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Reliability of sheet pile walls and the influence of corrosion – structural reliability analysis with finite elements

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ABSTRACT: The Finite Element Method is well accepted in design practice nowadays. It can be used for modeling complex structures and systems. The constitutive models are improving, which enables us to make more accurate predictions of the real world behavior. On the other hand, especially in the field of geotechnical engineering, the uncertainties in the input variables, namely the soil properties, are relatively high. Therefore, the use of probabilistic techniques is attractive. In this paper we present an approach that combines efficient reliability methods with FEM. A sheet pile wall serves as example. The structural reliability of this structure will be determined accounting for the uncertainties in the soil properties and the groundwater levels as well as the strength reduction of the structural elements due to corrosion. It is demonstrated that reliability techniques can be combined in combination with FEM and that reliability analysis can be carried out with reasonable effort.

1 INTRODUCTION

There is a trend in the development of safety concepts as well as in economical approaches to structural design to imply more probabilistic concepts. Recent developments were the introduction of partial safety concepts such as Load and Resistance Factor Design (semi-probabilistic) or risk based approaches like the Dutch regulations for dike safety. Also Life Cycle Cost Assessment (LCCA) or maintenance strategies are based on structural reliability considerations respectively the development of the structural reliability over time.

Probability and reliability theory form the foundation for these concepts. Furthermore, the determination of the reliability of a structural design respectively a structure is an essential subtask within these ideas. In this paper an attempt is made to contribute to this development by describing how structural reliability analysis can be carried out for sheet pile structures, respectively deep excavations.

The Finite Element Method is used to an increasing amount in design practice. We can use it for modeling complex structures and systems. The constitutive models are improving, which enables us to make more accurate predictions of the real world behavior. On the other hand, especially in the field of geotechnical engineering, the uncertainties in the input variables, namely the soil properties, are relatively high. Therefore, the use of probabilistic techniques is attractive. In this paper we present an approach that combines efficient reliability methods with FEM. It is demonstrated that reliability techniques can be combined in combination with FEM and that reliability analysis can be carried out with reasonable effort.

Current design codes are based on partial safety concepts. The load and material factors are ideally calibrated by means of probabilistic analysis. These factors might be suitable for a wide range of typical applications, but they are certainly not defined for specific, e.g. extreme cases like very deep excavations. Reliability analysis allows us in principle to determine the reliability of any structure directly and the suitability of the prescribed partial safety factors can be assessed. This way the target reliability levels of the design codes can be compared with the reliability obtained by the analyses.

2 RELIABILITY METHODS

TNO Built Environment and Geosciences have developed ProBox, a generic tool for reliability analysis.

The limit states to be analyzed may contain models in form of analytical expressions that can be defined, and alternatively external models, such as FEM codes can be used for the limit state evaluation (coupled analysis). In this case the FEM code Plaxis 8.2 was used for modeling the geotechnical structures respectively the sheet pile structure.

The following reliability techniques were available for the analyses:

- First Order Reliability Method (FORM)
- Second Order Reliability Method (SORM)
- Numerical Integration (NI)
- Directional Sampling (DS)
- Directional Adaptive Response Surface Sampling (DARS)
- Crude Monte Carlo Sampling (MC)
- Increased Variance Sampling

In principle all input and model variables can be assigned statistical distributions to account for the uncertainties. ProBox comprises 14 distribution types and allows the definition of distributions by means of tables. The correlations between the variables can be introduced in form of a (product moment) correlation matrix.

The program is under constant development and has generic couplings for some specific programs like FEM-codes as well as standard programs like Excel, Matlab or Mathcad already established. Also self-defined dll-routines can be used.

In this paper we will primarily make use of FORM (level II) and level III methods like Directional Sampling and DARS. The level III methods are either used to confirm the applicability of FORM, i.e. as validation tool, or for limit states that are non-linear and that include system effects. They have furthermore the advantage that their performance in terms of precision does not depend on subjective choices, like e.g. the choice of a response distribution as necessary for e.g. the Point Estimate Method or Latin Hypercube Sampling.

The use of Crude Monte Carlo Sampling is not always attractive for structural reliability problems. Its calculation effort depends on the probability of failure, which is for this kind of problem ideally in the order of magnitude of $P_f = 10^{-4}$.

