouda, the Netherlands, 1997
[14] Baars, S. van, Kuijper, H.K.T., e.a., *Handboek Constructieve Waterbouwkunde*, course CT3330| Delft, the Netherlands, 2003
[15] PIANC-publication MarCom Working group 50, *General Principals for the Design of Maritime Structures* | PIANC, Germany, 2005
[16] Blum, H., *Einspannuingsverhaltnisse bei Bohlwerken* | Wilhelm Ernst & Sohn, Berlin, Germany, 1931
[17] HMB, *Hydro Meteo Information,* volume 3 | Port of Rotterdam, Rotterdam, the Netherlands, 2nd issue 2004
[18] Report nr. 87-174/F, *Interpretatie onderzoeksresultaten zeekade, bewerkingkade and baggeren haven* | Public Works of Rotterdam, Rotterdam, the Netherlands, 1988
[19] Aanbesteding kadeconstructie Euromax, *Ontwerpnota aanbieding 2* | Delta Marine Consultants, Gouda, the Netherlands, 2005
[20] Project HH946, *Programma van Eisen Euromax* | Public Works of Rotterdam, Rotterdam, the Netherlands, 2005
[21] Project HH731, *Euromax grondonderzoek* | Public Works of Rotterdam, Rotterdam, the Netherlands, 2004

A.2 Websites

For literature:

[22]	www.nen.nl	Dutch Normalization Institute
[23]	www.cenorm.nl	Comité Européen de Normalisation
[24]	www.cur.nl	Civil Engineering Centre for Construction Research and Legislation
[25]	www.wsv.de/pianc	International Navigation Association
[26]	www.usace.army.mil	United States Army Corps of Engineers
[27]	www.wbdg.org	Whole Building Design Guide, publicities van NAVFAC
[28]	www.bbri.be	Centre for science and technique for building companies

For designing:

[29]	www.sheet-piling.arcelor.nl	Arcelor
[30]	www.spundwand.de	Hoesch
[31]	www.nedeximpo.nl	NEDEXIMPO B.V.
[32]	www.liebherr.com	LIEBHERR
[33]	www.kalmarind.nl	Kalmar
[34]	www.spannstall.ch	SpannStahl
[35]	www.fentek.nl	Fentek

In general :

[36]	www.library.tudelft.nl	Library TU Delft
[37]	www.google.nl	Google search
[38]	www.world.altaviste.com/tr	Altavsita Babel Fish translator

B Comparison of contents

G = Good, good information and/or good design rules and diagrams (**G+** = very good)
A = Adequate, adequate information and/or some design values and design rules
B = Basis, basis information and/or only design values
D = Only descriptive
\- = No description

	Subjects	**CUR 211**	**CUR 166**	**Eurocode 7-1**	**BS 6349 - 1 & 2**	**EAU 2004**	**ROM 0.2-90**	**Technical Standards In Japan**	**MIL-HDBK-1025/4**	**EM 1110-2-2504**
1	**Index**	G	G	G	G	G	G	G	G	G
2	**Symbol list**	-	G	G	G	G	G+	-	-	-
3	**Clarifying wordlist**	G	G	B	A	-	G	-	B	B
4	**Safety philosophy**	G+	G+	G	B	A	A/G	D	-	B
5	**Ship information**	A	-	-	A	A	A	A	-	-
6	**Environmental criteria:**									
6 a	Wind	D	-	-	D	B	B/A	D	-	D
6 b	Ice	B	A	D	D	G	D	-	-	D
6 c	Waves	D	A	D	G+	G	B	G+	D	D
6 d	Tide	G	B	D	D	G	A	B	D	D
6 e	Currents	D	-	-	D	-	B/A	B/A	-	-
6 f	Geotechnics	A	G	D	B	A	B	B	-	A
7	**Design aspects:**									
7 a	Loading	A/G	A	-	B/A	G	G	D	-	G
7 b	Earth pressures	A	G	G	A	G	G	A	B	G
7 c	Groundwater(flows)	G	G	D	D	G	A	-	-	D
7 d	Dredging works	D	D	-	-	A	-	-	-	-
7 e	Sheet piling	G	G+	B	A	G+	-	A	B	G
7 f	Piling	G	-	D	B	G	-	A/G	-	-
7 g	Anchoring	G	G+	D	B/A	G	A	A	B	G+
7 h	Relieving platform	G	A	-	B	A	B/A	B/A	B	-
7 i	Dolphins	-	-	-	B/A	G	-	D/B	-	-
7 j	Breakwaters	-	-	-	B	A	-	G	D	-
7 k	Embankments	-	-	D	D	G	-	-	D	-
7 l	Mooring	B	B/A	D	A	A	A	A	-	D
7 m	Quay wall equipment	D	-	-	B	G	-	B	-	-
7 n	Quay wall geometry	A	D	-	B	A/G	-	D	-	-
7 o	Seismic	D	B	D	D	A	-	A	-	D
7 p	Stability	G	G	D	B	G	-	B	-	A
8	**Type of quay wall treated:**									
8 a	Block wall	D	-	D	B	A	B	A	D	-
8 b	L-wall	D	-	D	B	-	B	A	D	-
8 c	Caisson	D	-	D	B	A	B	A	D	-
8 d	Cellular wall	D	D	D	B	B	-	A	D	-
8 e	Earth structure (Terre Armee wall)	D	D	D	-	-	-	-	D	-
8 f	Sheet pile structure	G	G+	D	A	G+	-	A	B	G
8 g	Cofferdam	D	G+	D	B	A	-	A	-	-
8 h	Sheet pile wall with relieving structure	G+	A	-	B	A	B	B/A	B	-
8 i	Open berth quay	D	-	-	A	G	-	A	D	-
9	**Construction**	G	G+	-	B	G+	-	-	-	B/A
10	**Cost**	B	B	-	-	-	-	-	-	-
11	**Monitoring**	D	D	D	D	A	-	-	-	D
12	**Maintenance**	G	G	D	D	A	-	B	D	D
13	**Experience**	G	A	-	-	G	-	-	-	-
14	**Examples**	G	G	-	-	-	-	-	-	-

C Determination of characteristic values

In this Annex the characteristic values mentioned and discussed in chapter 6 are worked out for a reference case. With these characteristic values the comparative calculations are carried out in Annex D and E.

C.1 Reference project "Euromax" for comparative calculations

For the design calculations the Euromax quay (in the Yangtze Harbor, Port of Rotterdam) is chosen as reference case. The construction of this large structure is still going on.

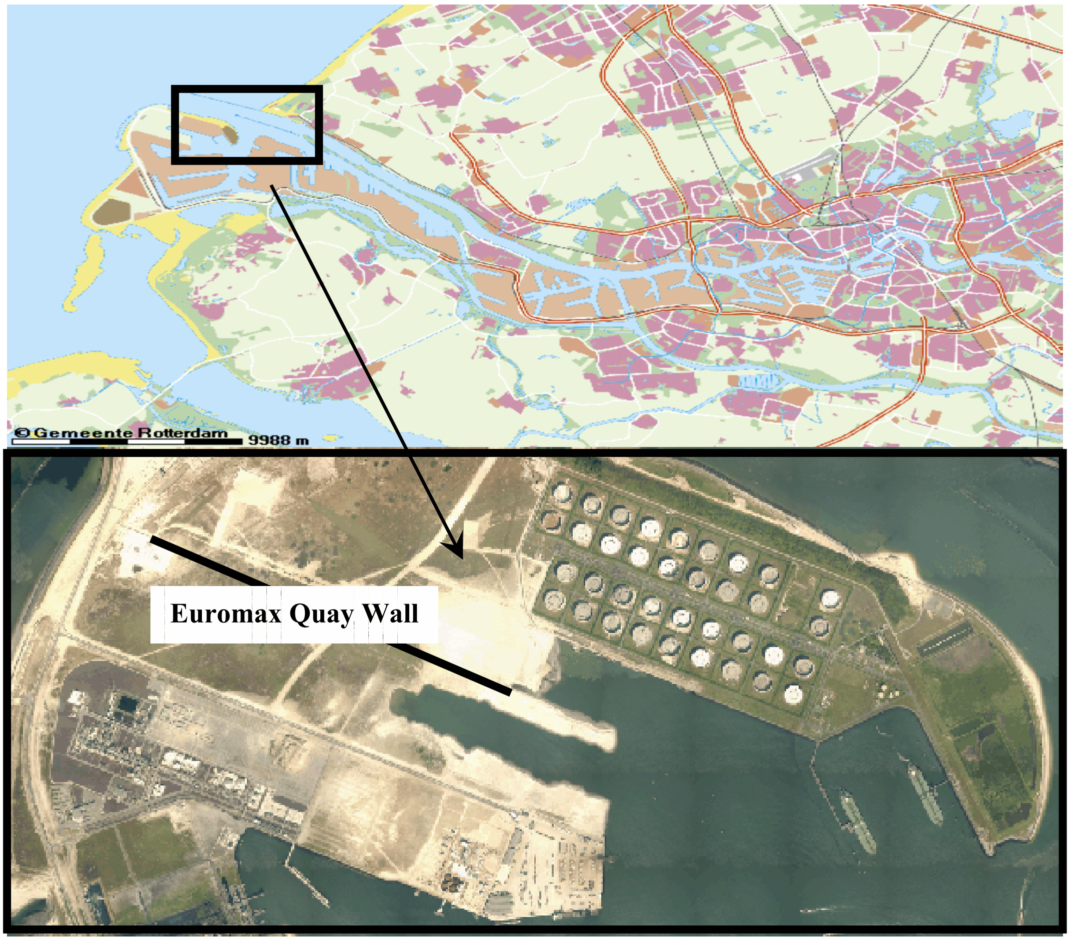


Figure C-1: Overview Port of Rotterdam, Yangtze Harbor with Euromax quay

C.2 Cases 1 & 2 for comparative calculations

Two design cases will be specified to overview the effect of the design recommendations on the design of a "small" and "large" quay wall structure. These two structures can be seen as typical structures.

For the comparative analysis no construction phases are taken into account. It is in contradiction with the real situation, where this plays an important role. However, for the reduction of the amount of calculations the construction is neglected.

C.2.1 Case 1: Sheet pile wall, tie rod and anchor wall (retaining height 12 m)

This quay will be constructing for container transshipment from inland navigation vessels. On top of the quay there is a mobile crane. The crane has 4 footprints, which spread the loads to the bottom and sheet pile wall. Around this crane the containers can be stored. On top of the quay there is a pavement which will be used for transporting the containers with special vehicles. The largest ship that will berth at the quay is a Large Rhine vessel.

Properties of the large Rhine vessel:

- Length (overall): 110,0 m
- Width: 11,4 m
- Draught: 4,5
- Block coefficient: 0,92 m

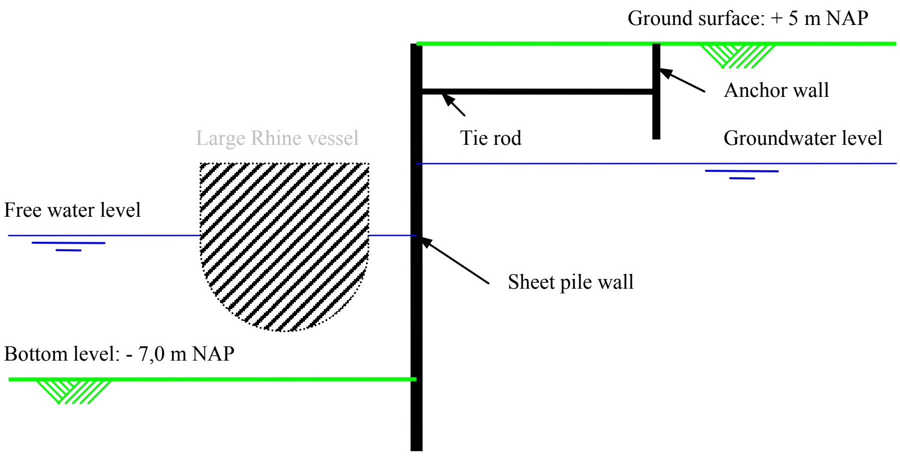


Figure C-2: Case 1, sheet pile wall with tie rod and anchor wall

C.2.2 Case 2: Combined sheet pile wall, relieving platform, pile foundation and MV-pile (retaining height 30 m)

This quay will be constructed for the 7th generation container vessels. On top of this quay there is a crane for transshipment of containers, which is adapted to the newest generation container vessels. The railway has a centre to centre distance of 30,48 m and at the waterside it has a foundation on the superstructure, the landward railway has a spread foundation. On top of the quay there is a pavement which will be used for transporting the containers with special vehicles. The largest vessel that will berth is the future 7th generation container vessel, which can probably transport 12500 TEU, called the Southampton ++.

Properties of the Southampton ++:

- Length (overall): 382 m
- Width: 57 m
- Draught: 17 m
- Block coefficient: 0,686 m

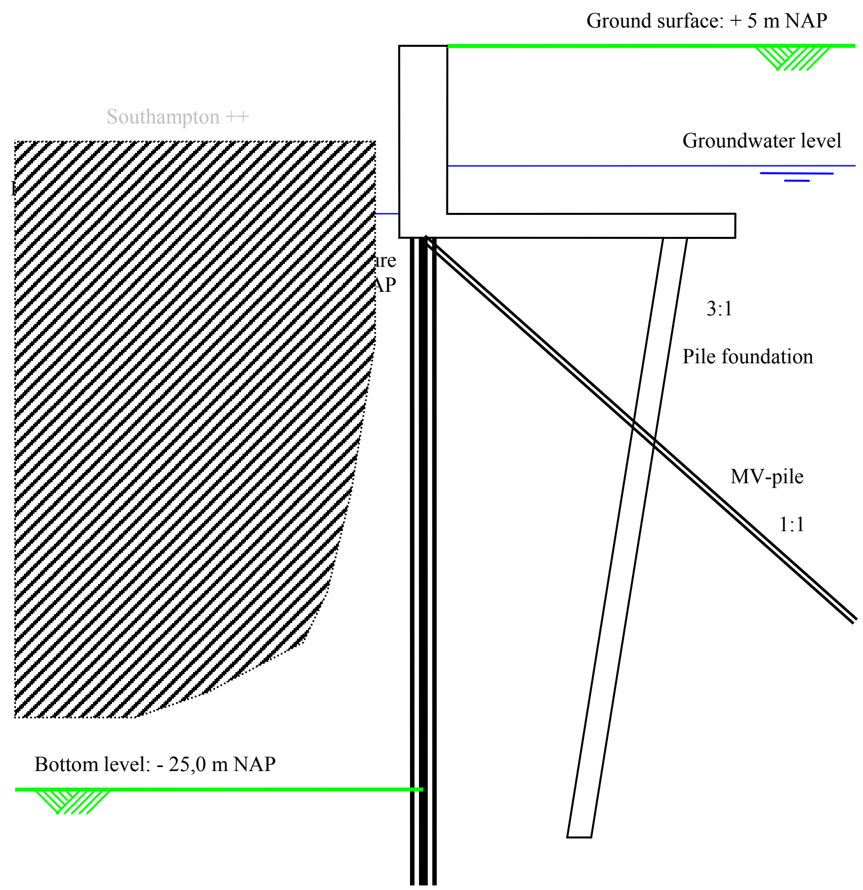


Figure C-3: Combined wall, with superstructure, MV-pile, pile foundation

Superstructure

The superstructure is a very important aspect in the design of this type of quay wall. It spreads the loads coming from crane, bollard, fender and other terrain loads to the foundation elements: combined wall, MV-pile and pile foundation. For the sheet pile calculation it is important in which way the loads will be introduced into the combined wall, because this can give extra bending moments in the wall due to eccentricity and 2nd order effect.

In the recommendations not much attention is paid to the redistribution of loads through the structure, except in Handbook Quay Walls which describes this topic very good. It is not the purpose of this comparative analysis to give a detailed calculation of the superstructure. The shapes and dimensions of the superstructure in the real Euromax case will be assumed reference.

The global dimensions of the superstructure are:

- Height wall: 6,5 m
- Depth underside of structure: - 1,5 m NAP
- Width floor: 20 m
- Thickness wall: 3 m
- Thickness relieving floor: 1,5 m

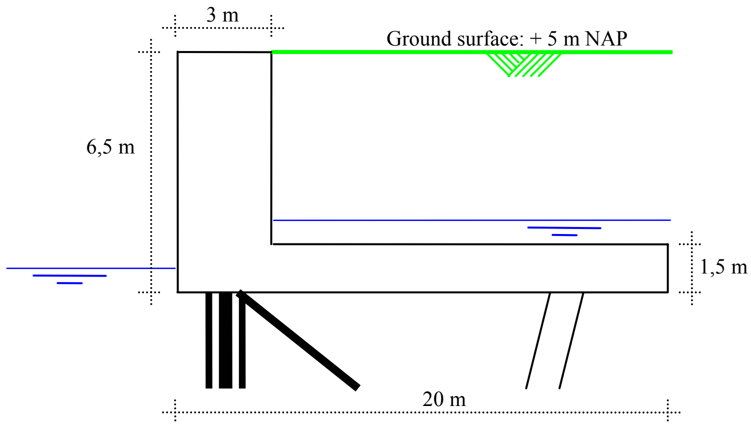


Figure C-4: Dimensions of the superstructure for case 2

C.3 Geometry parameters

For the geometry parameters, the Euromax quay is chosen as reference, but adapted to the type of quay corresponding with case 1 and 2.

C.3.1 Design depth of the bottom

The retaining height for case 1 is **12 m** and the retaining height for case 2 is **30 m**.

C.3.2 Top level of the quay

The level of the Maasvlakte ground surface is approximately **+ 5 m NAP**, as can be concluded from cone penetration tests. It is also a design condition given by the Harbor Authorities of Rotterdam. A water level higher than + 2,07 m NAP occurs only 1 % of the time (year). The top level of **+ 5 m** NAP seems to be safe enough.

C.4 Water levels

For the calculations of water levels see chapter 6.2.

C.4.1 General data

The three recommendations use a different method for calculating the most normative water levels. Measured water level changes in the Port of Rotterdam are described in the Hydro-Meteo-report [17] written by the Port Authority of Rotterdam for the Europa Harbor (near to the Yangtze Harbor).

% of HW that is larger or % of LW that is smaller [m NAP]

	1%	10%	50%	90%	99%
HW	+ 2,07	+ 1,64	+ 1,25	+ 0,87	+ 0,54
LW	- 1,23	- 0,98	- 0,71	- 0,36	+ 0,08

Table C-1: Mean water level measurements with percentage of exceedence

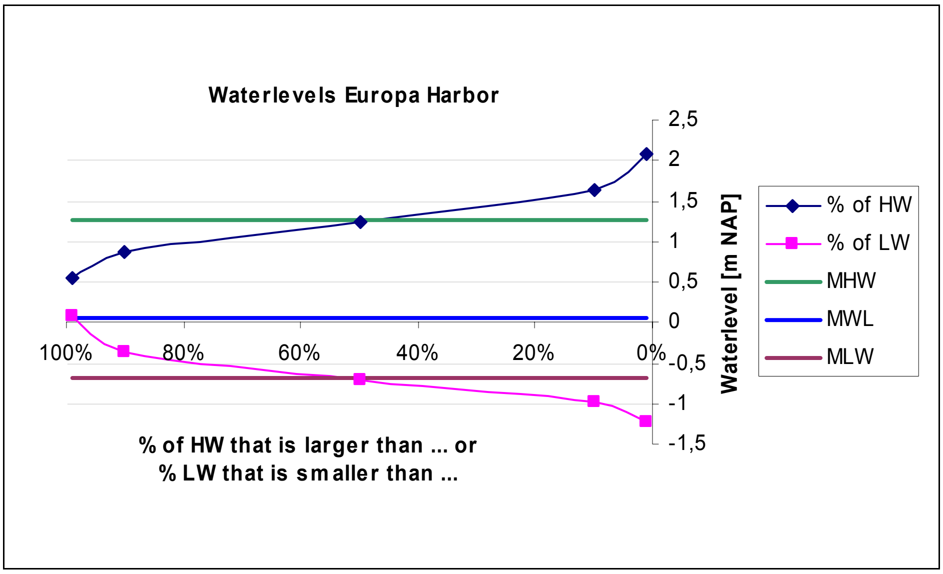


Figure C-5: Water levels in Europa Harbor, percentage of exceedence per year

Mean water level from measurements	**[m NAP]**
Mean High Water	+ 1,26
Mean Water Level	+ 0,06
Mean Low Water	- 0,69
Mean Low Water Spring	approx. - 0,80
Lowest Low Water Spring (≈ OLW)	- 0,90
Characteristic 5% water low level	- 1,10
Lowest Low Water	- 1,61

Table C-2: Water levels measured in the Europa Harbor

C.4.2 Handbook Quay Walls

In Handbook Quay Walls a special probabilistic water level analysis is described. This analysis is based on the probability distribution function of high and low waters. From this analysis mean values and standard deviations can be determined. From the free water analysis the groundwater level can be estimated. For this calculations a time shift of 2 hours is assumed. This is also done in an example for the Maasvlakte given in Handbook Quay Walls.

The design free water levels will be

$$h_{50,LW} = \mu_{50,LW} - \gamma_{sf} \cdot \sigma_{LW} = -1{,}22 - 0{,}6 \cdot 0{,}27 = -1{,}38 mNAP$$

$$h_{HW} = \mu_{HW} + \gamma_{sf} \cdot \sigma_{HW} = +1{,}26 + 0{,}6 \cdot 0{,}33 = +1{,}46 mNAP$$

The high groundwater level can be derived from the free water levels. As mentioned before a shift of 2 hours will be used. On the mean and standard deviation of the groundwater some safety factor should be applied.

$$h_{g,HW} = \mu_{HW} + \gamma_{sf} \cdot \sigma_{g,HW} = +0{,}80 + 2 \cdot 0{,}25 = +1{,}30 mNAP$$

All water levels according to Handbook Quay Walls are summarized in the table below.

