



Reliability of Quay Walls Using Finite Element Analysis

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During the last years the Finite Element Method (FEM) is increasingly applied in the design of quay walls. Especially in case of quay walls with relieving floors and bulk-storage as surcharge load, sub-grade reaction models are limited in their accuracy of modelling the situation. The Finite Element Method often is the only option to more detailed design calculations of quay walls. In the recent years the introduction of Eurocode and the increasing use of Finite Element analysis for design calculations has triggered the update of the CUR Quay walls handbook CUR 211. The latter second edition has recently been published. In advance of this second edition it was decided to look into more detail into the combination of FEM analysis in combination with the Eurocode which lead to the study that is described in this article. In order to infer a more fundamental base for the design method with FEM, two quay walls were examined to check the applicability of the existing FEM design method of the Dutch Handbook Sheet Pile structures (CUR 166) on quay walls with relieving floor. Furthermore, it was checked whether the current partial safety factors needed to be adapted. This research is done by performing probabilistic FEM calculations. The First Order Reliability Method is incorporated in the software Prob2B (Courage & Steenberg, 2007) to perform the calculations. It appeared that using the design method of CUR 166 for quay walls with relieving floor leads to an underestimation of the reliability of the structure. Therefore it is advised to adapt the design method. Furthermore, differences in partial safety factors are proposed to reach the required reliability index.

➤ During the last two years, CUR committee 183 has worked on the upgrade of the Dutch Quay Walls handbook (CUR 211), which was published in November 2013. Two of the main elements that are considered in this new edition are the addition of FEM analysis as a method for design, comparable to the description in the Dutch Handbook Sheet Pile Structures (CUR 166), and the calibration of partial safety factors design with FEM. With respect to the actuality of this update it must be remembered that with the new 2nd Maasvlakte and other changes in the Rotterdam harbour area, several quay walls are under construction, such as illustrated in Fig. 1, which configuration was a reference site for the CUR analyses.

One of the arguments for the further introduction of FEM analysis for quay wall design is that for the larger quay walls, relieve platforms are often used under heavy loaded conditions. In that situation

one must consider that the piles under the relieve platform may bear a part of the horizontal load that is normally taken by the retaining wall. Furthermore, if the load itself is bulk-storage there are limitations to the accuracy of modelling surcharge load with sub-grade reaction models that are normally applied for soil retaining wall design; horizontal components of bulk storage may be difficult to model; see Fig. 2.

The influence of relieve platforms and high surcharge loads on greenfield is one of the reasons to evaluate the design procedures in CUR 166 in the combination with Finite Element analysis, for different failure mechanisms. The method applied in this study is to put Finite Element analysis in a framework of probabilistic analysis. Within this framework, partial safety factors are inferred from the reliability indices and the influence coefficients of the variables of the FEM model, obtained with FORM analysis. This paper presents the method



Fig. 1: A quay wall under construction at Maasvlakte 2



Photo courtesy of CUR 211 (2013) Quay Walls, 2nd edition. Gouda

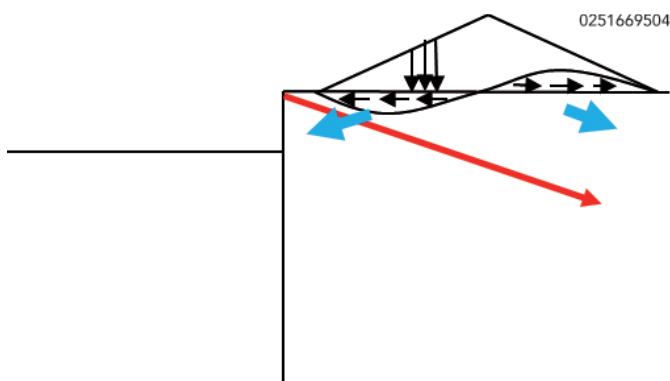


Fig. 2: Horizontal shear forces due to bulk-storage, that will increase the anchor force, are difficult to model with sub-grade reaction model