The expected number of calculations can be calculated with (see [Waarts 2000]):

$$N > 400\left(\frac{1}{P_f} - 1\right) \tag{1}$$

where P_f is the probability of failure (The expression holds for an accepted error of 10% of P_f.).

This means that even for a 'light' FEM-model that requires e.g. 10 seconds of calculation time, a reliability analysis with Crude Monte Carlo Sampling would require approximately 15 months!

3 COUPLED CALCULATIONS

In the proposed framework the calculations are controlled by the reliability algorithm (see fig 2).

After defining the models, assigning the statistical properties to the input variables and specifying the reliability method, the program determines the input for each model evaluation. The corresponding Plaxis data files are amended accordingly and the calculation is triggered. After each calculation the relevant outcomes are read from the corresponding Plaxis data files and the limit state function is evaluated. This

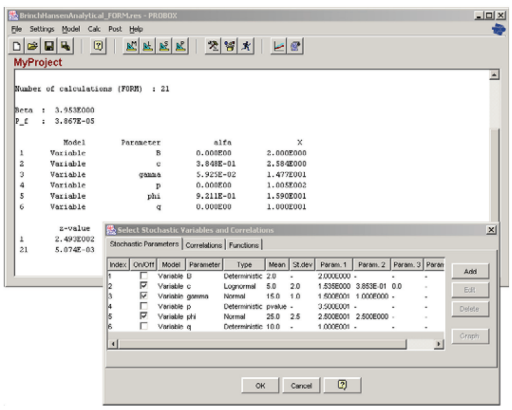

Figure 1. ProBox Screenshot.

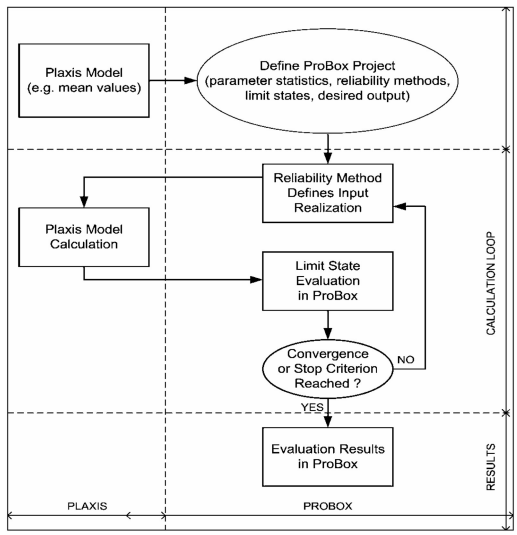

Figure 2. Coupling Scheme ProBox-Plaxis.

procedure is repeated until the pre-defined stop respectively convergence criteria are satisfied and the results are presented. These consist of:

- Reliability index β
- Probability of failure P_f
- Influence coefficients α_i
- Design point values

(For methods other than FORM/SORM approximations for the design point values and the influence factors are applied.)

4 LIMIT STATES

The definition of failure respectively the limit state function is crucial for a reliability analysis. It can have also considerable effects on the efficiency of the applied reliability analysis algorithm. In the following sections the most important failure mechanisms for sheet pile structures and according limit state formulations are discussed.

4.1 *Retaining system*

Fault trees can be used for system reliability analysis and provide an overview over the critical failure mechanisms. In figure 3 such a fault tree is presented for a sheet pile wall. From a design point of view there are Ultimate Limit State (ULS) and Serviceability Limit State (SLS) criteria that have to be fulfilled. We will concentrate on the ULS. There are basically three failure modes, for which the reliability respectively the failure probability is to be determined:

- Failure of the sheet pile (Z_1).
- Failure of the support (Z_2).
- Failure of the soil (Z_3).

Each of these failure modes consist of several failure mechanisms and the whole system can be considered a serial system:

$$P_{f,ULS} = P\{Z_1 < 0 \cup Z_2 < 0 \cup Z_3 < 0\} \qquad (2)$$

In this paper we will focus on the failure modes separately. the green color in figure 3 indicates that a failure mechanisms or mode can be determined directly by means of reliability analysis, the yellow objects can be determined based on these results and white ones either cannot be determined separately or are of minor interest.

For more information on system reliability considerations refer to [Schweckendiek 2006].