Water levels according Handbook Quay Walls	**[m NAP]**
Mean High Water, for a 4 years period	+ 1,26
Standard deviation MHW	*0,33*
Design value High Water (4 years) (γ=0,6)	***+ 1,46***
Mean Water Level	+ 0,27
Mean Low Water, for a 4 years period	- 0,69
Standard deviation MLW	*0,27*
Low Water, for a 50 year period	- 1,22
Design value Low Water (50 years) (γ=0,6)	***- 1,38***
High Groundwater Level (time shift 2 hours)	+ 0,80
Standard deviation HGW	*0,25*
Design value High Ground Water (γ=2,0)	***+ 1,30***

Table C-3: Water levels according to Handbook Quay Walls, based on a probabilistic approach of the probability distribution function of the water levels in the Europa Harbor

C.4.3 CUR 166

The CUR 166 refers to a water level analysis with measurements over a long time. For the determination of design water levels 2 situations can be considered for locations with tidal difference (see figure 6-1):

1. Without drainage system: Average of MHW and MLW plus 0,3 m to LLWS
   Δh = (1,26 m + 0,69 m)/2 + 0,3 m + (0,90 m - 0,69 m) = 1,49 m

   This looks like situation 3a of the EAU 2004, but for the CUR 166 the LLWS is used in stead of the MLWS as used in the EAU.

2. With drainage system: Underside drainage plus 0,3 m to LLWS
   Δh = 0,3 m + 0,90 m = 1,20 m

C.4.4 EAU 2004

For the EAU 2004 (like CUR 166) some normative situations are described with tidal conditions. These cases correspond with safety factors. The situation for the Europa Harbor looks like situation 3, described in the EAU 2004. This situation describes tidal conditions, divided into 4 sub-conditions (see figure 6-2):

a) Major water level fluctuations without drainage – normal case
   LC 1: Average of MHW and MLW plus 0,3 m to LLWS
   Δh = (1,26 m + 0,69 m)/2 + 0,3 m + (0,80 m - 0,69 m) = 1,39 m

b) Major water level fluctuations without drainage – limit case extreme low water level
   LC 3: Average of MHW and MLW to LLW
   Δh = (1,26 m + 0,69 m)/2 + (1,61 m - 0,69 m) = 1,90 m

c) Major water level fluctuations without drainage – limit case falling high water
   LC 3: Average of MHW plus 0,3 m to MLW
   Δh = 0,69 m + 1,26 m + 0,3 m = 2,25 m

d) Major water level fluctuations with drainage (a groundwater flow analysis is needed to find the reflux congestion; for this study a value of 0,3 m will be assumed)
   LC 1: 1,00 m plus reflux congestion of the groundwater
      Δh = MLW to 1,00 m + reflux congestion = 1,3 m
   LC 2: Average of MLWS and LLW to MLW plus 0,3 m plus reflux congestion of the groundwater
      Δh = (1,61 m + 0,80 m)/2 - 0,69 m + 0,3 m + reflux congestion = 1,12 m

The largest water level difference occurs in sub-situation c for load case 3. The largest water level difference for load case 1 occurs in sub-situation a. For load case 2 only one water level difference is described in sub-situation d.

C.4.5 Groundwater level

The groundwater level varies between + 4 m NAP in the centre of the harbor area to approximately NAP near the free water level. The groundwater follows the tide near the free water. But the fluctuation in the groundwater is less than the tide, because it needs some time for adaptation. It will be assumed that the tide (of the free water) has a **time shift of 2 hours** with the groundwater. This time shift is also used in an example done in Handbook Quay Walls for the Maasvlakte.

As mentioned before, no drainage systems will be applied for case 1 and 2 to simplify the comparative calculations.

The Pleistocene layer and the Holocene layers are separated by a clay layer. The groundwater in the Pleistocene layer has a water level head of + 0,5 m NAP. But the clay layers in the Holocene and Pleistocene package are not presented over the total longitudinal section. Transport of groundwater can easily occur over these layers.

The groundwater level according Handbook Quay Walls, is carried by a probabilistic analysis. It will be calculated from a probability distribution function of the free water level, with a time shift of 2 hours. In the EAU 2004 the groundwater level is as described for the free water level for 4 normative situations. The CUR 166 uses a similar approach as EAU 2004 from which a situation must be chosen.

Water levels from measurements	**[m NAP]**
Mean Free Water Level	+ 0,06
Groundwater level Pleistocene layer	+ 0,50
Water levels according Handbook Quay Walls	**[m NAP]**
Mean Groundwater Level (= MWL)	+ 0,27
Mean High Groundwater, due to tide (time shift of 2 hours)	+ 0,80
Design value High Groundwater Level	+ 1,30

Table C-4: Representative values of groundwater level

C.4.6 Waves

The quay in the Yangtze Harbor has an east-west direction and is protected against waves coming from sea. The only waves that can reach the quay are developed by wind in the harbor itself and stern waves, due to passing ships. The wind waves are very low and short, because the distance to develop the waves is very short. The waves due to passing ships will also be low, because the speeds of the vessels are very low and therefore the stern waves will be low. This are the reasons why waves will not be taking into account and are **neglected**. This is also done in the terms of reference written for the Euromax quay.

C.4.7 Currents

Currents due to passing ships will be **neglected**. The passing ships have a very low speed and the quay is high enough to withstand stern waves. The currents from propellers will also be neglected. The reason is that a design depth is assumed. In a detailed calculation it would probably be necessary to apply a bottom protection.

C.5 Soil parameters

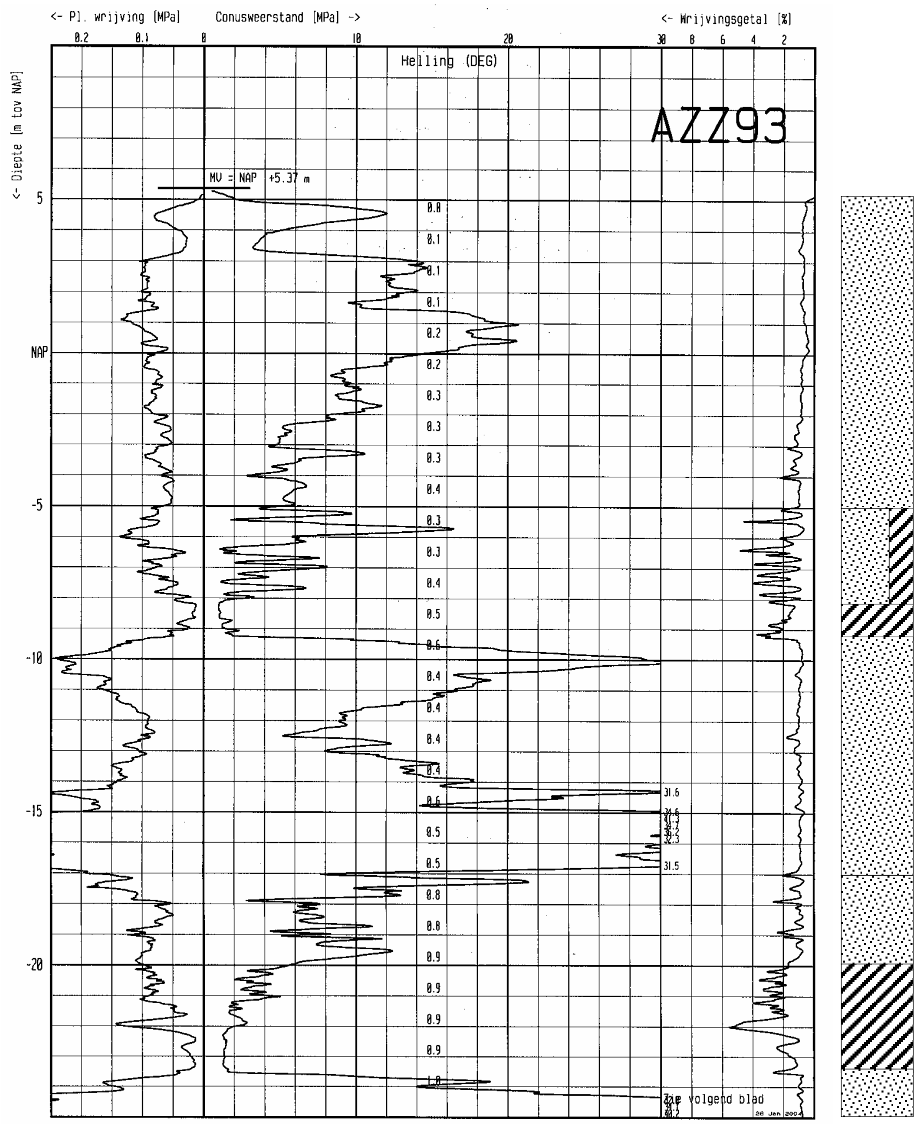
For the calculation of the soil properties see chapter 6.4.

C.5.1 General data for comparative analysis

For the Euromax quay a lot of cone penetration tests and boreholes are done at the Maasvlakte, to verify the soil properties. The Public Works of Rotterdam did an investigation [21] from these tests to find the layer profile and soil properties. This leads to a longitudinal section for the place the quay wall will be construct.

For the comparative analysis, one cone penetration test will be chosen for the sheet pile calculations. It is favorable to do this with a profile representative for the total area. In general 3 profiles can be determined, in which the profile with a clay layer at -20 NAP is the most representative. Approximately 1300 m of the longitudinal section has this clay layer. From this profile cone penetration test AZZ 93 is chosen.
With this penetration test and the general characteristic values presented in the recommendations, the soil properties are determined. Not all laboratory tests are studied, because then also other NEN and DIN codes must be used, what is not the purpose of this study for recommendations. The properties are determined from the tables CUR 166 and EAU 2004, because Handbook Quay Walls refers to the CUR 166 for the description. In both recommendations the starting point for the investigations is the cone resistance.

In general 7 layers can be determined. For this layer analysis, the soil properties can be estimated with tables 3.1 in CUR 166 (see table 6-2) and 9-1 in EAU 2004 (see table 6-4). Handbook Quay Walls refers to the CUR 166 for determination of soil properties. The table in CUR 166 gives representative values for properties in a large region of soil and EAU 2004 gives characteristic values, based on cautious mean empirical values for a large area of soil. These tables are both coming from codes used in the Netherlands (NEN) and Germany (DIN). For CUR 166 the starting point of finding the properties is the cone resistance. For the EAU 2004 this is the same, except for clay layers which use the liquid limit as reference point. The sand layers described in the EAU 2004 also use the grain size distribution as input for the table.



Layer	Layer thickness [m]	Cone resistance [MPa]
1	10	3 - 21
2	3	1 -16
3	1,3	1 - 3
4	7,7	5 - 41
5	3	3 - 21
6	3,5	1 - 5
7	> 20	> 30

Figure C-6: Cone penetration test AZZ 93, with soil layer separation and cone resistance

C.5.2 Soil layer separation from cone penetration test AZZ 93

Layer 1: Sand layer + 5 m / - 5 m NAP

A sand layer with sand from land reclamation for the construction of the Maasvlakte. The highest cone resistance occurs in the centre of this layer, this area (around NAP) is densely packet due to tidal effects. The mean cone resistance is 10 MPa.
EAU 2004: This corresponds to a fine, uniform sand layer with medium consistency.
CUR 166: The converted value for the effective soil pressure of the cone resistance is 12 MPa. This corresponds to a clean sand with a moderate consistency

Layer 2: Sand/clay layer - 5 m / - 8 m NAP

A sand layer with thin clay layers in between. The mean cone resistance is 5 MPa.
EAU 2004: This corresponds to a sand layer with clay and silt of low consistency. The sand layer has a grain size of d < 0,06mm < 15%, referring to a grain size diagram for a place near to the Euromax quay.
CUR 166: The converted value for the effective soil pressure of the cone resistance is 4 MPa. This corresponds to a sand layer with strong clay/silt influence.

Layer 3: Clay layer - 8 m / - 9,3 m NAP

This is a pure clay layer of 1,3 m thick. The mean cone resistance is 1,5 MPa. This layer is not present over the total longitudinal section.
EAU 2004: This corresponds to an an-organic clay layer. A medium plasticity is chosen, because no exact values are presented: Liquid Limit 35% < LL < 50%. The values are chosen at the safe side.
CUR 166: The converted value for the effective soil pressure of the cone resistance is 1 MPa. This corresponds to a clean clay layer with moderate consistency.

Layer 4: Sand layer - 9,3 m / - 17 m NAP

A sand layer which has a large difference in cone resistance, but is densely packet at some places. The mean cone resistance is 22 MPa.
EAU 2004: This corresponds to a fine, uniform sand layer with high consistency. It is fine sand, referring to some values near the future Euromax quay.
CUR 166: The converted value for the effective soil pressure of the cone resistance is 13 MPa. This corresponds to a clean sand layer with moderate consistency.

Layer 5: Sand layer - 17 m / -20 m NAP

A sand layer which is loosely packet. The mean cone resistance is 10 MPa.
EAU 2004:
This corresponds to a fine, uniform sand layer with medium consistency. It is fine sand, referring to some values near the future Euromax quay.
CUR 166:
The converted value for the effective soil pressure of the cone resistance is 5 MPa. This corresponds to a clean sand layer with low consistency.

Layer 6: Clay layer - 20 m / -23,5 m NAP

This is the clay layer on top of the Pleistocene sand which is familiar for the Maasvlakte. The mean cone resistance is 2 MPa.
EAU 2004: This corresponds to an an-organic clay layer. A medium plasticity is chosen, approximated values are presented from a boring in the neighborhood: Liquid Limit is approximately 48%, which is 35% < LL < 50%. The values are chosen at the safe side.
CUR 166: The converted value for the effective soil pressure of the cone resistance is 1 MPa. This corresponds to a moderate clay layer with low consistency.

Layer 7: Pleistocene sand layer - 23,5 m / -40 m NAP

The Pleistocene sand layer, with mean consistency over the first 40 m is 50 MPa.
EAU 2004: This corresponds to a coarse sand layer with high consistency. It is moderate coarse sand, referring to some values near the future Euromax quay.
CUR 166: The converted value for the effective soil pressure of the cone resistance is 20 MPa. This corresponds to a clean clay layer with dense consistency.

C.5.3 Soil properties for CUR 166 and EAU 2004

The properties of both recommendations will be given below.

Layer [m NAP]		**Soil type according CUR 166**			**$q_{c\ nominal}$ [Mpa]**	**γ [kN/m³]**	**γ sat [kN/m³]**	**E_{100} [MPa]**	**φ' [°]**	**c' [kPa]**	**c_u [kPa]**
top	*bottom*	*Main name*	*Mixture*	*Consistency*	*mean*	*low*	*low*	*low*	*low*	*low*	*low*
5	-5	sand	clean	moderate	12	18	20	45	32,5	0	0
-5	-8	sand	strong clay silt		4	18	20	15	25	0	0
-8	-9,3	clay	clean	moderate	1	17	17	2	17,5	5	50
-9,3	-17	sand	clean	moderate	13	18	20	45	32,5	0	0
-17	-20	sand	clean	loose	5	17	19	15	30	0	0
-20	-23,5	clay	clean	moderate	1	17	17	2	17,5	5	50
-23,5	-50	sand	clean	dense	20	19	21	75	35	0	0

Table C-5 : Conservative values for soil properties, according to CUR 166 (representative values)

Layer [m NAP]		Soil type according EAU 2004			q_c [Mpa]	γ [kN/m³]	γ' [kN/m³]	E_s [kN/m²]	φ' [°]	c' [kPa]	c_u [kPa]
top	*bottom*	*Soil type*	*Soil group*	*Consistency*	*mean*	*low*	*low*	*low*	*low*	*low*	*low*
5	-5	sand	uniform, fine sand	medium	10	17	9,5	19.462	32,5	0	0
-5	-8	sand d < 0,06 < 15%	sand clay/silt	low	5	16	8,5	20.192	30	0	0
-8	-9,3	Clay	an organic 35 % < LL < 50 %	safe	1,5	18,5	8,5	1.650	22,5	5	5
-9,3	-17	sand	uniform, fine sand	high	22	18	10,5	46.688	35	0	0
-17	-20	sand	uniform, fine sand	medium	10	17	9,5	41.766	32,5	0	0
-20	-23,5	clay	an organic 35 % < LL < 50 %	safe	2	18,5	8,5	2.840	22,5	5	5
-23,5	-50	sand	uniform, coarse sand	high	50	18	10,5	147.346	35	0	0

Table C-6 : Conservative values for soil properties, according to EAU 2004 (characteristic values)

It is clear to see that there are differences between the CUR and EAU. The densities are almost the same, taken into account that the saturated values are given for drained circumstances in the EAU. The angle of internal friction in the EAU is in most of the layers higher than the CUR-values. A reason is that some layers have a lower cone resistance due to the conversion of effective soil pressures in the CUR 166, especially the layers between - 9,3 m and - 20 m NAP. The E parameter in CUR 166 is given in MPa, while for the EAU it is given in kN/m². Some large differences occur for E-values, sometimes with a difference of a factor 2. A reason is that also these values are converted with the effective soil pressure.

C.5.4 Modulus of sub-grade reaction for CUR 166 and Handbook Quay Walls

If a beam on elastic foundation model is used for calculations, the stiffness of the soil is taken into account by modulus of sub-grade reaction parameters. This parameter is the schematized soil stiffness (see figure 6-3). For CUR 166 high and low values (see table 6-3) of the modulus of sub-grade reaction are given, determined from experience of bottom conditions in the Netherlands. In Handbook Quay Walls, the mean CUR-values of the high and low modulus of sub-grade reaction are used. In the EAU 2004 only the E-value is given for the compressibility, probably because they use the Blum method, which doesn't use a modulus of sub-grade reaction parameter.

The CUR determination of the modulus of sub-grade reaction is based on cone resistance values, but this time no nominal value is taken, just the mean value from the cone penetration diagram.

Layer [m NAP]		k_{h1} [kN/m³]			k_{h2} [kN/m³]			k_{h3} [kN/m³]		
top	*bottom*	*low*	*mean*	*high*	*low*	*mean*	*high*	*low*	*mean*	*high*
5	-5	20000	32500	45000	10000	16250	22500	5000	8125	11250
-5	-8	12000	19500	27000	6000	9750	13500	3000	4875	6750
-8	-9,3	4000	6500	9000	2000	3250	4500	800	1300	1800
-9,3	-17	40000	65000	90000	20000	32500	45000	10000	16250	22500
-17	-20	20000	32500	45000	10000	16250	22500	5000	8125	11250
-20	-23,5	4000	6500	9000	2000	3250	4500	800	1300	1800
-23,5	-50	40000	65000	90000	20000	32500	45000	10000	16250	22500

Table C-7: Modulus of sub-grade reaction, based on experience values described in CUR 166 (high and low values) and Handbook Quay Walls (mean values)

C.6 Wall parameters

The properties of the sheet pile wall are mainly kept the same for a good comparative analysis. The change of profiles can be investigated for specific circumstances. There are a few profiles from which sheet pile wall parameters can be chosen: Hoesch, Arcelor and Larsen. Tables are available with dimensions, the moment of inertia and the section modulus. Sometimes these moments must be reduced due to the type of profile and local circumstances.

C.6.1 Oblique bending, according to CUR 166

For an explanation of oblique bending see chapter 6.5.3.

For a single sheet pile profile the **0,55-factor** can be increased with:

- **0,10** moderate strength of the cohesive and non-cohesive layers
- **0,05** resistance in the direction perpendicular due to the anchor which is installed
- **0,05** resistance in longitudinal direction due to the concrete beam
- **0,05** resistance in vertical direction due to the concrete beam on top of the wall
- **0,10** installation of the sheet pile without special measures
- **0,10** existence of a sand layer (almost 5 m thick) above groundwater level

$$\beta_B = \beta_{B;0} + \sum_{i=1}^{6} \Delta\beta_{B;i} = 0{,}55 + (0{,}10 + 0{,}05 + 0{,}05 + 0{,}05 + 0{,}10 + 0{,}10) \approx 1{,}0$$

$$\beta_D = \beta_{D;0} + \sum_{i=1}^{6} \Delta\beta_{D;i} = 0{,}55 + (0{,}10 + 0{,}05 + 0{,}05 + 0{,}05 + 0{,}10 + 0{,}10) \approx 1{,}0$$

These factors are more or less valid for the two cases that will be studied. This results in a reduction factor for the moment of inertia and the section modulus of 1,0. So no reduction of these moments have to take place. For a combined wall these calculations do not have to be taken into account, because of the stiffness of the tube in all directions.

C.6.2 Profile for case 1: U-shaped sheet pile wall

In general there are U-shaped and Z-shaped profiles for this case. A choice must made between these two types. A disadvantage of the U-shaped profiles is that they can have bending moments in the longitudinal direction, called oblique bending. This is not the case for Z-shaped profiles, but they have the disadvantage that the highest bending stresses are situated at the places of the interlocks and that the interlocks will be pushed open. This is not the case for U-shaped profiles, the interlocks will be push together when installed correctly. Therefore **an U-shaped profile will be chosen with a high moment of inertia, so that this will be sufficient for all recommendations**.

LARSSEN 607 n — **Section width per D = 1200 mm**

		Unit	Per m wall	Single pile	Double pile	Triple pile
				E	D	Dr
Elastic section modulus[1]	W_y	cm³	**3200**	649	3840	4330
	W_z	cm³	–	1730	–	–
Plastic section modulus[1]	W_y	cm³	3620	–	–	–
Weight		kg/m	**190.0**	114.0	228.0	342.0
Cross sectional area		cm²	241.7	145.0	290.0	435.0
Circumference[2]		cm	293	203	380	554
Coating area[3]		m²/m	2.93	1.91	3.67	5.43
Static moment	S_y	cm³	1810	–	–	–
Second moment of inertia	I_y	cm⁴	**72320**	11280	86790	119400
	I_z	cm⁴	–	55070	–	–
Radius of gyration	i_y	cm	17.30	8.73	17.30	16.55

Table C-8: Larsen L607n properties for calculations of CASE 1

C.6.3 Profile for case 2: Combined sheet pile wall, U-shaped infill piles

Tubes can be delivered in the sizes that are needed, no real standards are described. This is not the case for the infill piles, which can be chosen from standard profiles like Hoesch, Arcelor and Larsen. The advantage is that less tubular profiles are needed.