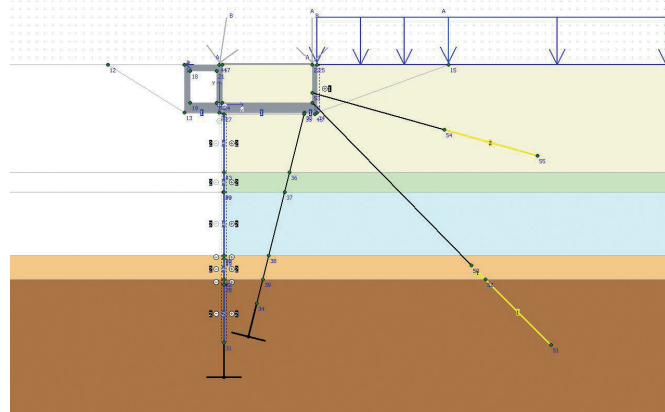


Fig. 3: Quay wall with relieve platform, modelled in PLAXIS

that is used to obtain the reliability indices and partial safety factors and compares the results from these calculations with the 2003 edition of CUR 211.

Quay Walls and Failure Mechanism

The research is done for two different types of quay wall, modelled in Plaxis. To begin with, an anchored sheet-pile with two different sheet-pile lengths (21m and 23m) was analysed. The sheet-pile is anchored 2 m below ground level (ground level is NAP) and has an AZ36-700N profile. The wall is excavated till NAP -12 m and a surcharge load of 30 kN/m² is present. The upper sand layer reaches till NAP -10 m. Below there is a clay layer till NAP -15 m followed by another sand layer. The soil and structural parameters can be found in the report of Wolters (2012, pp. 83-84). This first configuration was used to check the method.

Secondly, a heavier quay wall with relieving floor was modelled; see Fig. 3. In this article

the main focus is on this second analysis. The model is based on the quay wall from Fig. 1. The quay wall is anchored by a double anchor that must guarantee a top displacement of less than 50 mm. The combi-wall consists of tubular piles with 1420 mm diameter and 18 mm wall thickness. In between the piles there are three sheet-piles with profile AU20. The wall is excavated till NAP -19 m which implies 24 m retaining height. A bollard force of 70 kN/m and a surcharge load of 40 kN/m² behind the quay wall are taken into account. The level of the top of the quay wall is NAP +5,0 m. The soil configuration is based on Maasvlakte conditions. The upper sand layer reaches till NAP -8,50 m. Below there is a clay layer till NAP -11,0 m, a sand layer till NAP -19,0 m, another clay layer till NAP -22,0 m and Pleistocene sand. The soil and structural parameters can be found in the report of Wolters (2012, pp. 137-138).

The PLAXIS Hardening Soil model was used to model the soil, because this enables a better

description of the unloading behaviour of the soil behind the wall and gives a description of the soil deformations under the relieving platform. In contrast to the normal design procedure, here for the probabilistic analysis, mean values of the parameters where used. Normally, characteristic values would need to be applied, according to the Eurocode. The difference between mean values and characteristic values was discounted for afterwards when partial safety factors where derived.

As a starting point for the analysis the FEM procedures, as described in CUR 166, were taken. The analysed failure mechanisms of these structures are anchor failure in tension (ULS), wall failure in bending (ULS), soil mechanical failure (ULS) and excessive deformations (SLS). For each failure mechanism a reliability function is defined of the form:
 $Z = \text{Resistance}(R) - \text{Solicitation}(S)$,
 which implies that failure is assumed for $Z < 0$. The

output that is searched for are the reliability index β , linked to the probability of failure ($p_f = \Phi(-\beta)$) and the influence coefficients per variable (α_i). As tool for the probabilistic analysis the PLAXIS kernel was linked in with software (i.e. Prob2B (Courage & Steenberg, 2007)), which controlled the variations that are necessary to derive the influence coefficients α_i .

