Kennisplan Natte Kunstwerken 2020

Bestaand object

Probabilistische Tools

Quantifying reliability of sheetpile walls

Liesette la Gasse, Diego Allaix (TNO)

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In het **Kennisprogramma Natte Kunstwerken** (KpNK) werken Deltares, MARIN, Rijkswaterstaat en TNO samen aan de kennisontwikkeling om de vervangings- en renovatieopgave bij natte kunstwerken (stuwen, sluizen, gemalen en stormvloedkeringen) efficiënt en kostenbesparend aan te pakken.

Voor het kennisprogramma wordt er jaarlijks een inhoudelijk **Kennisplan** inclusief bijbehorend financieringsplan opgesteld. Andere partijen (zoals waterschappen en marktpartijen) worden nadrukkelijk uitgenodigd om deel te nemen.

Meer informatie over het Kennisprogramma Natte Kunstwerken vindt op www.nattekunstwerkenvandetoekomst.nl waar ook de onderzoeksresultaten ter beschikking worden gesteld.

De samenwerking binnen het Kennisprogramma Natte Kunstwerken vormt de uitwerking van de onderzoekslijn "Toekomstbestendige Natte Kunstwerken" binnen het **Nationaal Kennisplatform voor Water en Klimaat** (NKWK). Dit kennisplatform brengt Nederlandse overheden, kennisinstellingen en bedrijven bij elkaar om samen te werken aan pilots, actuele vraagstukken en lange termijn-ontwikkelingen op gebied van water- en klimaatvraagstukken.

Meer informatie staat op www.waterenklimaat.nl.

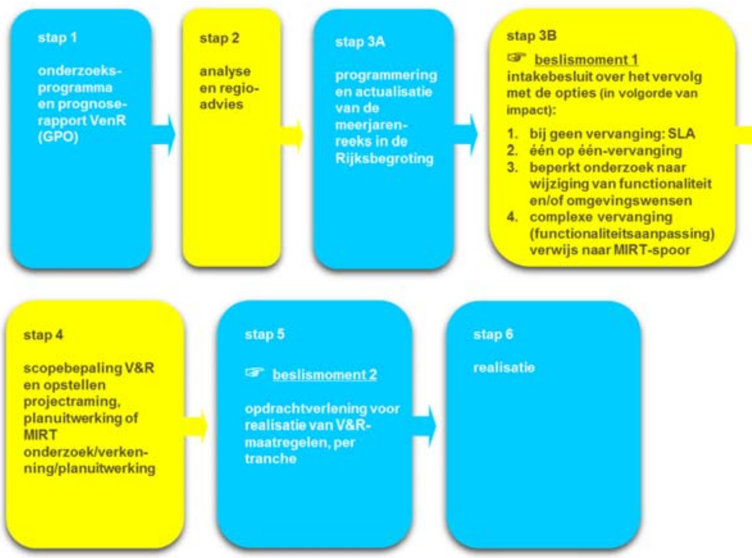
Voor vragen met betrekking tot het rapport kunt u terecht bij:
Diego Allaix - diego.allaix@tno.nl

Voor vragen met betrekking tot het Kennisplan Natte Kunstwerken 2020 kunt u terecht bij:
Daan Dunsbergen - daan.dunsbergen@rws.nl

Voorwoord

Sluizen, stuwen, gemalen en stormvloedkeringen zijn belangrijke assets van beheerders zoals Rijkswaterstaat en de waterschappen. Een groot deel van deze natte kunstwerken bereikt komende decennia het einde van de (technische) levensduur waarvoor het is ontworpen. Er dient zich dan ook een aanzienlijke vervangings- en renovatieopgave van deze kunstwerken aan.

De laatste jaren wordt steeds meer gezocht naar mogelijkheden om levensduur van kunstwerken te verlengen, en om bij einde levensduur (noodzakelijke) ingrepen te koppelen aan gebiedsontwikkelingen en/of functionele-/netwerk ontwikkelingen. RWS heeft daartoe als asset manager een vernieuwde werkwijze voor het Vervanging en Renovatie (VenR) proces opgesteld, welke de basis vormt voor de inrichting van het kennisprogramma Natte Kunstwerken (zie onderstaand figuur).



Figuur 1. Vernieuwde RWS-werkwijze Vervanging en Renovatie.

In het kennisprogramma Natte Kunstwerken wordt kennis ontwikkeld die bijdraagt aan de verschillende stappen binnen deze vernieuwde VenR-werkwijze, met als focuspunten stap 1 prognoserapport en stap 2 regioanalyse en - advies. Het prognoserapport richt zicht op de (einde) technische levensduur, het regio-advies brengt met name in kaart de relatie object-netwerk-gebied.