Pipe Dimensions		Intermediate Sheet Piles = double AZ18				Intermediate Sheet Piles = triple PU18			
Diameter (mm)	Thickness (mm)	M60% (kg/m^2)	M100% (kg/m^2)	I (cm^4/m)	W (cm^3/m)	M60% (kg/m^2)	M100% (kg/m^2)	I (cm^4/m)	W (cm^3/m)
863	10	138	165	132003	3059	129	163	113528	2631
	12	157	184	153502	3557	144	178	130748	3030
	14	176	203	174696	4049	159	193	147724	3423
914	10	140	167	149832	3279	131	164	128215	2806
	12	160	187	174896	3827	147	180	148383	3247
	14	180	207	199625	4368	163	196	168280	3682
1016	12	166	192	223588	4401	152	184	188843	3717
	14	187	213	256351	5046	169	201	215433	4241
	16	208	234	288719	5683	186	218	241703	4758
1220	14	200	223	398241	6529	181	211	335084	5493
	16	223	247	450554	7386	200	230	378196	6200
	18	246	270	502341	8235	219	249	420874	6900
1420	16	236	257	652832	9195	212	240	551496	7768
	18	261	282	729430	10274	233	261	615445	8668
	20	286	307	805367	11343	253	282	678842	9561
1520	16	241	262	770638	10140	217	245	653432	8598
	18	267	288	861705	11338	239	266	729907	9604
	20	293	314	952039	12527	261	288	805766	10602
1620	18	273	293	1006693	12428	245	271	856130	10570
	20	300	320	1112824	13739	267	294	945745	11676
	22	326	347	1218152	15039	290	316	1034681	12774
1820	18	284	303	1335351	14674	256	281	1144634	12578
	20	312	331	1477344	16235	280	305	1265735	13909
	22	340	359	1618384	17784	304	329	1386022	15231
2020	20	323	341	1899968	18812	291	315	1640324	16241
	22	352	370	2082494	20619	316	340	1797381	17796
	24	382	399	2263915	22415	341	365	1953488	19341
2500	21	360	376	3309036	26472	327	348	2903123	23225
	23	392	408	3614423	28915	355	376	3170600	25365
	25	424	440	3918320	31347	383	404	3436772	27494
3000	21	379	392	5068650	33791	347	366	4508668	30058
	23	413	426	5539330	36929	377	396	4926943	32846
	25	446	460	6008102	40054	407	426	5343522	35623

Table C-9: Combinations of primary and secondary (/infill) piles for calculation of CASE 2

	Sectional area	Mass per m	Moment of inertia	Section modulus	Radius of gyration	Coating area *
	cm^2	kg/m	cm^4	cm^3	cm	m^2/m
Per S	98.0	76.9	7220	484	8.58	0.87
Per D	196.0	153.8	46380	2160	15.38	1.72
Per T	294.0	230.7	64240	2495	14.78	2.58
Per m of wall	163.3	128.2	38650	1800	15.38	1.43

Table C-10: PU 18 properties for calculations with the infill pile

C.7 Anchorage parameters

C.7.1 Anchor for case 1: Single anchor, with tie rod and anchor wall

For structures with a moderate retaining height a lot of anchor systems are available. For example: steel bars, steel cables, steel screw anchors and anchors with gout injection. For case 1, a simple anchor system will be chosen which is applied in a lot of cases: the steel bar or tie rod. The bar diameters can vary between 16 and 63 mm and can have a yielding stress of 700 N/mm^2. At the end of the anchor a wall will be placed. The length of the tie rod, the distance between tie rods and the height of the anchor wall will have influence of on the stiffness of the anchor system. **A stiff system is chosen so that it satisfies in all cases.**

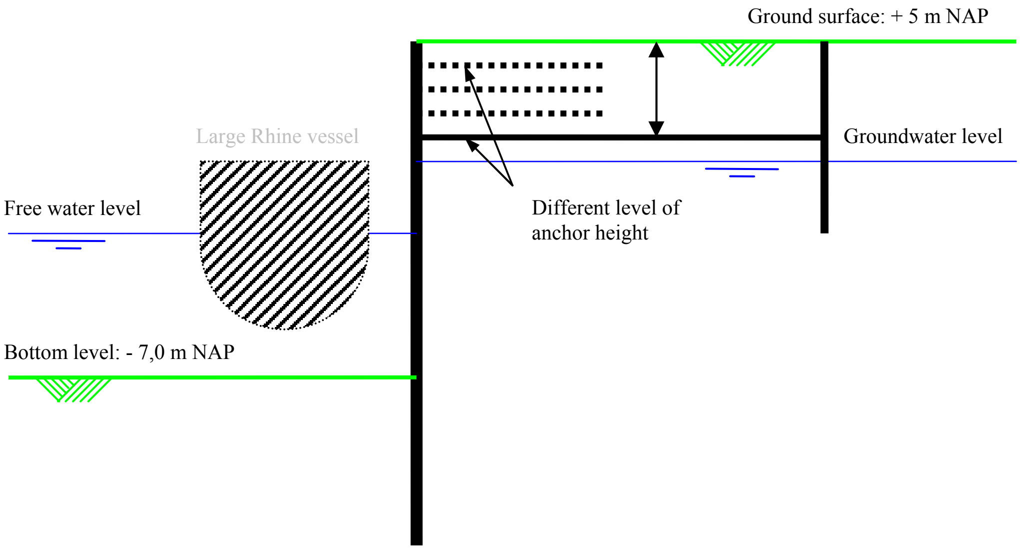


Figure C-7: Support height of anchor in the wall, see arrows

The height of the support of the anchor to the sheet pile wall is favorable above the free and groundwater level, for installation purposes. The height of the anchor support in the sheet pile wall can also reduce or increase the bending moments. After some calculations a height 2 m looks like the optimal height of the anchor support at the sheet pile wall.

Support height [m NAP]	Maximum moment [kNm/m]	Anchor force [kN/m]
+ 5	- 1045,4	209,1
+ 4	- 950,8	269,7
+ 3	- 806,9	370,2
+ 2	- 643,3	430,8
+ 1	521,1	439,0
0	442,9	369,4

Table C-11: Maximum moments and anchor forces, for several support heights for the anchors (CUR 166 calculation, design values, toe level - 12 m NAP)

C.7.2 Anchor for case 2: MV-pile

For a large retaining height a large tensile element is needed. A type which is famous in Rotterdam is the MV pile. This type of pile is coming from Germany and is called after Müller Verfahren. The pile that is chosen is a H-profile with grout-pipes. From these pipes, grout will be injected and surrounds the total profile. This gives a large tensile strength of the pile.

In case of a quay wall with superstructure, the pile will be connected in a way that the pile, the wall and the superstructure meet in one point. For case 2, this means at **- 1,5 m NAP** at the front of the superstructure. **A stiff anchor system will be chosen so that it satisfies for all conditions.**

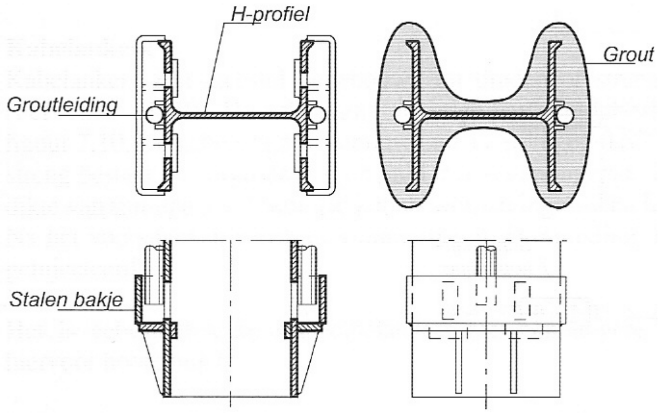


Figure C-8: MV-pile cross-section (Handbook Quay Walls)

C.8 Loads working on the quay wall structure

Loads working on the structure are coming from permanent actions, as water and soil pressure. They can also be due to extern variable loads working horizontal and vertical at the quay wall. In the recommendations there is general data given about the variable loads. The data of Euromax, given in the terms of reference, will be discussed compared to the data from the recommendations.

C.8.1 Terrain loads

For characteristic terrain loads see chapter 6.6.1.

Terrain loads for case 1

In case 1 a mobile crane is considered. The total quay can be used for storage, except the area where the crane is placed. A stack of 5 layers gives a load of 50 kN/m^2 (according Handbook Quay Walls) or 55 kN/m^2 (according EAU 2004). In the terms of reference for the Euromax a value of **60 kN/m^2** is determined, which looks a reliable value. For this case, only the cross-section with a crane and storage behind the crane will be calculated. Then it is wise to apply a load between the footprints of general traffic, according the EAU 2004: **10 kN/m^2**.

Terrain loads for case 2

Between the waterside of the quay wall and the landward side of the railway the quay is loaded by general traffic for container transport and storage of containers to be transported. In the situation of case 2, it is assumed that no containers will be stacked between the crane railways. If in an emergency this will be the case then only 2 layers of full containers will be assumed. Two full stacked containers give a load of 35 kN/m^2, recommended by the EAU 2004, the Handbook Quay Walls gives a smaller load. In the terms of reference of the Euromax quay a value of **40 kN/m^2** is given. This value will be assumed for case 2.
The area behind the landward railway could be used for storage. The containers in this area can be stacked higher. The Handbook Quay Walls uses stacks of 5 layers high, with a load of 50 kN/m^2. Included in this load is a percentage (17%) of the containers that is empty. The EAU 2004 recommends full containers, stacked 4 high with a load of 55 kN/m^2. For the Euromax case a value of **60 kN/m^2** is recommended, also because of the future development. This looks a reliable assumption.

C.8.2 Traffic load

The traffic load can be due to transport vehicles and other transport systems. They drive where there are no cranes or containers. Because a large terrain load is applied for the storage of containers, it is assumed that the **traffic loads are included in the terrain loads**.

C.8.3 Crane loads

For characteristic crane loads see chapter 6.6.3.

Case 1: Mobile crane

The crane for case 1 is a mobile crane and must have an outreach from the wall of the quay of approximately 12 m. This results in a Liebherr LHM 150, with a maximum dynamic area pressure (including wind) of 296 kN/m^2.

The waterside footprint is placed at 2 m from the side of the wall. This results in a line load of approximately **170 kN/m^2** and a load of the landward (center to center distance 9 m) footprint is approximately **60 kN/m^2**.

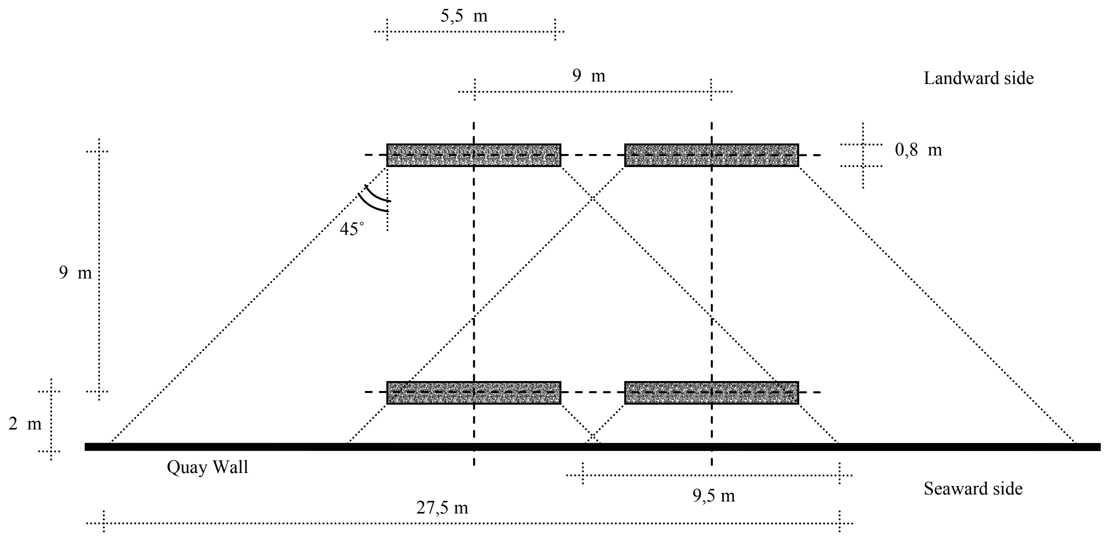


Figure C-9: Redistribution of loads over the quay, due to mobile crane footprints

Case 2: Large container crane

For the future container vessels no general values are give. For the Euromax quay (the design situation) is a crane load taken into account for these future ships. These value will be assumed for the calculations and are valid under storm and operational conditions: vertical load 1600 kN/m, horizontal load +/- 60 kN/m, railway width of 30,48 m.

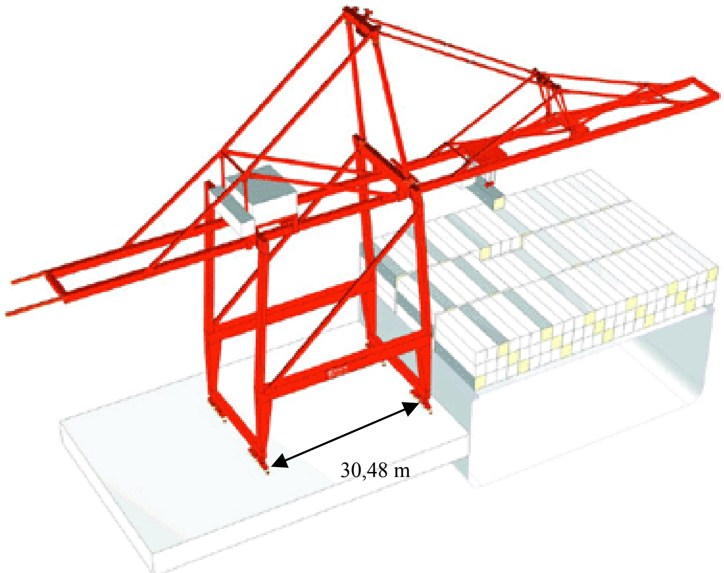


Figure C-10: A Kalmar container crane

This means for the landward crane side that the load is redistributed over the 45° in both sides, resulting in a width of 2*(30,48 m+ 2 m) + 10 m(=width of the foot) ≈ 75 m. Normally the 1600 kN/m works over a length of 10 m. The line load corresponding to 75 m is **213 kN/m**, this also holds for the horizontal landward crane load, which results in **8** kN/m. All these values are valid when 1 crane is operating and other cranes do not influence these loads.

C.8.4 Mooring loads on the bollard

For characteristic mooring loads see chapter 6.6.4.

Mooring loads for case 1
For case 1, when a inland navigation ship is berthing at the quay approximately an amount of 4500 ton water is displaced, which corresponds to a berthing load of 163 kN from the general tables with water displacement. But this is not a sea going vessel. For inland navigation vessels there are classes defined (according to CUR 166), for which the Large Rine vessel belongs to Class V and has a representative bollard load of **250 kN**.
This load can not redistribute through a superstructure, only through a beam on top of the sheet pile. This load can possibly redistribute over a few meters, especially when the beam is not very stiff. As an assumption a length of 2,5 m is chosen, which results in **100 kN/m**.

Mooring loads for case 2
For case 2 a container vessel displaces more than 200.000 tons of water. The force on the berthing vessel that corresponds with this tonnage is approximately **2000 kN**. Normally this load will redistribute **over an angle of 45°** trough the concrete of the superstructure to a line load. The bollard is placed in the centre of the width of the superstructure. This results in a horizontal line load of approximately **175 kN/m** redistributed over a length of 11,5 m.

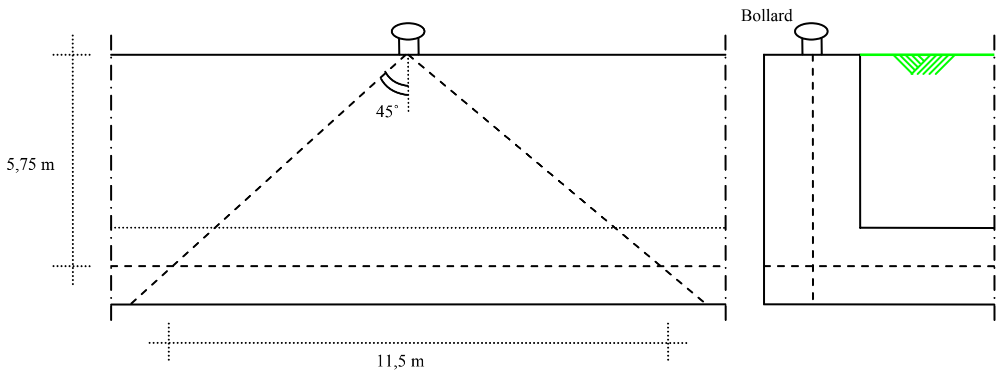


Figure C-11: Redistribution of loads over the quay, due to bollard loads

C.8.5 Fender loads

For calculation rules for fender loads see chapter 6.6.5.

Case 1: Foam Fender
The loads of the fender system in case 1, is directly placed at the sheet pile wall. This load can be applied directly on the wall, so that the structure and the ships skin must take the energy. Also a fender can be chosen which redistributes the loads as much as possible over the wall. The result is a foam fender of **2500 * 4000 from a FF 50 series**, with a reaction force of 1760 kN. When a ship berths, the reaction will redistribute over a surface of approximately 3,38 * 4 is 13,5 m^2. This results in an surface load of **130 kN/m^2**. This load will also work at + 1,5 m NAP. For case 1, the load combination with the fender loads will probably not be the normative combination.

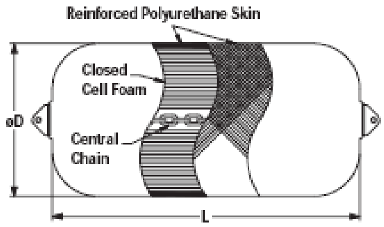


Figure C-12: Fentek Foam Fender

Case 2: Super Cone Fender

For the Euromax quay a Fentek fender system is chosen: Super Cone Fender. Some calculations are done for a single cone. The results for the quay with superstructure, case 2, give a reaction force of 2539 kN, for **SCN 1800 – E 1.3**. These loads can also redistribute under the same assumption as the bollard. The centre of the fender in case 2, is at + 1,5 m NAP. With a redistribution angle of 45° the length will be approximately 10 m, which results in a horizontal line load on the sheet pile wall of approximately **250 kN/m**.

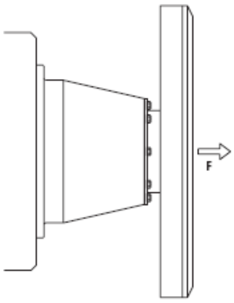


Figure C-13: Fentek Super Cone Fender

D Sheet pile calculations case 1

The calculations following the design philosophies described in CUR 166, Handbook Quay Walls and EAU 2004 (chapter 7, 8 and 9) are carried out in this annex for case 1 (retaining height 12 m). Not all aspects of the design process will be treated. The calculations will focus on the sheet pile calculation. For a good comparative analysis the internal forces and anchor forces are calculated for a wall with a toe level of -12 m NAP. This is assumed as a realistic depth and is lower than the minimum toe levels, which are also calculated and compared. The representative/ characteristic values are schematized below.

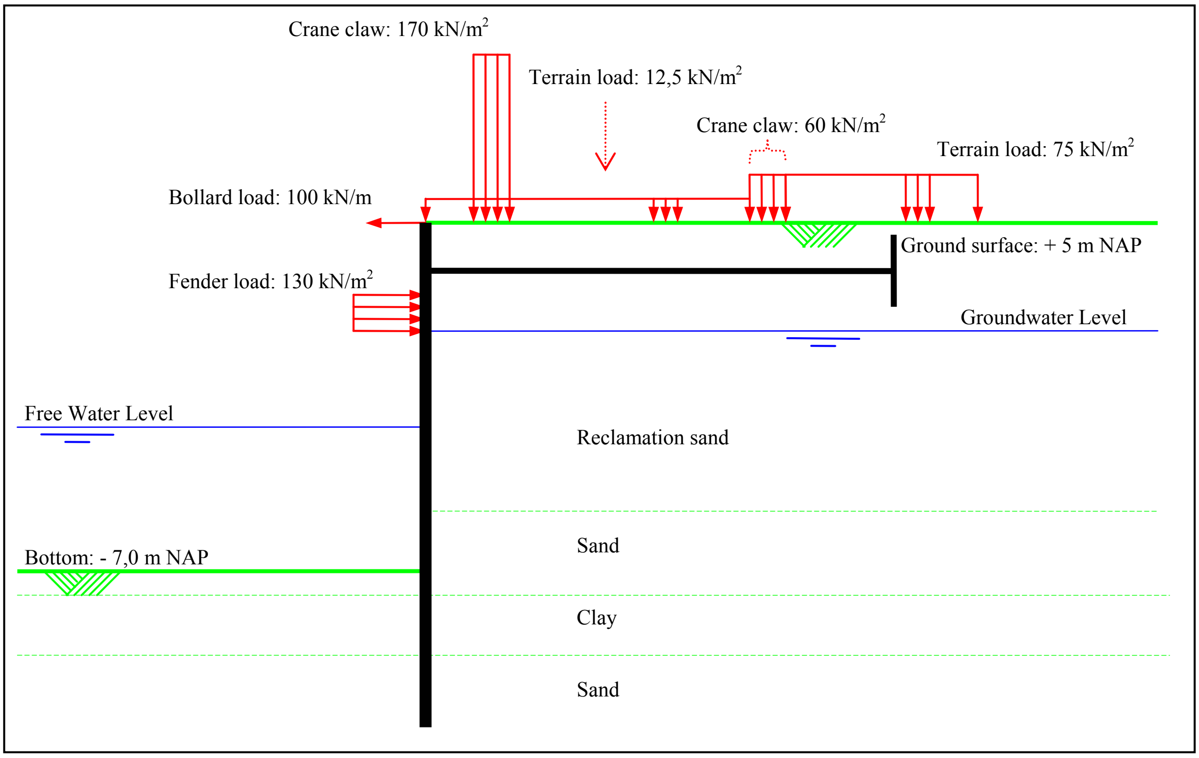


Figure D-1: Characteristic and representative values for the sheet pile calculations

The material properties are kept the same for the comparative calculations.

Material properties		
Steel quality	S355	
Elasticity modulus steel	$210*10^3$	N/mm^3
Yielding force	355	N/mm^2
Tensile strength	510	N/mm^2
Sheet pile wall		
Profile	Larsen 607n	
W	3200	cm^3/m
W triple	4330	cm^3
Width single sheet pile	600	mm
Tie rod anchor		
Diameter tie rod	63	mm
Supporting height anchor	+ 2	m NAP
Length tie rod	20	m
Centre to centre distance (longitudinal)	1	m
Anchor wall height	6	m

Table D-1: Material properties for sheet pile wall and anchor for CASE 1

D.1 CUR 211, Handbook Quay Walls

The design philosophy of Handbook Quay Walls is described in chapter 7.