First Order Reliability Method (FORM) & Prob2B

The used probabilistic method FORM is an iterative level II approach, entailing linearization in the design point to derive influence coefficients. Using this method the uncertainty in the output of the analysis can be objectively weighted with the uncertainty in the input parameters discounting these objectively with the variation coefficient of a parameter that is uncertain. The variation coefficient V_i being defined as:

$$V_i = \frac{\sigma_i}{\mu_i}$$

The approach that is applied in Prob2B can be described in different steps:

- The calculation starts with a 'normal' PLAXIS calculation using mean parameter values. The parameter values are transformed into standard-normal u-space by the transformation:

$$u_i = \frac{x_i - \mu_i}{\sigma_{x_i}}$$

- For each stochastic variable an additional calculation is made by increasing the variable by a fraction of the standard deviation. This variation is made in order to check the variables influence on the Z-value. This influence is expressed in the derivative dZ/dx_i , in which x is a parameter value.
- This derivative is important in the FORM procedure to determine the standard deviation and mean value of Z, which define the reliability index:

$$\beta = \frac{\mu_z}{\sigma_z} = - \frac{\sum u_i * \left(\frac{dZ}{du_i} \right)}{\sqrt{\sum \left(\frac{dZ}{dx_i} \right)^2}}$$

- From the derivatives for each variable an influence coefficient, α_i , can be determined by:

$$\alpha_i = \frac{\frac{dZ}{du_i}}{\sqrt{\sum \left(\frac{dZ}{dx_i} \right)^2}} \quad \text{The sum of } \alpha_i^2 \text{ is equal to 1.}$$

- For each parameter a possible design point value can be determined by:

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

- This procedure is repeated by iterating with the new obtained parameter values instead of the initial mean values. After some iterations, convergence may be found and the design point and corresponding β and α_i can be determined. For the failure mechanisms the Z function should approach zero.

This FORM procedure is incorporated in Prob2B. However, not all stochastic parameters can be included. Variations in surcharge loads, water levels and bottom level (dredging depth) need to be made manually. With these manual variations dZ/dx is calculated and from there the uncertainty in those parameters can be incorporated in the reliability index and influence factors can be derived.

Number of calculations (FORM): 78				
β : 2.646				
Pf: 4.068*10-3				
Parameter (X)	V = σ / μ	Λ	X* (design point)	Unit
Gsilty moderately packed sand	0.3	-0.12	43310	[kPa]
Gpleistocene sand	0.3	0.48	106400	[kPa]
sin(ϕ)silty moderately packed sand	0.18	0.67	0.48	[-]
sin(ϕ)pleistocene sand	0.18	0.46	0.38	[-]
ysat,silty moderately packed sand	0.05	-0.30	20.15	[kN/m ³]
calc.	Z-value			
1	170400			
78	8110			