4.2 *Sheet pile*

In common design codes the principal load on the sheet piles is considered to be the bending moment due to horizontal loads generated by soil and groundwater.

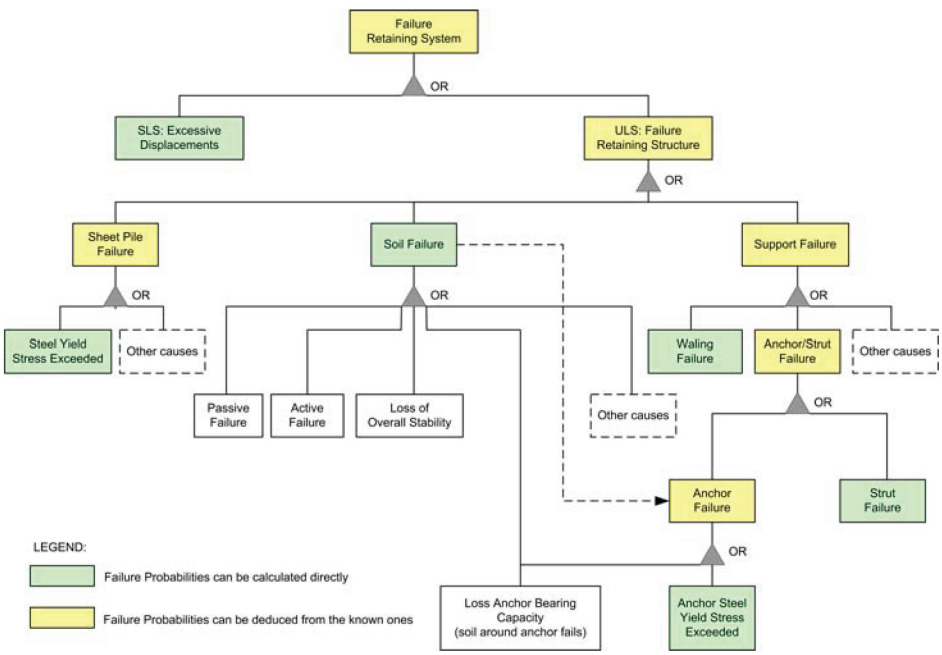

Figure 3. Fault Tree of a Sheet Pile Wall.

The design moment (in [kNm/m]) is usually determined by:

$$M_d = W_{el} \cdot f_y \tag{3}$$

where W_{el} [m^3/m] is the elastic section modulus and f_y [kPa] is the steel yield strength.

In fact, using this expression, we consider the exceedance of the yield stress in the outer fibre as failure. In anchored sheet pile walls there is also an axial force contribution to the stresses in the wall, mainly due to the vertical component of the anchor force. Considering these two components, bending moments and axial forces, the stresses in the outer fibre of the sheet pile are determined by:

$$\sigma = \frac{M}{W_{el}} + \frac{F_N}{A_{SP}} \tag{4}$$

where M [kNm/m] is the bending moment, F_N [kN/m] the axial force and A_{SP} [m^2/m] the cross sectional area of the sheet pile wall.

For the reliability analysis it is convenient to determine the limit state function as:

$$Z_1(z) = f_y - \max[\sigma(z)] = f_y - \max\left[\left(\frac{M(z)}{W_{el}(z)} + \frac{F_N(z)}{A_{SP}(z)}\right)\right] \tag{5}$$

That means that using this limit state function Z_1 we determine the probability that the yield strength is exceeded anywhere in the sheet pile. Note that all the relevant variables are depth (z-direction) dependent. The load variables M and F_N have a distribution over depth and the strength variables, the geometrical properties of the sheet piles W_{el} and A_{SP}, can also be variable over depth. Especially as we will consider corrosion models that determine the thickness loss $\Delta e(z)$*[mm]* as a function of depth.

The section modulus decreases almost linearly with the decreasing thickness of the sheet pile wall and can therefore be approximated by:

$$W_{el} = W_{el,0} - 1.6 \cdot 10^{-4} \cdot \Delta e \tag{6}$$

where $W_{el,0}$ [m^3/m] is the initial section modulus, i.e. before corrosion (see [Schweckendiek 2006]).

The cross sectional area including corrosion can be approximated by:

$$A_{SP} = A_{SP,0} \frac{e - \Delta e}{e} \tag{7}$$

where $A_{SP,}0$ [m^2/m] is the initial cross sectional are and e the thickness of the flanges for sake of simplicity.