D.1.1 Determination representative and design values

The nominal conditions are described in the chapter 6 and Annex C. The design philosophy is based on:

- Safety Class 2 (β=3,4)
- Life time 50 years

The design philosophy in Handbook Quay Walls is based on **representative soil properties** for the sheet pile calculation. The angle of inclination is a fraction of the angle of internal friction of the soil layer: 0,67*φ_d.

In contradiction with CUR 166, the Handbook Quay Walls applies the **mean modulus of sub-grade reaction** for the stiffness of the soil. With these mean values the stiffer and less stiff parts of the soil behind the wall over the longitudinal direction are redistributed.

Layer [m NAP]		**k_{h1} [kN/m^3]**	**k_{h2} [kN/m^3]**	**k_{h3} [kN/m^3]**
top	*bottom*	*mean*	*mean*	*Mean*
5	-5	32500	16250	8125
-5	-8	19500	9750	4875
-8	-9,3	6500	3250	1300
-9,3	-17	32500	16250	8125
-17	-20	19500	9750	4875
-20	-23,5	6500	3250	1300
-23,5	-50	65000	32500	16250

Table D-2: Mean modulus of sub-grade reaction

The geometrical conditions are based on probability calculations. The free water level must be analyzed and a probability distribution function must be made. A safety factor should be applied on the standard deviations of the free and groundwater level. For the harbor bottom depth a standard value is added to the mean harbor depth.

Geometrical		**Safety** γ	Δ	*mean*	*design*
Harbor bottom	m NAP	-0,4 m	-	-12	-12,40
Free water LW (50 years)	m NAP	0,6	0,27	-1,22	-1,38
Groundwater HW	m NAP	2	0,25	0,8	1,30

Table D-3: Geometrical design values, from probability analysis

No safety factors are applied on the variable unfavorable loads working on the superstructure, because the axial load from this calculation should be applied on top of the wall for the sheet pile calculation. However, the terrain load behind the superstructure must be valid for an extreme case. In chapter 6 a characteristic terrain load is determined, but no extreme loads are given. Therefore the characteristic load will be increased with a factor 1,25 to get an extreme terrain load. This factor can be seen as safety factor.

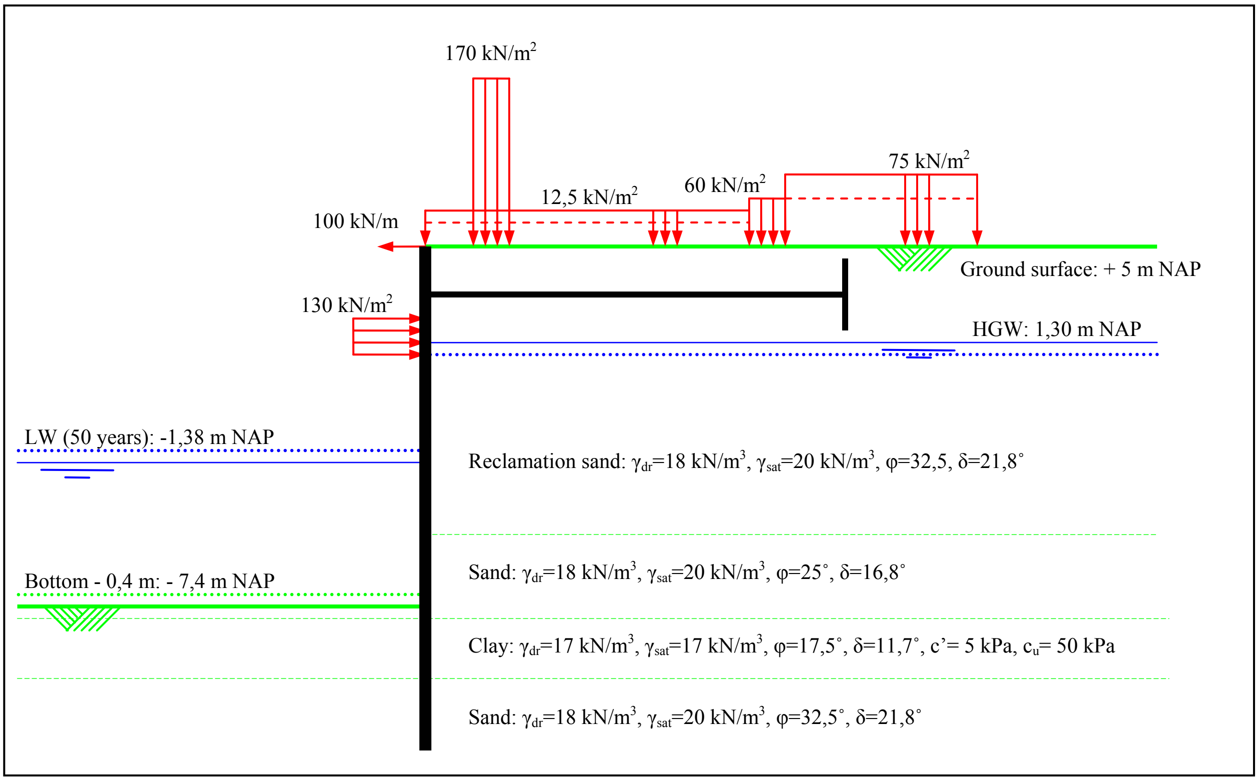


Figure D-2: Design values (situation with high groundwater and low free water level) for sheet pile calculation

D.1.2 Calculating minimum toe level

Normally the minimum toe level is based on the vertical bearing capacity, but due to the absence of an axial load, the depth must be based on horizontal bearing capacity. A safety factor of 1,3 must be maintained between mobilized and maximum passive earth pressure. The minimum toe level is reached when 77% of the passive earth pressure is mobilized. This leads to a toe level of -9,80 m NAP. This toe level belongs to a special load combination with extreme scour of 1 m with terrain and crane load. The fundamental situation leads to a lower minimum toe level of -9,60 m NAP for a combination with terrain and crane load.

D.1.3 Toe level of - 12 m NAP

For the assumed toe level of -12 m NAP fundamental en special load combinations are calculated to find the normative loading situation.

Load combination	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Fundamental (including combination factor 0,7)		
Terrain, Crane	513	407
Terrain, Crane, Bollard	496	410
Special (including combination factor 0,5)		
Terrain, Crane	**600**	**445**
Terrain, Crane, Bollard	586	444

Table D-4: Calculations WITHOUT safety factors on action effects

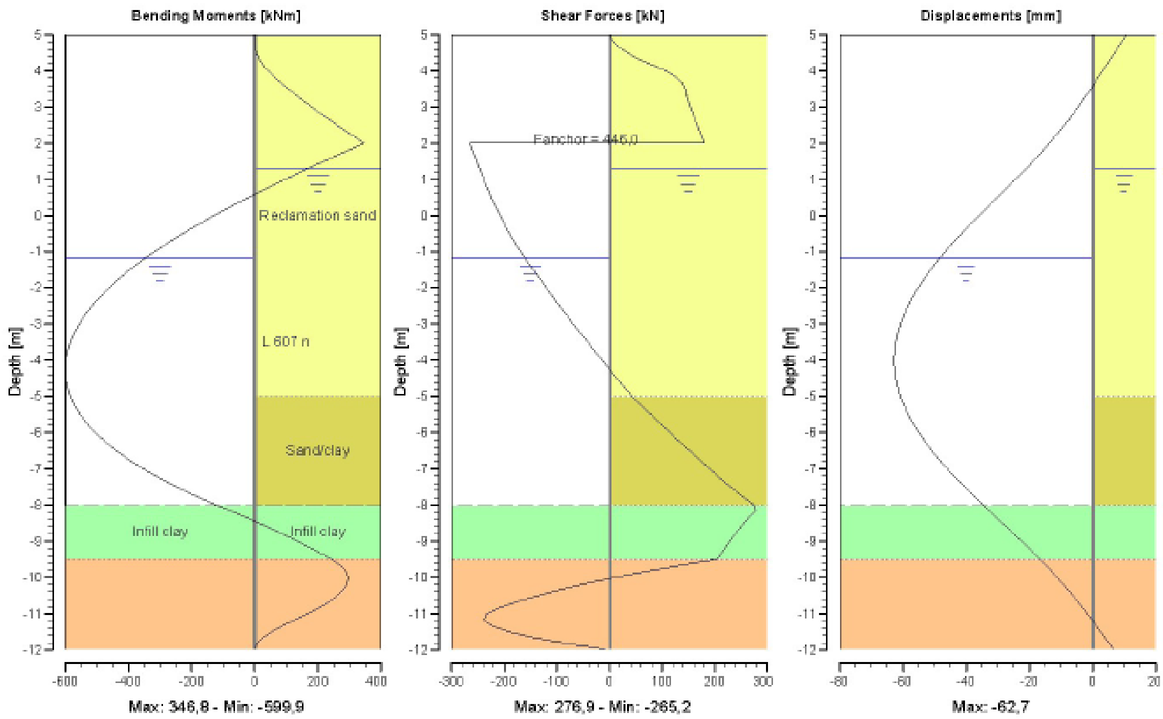


Figure D-3: Calculation with toe level of -12 m NAP, special situation with extreme scour, terrain and crane load

The design values must be calculated with partial safety factors on the action effects.

$$M_d = \gamma_{PF} \cdot M_{ch} = 1{,}3 \cdot 600 = 780 kNm/m$$
$$F_{a,d} = \gamma_{PF} \cdot \gamma_{PF,overall} \cdot F_{a,ch} = 1{,}2 \cdot 1{,}2 \cdot 445 = 641 kN/m$$

Stresses must be checked:

$$\frac{\sigma_{yielding}}{\gamma_{steel}} \geq \frac{M_{\max;d}}{W} + \frac{N'_d}{A} = \frac{780kNm/m}{3200cm^3/m} + 0N/mm^2 = 244N/mm^2 \leq \frac{355N/mm^2}{1{,}0}$$

D.2 CUR 166, Sheet pile structures

The design philosophy of CUR 166 is described in chapter 8. ***This guideline also refers to Handbook Quay Walls for the design of quay wall structures.*** But in this study the quay wall is calculated according to the CUR 166.

D.2.1 Step 1,2,3: Determine normative conditions, representative and design values

The nominal conditions are described in the chapter 6 and annex C and will be summarized. The design philosophy is based on:

- Safety Class III (β = 4,2)
- Life time of 50 years

Design values

In CUR 166 safety factors for limit state 1A must be applied on the **soil strength parameters: tan(φ), δ and c'**. The angle of inclination is determined by the design value of the internal friction: 0,67*φ_d. The result of these safety factors is an **increase of the active earth pressure coefficient and a reduction of the passive earth pressure** coefficient.

For the beam on elastic foundation program the modulus of sub-grade reaction is used and based on Dutch experience values. Low and high values are given to find the normative design situation. On the low bedding constants a safety factors of 1,3 can be applied.

Layer [m NAP]		**k_{h1} [kN/m^3]**			**k_{h2} [kN/m^3]**			**k_{h3} [kN/m^3]**		
top	*bottom*	*low rep*	*low design*	*high rep/design*	*low rep*	*low design*	*high rep/design*	*low rep*	*low design*	*high rep/design*
5	-5	20000	15385	45000	10000	7692	22500	5000	3846	11250
-5	-8	12000	9231	27000	6000	4615	13500	3000	2308	6750
-8	-9,3	4000	3077	9000	2000	1538	4500	800	615	1800
-9,3	-17	40000	30769	90000	20000	15385	45000	10000	7692	22500
-17	-20	20000	15385	45000	10000	7692	22500	5000	3846	11250
-20	-23,5	4000	3077	9000	2000	1538	4500	800	615	1800
-23,5	-40	40000	30769	90000	20000	15385	45000	10000	7692	22500

Table D-5: Design values with safety factor 1,3 for low bedding parameters and 1,0 for high bedding parameters

Safety is applied on the retaining height and the groundwater level at the high and the low side of the sheet pile wall. For the retaining height this means a lowering of the bottom of 0,35 m till - 7,35 m NAP. The groundwater level at the free water side can be neglected, because of the free water level. On the groundwater level of the high side a safety factors of 0,05 m is applied. This results in a groundwater level of 0,64 m.

Only for variable, unfavorable loads a safety factor of 1,25 is available for safety level III, which belongs to quay walls.

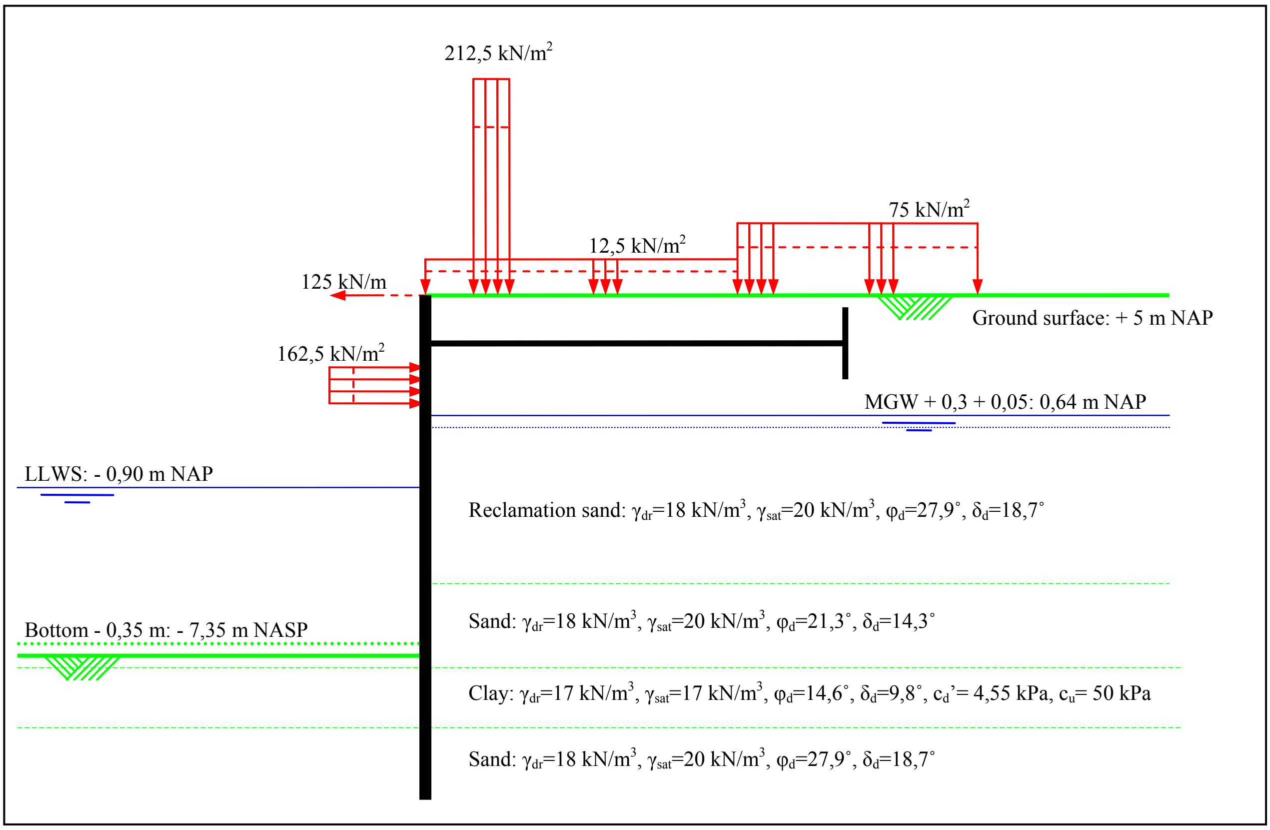


Figure D-4: Design values (situation without drainage system) for sheet pile calculation for limit state 1A

D.2.2 Step 4: Choose a design scheme

The calculation will be done for scheme B, because this will probably give the most optimized sheet pile profile. However, no construction phases are calculated, which will result in a calculation with only design values.

D.2.3 Step 5: Calculation of the minimum toe level

The minimum toe level occurs when 100% of the passive earth pressure is mobilized. This will be reached at a depth of - 9,55 m NAP for application of the terrain and crane load.

D.2.4 Step 6: Calculations for dimensioning of the wall

The maximum moments and anchor forces are calculate for the assumed toe level of -12 m NAP, design low and high modulus of sub-grade reaction and representative values.

Load combination	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Low modulus of sub-grade reaction		
Terrain	**648**	433
Terrain, Crane	632	506
Terrain, Crane, Bollard	582	**531**
High modulus of sub-grade reaction		
Terrain	537	394
Terrain, Crane	524	458
Terrain, Crane, Bollard	474	511
Representative values, low representative modulus of sub-grade reaction		
Gives in all situations smaller moments and anchor forces		

Table D-6: Calculations WITHOUT safety factors on action effects for anchors

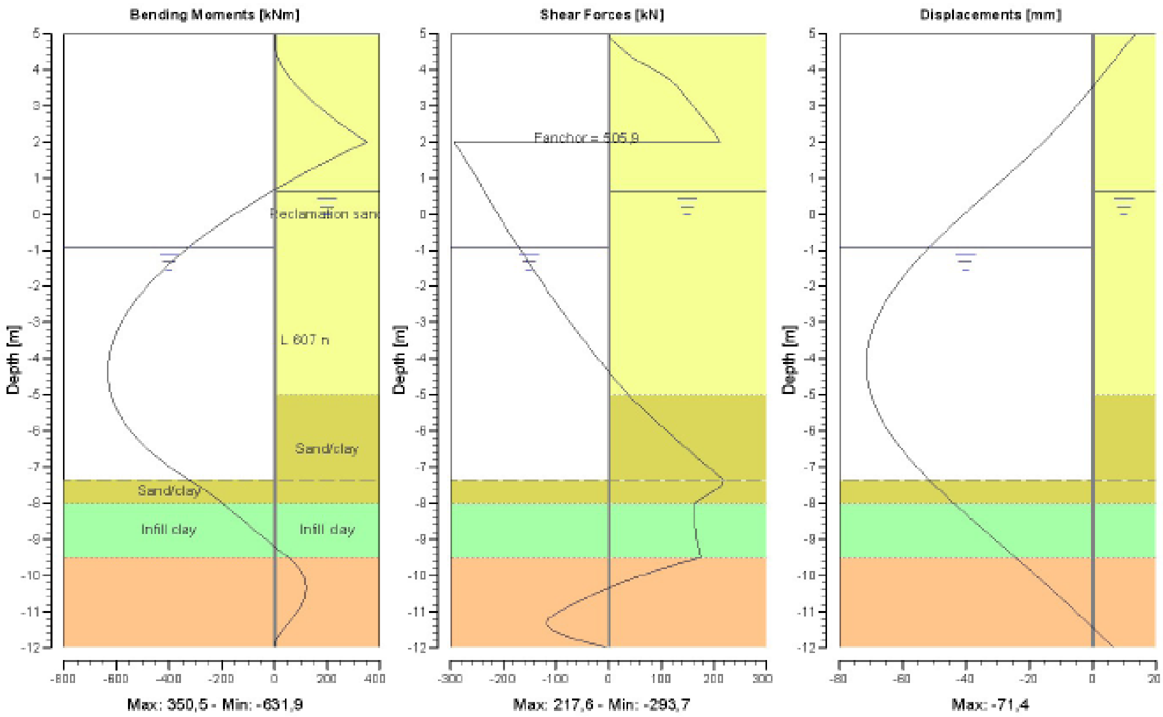


Table D-7: Calculation with toe level of -12 m NAP, with terrain and crane load

For the anchor profile the anchor force must be increased with a safety factor:

$$M_d = 648 kNm/m$$

$$F_{a,d} = F_{a,ch} \cdot \gamma_{PF} = 1{,}25 \cdot 531 = 663 kN/m$$

D.2.5 Step 7: Check of the moment

The maximum bending moment must be checked:

$$M_{r;d} = \frac{\beta_B \cdot W \cdot f_y}{\gamma_{steel}} = \frac{1{,}0 \cdot 3200 cm^3/m \cdot 355 N/mm^2}{1{,}0} = 1136 kNm/m \geq M_{s;d} (= 648 kNm/m)$$

D.3 EAU 2004, Waterfront Structures

The design philosophy of EAU 2004 is described in chapter 9. These calculations will be done with a beam on elastic foundation program, but are also compared with calculations done with program based in Blum.

D.3.1 Blum model in beam on elastic foundation program

The Blum schematization requires in all situations fully developed active and passive earth pressures. The displacements in a beam on elastic foundation program must be large enough to reach this fully developed state.

One method to achieve enough displacements is to enlarge the stiffness of the soil. So that a very small displacement causes a plastic earth pressure. The "line" of the soil stiffness becomes very steep in this way. This leads to iteration problems in the computer program, because it is easy for the program to switch between active and passive earth pressure. The reason for this problem is that the distance of passive and active zone is very small due to the steep line of elasticity.

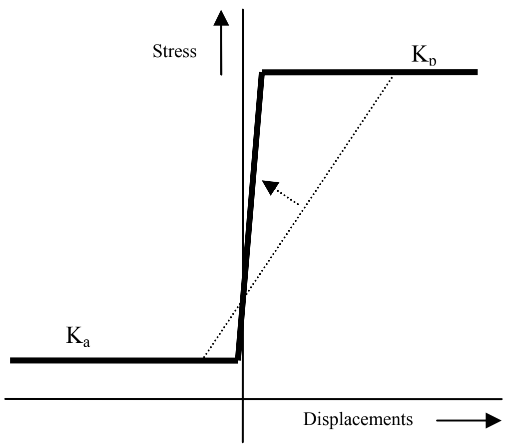


Figure D-5: Schematization of soil stiffness for a beam on elastic foundation model

For a fixed wall the displacements at the toe become very small. For this depth the Blum schematization is not valid anymore.

D.3.2 Determination characteristic, design values

The nominal conditions are described in the chapter 6 and annex C. The design philosophy is based on several loading cases with associate water level situations. For the sheet pile calculations, the safety factors in LC 1 must be applied on the permanent and variable unfavorable actions effects and the resistance. For LC 3 only a safety factor has to be applied on the passive earth pressure. The characteristic values of soil properties will be used for the loading cases in limit state 1B. The angle of inclination will be determined with 0,67*φ.