Het onderzoek in het kennisprogramma vindt plaats langs de onderstaande 3 onderzoekssporen en heeft tot doel om een effectieve en efficiënte aanpak van de vervanging- en renovatieopgave en nieuwbouw van natte kunstwerken mogelijk te maken:

- bestaand object	- inzicht in (einde) technische levensduur - levensduurverlenging
- object-systeem	- inzicht in (einde) functionele levensduur en object-systeemrelaties
- nieuw(e) object/objectonderdelen	- toepassen innovaties - inspelen op toekomstige ontwikkelingen.

Sinds enkele jaren is er het Nationaal Kennisplatform voor Water en Klimaat (NKWK). Hieronder lopen diverse onderzoekslijnen. Eén van de onderzoekslijnen is Toekomstbestendige Natte Kunstwerken. Voor het praktisch laten functioneren van deze onderzoekslijn is er een Samenwerkingsovereenkomst Natte Kunstwerken en een Kennisprogramma Natte Kunstwerken opgesteld:

- Samenwerkingsovereenkomst Natte Kunstwerken. De partijen die momenteel binnen deze overeenkomst samenwerken aan onderwerpen op het gebied van natte kunstwerken (stuwen, sluizen, gemalen en stormvloedkeringen) zijn Deltares, TNO, Marin en RWS.
- In het kader van de bovengenoemde Samenwerkingsovereenkomst Natte Kunstwerken en de 3 onderzoekssporen van het Kennisprogramma Natte Kunstwerken wordt er jaarlijks een inhoudelijk Kennisplan Natte Kunstwerken inclusief bijbehorend financieringsplan opgesteld.

Naast de genoemde partijen zijn en worden andere partijen nadrukkelijk uitgenodigd om deel te nemen aan de Samenwerkingsovereenkomst en/of Kennisplan Natte Kunstwerken. Inzet kan zowel in kind en/of financieel zijn.

Resultaten uit het Kennisplan Natte Kunstwerken worden gedeeld met de gehele sector via onder andere de site www.nattekunstwerkenvandetoekomst.nl.

Het na dit voorwoord beschreven onderzoek en rapportage op het gebied van betrouwnbaarheidsanalyses voor de beoordeling van damwanden is uitgevoerd in het kader van het Kennisplan Natte Kunstwerken 2020.

Summary

Quantifying reliability of sheetpile walls

Aanleiding

In 2019 TNO and Deltares cooperated in the research about the use of Structural Health Monitoring (SHM) for system identification of sheet pile walls. The investigation led, among other results, to (i) the development of a sound and robust procedure for the design of the sensor layout, (ii) a remarkable gain in terms of structural reliability and (iii) a significant reduction of the computational effort by using adaptive surrogate models and advanced sampling methods for Bayesian inference.

Additionally, in 2019 a research within this Knowledge Program a research was conducted where TNO proposed a reliability framework to evaluate the evolution of the annual reliability of existing sheet pile walls through the design lifetime using a simplified model in Blum's method not taking into account the soil-structure interaction.

Onderzoeksvraag en -opzet (WAT)

The aim of the research done in 2020 is the evaluation of the added value of measuring the structural response of sheet pile walls and anchors. The added value is assessed in terms of the probability of failure with respect to the ULS conditions of yielding of the sheet pile wall and tension failure of the anchor. The following research questions were stated at the start of the project:

- given that corrosion affects the safety of sheet pile walls, how does the added value of system identification change during the lifetime?
- given information about the serviceability limit state (SLS) related behaviour of the wall, what could be concluded about the reliability of the wall and the anchors at the ultimate limit state (ULS)?
- given information about the SLS behaviour of the anchor, what could be concluded about the reliability of the wall and the anchors at the ULS?

Onderzoek zelf (aanpak, methode; het HOE)

Within this Knowledge Program TNO and Deltares investigate the effect of corrosion on the annual probability of failure of sheet pile walls by combining numerical models of the soil-structure interaction and reliability methods. Furthermore, reliability methods will be compared aiming at an efficient estimation of the reliability over the lifetime of the structures in addition to methods proposed in 2019. Last, the added value of measuring the structural response of sheet pile walls and anchors should be evaluated.

The foreseen activities are (in order of execution):

- Further development parameter identification tool prob_taralli
- Case definition

- Prior reliability assessment with respect to the ULS of yielding of the sheet pile wall and to the ULS of tension failure of the anchor
- Parameter identification
- Posterior reliability assessment with respect to the ULS of yielding of the sheet pile wall and to the ULS of tension failure of the anchor
- Reporting

Onderzoeksresultaten en synthese

The project plan, in hindsight, seemed to be a bit ambitious. Therefore, not all research questions stated at the start of the project are addressed in this report and should be further considered in future research. The activities described in this report are the case definition and the prior reliability assessment with respect to the ULS of yielding of the sheet pile wall using different implementations of FORM. Also, a plan of activities is proposed for the parameter identification, using a toolbox developed at TNO.