Using the above expressions we can determine all the load variables in the limit state function (equation 5) by means of the FEM-analysis and the strength variables can be updated using the corrosion model outcomes $\Delta e(z)$*[mm]*.

Also plastic moments could be used, if one wants to go beyond the limits of elasticity. This is just a matter of the failure respectively the limit state definition.

4.3 *Anchors*

For anchors we can follow a similar approach, using the exceedance of the yield stress as failure criterion. The stresses in an anchor are determined by:

$$\sigma = \frac{F_A}{A_A} \tag{8}$$

where F_A [kN/m] is the anchor force that is assumed to be constant over the free anchor length and A_A [m^2]is the cross sectional area of the anchor.

Similarly to eq. 5 the limit state function can be defined as:

$$Z_2(x,z) = f_y - \max[\sigma(x,z)] = f_y - \max\left[\frac{F_A}{A_A(x,z)}\right] \tag{9}$$

Note that the limit state function is again a spatially variable function due to the fact that the strength reduction by corrosion is not necessarily uniform over the whole anchor.

The determination of the waling failure limit state is trivial in most cases, because the waling design is usually based on the design anchor force, which itself has already a certain exceedance probability. Therefore the waling reliability is usually higher than the anchor reliability.

4.4 *Soil shear failure*

In any retaining structure the soil forms part of the structural system. Figure 4 gives an overview over the most relevant failure mechanisms involving the soil not only as load element, but where also the strength is determined by the soil properties.

The determination of the failure probabilities for these mechanisms is difficult and these probabilities can usually not be determined separately. There are failure mechanisms that are more dominant than others. The consequence is that using FEM we cannot assess the less dominant ones, because the dominant ones will occur first and the calculation does not converge anymore.

What we can do is determine the common failure probability of all these mechanisms, i.e. the probability that any of these mechanisms occurs or its converse, the reliability against *soil failure*.

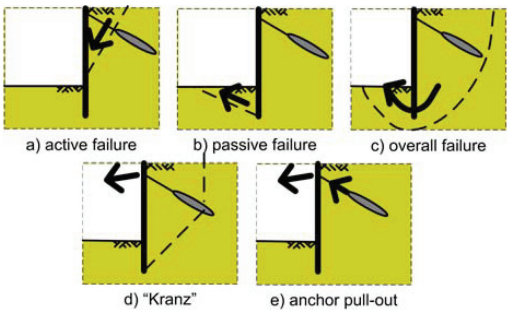

Figure 4. Soil Shear Failure Mechanisms in Retaining Walls.

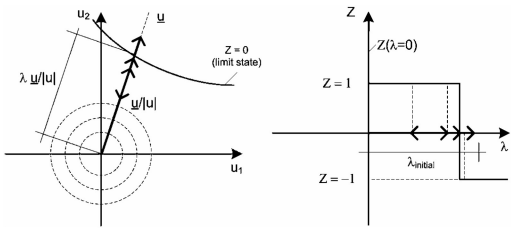

Figure 5. Directional Sampling with Limit Equilibrium.

Some approaches to determine $P_{f,soil}$ are:

- Deformation-based limit state functions.
- Safety-factor-based limit state functions. (e.g. phi-c-reduction, see [Brinkgreve 1991])
- Mobilized shear strength in potential slip planes.
- Limit equilibrium criteria (convergence of the FEM-calculation).

In this paper we will restrict ourselves to the last approach using limit equilibrium criteria. For information on the other possibilities refer to [Schweckendiek 2006].

The basic idea of the approach is that we consider failure to occur, if an FEM-calculation does not reach equilibrium, i.e. does not converge. Consequently all FEM-calculations with parameter combinations respectively realizations that reach equilibrium in all construction stages are considered to be in the safe domain. In fact similar definitions are used for the phi-c-reduction technique and even in the Dutch Technical Recommendation for sheet pile walls CUR166. The problem, as in all reliability analysis problems, is to integrate the probability density either over the safe or over the failure domain. To this end the Directional Sampling (see [Deak 1980]) method is adopted to the needs of this binary criterion (see fig. 5).