A safety factor must be applied on the retaining height. The harbor bottom should be lowered with 0,5 m till - 7,5 m NAP.

The water levels are determined from standard (see chapter 6 and annex C) water level situation 3a (LC 1), 3b (LC 3) and 3c (LC 3). Situation 3d (LC 1 and LC 2) will be neglected, because no values about the reflux congestion are available.

The loads on the terrain will be used as characteristic values. Safety factors will be applied on the action effects.

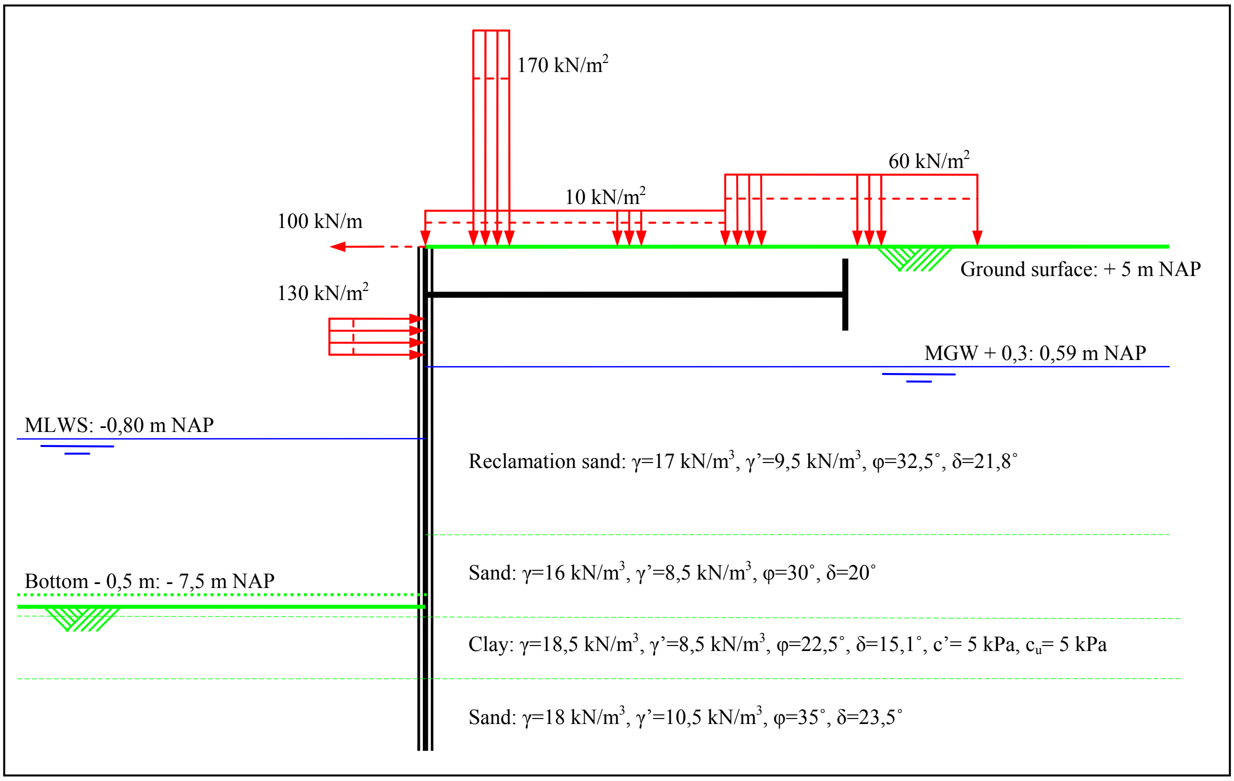


Figure D-6: Design values for the sheet pile calculation for LC1, situation 3a

D.3.3 Reduction of safety factor for hydrostatic pressure

No clear dependencies of the free water and the groundwater are proved, no numerical model of the free water is applied and no clear geometrical boundary is present. Non of the conditions necessary for the reduction of the safety factor on the hydrostatic pressure satisfies, so **no reduction** will be applied.

D.3.4 Reduction of the safety factor for passive earth pressure

For the reduction of the passive earth pressure safety factor there are 4 cases available. The toe of the wall is mainly placed in a clay layer with soft consistency. This corresponds with case 4, which describes that there are layers of lower strength and consistency available. For this case **no reduction** of the passive earth pressure safety factor have to be applied.

D.3.5 Load combinations

The normative situation must be determined for water level situation 3. For situation 3d the reflux congestion of the groundwater is needed. This reflux congestion is not available, because groundwater levels are not measured. Therefore this situation will be skipped for the comparative analysis. The loading cases LC 1 (3a) and LC 3 (3b and 3c) are taken into account for the sheet pile calculation.

D.3.6 Redistribution active earth pressure (in German: "Umlagerung")

The anchor support is placed at + 2 m NAP, with a retaining height of 12 m. This distance between top of the wall and anchor is 3 m. The construction method in this case is trenching in front of the wall. This results in redistribution diagram "case 3".

Excavation has taken place in front of the wall, the cohesive layer behind the wall are consolidated over many years and the sheet pile wall is has certain deformations. With these conditions a **redistribution of active earth pressure may take place**.

This kind of redistribution belongs to the Blum method. It is difficult to apply a redistribution in a beam on elastic foundation model, because distribution of earth pressure is automatically determined by the soil layers. So this **redistribution will be considered but not applied**.

D.3.7 Minimum toe level

With the beam on elastic foundation program it is not possible to calculated a correct minimum toe level. In the Blum program a minimum toe level is reached for a depth of -13,22 m NAP. This is a much lower level than Handbook Quay alls and CUR 166, but in this calculation the passive earth pressure at the place of the anchor support is not taken into account.

D.3.8 Toe level of - 12 m NAP (- 13,22 m NAP)

First the EAU 2004 partial safety factor approach is applied in the beam on elastic foundation model. For this calculation the stiffness is schematized with high and low modulus of sub-grade reaction for the CUR 166. This leads to smaller bending moments and anchor forces than the CUR 166 and Handbook Quay Walls. The Blum schematization does not satisfy for this case and does not lead to reliable results.

Load combination	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Low modulus of sub-grade reaction form CUR 166		
Terrain and crane load	560	489
High modulus of sub-grade reaction from CUR 166		
Terrain and crane load	497	454

*Table D-8: Calculation with high and low CUR 166 **modulus of sub-grade reaction** in combination with EAU 2004 partial safety factors for LC1*

It is not possible to make calculations for a toe level of -12 m NAP with the Blum program, because for this level the minimum level is not yet reached. Therefore the moments and anchor forces are calculated for a depth of - 13,22. Different safety factors on action effects are available for permanent and variable effect. The largest bending moments are resulting from water level situation 3a with loading case 1.

Load combination	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Calculations with GGU retain without redistribution		
Terrain and crane load	1786	550
Calculations with GGU retain with redistribution according to EAU 2004		
Terrain and crane load	1483	586

Table D-9: Calculation for the GGU Retain software packages, with and without redistribution of active earth pressure according to EAU 2004

The permanent and variable contributions to the action effects can be calculated as follows:

$$M_d = M_{P,ch} \cdot \gamma_{PF} + M_{V,ch} \cdot \gamma_{PF} = 786 \cdot 1{,}35 + 281 \cdot 1{,}5 = 1483 kNm/m$$
$$F_{a,d} = F_{P,ch} \cdot \gamma_{PF} + F_{V,ch} \cdot \gamma_{PF} = 290 \cdot 1{,}35 + 130 \cdot 1{,}5 = 586 kN/m$$

This large bending moment in GGU Retain occurs due to the fact that no passive earth pressure is taken into account for the high side of the wall, at the place of the anchor support.

D.3.9 Verification of sheet pile profile

The bending moments from the beam on elastic foundation program and the GGU Retain program will be checked:

$$M_{r;d} = W \cdot f_y = 3200 cm^3/m \cdot 355 N/mm^2 = 1136 kNm/m \leq M_{s;d} (= 1483 kNm/m)!!!!$$
$$M_{r;d} = W \cdot f_y = 3200 cm^3/m \cdot 355 N/mm^2 = 1136 kNm/m \geq M_{s;d} (= 560 kNm/m)$$

D.4 Final considerations of the design recommendations

All calculation results are summarized in the table D-10. The results are compared for relative safety with the CUR 166. Conclusions about these results are drawn in chapter 10.

CASE 1 (annex D)	Minimum toe level [m NAP]	Toe level [m NAP]	Maximum bending moment [kNm/m]	Percentage difference too CUR 166	Anchor force for anchor profile [kN/m]	Percentage difference too CUR 166	Anchor force for bearing capacity [kN/m]
CUR 166	- 9,55	- 12,00	648	-	663	-	583
Handbook Quay Walls	- 9,80	- 12,00	780	+ 20%	641	- 3%	534
EAU 2004 (spring system)	-	- 12,00	560	- 16%	489	- 36%	489
EAU 2004 (GGU retain)	-13,22	-13,22	1483	+ 129%	586	- 13%	586

Table D-10: Results of the sheet pile calculations and optimization in previous calculations

E Sheet pile calculations case 2

The calculations as described before in CUR 166, Handbook Quay Walls and EAU 2004 (chapter 7, 8 and 9) are carried out in this annex for case 2 (retaining height 30 m and superstructure). Not all aspects of the design process will be treated. The calculations will focus on the sheet pile calculation. For a good comparative analysis the internal forces and anchor forces are calculated for a wall with a toe level of -35 m NAP. This is assumed as a depth that gives enough vertical bearing capacity for a wall with a large axial load on top. The representative/ characteristic values are schematized below.

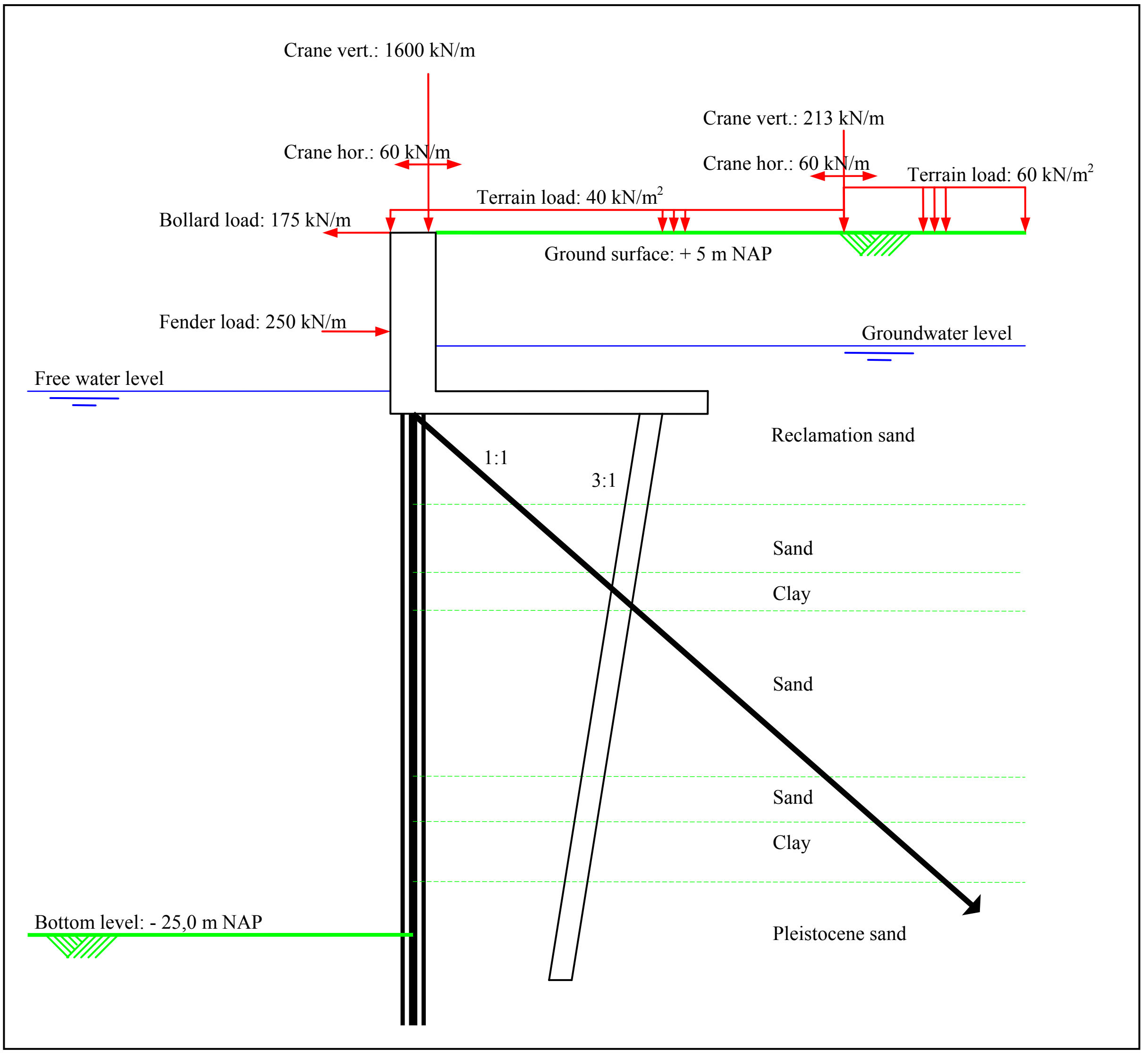


Figure E-1: Representative/ Characteristic values for the sheet pile calculation of case 2

For the comparative analysis the wall properties are kept the same.

Material properties	
Elasticity modulus steel	$210*10^3$ N/mm3
Steel quality steel tube	X70
Yielding force for steel tube	483 N/mm2
Combined sheet pile wall:	
Tubes:	
Diameter	2500 mm
Thickness	24 mm
Infill piles:	
Profile	PU 18
W	1800 cm^3/m
Depth (due to piping criteria)	-27 M NAP
Combined wall:	
W	19341 cm^3/m
MV-pile:	
Profile	HP 360 × 152
Cross-section	193,8 cm^2
Perimeter	2,153 m
Angle of with wall anchor	45 °
Centre to centre distance (longitudinal)	5 m

Table E-1: Material properties

E.1 CUR 211, Handbook Quay Walls

The design philosophy of Handbook Quay Walls is described in chapter 7.

E.1.1 Determination representative and design values

The nominal conditions are described in the chapter 6 and Annex C. The design philosophy is based on:
- Safety Class 2 (β=3,4)
- Life time 50 years

The design philosophy in Handbook Quay Walls is based on **representative soil properties** for the sheet pile calculation. The angle of inclination is a fraction of the angle of internal friction of the soil layer: 0,67*φ_d.

In contradiction with CUR 166, the Handbook Quay Walls applies the **mean modulus of sub-grade reaction** for the stiffness of the soil. With these mean values the stiffer and less stiff parts of the soil behind the wall over the longitudinal direction are redistributed.

Layer [m NAP]		**k_{h1} [kN/m^3]**	**k_{h2} [kN/m^3]**	**k_{h3} [kN/m^3]**
top	*bottom*	*mean*	*mean*	*Mean*
5	-5	32500	16250	8125
-5	-8	19500	9750	4875
-8	-9,3	6500	3250	1300
-9,3	-17	32500	16250	8125
-17	-20	19500	9750	4875
-20	-23,5	6500	3250	1300
-23,5	-50	65000	32500	16250

Table E-2: Mean modulus of sub-grade reaction

The geometrical conditions are based on probability calculations. The free water level must be analyzed and a probability distribution function must be made. A safety factor should be applied on the standard deviations of the free and groundwater level. For the harbor bottom a standard value is added to the mean harbor bottom.

Geometrical		**Safety** γ	Δ	*mean*	*design*
Harbor bottom	m NAP	-0,4 m	-	-25	-25,40
Free water LW (50 years)	m NAP	0,6	0,27	-1,22	-1,38
Groundwater HW	m NAP	2	0,25	0,8	1,30

Table E-3: Geometrical design values, from probability analysis

No safety factors are applied on the variable unfavorable loads working on the superstructure, because the axial load from this calculation should be applied on top of the wall for the sheet pile calculation. However, the terrain load behind the superstructure must be valid for an extreme case. In chapter 6 a characteristic terrain load is determined, but no extreme loads are given. Therefore the characteristic load will be increased with a factor 1,25 to get an extreme terrain load. This factor can be seen as safety factor.

E.1.2 Redistribution of loads through the superstructure for normative load combination

The loads working on the superstructure must be transferred to the foundation elements. The stiffness of the superstructure gives the degree of distribution of the loads: a stiffer structure gives larger redistributions. The stiffness of the structure must be determined in the plane of the wall and perpendicular to the wall. This could be done with the application of a reference load of 100 kN/m. This is not done for the comparative analysis, because it needs some detailed calculations with for example a finite element program. It is assumed that the loads redistribute over an angle of 45˚ to the stiff relieving platform of the superstructure.

From this redistribution the loads working on the foundation elements can be determined. The normative load combination is needed for calculation of the combined wall, MV-pile and pile foundation. The superstructure is assumed as stiff static structure, so that the distribution of loads can be calculated with simple equations.

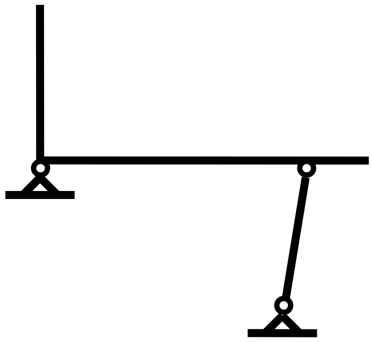


Figure E-2: Static superstructure for calculations of the foundation elements

For the comparative analysis only the load situation of the combined wall and the anchor force are important. This is carried out for case 2. The axial load resulting from the superstructure calculation must be increased with the anchor force coming from the sheet pile calculation. This is an iterative process and will be carried out in the next paragraphs.

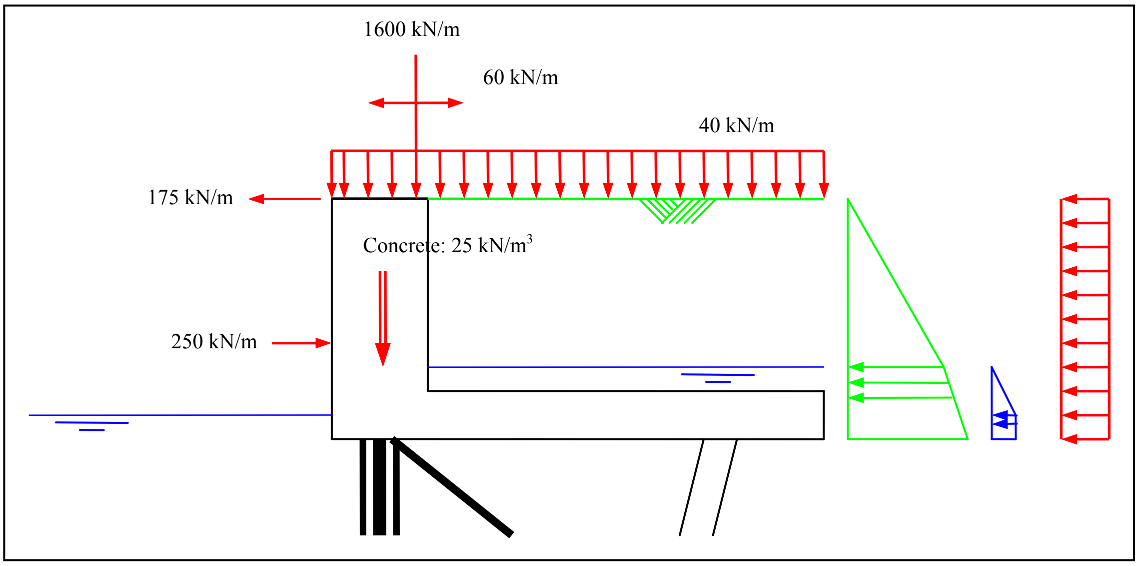


Figure E-3: Loads for the superstructure calculation

Earth and water pressure	Terrain load	Crane load	Bollard load	Fender load	Axial load on combined wall (including vertical anchor addition) [kN/m]	Horizontal load on anchor [kN/m]
Fundamental						
High groundwater and low free water level						
1	0,7	0,7			2530,7	-52,5
1	0,7				1449,5	-30,7
1	**0,7**		**0,7**		**1636,4**	**-169,4**
1		0,7	0,7		2294,6	-88,8
1	0,7			0,7	1238,3	153,4
1		0,7		0,7	1869,1	233,91
Same low ground and low free water level						
1	**0,7**	**0,7**			**2684,5**	**-51,5**
1	0,7		0,7		1790,5	-168,4

Table E-4: Calculation of normative load combination for axial loads on the wall and tensile anchor force according to Handbook Quay Walls

The normative load combination for the axial load is a combination of earth and water pressure together with a terrain and crane load with combination factor 0,7, for a low free and groundwater level. For the anchor force the normative situation occurs for a terrain and bollard load with combination factor 0,7, for a large gradient in free and groundwater level.

These loads must be added, together with the anchor force, to the total axial load working on top of the sheet pile wall.

E.1.3 Redistribution of loads through the sheet pile wall for normative load combination

For the loads working on the sheet pile wall the normative combination can be determined. In these calculations the axial load from the superstructure must be included. As for case 1 fundamental and special load combinations will be overviewed. It is assumed that a wall with a toe level of -35 m NAP gives enough vertical bearing capacity. The sheet pile calculation will be based on this toe level.