A case study is used that makes use of a nonlinear model also considering soil-structure interaction implemented in DSheetPiling additional to the simplified Blum's method used in 2019. The sheet pile case study is taken from (Post, 2019). Two implementations of FORM were compared for the prior analysis. The FORM analysis using the implementation in UQLab did not converge, while the analysis using the implementation by Prob2B did.

In addition, the effect of measurements of the residual thickness and outcomes of proof load tests on the annual reliability of sheet pile walls with respect to the ultimate limit state of yielding of the steel profile has been investigated. This work is an addition to the reliability framework to evaluate the evolution of the annual reliability of existing sheet pile walls through the design lifetime proposed in the work of 2019.

The work of 2020 therefore consists of two reports:

1. An extension of the report of 2019 adding the effect of measurements and outcomes of proof load tests on the annual reliability of sheet pile walls based on the simplified Blums model
2. The prior annual reliability analysis of sheet pile walls using a nonlinear model including soil-structure interaction and comparison of reliability methods.

This sheet accompanies report 1.

Evaluatie en vooruitblik

The prior reliability analysis for the case study and comparison of different reliability tools turned out to be more challenging and time-consuming than expected. Future research should continue with current analyses extending the comparison with AK-MCS and SDARS. Also the parameter identification and posterior reliability analysis should start once the prior reliability results for all 75 years are available.

The reliability analyses accounting for measurements of the residual thickness and outcomes showed a reliability gain. The outcomes of such analysis could be used for which information and at which point in time should be gathered from the structure for an optimal assessment of the structural reliability.

Stieltjesweg 1
2628 CK Delft
P.O. Box 155
2600 AD Delft
The Netherlands

www.tno.nl

T +31 88 866 20 00
F +31 88 866 06 30

TNO report

Reliability analysis of existing sheet pile walls based on the 1-year reference period

Date	10 November 2020
Author(s)	ir. L.C. la Gasse dr. ir. D.L. Allaix
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Summary

Deterioration caused by corrosion is a concern for asset management of steel sheet pile walls at the end of their design lifetime. Aiming to support decision-making related to lifetime extension, TNO has performed research within the Kennisprogramma Natte Kunstwerken on the reliability assessment of deteriorating sheet pile walls and on the reliability updating based on inspections and load testing.

The structural reliability of existing sheet pile walls has been investigated in terms of the annual probability of failure. The 1-year reference period enables the updating of the structural reliability in a rigorous way by accounting for available structure-specific information and the point in time when the information is gathered.

The annual probability of failure has been estimated by using the First Order System Reliability method and the Equivalent Plane method in combination with a simplified model of the behaviour of the soil-structure system. The focus of the reliability investigation is on the ultimate limit state of yielding of the steel profile.

Measurements of the residual thickness and outcomes of load tests are the information used to update the reliability of the structure. For this purpose, specific performance functions have been formulated.
For both sources of information, it has been shown that there is a reliability gain. However, the conclusion based on the application example cannot be generalized to the whole population of sheet pile walls. It is suggested to perform a sensitivity analysis of the reliability gain with respect to the outcomes of inspections and load tests in the form of what-if-scenario analysis.

Contents

1 Introduction

1.1 Background

The thickness reduction induced by corrosion is the most relevant cause of lack of safety of sheet pile walls after many years of exposure to aggressive environments. In the design of new structures, the effect of corrosion is taken into account by increasing the thickness of the sheet pile wall according to the recommendations of design standards and guidelines. In the assessment of existing sheet pile walls, information related to the actual condition is of utmost importance for decision making regarding replacement and lifetime extension of assets.
The use of structure-specific information for the reliability assessment of existing sheet pile walls is investigated in this research project.

1.2 Objectives

The main goal of the research is to investigate the reliability of sheet pile walls subjected to corrosion taking into account structure-specific information. The sources of information considered in this investigation are the survival of the structure to a number of years, measurements of the residual thickness and the survival to load tests.

1.3 Approach

Aiming at the lifetime extension of existing sheet pile walls from 75 to 90 years, knowledge of the time-dependent variation of the structural reliability (e.g. due to degradation of the resistance) is of outmost importance. Therefore, the reliability assessment is performed considering a reference period of 1 year instead of using the lifetime. This choice allows not only to consider the year-by-year effect of corrosion, but also the effect structure-specific information. For example, the observation that the structure has survived a number of year after construction is particularly useful for retaining structures, because the reliability of those structures is governed by time-invariant parameters (e.g. soil properties and uncertainties of the soil-structure models).