For each directional sample the 'distance' λ between the origin of parameter space and the limit state has to be determined. Since the only information we have about the value of the limit state function after an FEM-analysis is, whether it is positive (safe) or negative (failed), we assign the value $Z = 1$ to the evaluations in the safe domain and $Z = -1$ to the failures. A bisection algorithm is used as iterative procedure to determine λ.

In Directional Sampling the failure probability is determined by:

$$P_f = \frac{1}{N}\sum_{j=1}^{N}\left(1-\chi^2\left\{\lambda_j^{\,2},n\right\}\right) \tag{12}$$

where $\chi^2(X,n)$ is the chi-squared distribution with n degrees of freedom and N is the number of (directional) samples.

As convergence criterion for the reliability analysis we can use the variance of the failure probability (estimate):

$$\sigma_{P_f}^{\;2} = \frac{1}{N(N-1)}\sum_{j=1}^{N}\left(P_j - P_f\right)^2$$

$$\text{with } P_j = 1-\chi^2(\lambda_j^{\,2},n) \tag{13}$$

When the variance respectively the variation coefficient of the estimate of the failure probability drops below a pre-defined acceptable value the analysis is stopped.

This criterion does not require any information from the FEM-analysis apart from the fact, whether equilibrium has been reached in all phases or not. This makes it a robust method. For more information on its application refer to [Schweckendiek 2006].

4.5 *Summary*

The limit state definitions in the previous sections are based on quantities that are either input or output variables of an FEM-analysis or they can be determined by analytical or other simple models. That means that we have gathered all the ingredients for a reliability analysis of a sheet pile structure using FEM-analysis.

5 CALCULATION EXAMPLE

The application of the presented methodology is demonstrated by a sheet pile wall in layered soil with one anchor layer. Two variants are discussed. Variant 1 treats only the soil properties, i.e. the load on the structure, as stochastic quantities. The rest of the variables assume either nominal or mean values. In variant 2 a stochastic thickness loss on the sheet pile due to natural corrosion is considered.

The structural dimensions are the result of a structural design calculation. The results of the reliability analysis are compared with the target reliability of the design guideline.

5.1 *Description*

The calculation example is a sheet pile wall in layered soil and one anchor layer. The top layers are soft

(peat and clay) down to a depth of −11.0 m. The base layer is a stiff dense sand layer. Groundwater is present. Figure 6 shows the geometry in the final excavation stage.

The pit will is excavated stepwise and the groundwater level inside the pit is lowered before the final excavation.

5.2 *Parameters*

For the soil parameters typical values for Dutch soil conditions were assumed. Their distributions are summarized in table 1.

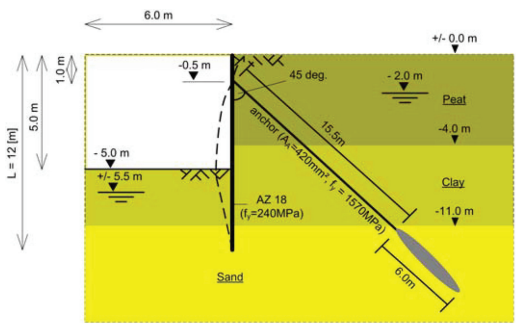

Figure 6. Geometry Calculation Example.

It is not accounted for any correlations between the variables in this example. The structural parameters were determined in a deterministic design based on the CUR166 (see [CUR166]) with safety class two, which is based on a target reliability of β=3.4. The structural dimensions used in the FEM – model are indicated in figure 6. The water levels were set to the design water levels used in the CUR166 design at −5.84 m respectively −1.5 m.

5.3 *FEM-model*

The Finite Element Model was made with the code Plaxis 8.2. The model and the mesh are illustrated in figure 7.

The construction stages are simulated as follows:

- Gravity loading (generation of initial stresses).
- Excavation to −1.0 m.
- Placing of the anchor and prestress with 80 kN/m.
- Groundwater lowering inside the excavation and final excavation to −5.0 m.

The Mohr-Coulomb Model was used for the calculations with the respective soil parameters from table 1. The structural elements were modeled as linear elastic.

Table 1. Soil Parameter Distributions.