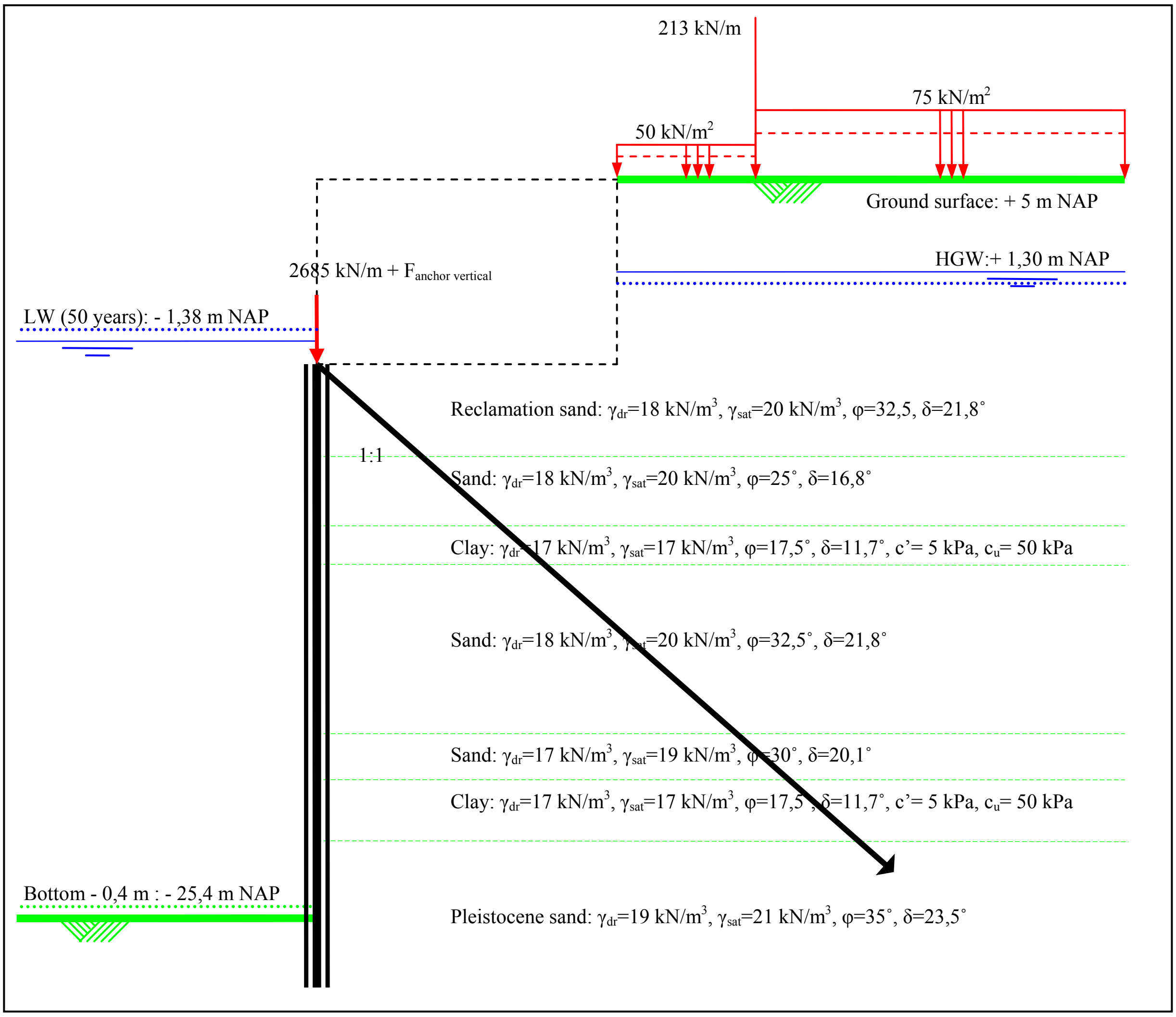


Figure E-4: Design values (situation with high groundwater and low free water level) for sheet pile calculation

E.1.4 Toe level of - 35 m NAP

The bending moments and anchor forces are calculated for a toe level of -35 m NAP.

<u>Load combination</u>	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Fundamental (including combination factor 0,7)		
Terrain, Crane	3602	<u>**752**</u>
Special (including combination factor 0,5)		
Terrain, Crane	<u>**3658**</u>	740

Table E-5: Calculations <u>WITHOUT</u> safety factors on action effects and eccentricity effects

Due to eccentricity the moment will be enlarged with 172 kNm/m. This leads to a maximum bending moment of 3830 kNm/m.

$$M_d = \gamma_{PF} \cdot M_{ch} = 1{,}3 \cdot 3830 = 4979 kNm/m$$

$$F_{a,d} = \gamma_{PF} \cdot \gamma_{PF,overall} \cdot F_{a,ch} = 1{,}2 \cdot 1{,}2 \cdot 752 = 1083 kN/m$$

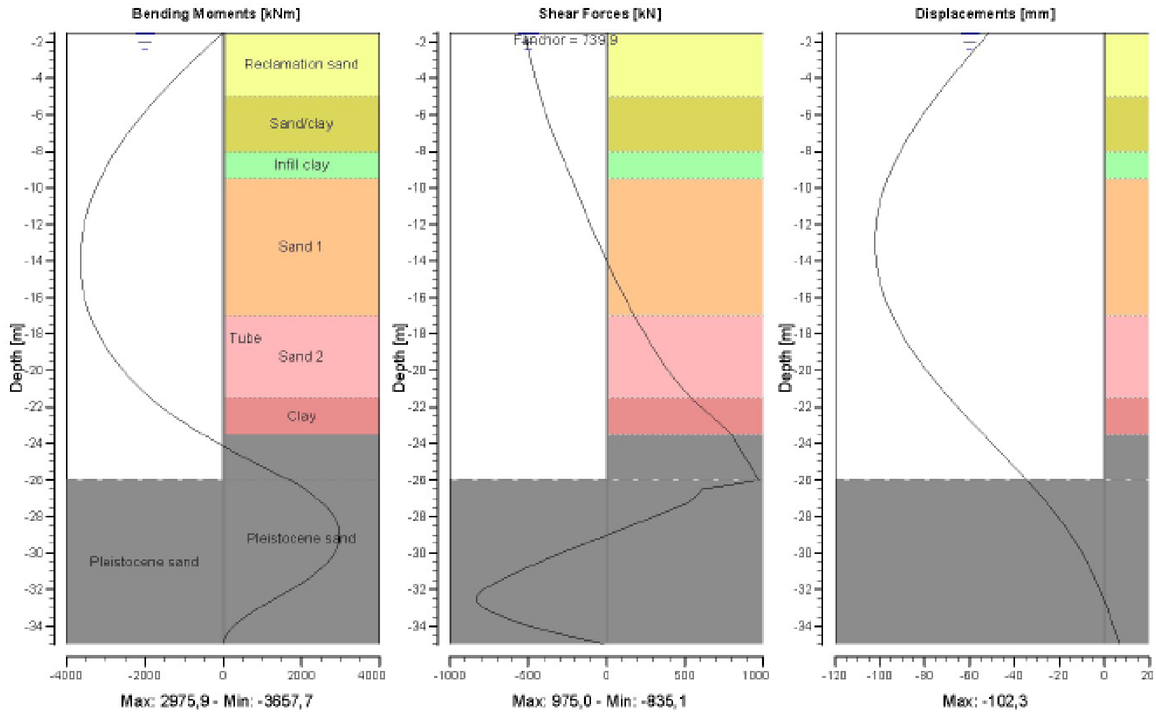
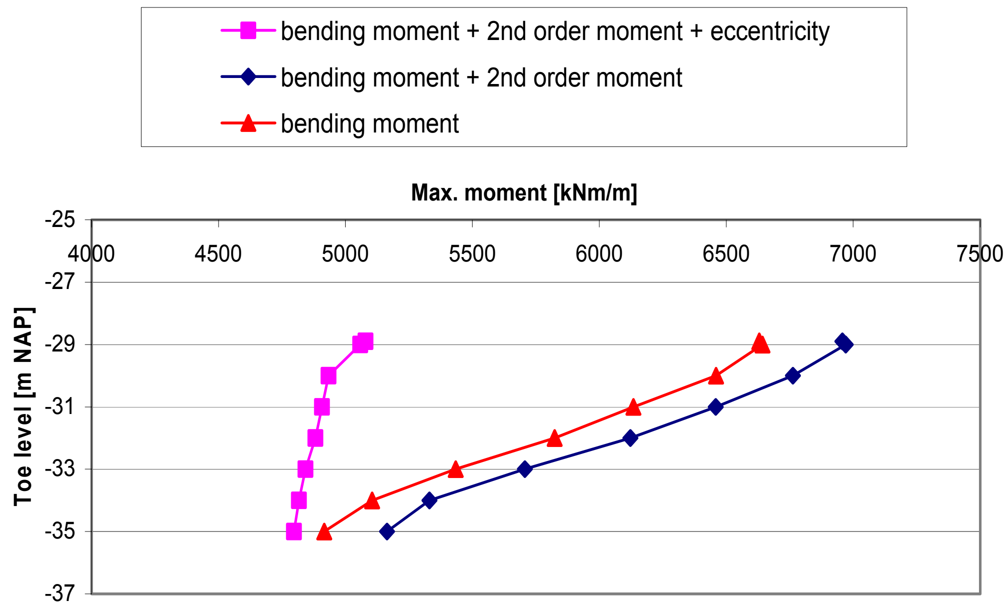


Figure E-5: Calculation with toe level of - 35 m NAP, special situation with extreme scour, terrain and crane load

With the considered axial load from the previous calculations a sheet pile calculation can be done. The representative axial load will be applied at the top of the sheet pile. This will cause a second order moment and depending on the structure also an eccentricity. The eccentricity introduces a bending moment due to the axial load coming from the superstructure and vertical load from the anchor force. It is assumed that the summation of loads is positioned at 1 m from the centre of the profile (tube), so that the **eccentricity works at 1 m from the centre of the wall**. This calculation is an iterative process.



Graph E-1: Maximum bending moment for different toe levels, including eccentricity and 2nd order moment, with the axial load from the superstructure calculation (2685 kN/m)

It is also possible to apply a plastic spring at the toe of the wall. The load working on the toe is a multiplication of the vertical toe resistance force with the tangent of the angle of internal friction of the foundation layer (see figure E-6). This will not be taken into account, because no calculations are made for the vertical bearing capacity.

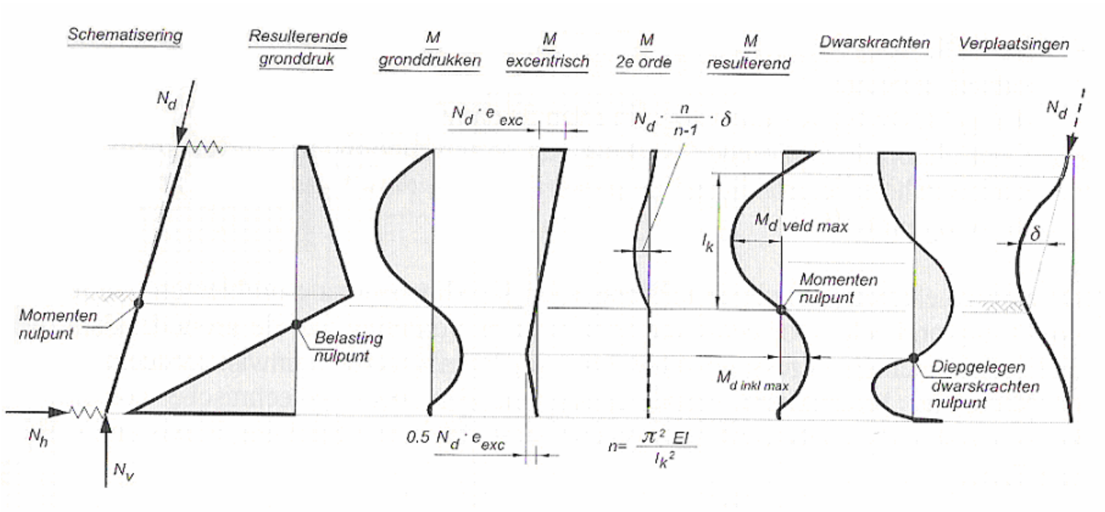


Figure E-6: Special additions for quay walls with superstructure: plastic spring, eccentricity, 2nd order moment

It is clear that the eccentricity reduces the maximum moment. In this case the moment due to eccentricity at the top of the wall is very large. It is higher than the moments in the field and at the toe. **For the comparative analysis the eccentricity will not be taken into account. Only second order moments will be added to the bending moment.**

The stiffness of the combined profile should be taken into account for the calculation of the sheet pile wall. The resistance of the bending moment will only be taken by the tubes in the wall. The difference in stiffness between the infill piles and the tubes is so large that probably arching will occur, so that the tubes take almost all the internal forces. The infill piles are mainly loaded by the hydrostatic pressure.

$$I_{tube} = \frac{\pi}{64} \cdot (D_o^4 - D_i^4) = \frac{\pi}{64} \cdot (2{,}5125^4 - 2{,}4875_i^4) = 0{,}0767 m^4$$

$$\Rightarrow \frac{0{,}0767 m^4}{4{,}3m} = 0{,}0178 m^4 / m$$

$$W = \frac{I}{0{,}5 \cdot D} = \frac{0{,}0178 m^4 / m}{0{,}5 \cdot 2{,}5m} = 0{,}01458 m^3 / m$$

$$\sigma_{yielding} = \frac{M_{\max;d}}{W} + \frac{N'_d}{A} = \frac{1{,}3 \cdot 3830 kNm / m}{0{,}01458 m^3 / m} + \frac{1{,}3 \cdot (752 + 2685) kN / m}{0{,}04566 m^2 / m}$$

$$\sigma_{yielding} = 341495 + 97856 = 439351 kN / m^2 = 440 N / mm^2 \leq 483 N / mm^2$$

This profile satisfies for this type of wall. The yielding stress is not yet reached, but this extra thickness of the wall can be used for corrosion.

E.2 CUR 166, Sheet pile structures

The design philosophy of CUR 166 is described in chapter 8. This guideline also refers to Handbook Quay Walls for the design of quay wall structures. But in this study the quay wall is calculated according to the CUR 166.

For the calculation of the sheet pile wall the axial load is used that is calculated for the Handbook Quay Walls. No determination of loads from a superstructure is mentioned in the CUR 166.

E.2.1 Step 1,2,3: Determine normative conditions, representative and design values

The nominal conditions are described in the chapter 6 and annex C will be summarized. The design philosophy is based on:

- Safety Class III (β = 4,2)
- Life time of 50 years

Design values

In CUR 166 safety factors for limit state 1A must be applied on the soil strength parameters: tan(φ), δ and c'. The angle of inclination is determined by the design value of the internal friction: $0,67*\varphi_d$. The result of these safety factors is an increase of the active earth pressure coefficient and the a reduction of the passive earth pressure coefficient.

For the beam on elastic foundation program bedding parameters are used and based on Dutch experience values. Low and high values are given to find the normative design situation. On the low bedding constants a safety factors of 1,3 can be applied.

Layer [m NAP]		**k_{h1} [kN/m^3]**			**k_{h2} [kN/m^3]**			**k_{h3} [kN/m^3]**		
top	*bottom*	*low rep*	*low design*	*high rep/design*	*low rep*	*low design*	*high rep/design*	*low rep*	*low design*	*high rep/design*
5	-5	20000	15385	45000	10000	7692	22500	5000	3846	11250
-5	-8	12000	9231	27000	6000	4615	13500	3000	2308	6750
-8	-9,3	4000	3077	9000	2000	1538	4500	800	615	1800
-9,3	-17	40000	30769	90000	20000	15385	45000	10000	7692	22500
-17	-20	20000	15385	45000	10000	7692	22500	5000	3846	11250
-20	-23,5	4000	3077	9000	2000	1538	4500	800	615	1800
-23,5	-40	40000	30769	90000	20000	15385	45000	10000	7692	22500

Table E-6: Design values with safety factor 1,3 for low bedding parameters and 1,0 for high bedding parameters

Safety is applied on the retaining height and the groundwater level at the high and the low side of the sheet pile wall. For the retaining height this means an lowering of the bottom of 0,35 m till - 25,35 m NAP. The groundwater level at the free water side can be neglected, because of the free water level. On the groundwater level of the high side a safety factors of 0,05 m is applied. This results in a depth of 0,64 m.

Only for variable, unfavorable loads a safety factor of 1,25 is available. For other external loads, representative values must be used.

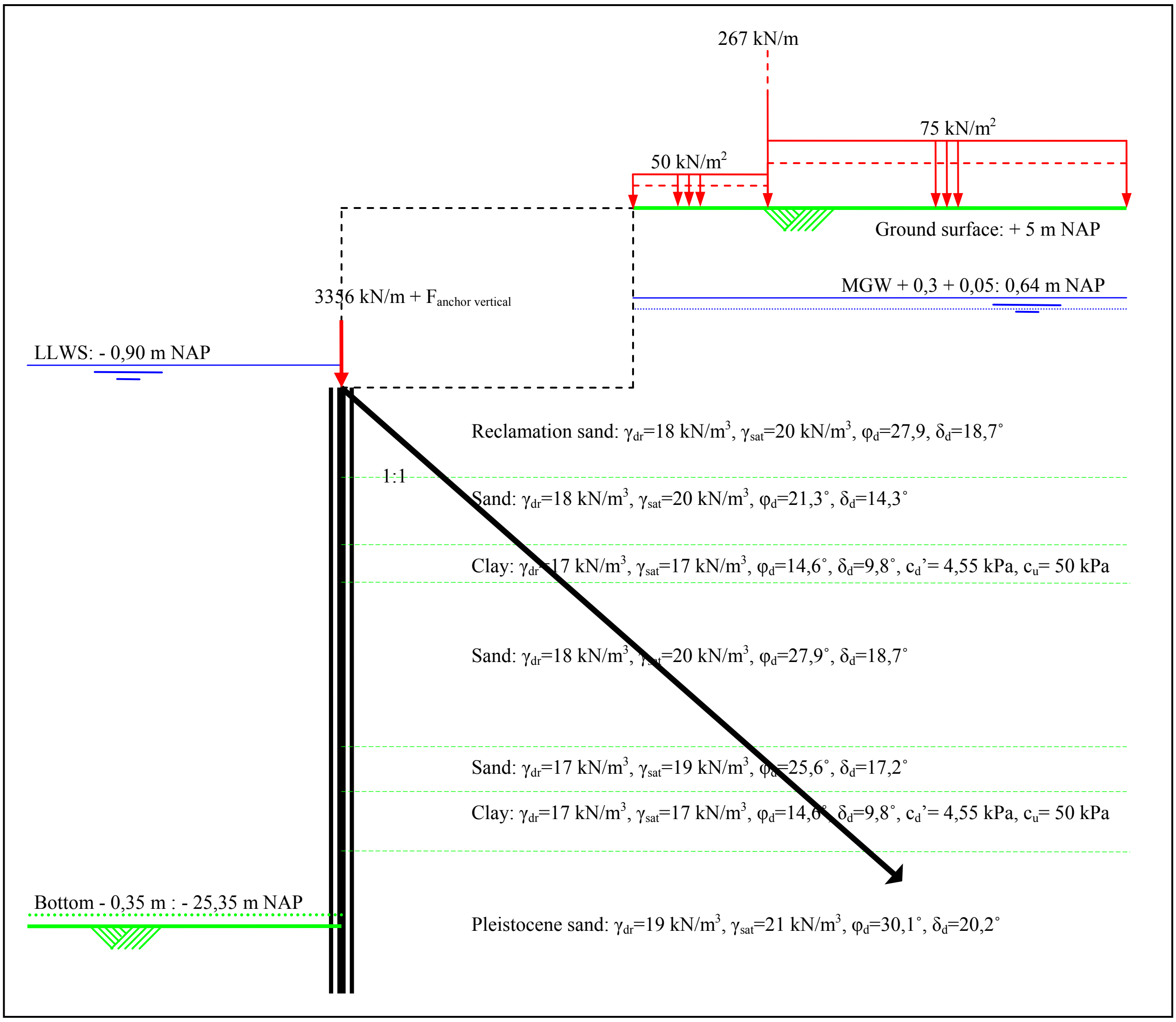


Figure E-7: Design values (situation without drainage system) for sheet pile calculation according CUR 166

The 2nd order moment and eccentricity are mentioned in CUR 166, but only 2nd order moments are described in detail. The axial load that works on the top of the wall is a combination of permanent and variable loads. The CUR 166 applies a **safety factor on unfavorable variable loads**. The **axial load** in this case is assumed as variable unfavorable, because the crane load which has a lot of influence on the axial load is a variable unfavorable load.

E.2.2 Step 4: Choose a design scheme

The calculation will be done for scheme B, because this will probably give the most optimized sheet pile profile. However, only one stage is calculated, which will result in a calculation with only design values.

E.2.3 Step 5: Calculation of the minimum toe level

The calculation of the minimum toe level is based on 100% mobilized passive soil pressure. This lead to a minimum toe level of - 29,55 m NAP. However, this is not the real minimum level, because that level is based on the vertical bearing capacity.

E.2.4 Step 6: Calculations for dimensioning of the wall

The maximum moments and anchor forces are calculate for the assumed toe level of -35 m NAP, design low and high modulus of sub-grade reaction and representative values.