Table 1: Example of the output of a FORM calculation

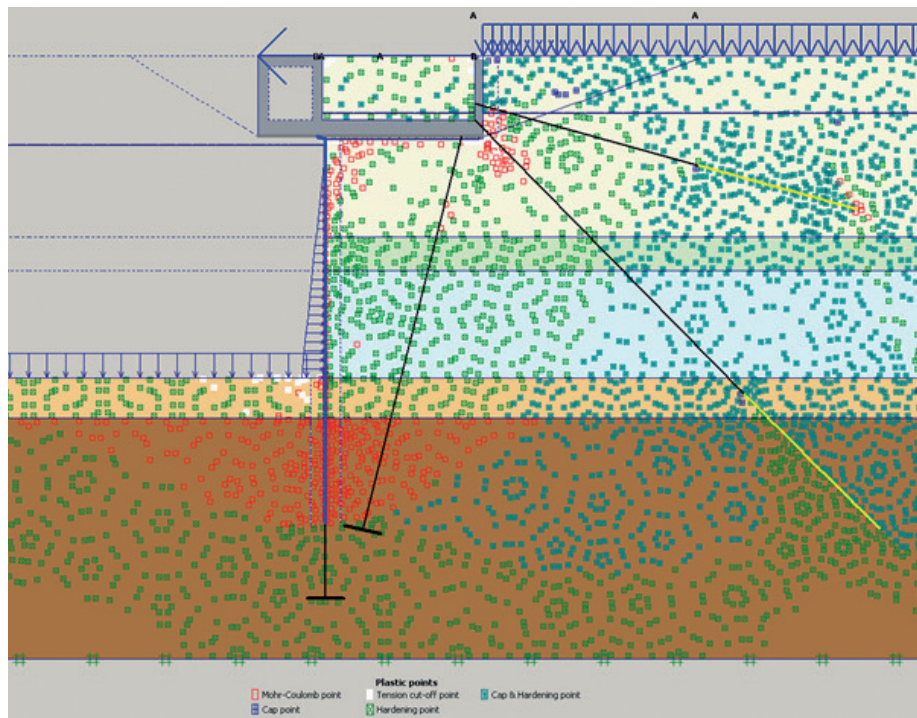


Fig. 4: Example of a plastic point plot in the design point of wall failure in bending (Red: Mohr-Coulomb point, Green: Hardening point, Blue: Cap and Hardening point)

From the output of Prob2B partial safety factors can be derived using the formulas:

$$\gamma_{k,S} = \frac{1 + 1.64V_i}{1 - \alpha_i \beta V_i} \quad (\text{load}) \text{ and}$$

$$\gamma_{k,R} = \frac{1 - 1.64V_i}{1 - \alpha_i \beta V_i} \quad (\text{resistance}),$$

The coefficients of variation for the soil variables and their correlations with other variables are derived from a database of Rotterdam Public Works, containing 3000 tri-axial tests and added by information from the Dutch national annex to NEN-EN 1997 (former NEN 6740). For the structural variables the prescriptions of the Joint Committee on Structural Safety (2002) are followed.

Output

For each FORM calculation output is generated by the software. Table 1 is an example of the output for the limit state function of wall failure in bending. The table includes the variation coefficients (input) the influence factors, design point values and the failure probability (reliability index). When implementing the design values a plastic point plot can visualize the failure mechanism.

An example of such a plot is given in Fig. 4. In this plot it can be seen that wall failure occurs due to failure of the soil elements at the lower part of the piles. This soil failure leads to a reduced fixing moment which causes an increase in the maximum

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

Table 2: Reliability calculated for the simple Quay wall

	β calculated	β CUR 211
Upper Anchor failure	4.40	3.828
Lower Anchor failure	2.99	3.828
Wall failure	2.65	3.872
Soil mechanical failure	2.78	4.396
Deformations	2.44	1.800

Table 3: Reliability calculated for the Quay wall with relieve platform

Parameter		Reliability / Consequence Class
		RC2
Angle of internal friction	γ_{ϕ}'	1,25
Effective cohesion	γ_c'	1,25
Density	γ_g	1,0
Surcharge load	γ_F	1,05
Soil Stiffness	γ_E	1,0

Table 4: Partial safety factors with respect to the characteristic values of parameters

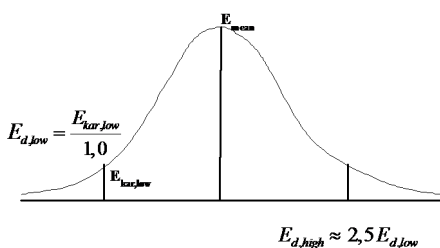


Fig. 5: Distribution of variation in soil stiffness, (CUR 166)

moment in the piles.