Peat, medium		COV	Mean	STD	Lower Bound	Upper Bound	Distribution	Unit
saturated volumetric weight	γ_{sat}	5%	13.1	0.65	–	–	Normal	[kN/m^3]
cohesion	c	20%	7.5	1.5	0.0	–	Lognormal	[kPa]
friction angle	φ,	10%	23.9	2.39	–	–	Normal	[deg]
dilation angle	ψ	–	0.0	–	–	–	deterministic	[deg]
Young's modulus	E	25%	850	212	–	–	Normal	[kPa]
Poisson ratio	ν	10%	0.35	0.035	0.0	0.5	Beta	[−]
interface strength	R_{inter}	20%	0.6	0.12	0.0	1.0	Beta	[−]
Clay, medium		**COV**	**Mean**	**STD**	**Lower Bound**	**Upper Bound**	**Distribution**	**Unit**
saturated volumetric weight	γ_{sat}	5%	18.5	0.93	–	–	Normal	[kN/m^3]
cohesion	c	20%	14.9	2098	0.0	–	Lognormal	[kPa]
friction angle	φ,	10%	20.9	20.09	–	–	Normal	[deg]
dilation angle	ψ	–	0.0	–	–	–	deterministic	[deg]
Young's modulus	E	25%	3400	850	–	–	Normal	[kPa]
Poisson ratio	ν	10%	0.35	0.035	0.0	0.5	Beta	[−]
interface strength	R_{inter}	20%	0.6	0.12	0.0	1.0	Beta	[−]
Sand, dense		**COV**	**Mean**	**STD**	**Lower Bound**	**Upper Bound**	**Distribution**	**Unit**
saturated volumetric weight	γ_{sat}	–	19.0	–	–	–	deterministic	[kN/m^3]
cohesion	c	–	7.5	–	–	–	deterministic	[kPa]
friction angle	φ,	10%	35.0	3.50	–	–	Normal	[deg]
dilation angle	ψ	–	$\varphi' - 5$	–	–	–	deterministic	[deg]
Young's modulus	E	–	125,000	–	–	–	deterministic	[kPa]
Poisson ratio	ν	–	0.3	–	–	–	deterministic	[−]
interface strength	R_{inter}	–	0.7	–	–	–	deterministic	[−]

5.4 *Results – Sheet pile failure*

5.4.1 *Variant 1 – Stochastic soil properties*

In variant 1 we consider only the relevant soil properties as stochastic variables, whilst the geometry, the pore pressures and the properties of the structural members are treated deterministically. The calculations were carried out with FORM.

Based on the results in table 2, the calculations were repeated with a reduced number of random variables. The 5 most influential parameters were kept and the rest of the variables was set to their expectations and treated deterministically. Due to the decrease in input uncertainty, the reliability index increased to $\beta = 4.38$ ($P_f = 5.97$ E-6). This confirms that the impact of the neglected variables on the reliability was small and it shows that previous sensitivity analysis can decrease the calculation effort considerably, in this case from 99 to 37 limit state function evaluations with FORM.

These level II results were also assessed with a DARS-analysis (level III). It gave basically the same result using about 1,500 limit state function evaluations. Hereby the applicability of FORM (linearization of the limit state) could be confirmed and the calculation effort was still acceptable considering that with Crude Monte Carlo several millions of evaluations would have been necessary for the same accuracy.

The influence factors (also called 'importance factors') in table 3 also reveal that for this limit state the clearly dominant parameters are the stiffness parameters of the clay layer in this case. A design point is calculated in which the he sheet pile fails before the strength parameters of the soil become important. The problem remains in the elastic domain for this limit state. That is especially important considering that the current design codes mostly focus on the strength properties of the soil for the calibration of partial safety factors. In fact, for using FEM in design the sets of load and material factors should be calibrated separately.

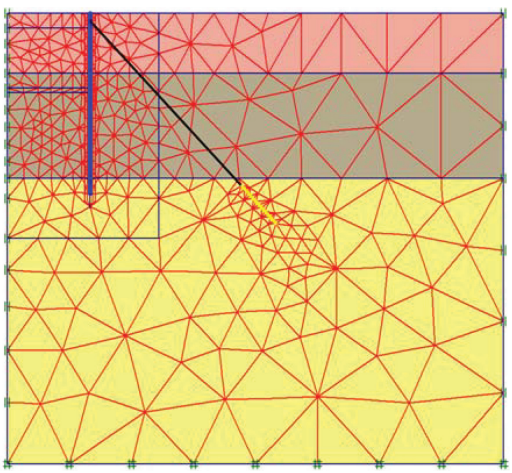

Figure 7. Finite Element Model (Plaxis 8.2).