Modulus of sub-grade reaction *without 2nd order effect*	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Low	**5538**	**870**
High	4728	791
Rep	1,2·2800	1,2·558

Table E-7: Calculations for different modulus of sub-grade reaction, excluding eccentricity

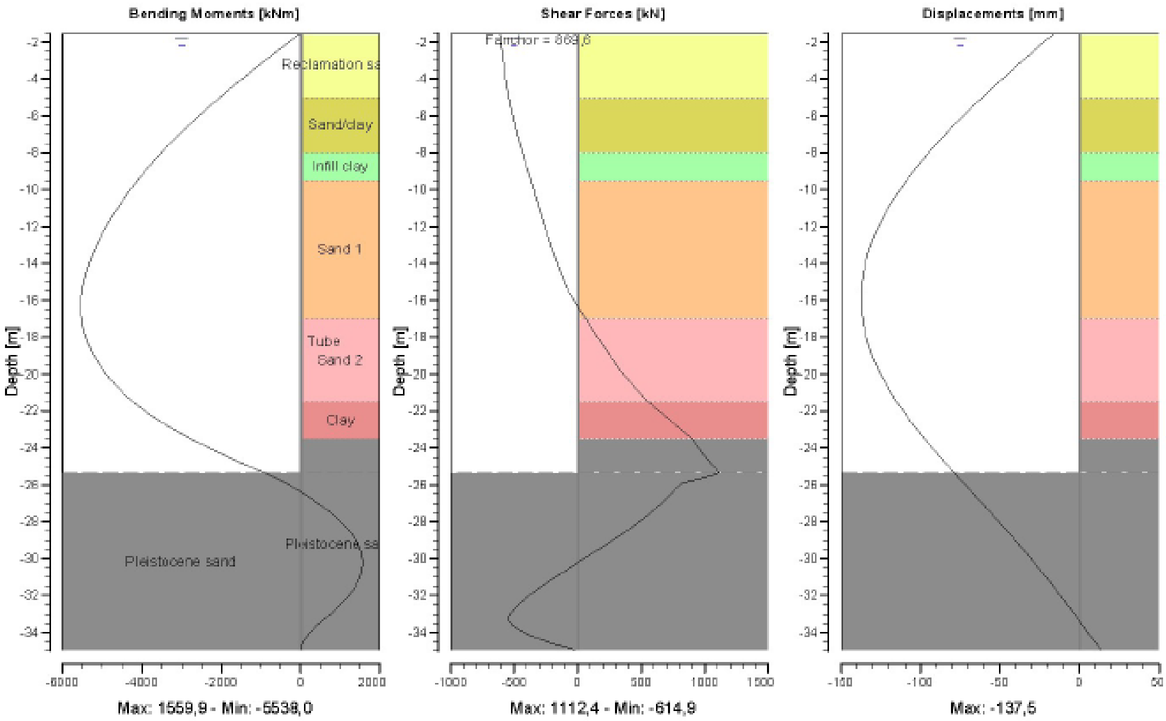


Figure E-8: Calculation with a toe level of -35 m NAP and low modulus of sub-grade reaction (design parameters)

Due to eccentricity the moment will be enlarged with 423 kNm/m. This leads to a maximum bending moment of 5961 kNm/m.

$$M_d = 5961 kNm/m$$

$$F_{a,d} = \gamma_{PF} \cdot F_{a,ch} = 1{,}25 \cdot 870 = 1088 kN/m$$

E.2.5 Step 7: Check of the moment

The maximum bending moment must be checked:

$$I_{tube} = \frac{\pi}{64} \cdot (D_o^4 - D_i^4) = \frac{\pi}{64} \cdot (2{,}5125^4 - 2{,}4875_i^4) = 0{,}0767 m^4$$

$$\Rightarrow \frac{0{,}0767 m^4}{4{,}3m} = 0{,}0178 m^4/m$$

$$W = \frac{I}{0{,}5 \cdot D} = \frac{0{,}0178 m^4/m}{0{,}5 \cdot 2{,}5m} = 0{,}01458 m^3/m$$

$$\sigma_{yielding} = \frac{M_{\max;d}}{W} + \frac{N'_d}{A} = \frac{5961 kNm/m}{0{,}01458 m^3/m} + \frac{(1{,}1 \cdot 870) + (1{,}25 \cdot 2685) kN/m}{0{,}04566 m^2/m}$$

$$\sigma_{yielding} = 408848 + 94465 = 503313 N/m^2 = 503{,}3 N/mm^2 \geq 483 N/mm^2 !!!$$

The profile does not satisfy and a larger profile must be applied.

E.3 EAU 2004, Waterfront structures

The design philosophy of EAU 2004 is described in chapter 9. These calculations will be done with a beam on elastic foundation program, but are also compared with calculations done with program based in Blum.

E.3.1 Blum model in beam on elastic foundation program

The Blum schematization requires in all situations fully developed active and passive earth pressures. The displacements in a beam on elastic foundation program must be large enough to reach this fully developed state.

One method to achieve enough displacements is to enlarge the stiffness of the soil. So that a very small displacement causes a plastic earth pressure. The "line" of the soil stiffness becomes very steep in this way. This leads to iteration problems in the computer program, because it is easy for the program to switch between active and passive earth pressure. The reason for this problem is that the distance of passive and active zone is very small due to the steep line of elasticity.

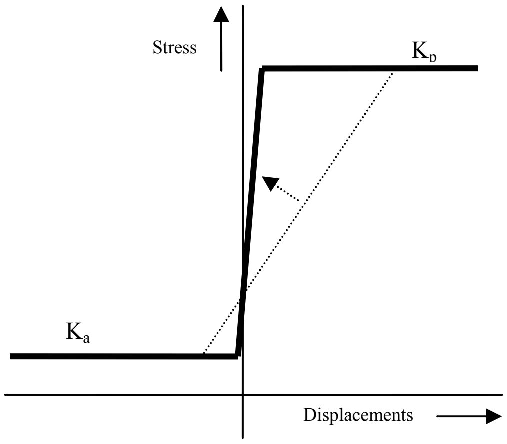


Figure E-9: Schematization of soil stiffness for a beam on elastic foundation model

For a fixed wall the displacements at the toe become very small. For this depth the Blum schematization is not valid anymore.

E.3.2 Determination characteristic, design values

The nominal conditions are described in the chapter 6 and annex C. The design philosophy is based on several load cases with associate water level situations. For the sheet pile calculations, the safety factors in LC 1 must be applied on the permanent and variable unfavorable actions effects and the resistance. For LC 3 only a safety factor has to be applied on the passive earth pressure. The characteristic values of soil properties will be used for the loading cases in limit state 1B. The angle of inclination will be determined with 0,67*φ.

A safety factor must be applied on the retaining height. The harbor bottom should be lowered with 0,5 m till - 25,5 m NAP.

The water levels are determined from standard (see chapter 6 and annex C) water level situation 3a (LC 1), 3b (LC 3) and 3c (LC 3). Situation 3d (LC 1 and LC 2) will be neglected, because no values about the reflux congestion are available. A safety factor must be applied on the hydrostatic pressure for LC 1 and 2.

With the application of a superstructure there is an axial load on top of the wall. This load in mainly due to unfavorable variable loads. On the effect of this axial load a safety factor must be applied. The loads on the terrain will be used as characteristic values. Safety factors will be applied on the action effects.

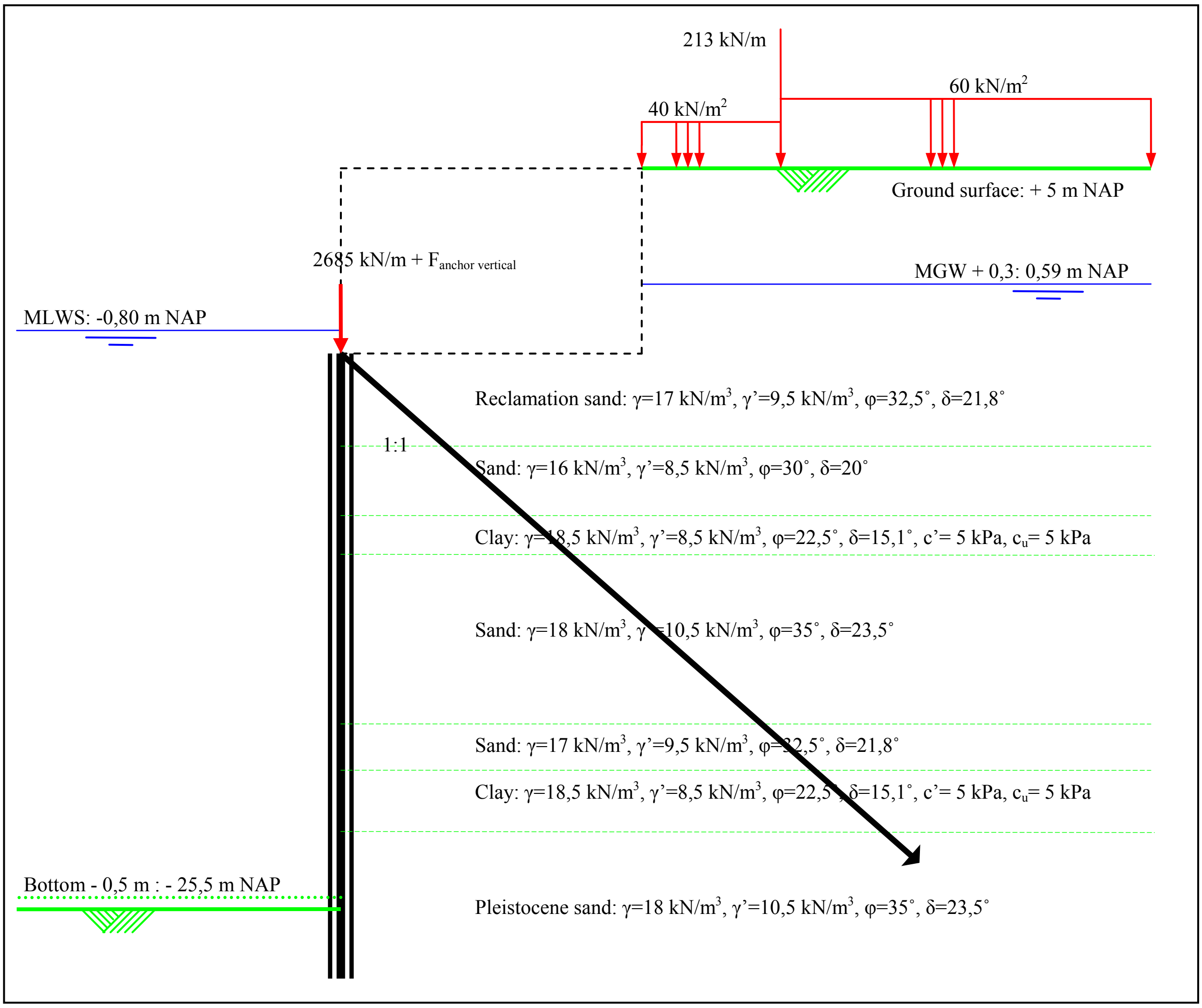


Figure E-10: Design values (situation 3a LC1) for sheet pile calculation

E.3.3 Reduction of safety factor for hydrostatic pressure

No clear dependencies of the free water and the groundwater are proved, no numerical model of the free water is applied and no clear geometrical boundary is present. Non of the conditions necessary for the reduction of the safety factor on the hydrostatic pressure satisfies, so **no reduction** will be applied.

E.3.4 Reduction of the safety factor for passive earth pressure

For the reduction of the passive earth pressure safety factor there are 4 cases available. The toe of the wall is placed in the Pleistocene layer with stiff consistency. This corresponds with case 2, which describes that there are at least layers of medium strength or rather stiff consistency present under the calculation bottom (harbor bottom). For this case **reduction of the passive earth pressure safety factor** have to be applied for calculation of the bending moment.

E.3.5 Load combinations

<u>**Load combinations for superstructure**</u>
No axial load will be calculated according EAU 2004. For the calculations the axial load of Handbook Quay Walls will be applied, because in that recommendation a description is given how this load must be calculated.

<u>**Load combination for sheet pile calculation**</u>
The normative situation must be determined for water level situation 3. For situation 3d the reflux congestion of the groundwater is needed. This reflux congestion is not available, because groundwater

levels are not measured. Therefore this situation will be skipped for the comparative analysis. The loading cases LC 1 (3a) and LC 3 (3b and 3c) are taken into account for the sheet pile calculation.

E.3.6 Redistribution active earth pressure (in German: "Umlagerung")

The anchor support is placed at the top of the wall, with a retaining height of 30 m. This distance between top of the wall and anchor is 0 m. The construction method in this case is trenching in front of the wall. This results in redistribution diagram "case 1".

Excavation has taken place in front of the wall, the cohesive layer behind the wall are small and consolidated over many years and the sheet pile wall has certain deformations, because of the large retaining height. Under these conditions a **redistribution of active earth pressure could take place**.

This kind of redistribution belongs to the Blum method. It is difficult to apply a redistribution in a beam on elastic foundation model, because distribution of earth pressure is automatically determined by the soil layers. So this **redistribution will be considered but not applied**.

E.3.7 Minimum toe level

The normative toe level will be reached for **LC 1 in combination with water level situation 3a**, because the safety factors on action effects are much higher in this LC than for LC 3. For this situation (3a LC1) the minimum toe level is calculated.

The minimum toe level will be calculated in the same way as for case 1, the stiffness of the soil does not play a role for the calculation of the minimum toe level. Only safety factors have to be applied on the resistance (passive earth pressure) at the passive side.

The minimum wall depth for a simply supported wall is reached for a toe level of - 31,7 m NAP. It is clear that no real Blum schematization exist for this case, because a strange loading diagram occurs at the toe of the wall. This is probably due to the difference in stiffness of the wall between the part with and without infill piles. The EAU 2004 uses a staggered toe level for a combined wall. But this method describes staggering lengths of less than 1 m. For larger staggering lengths the load bearing capacity of the longer piles have to be checked. The staggering length of case 2 is much larger than 1 m. Therefore this EAU 2004 approach about staggering is not useful in this case.

E.3.8 Toe level of - 35 m NAP

First the EAU 2004 partial safety factor approach is applied in the beam on elastic foundation model. For this calculation the stiffness is schematized with high and low modulus of sub-grade reaction for the CUR 166. This leads to smaller bending moments and anchor forces than the CUR 166 and Handbook Quay Walls. The Blum schematization does not satisfy for this case and does not lead to reliable results.

Load combination	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Low modulus of sub-grade reaction from CUR 166		
Terrain and crane load	4219	739
High modulus of sub-grade reaction from CUR 166		
Terrain and crane load	3833	697

*Table E-8: Calculation with high and low CUR 166 **modulus of sub-grade reaction** in combination with EAU 2004 partial safety factors for LC1, including 2nd order effect*

The moments and anchor forces are also calculated with the Blum program (GGU Retain). Different safety factors on action effects are available for permanent and variable effect. The largest bending moments are resulting from water level situation 3a with loading case 1 (see table E-9).

Load combination	Maximum bending moment [kNm/m]	Anchor force [kN/m]
Calculations with GGU retain without redistribution		
Terrain and crane load	9912	921
Calculations with GGU retain with redistribution according to EAU 2004		
Terrain and crane load	9314	1078

Table E-9: Calculation for the GGU Retain software packages, with and without redistribution of active earth pressure according to EAU 2004, excluding the influence of the 2nd order effect

Due to eccentricity the moment will be enlarged with 344 kNm/m. The permanent and variable contributions to the action effects can be calculated as follows:

$$M_d = M_{P,ch} \cdot \gamma_{PF} + M_{V,ch} \cdot \gamma_{PF} = (6169 + 344) \cdot 1{,}35 + 657 \cdot 1{,}5 = 9778 kNm/m$$

$$F_{a,d} = F_{P,ch} \cdot \gamma_{PF} + F_{V,ch} \cdot \gamma_{PF} = 752 \cdot 1{,}35 + 42 \cdot 1{,}5 = 1078 kN/m$$

E.3.9 Verification of sheet pile profile

The moments for this method is much larger than the moment for CUR 166. Therefore no more check is done for the cross-section, because the profile for CUR 166 does not satisfies. A profile with larger moment of inertia should be chosen.

E.4 Final considerations of the design recommendations

The calculation results are summarized in table E-10. The results are compared for relative safety with the Handbook Quay Walls. The conclusions about these results will be drawn in chapter 10.

CASE 2 (annex E)	Toe level [m NAP]	Maximum bending moment [kNm/m]	Percentage difference too Handbook Quay Walls	Anchor force for anchor profile [kN/m]	Percentage difference too Handbook Quay Walls	Anchor force for bearing capacity [kN/m]
CUR 166	- 35,00	5961	+ 20%	1088	+ 1%	957
Handbook Quay Walls	- 35,00	4980	-	1082	-	902
EAU 2004 (spring system)	- 35,00	4219	- 18%	739	- 46%	739
EAU 2004 (GGU Retian)	- 35,00	9778	+ 96%	1078	- 1%	1078

Table E-10: Summary of calculation results for sheet pile calculations including 2nd order effect, excluding eccentricity and excluding the forming of a vault

F Influence of safety factors on soil strength parameters

It is clear that the safety factors working on the soil strength parameters have large influence on the maximum bending moment and anchor force. Therefore the cases 1 and 2 will be overviewed for all guidelines under the same conditions, with only different safety factors on actions (soil strength parameters), action effects (internal forces) and for the EAU 2004 and CUR 166 also safety factors on the resistance (passive earth pressure). The **variable actions** working behind the wall are also included, which are transferred to the wall by the effective earth pressure.

F.1 Influence of safety factors for case 1

The characteristic situation of case 1 is adapted to design values with safety factors prescribed by the guidelines.

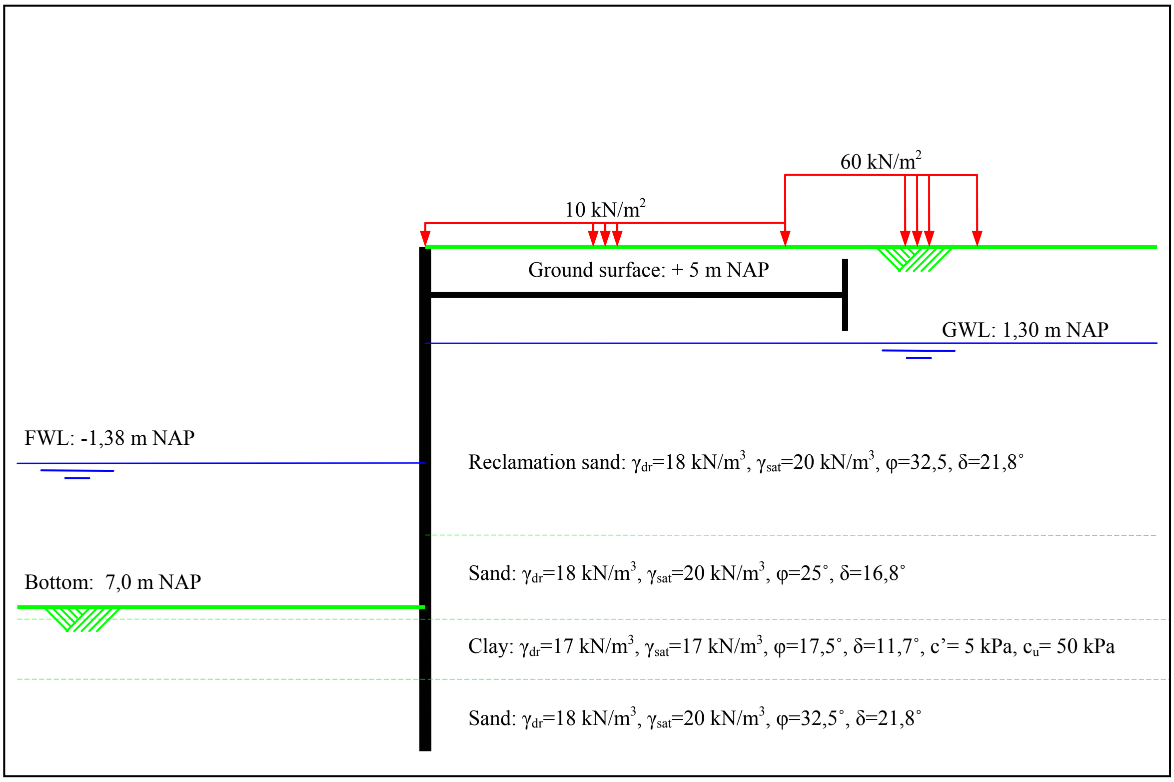


Figure F-1: Characteristic values of CASE 1

F.1.1 Bending moment for case 1

The results for the bending moments are outlined in the graph below (figure F-2). The relative safety in comparison with the characteristic situation is given in the table below (table F-1).

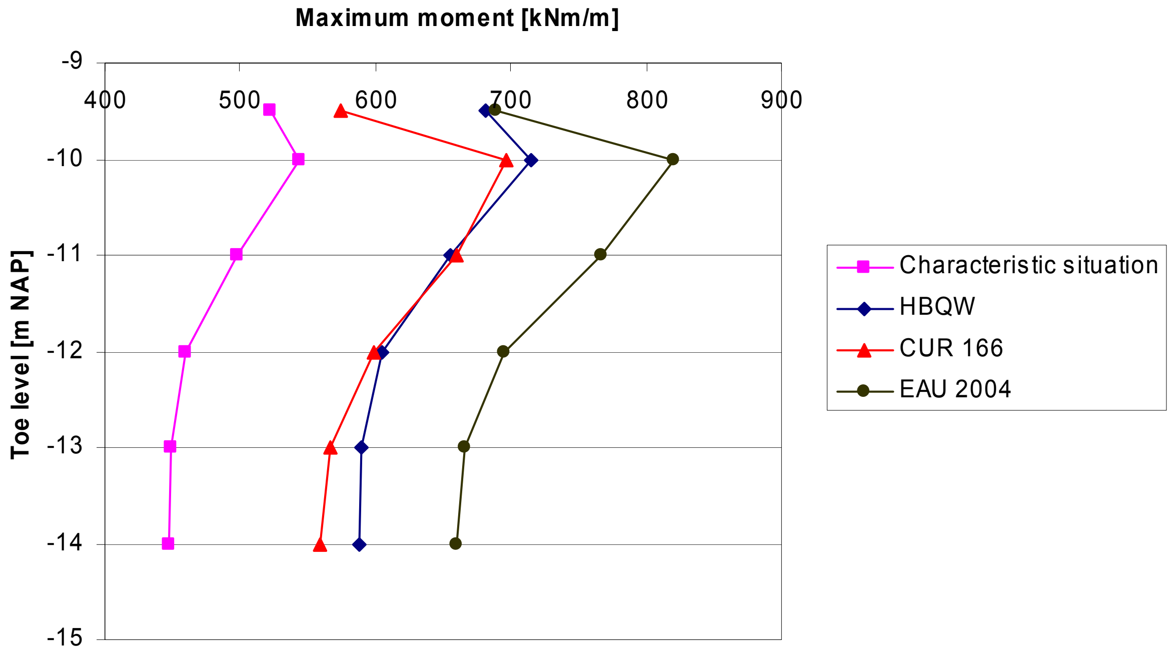


Figure F-2: Maximum bending moments for different guidelines and characteristic situation [harbor bottom -7 m NAP]

Toe level	**-10 m NAP**		**-12 m NAP**		**- 14 m NAP**	
	Moment [kNm/m]	Difference to characteristic situation	Moment [kNm/m]	Difference to characteristic situation	Moment [kNm/m]	Difference to characteristic situation
Characteristic situation	544	1,00	460	1,00	448	1,00
Handbook Quay Walls	714	1,31	605	1,31	588	1,31
CUR 166	696	1,28	599	1,30	559	1,25
EAU 2004	821	1,48	695	1,51	660	1,47

Table F-1: Bending moments and relative safety in comparison with the characteristic situation

F.1.2 Anchor force for case 1

The results for the anchor force are outlined in the graph below (figure F-3). The relative safety in comparison with the characteristic situation is given in the table below (table F-2).