Results – reliability indices

The calculated reliability indices for the different failure mechanisms are compared to the prescribed target reliability indices in the first edition of CUR 211 to see whether the FEM design method of CUR 166 leads to sufficient reliable quay walls. Table 2 shows that this target reliability index is reached for the failure modes anchor failure and wall failure. Furthermore the reliability level is by far reached for the deformations requirement (maximum displacement of 50 mm). However, the obtained reliability for soil mechanical failure is too low. This can be explained to some extent by the applied calculation method, but is also due to the

relatively high prescribed target reliability in CUR 211 (Wolters, 2012 pp. 170-174).

For the quay wall with relieving floor the differences are shown in Table 3. The target reliability indices for the lower anchor failure, wall failure and soil mechanical failure are not reached.

Results – Partial Safety Factors

Compared to the method as described in CUR 166 for Sheet Pile walls, for Quay walls with relieving floor the partial safety coefficients need to be adapted. A summary as proposed for CUR 211 for Consequence class 2, i.e. $\beta = 3.8$ is indicated in Table 4.

With respect to the analyses as performed, compared to the results based on point variation values of parameters, it appeared to be necessary to include spatial correlation in soil layers in order to derive realistic values of the partial safety factors.

The main consequences with respect to the partial safety factors are:

- Theoretically the partial safety factors should be subdivided in factors per failure mechanism to be checked. For practical purposes one set of safety factors is proposed for all mechanisms that cover the safety that needs to be realized.
- In deviation from the structural mechanisms for the design of walls and anchors, where a factor of 1.25 for soil strength suffices, for soil mechanical stability a higher partial safety factor on the internal angle of friction is necessary, γ_{ϕ} should be increased for this mechanism up to 1.35.
- For practical purposes, in addition to the structural design, stability needs to be checked for a higher value of MSF (in a $\phi - c$ reduction procedure). All procedures further in agreement with Eurocode being related to characteristic values of the parameters
- With respect to stiffness, for the structural design, characteristic low values of the soil stiffness need to be applied, however for anchor failure in addition to that a check needs to be done with a partial safety coefficient of 0.4.
- The latter means that for anchor design the results of an analysis with a high characteristic value of soil stiffness needs to be checked.
- The analysis indicates that for the geometrical parameters an additional retaining height and water level difference of 15 cm needs to be taken into account for both variables.
- In engineering practice other uncertainties such as dredging depth are more uncertain and it is advised to account for these small differences by including them in the definition of the design shape of the geometry before doing a Finite Element analysis.

Discussion

The influence factors combined with failure plots (e.g. see Fig. 4) explain how a failure mechanism occurs. Anchor failure in tension mostly occurs due to exceedance of the soil shear resistance in the layer where the anchor bonding element is positioned, combined with reduction of the passive shear resistance in the lowest sand layer. Sometimes this passive shear resistance is not fully developed due to the higher soil stiffness, which gives the elastic parameters more influence.

Finally, the uncertainty in anchor parameters can contribute significantly to the anchor failure.

Wall failure is mainly induced due to reduction of the internal angle of friction and therewith the passive shear resistance in the Pleistocene sand layer. The fixing moment is reduced, which causes the bending moment in the wall to increase. The bending moment gets even larger by failure of soil elements in upper layers, which act as an additional load on the wall.

Soil mechanical failure (Fig. 5) is also induced by reduction in passive shear resistance, which implies that this mechanism is correlated with the wall failure mechanism.

Conclusions

For the anchored sheet-pile the target reliability is well approached for the mechanisms wall failure in bending and anchor failure in tension, when designing according to the FEM design prescriptions of CUR 166. The obtained reliability index for soil mechanical failure is too low, but this is mainly due to the used calculation method.

In case of quay walls with relieving floor, the obtained reliability indices are too low. It is therefore necessary to adapt the FEM design procedure for this more complicated type of quay wall and subsequently to modify the partial safety factors, in order to design the structure with an acceptable failure probability.

Note that the results are obtained for structures in the Maasvlakte area of the Port of Rotterdam. Therefore the conclusions are only valid for those specific circumstances. The applied method of analysing the quay walls by probabilistic FEM calculations, however, can be used for quay walls in different situations.

References

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