Table 2. Reliability Results Sheet Pile Failure.

Number of Calculations (FORM): 99		
β : 4.207		
P_f: $1.293*10^{-5}$		
Variable X_i	Infl. factor α_{Xi}	Design Point X_i*
E_clay	+0.878	1,328 [kPa]
E_peat	+0.114	733.2 [kPa]
ν _clay	−0.320	0.396 [−]
ν _peat	+0.107	0.335 [−]
γ _sat_clay	−0.037	18.64 [kN/m^3]
γ _sat_peat	−0.174	13.58 [kN/m^3]
ϕ _clay	+0.037	20.48 [deg]
ϕ _peat	−0.000	23.78 [deg]
ϕ _sand	+0.064	33.90 [deg]
c_clay	+0.079	13.69 [kPa]
c_peat	−0.002	7.36 [kPa]
R_inter_clay	+0.240	0.474 [−]
R_inter_peat	−0.007	0.608 [−]

5.4.2 *Variant 2 – Stochastic corrosion*

Variant 2 is an extension of variant 1, in which we also account for the uncertainty in the strength reduction due to natural corrosion of the sheet pile. To this end the following distributions for the thickness loss in 100 years were assumed, based on 95%-quantiles (characteristic values) from EC3:

Taking this thickness loss into account we obtain the results as presented in table 4, in an similar analysis

Table 3. Thickness Loss (Δe) Distributions per Soil Type.

Soil	distribution	μ	σ	95%-quantile	
peat	lognormal	0.6	0.32	1.2	[mm/100y]
clay	lognormal	2.0	0.67	3.25	[mm/100y]

Table 4. Reliability results sheet pile failure.

Number of Calculations (FORM): 99		
β : 1.981		
P_f: $2.379*10^{-2}$		
Variable X_i	Infl. factor α_{Xi}	Design Point X_i*
E_clay	+0.500	2,584 [kPa]
E_peat	+0.094	787.7 [kPa]
ν _clay	−0.184	0.364 [-]
ν _peat	+0.109	0.344 [-]
γ _sat_peat	−0.085	13.21 [kN/m^3]
Δ e_peat	−0.829	3.24 [mm/100y]
Δ e_clay	−0.012	1.24 [mm/100y]

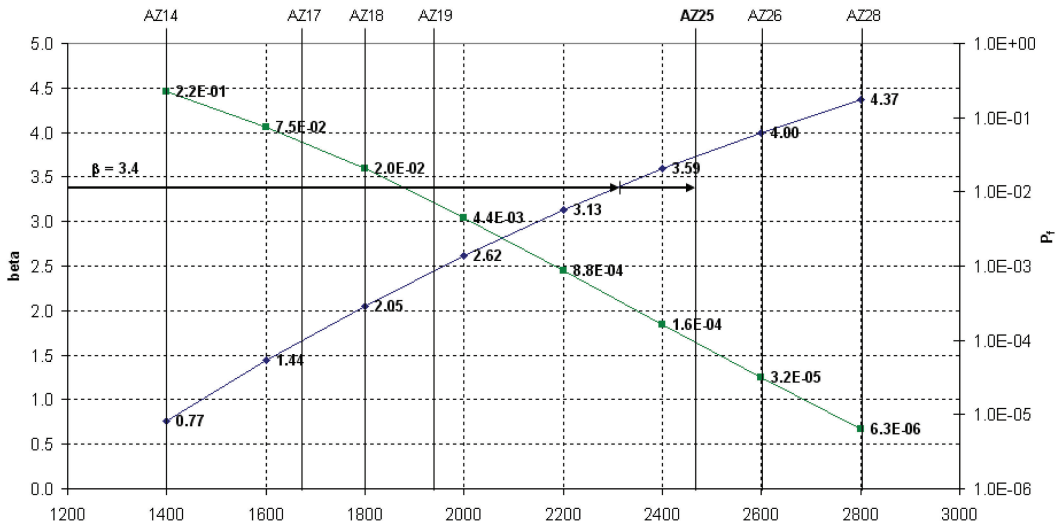

Figure 8. Reliability Index vs. Initial Section Modulus.

as carried out in 5.4.1 with FORM with the reduced number of variables.