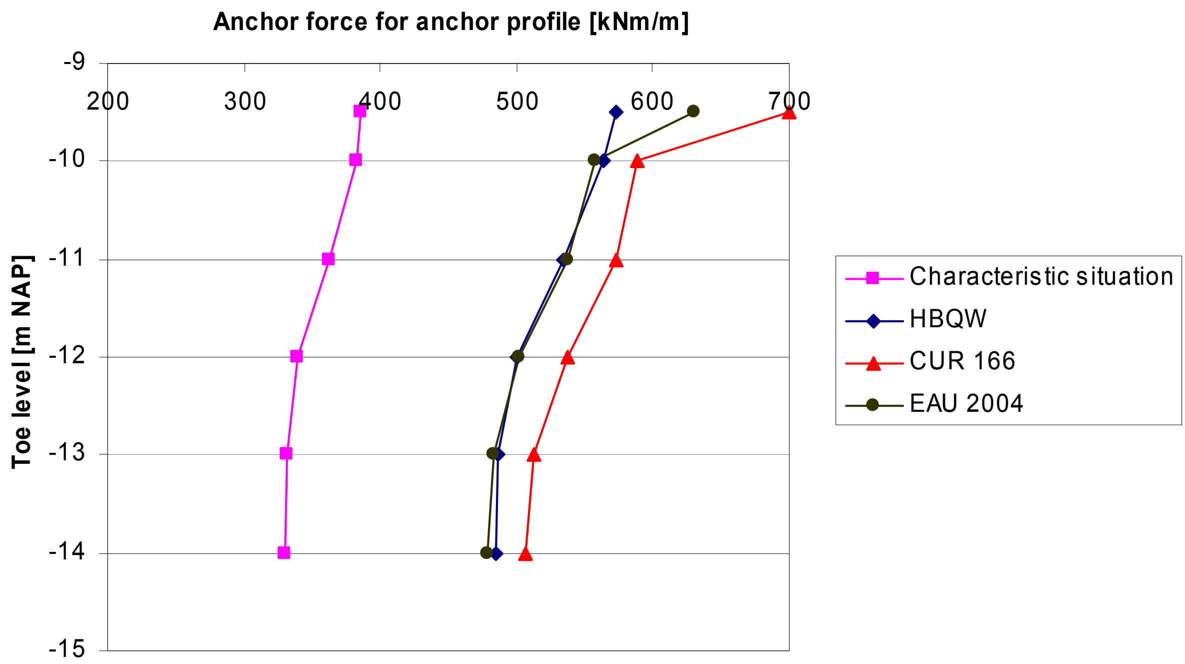


Figure F-3: Anchor forces for different guidelines and characteristic situation [harbor bottom -7 m NAP]

Toe level	**-10 m NAP**		**-12 m NAP**		**- 14 m NAP**	
	Anchor force [kN/m]	Difference to characteristic situation	Anchor force [kN/m]	Difference to characteristic situation	Anchor force [kN/m]	Difference to characteristic situation
Characteristic situation	383	1,00	340	1,00	330	1,00
Handbook Quay Walls	563	1,47	502	1,47	486	1,47
CUR 166	588	1,54	537	1,58	506	1,53
EAU 2004	558	1,46	502	1,47	479	1,45

Table F-2: Anchor forces and relative safety in comparison with the characteristic situation

F.2 Influence of safety factors for case 2

The characteristic situation of case 2 is adapted to design values with safety factors prescribed by the guidelines. The calculations are done without the 2^{nd} order effect and eccentricity. The 2^{nd} order effect will in general result in higher moments (order of magnitude: ± 5%).

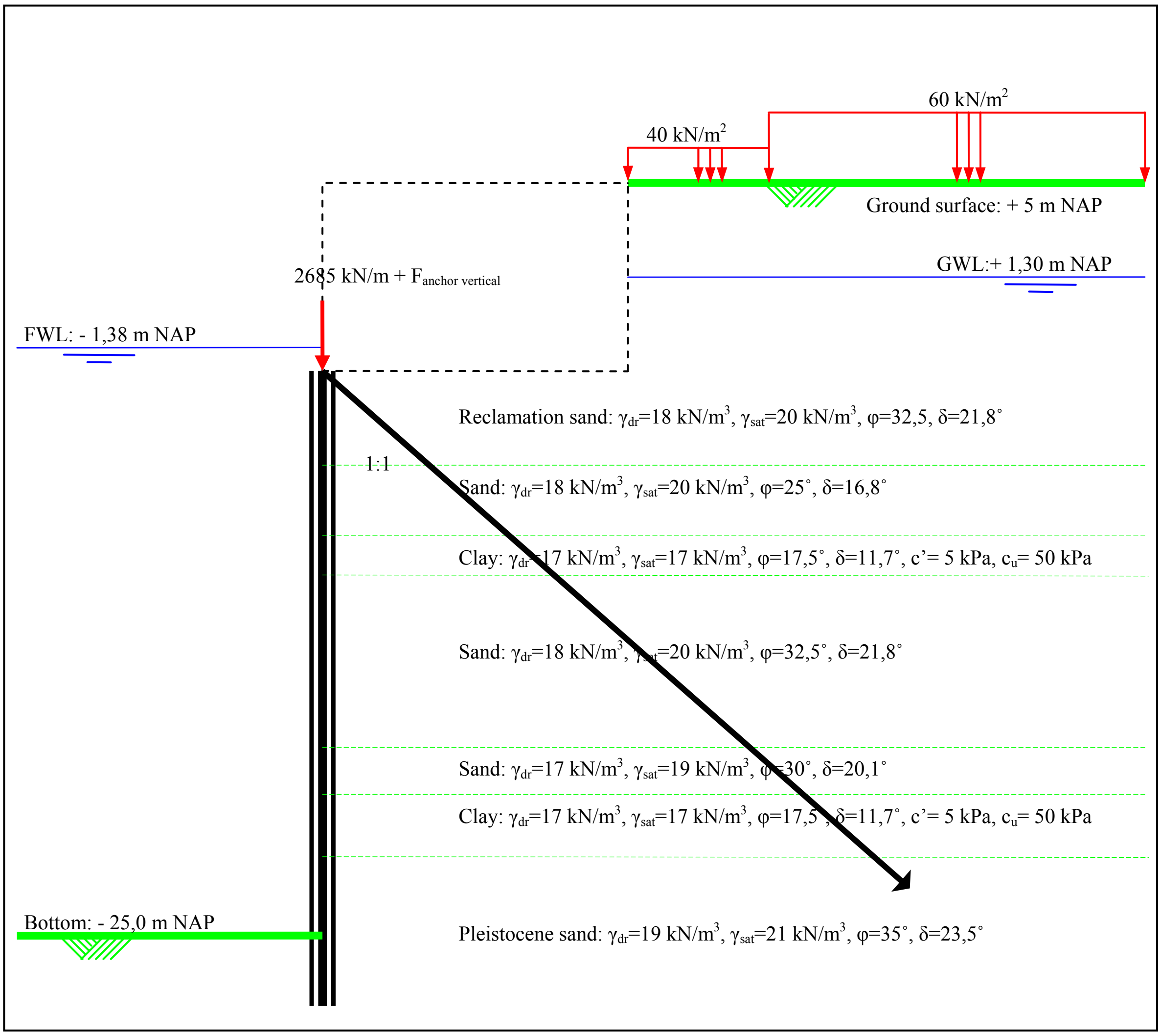


Figure F-4: Characteristic values of CASE 2

F.2.1 Bending moment for case 2

The results for the bending moment are outlined in the graph below (figure F-5). The relative safety in comparison with the characteristic situation is given in the table below (table F-3).

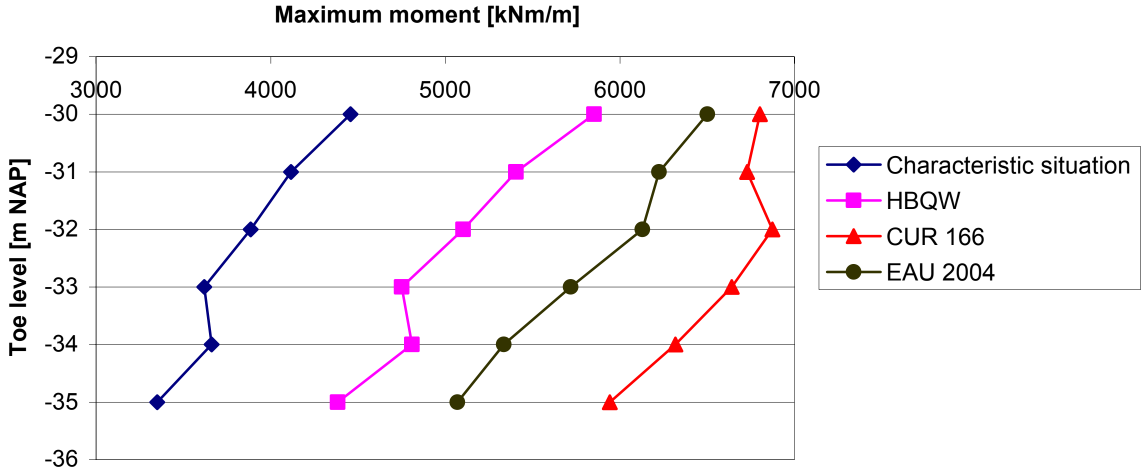


Figure F-5: Maximum bending moments for different guidelines and characteristic situation [harbor bottom -25 m NAP]

Toe level	**-30 m NAP**		**-32 m NAP**		**- 34 m NAP**	
	Moment [kNm/m]	Difference to characteristic situation	Moment [kNm/m]	Difference to characteristic situation	Moment [kNm/m]	Difference to characteristic situation
Characteristic situation	4458	1,00	3886	1,00	3661	1,00
Handbook Quay Walls	5852	1,31	5103	1,31	4809	1,31
CUR 166	6802	1,53	6874	1,77	6319	1,73
EAU 2004	6502	1,46	6129	1,58	5335	1,46

Table F-3: Bending moments and relative safety in comparison with the characteristic situation

F.2.2 Anchor force for case 2

The results for the anchor force are outlined in the graph below (figure F-6). The relative safety in comparison with the characteristic situation is given in the table below (table F-4).

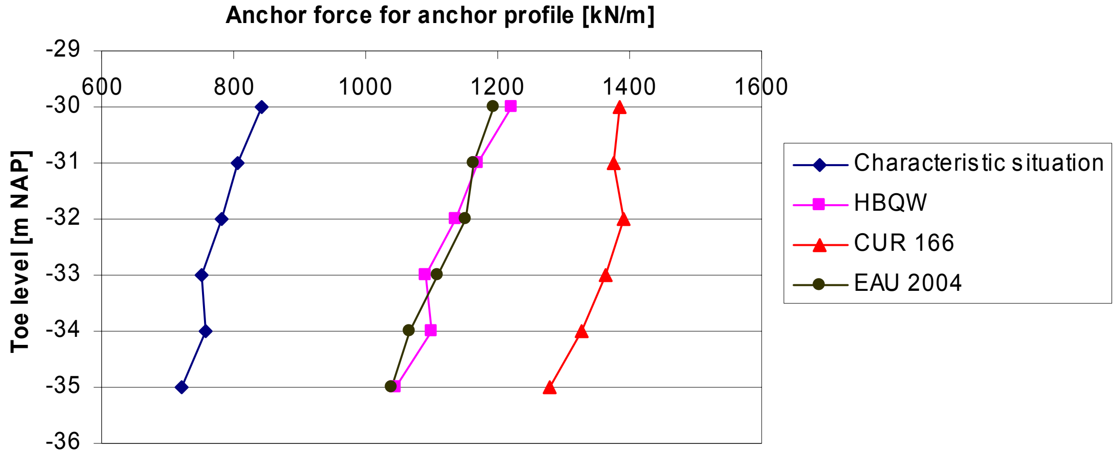


Figure F-6: Anchor forces for different guidelines and characteristic situation [harbor bottom -25 m NAP]

Toe level	**-30 m NAP**		**-32 m NAP**		**- 34 m NAP**	
	Anchor force [kN/m]	Difference to characteristic situation	Anchor force [kN/m]	Difference to characteristic situation	Anchor force [kN/m]	Difference to characteristic situation
Characteristic situation	843	1,00	783	1,00	757	1,00
Handbook Quay Walls	1218	1,44	1136	1,45	1100	1,45
CUR 166	1384	1,64	1392	1,77	1326	1,75
EAU 2004	1193	1,42	1153	1,47	1068	1,41

Table F-4: Anchor forces and relative safety in comparison with the characteristic situation

F.3 Influence of the relieving platform

As is visible in the previous calculations for case 2, the CUR 166 and Handbook Quay Walls method show a lot of difference. With a calculation under the same conditions, but without a relieving platform and the ground surface at the level of the platform height, the effect of the relieving capacity is overviewed. ***This is not a realistic case, but only shows the effect without the relieving platform.***

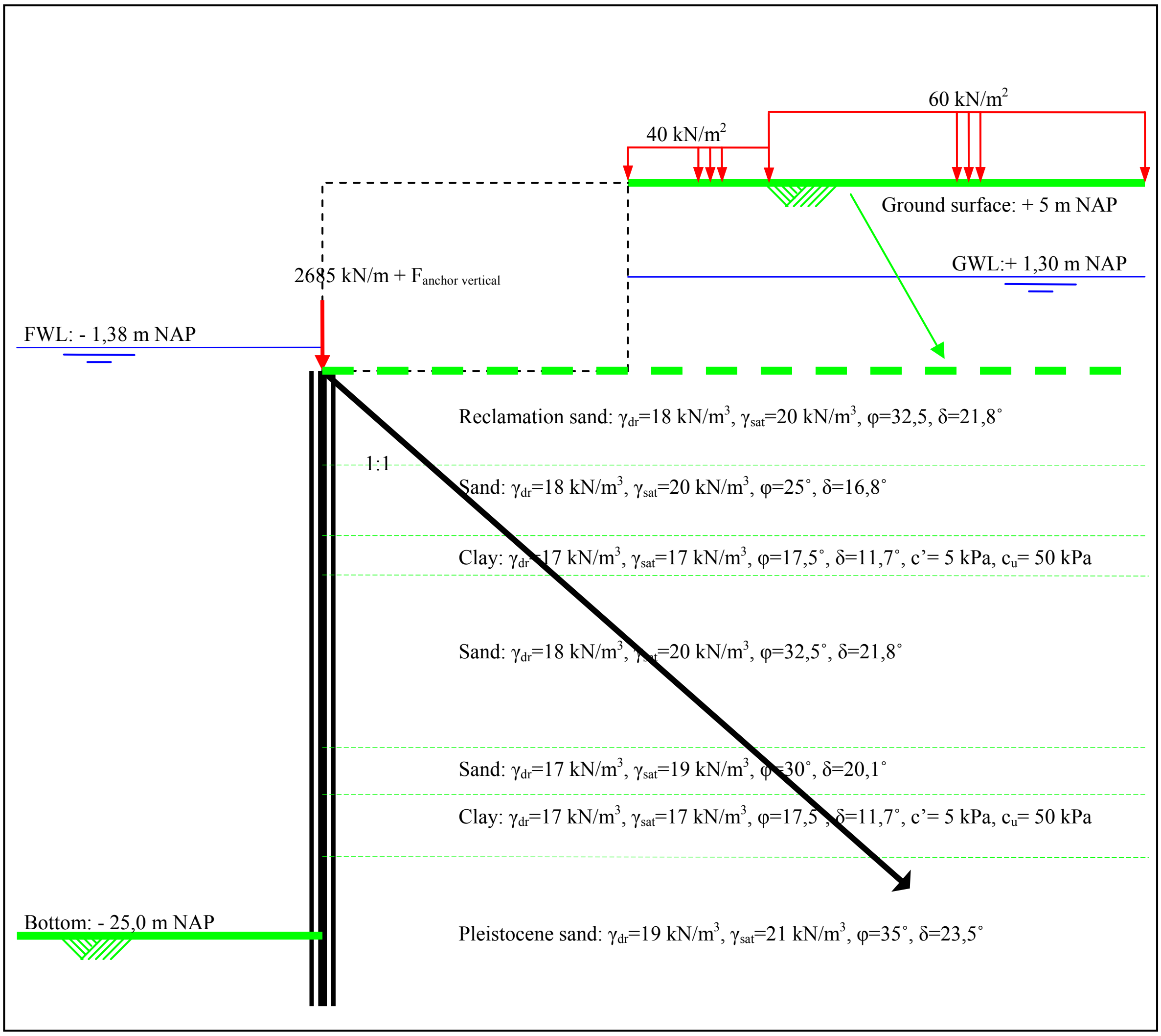


Figure F-7: Characteristic values of CASE 2, the thick dashed line gives the new ("imaginary") ground surface

F.3.1 Bending moment for case 2, without relieving platform

The results for the bending moment are outlined in the graph below (figure F-8). The relative safety in comparison with the characteristic situation is given in the table below (table F-5).

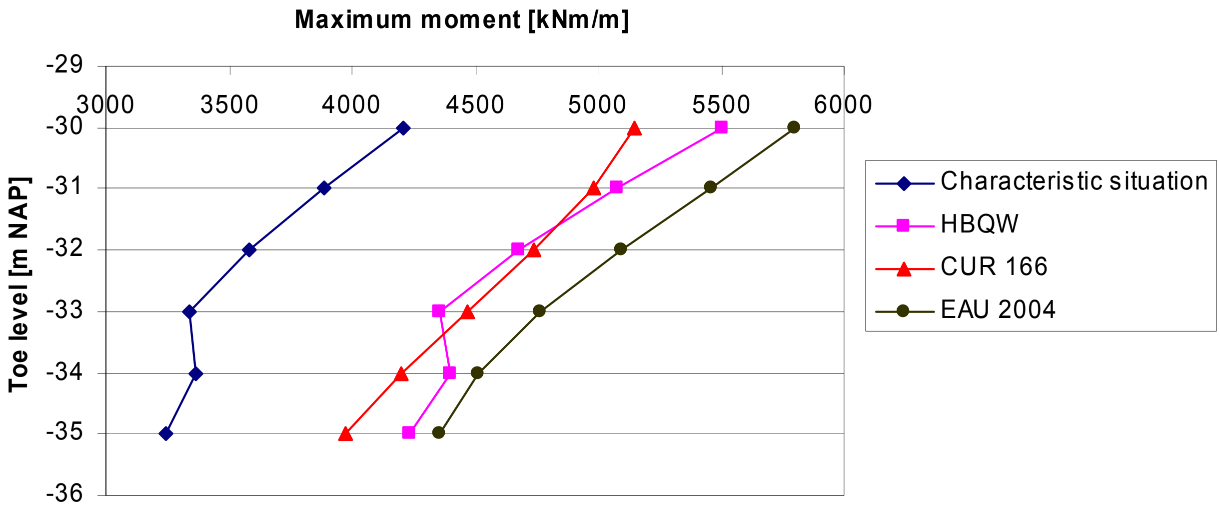


Figure F-8: Maximum bending moments with relative safety to the characteristic situation; WITHOUT relieving platform [harbor bottom -25 m NAP]

Toe level	**-30 m NAP**		**-32 m NAP**		**- 34 m NAP**	
	Moment [kNm/m]	Difference to characteristic situation	Moment [kNm/m]	Difference to characteristic situation	Moment [kNm/m]	Difference to characteristic situation
Characteristic situation	4209	1,00	3581	1,00	3365	1,00
Handbook Quay Walls	5500	1,31	4683	1,31	4399	1,31
CUR 166	5151	1,22	4742	1,32	4198	1,25
EAU 2004	5801	1,38	5099	1,42	4510	1,34

Table F-5: Bending moments and relative safety in comparison with the characteristic situation

F.3.2 Anchor force for case 2, without relieving platform

The results for the anchor force are outlined in the graph below (figure F-9). The relative safety in comparison with the characteristic situation is given in the table below (table F-6).

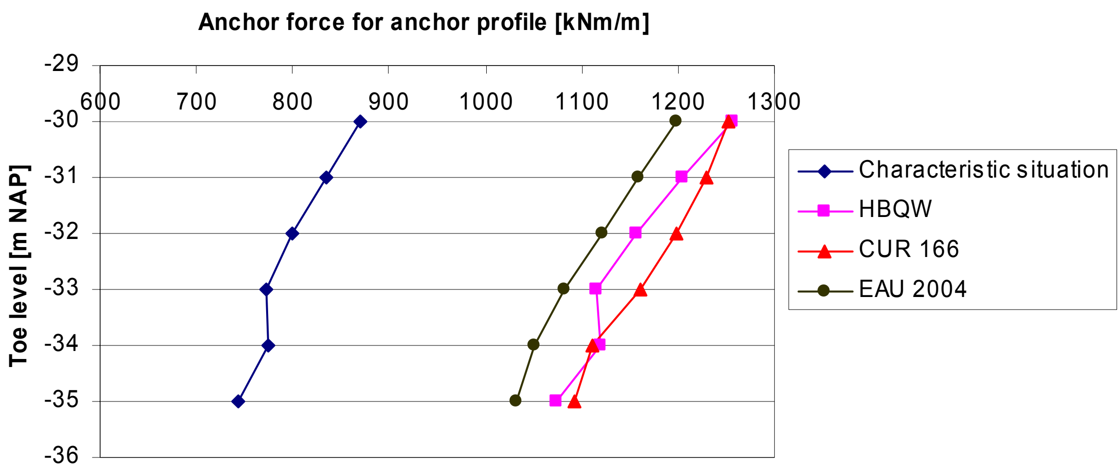


Figure F-9: Anchor forces for anchor profile with relative safety to the characteristic situation; WITHOUT relieving platform [harbor bottom -25 m NAP]

Toe level	-30 m NAP		-32 m NAP		- 34 m NAP	
	Anchor force [kN/m]	Difference to characteristic situation	Anchor force [kN/m]	Difference to characteristic situation	Anchor force [kN/m]	Difference to characteristic situation
Characteristic situation	870	1,00	800	1,00	775	1,00
Handbook Quay Walls	1253	1,44	1156	1,45	1120	1,45
CUR 166	1253	1,44	1199	1,50	1111	1,43
EAU 2004	1197	1,38	1121	1,40	1052	1,35

Table F-6: Anchor force for the calculation of the anchor profile and relative safety in comparison to the characteristic situation