The reliability decreases considerably (from $\beta = 4.38$ to $\beta = 1.98$) and the corrosion in the peat layer becomes the most influential parameter. In fact, the maximum stresses were calculated at the lower boundary of the peat layer, where relatively high bending moments are combined with the high thickness loss in the aggressive peat environment.

Figure 8 shows the results of a parametric study on the sheet pile type. According to this study an AZ25 is required instead of the AZ18 without corrosion allowance.

5.5 *Results – Anchor failure*

The anchor reliability was determined by FORM and checked with DARS, similarly to the procedure described in 5.4.1. The results are presented in table 5.

The anchors were over-designed with respect to the target reliability of $\beta = 3.4$, which is basically a result of the design process and the limited choices of anchor sizes. The influence factors show a similar picture as for the sheet pile.

5.6 *Results – Soil shear failure*

The probability of soil shear failure was determined with the limit equilibrium approach that was described in 4.4. The Directional Sampling analysis required 1,653 model evaluations. The results are presented in table 6.

For this limit state there is clearly a shift in the influence coefficients from the stiffness parameters that dominate the horizontal load on the wall to a combination of this horizontal load and the shear strength of the soil. It should be noted that the shear strength not solely depends on the strength parameters but also on the effective stress field, and therefore implicitly also on the weight of the soft top layers.

Table 5. Reliability results anchor failure.

Number of Calculations (FORM): 99		
β: 5.645		
P_f: $8.273*10^{-9}$		
Variable X_i	Infl. factor α_{Xi}	Design Point X_i *
E_clay	+0.828	1,043 [kPa]
E_peat	+0.135	683.7 [kPa]
ν _clay	−0.288	0.404 [−]
ν _peat	−0.247	0.398 [−]
γ _sat_clay	−0.349	18.68 [kN/m^3]
γ _sat_peat	−0.310	14.24 [kN/m^3]
ϕ _clay	+0.019	20.58 [deg]
ϕ _peat	−0.000	23.78 [deg]
ϕ _sand	+0.042	34.01 [deg]
c_clay	+0.056	13.73 [kPa]
c_peat	−0.002	7.37 [kPa]
R_inter_clay	+0.220	0.445 [−]
R_inter_peat	−0.002	0.603 [−]

6 CONCLUSIONS

We can draw the following conclusions:

– A fully probabilistic reliability analysis of the relevant limit states of a sheet pile structure has been

Table 6. Reliability results soil shear failure.

Number of Calculations (DS): 1,653		
β : 3.360		
P_f: $3.900*10^{-4}$		
Variable X_i	Infl. factor α_{Xi}	Design Point X_i*
E_clay	+0.334	2,501 [kPa]
E_peat	+0.372	606.5 [kPa]
ν _clay	−0.013	0.353 [−]
ν _peat	+0.069	0.343 [−]
γ _sat_clay	+0.628	16.54 [kN/m^3]
γ _sat_peat	+0.114	12.85 [kN/m^3]
ϕ _clay	+0.230	19.25 [deg]
ϕ _peat	+0.429	23.44 [deg]
ϕ _sand	+0.297	31.53 [deg]
c_clay	+0.095	13.72 [kPa]
c_peat	−0.289	8.91 [kPa]
R_inter_clay	−0.109	0.650 [−]
R_inter_peat	−0.183	0.679 [−]

carried out, taking uncertainties in the soil properties and the strength reduction by corrosion into account.

- The reliability analysis provides valuable information in form of influence coefficients, which can be used in optimization and to better understand the physical problem itself. E.g. the sheet pile failure was clearly dominated by the stiffness parameters in this example.
- The presented methodology proved to work well for limit states where the soil represents the load on the structure, whilst for soil failure further research is necessary.
- The presented approach can be used in probabilistic and risk-based design concepts. Furthermore, it allows us to compare the target reliability of design codes with the 'actual' (calculated) reliability. Therefore it can be used for calibration of load and resistance factors, when FEM is used for design.
- This probabilistic approach is not restricted to structural reliability problems. It can be applied to all kinds of problems, where input uncertainty has to be propagated through a model and especially for the computation of small failure probabilities.

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