The assessment of the annual probability of failure is performed using a first-order system reliability method and the Equivalent Plane method, which allow to obtain a reasonably approximation of the probability of failure. The ULS condition of yielding of the wall is considered as the only relevant failure mode. The reason of this choice is that corrosion is the main degradation mechanism affecting the reliability of sheet pile walls. The bending moment and the axial force in the wall are obtained by using Blum’s model.

The probabilistic framework for the reliability updating of existing sheet pile walls is applied to a simple case of a sheet pile wall with one order of anchors. This structure has been already investigated in 2015 by TNO and Deltares in the Kennis Programma Natte Kunstwerken.

1.4 Scope and limitations

The use of a 1-year reference period is useful for existing structures, because it allows to estimate in a rigorous way the residual life and to account for information gathered from the structure for the prediction of the structural reliability. In case of retaining structures, the reliability is governed by the uncertainties of the soil parameters and the corrosion-induced loss of thickness, especially in the last years if the design lifetime. Therefore, the outcomes of inspections aiming at measuring the residual thickness or performing load tests and the observation that the structure has survived many years provide information for the reduction of the uncertainties of the governing parameters. The reliability gain provided by this kind of information is in general more relevant for sheet pile walls than for other structures that are more sensitive to time-variant loads.

1.5 Reading guide

Chapter 2 describes the reliability approach for existing sheet pile walls. The choice of the reference period, the approach used for assessing the structural reliability and the reliability updating based on structure-specific information are presented.

The case study is presented in Chapter 3. The structure under investigation, the model used for assessing the structural behaviour, the probabilistic model of the governing parameters are presented.

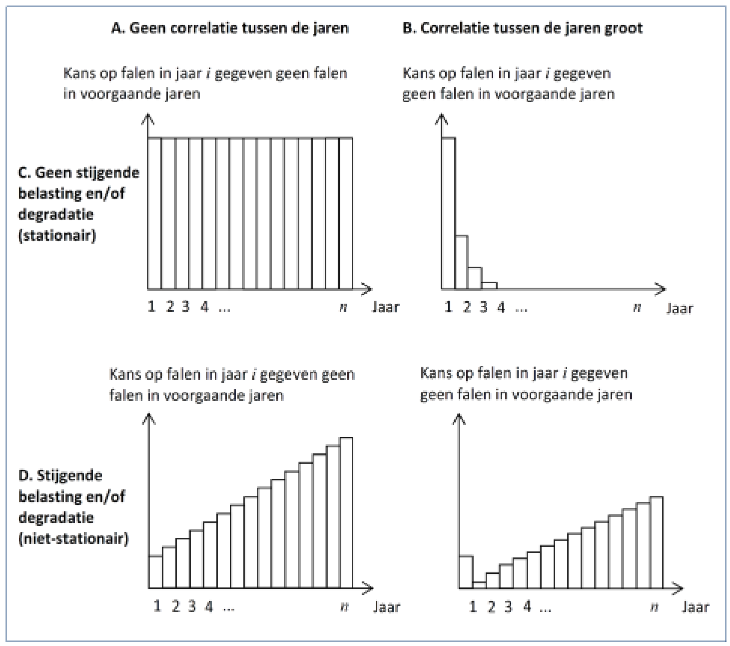
The results of the investigation are outlined in Chapter 5.

The conclusions of the study and the recommendations for future investigations are outlined in Chapter 6.

2 Reliability analysis for existing sheet pile walls

2.1 Introduction

A reliability framework for the assessment of existing sheet pile walls is presented in the following. The framework is based on the 1 year reference period. This choice allows to estimate the effect of past performance, inspections, load tests etc. on the structural reliability for the remaining lifetime. Since the time-development of the reliability is estimated on a yearly basis, the framework enables also the optimization of inspections, maintenance and renovation of sheet pile walls.



2.2 Annual probability of failure

Suppose that a sheet pile walls has been designed for a lifetime of n years. The annual probability of failure in year i can be calculated by:

$$P_{f,i} = P(F_{\boldsymbol{i}} \cap S_{\mathbf{1}} \cap S_2 \cap \dots \cap S_{i-1}) \tag{2.1}$$

where:
- F_i denotes the failure event during year i;
- S_1, S_2, S_{i-1} are the survival events during the first year, the second year and year *i-1*.

The failure probability of Eq.(2.1) is usually called unconditional failure probability or the unconditional failure rate.

Afterwards, the structure has been built and after a number of years (e.g. i years) the estimation of the reliability for the remaining lifetime has to be performed. The task is the estimation of the annual probability of failure for the years i+1, i+2,… until n, knowing that the structure is in service already for i of years.

The probability of failure in year i+1 is defined as the probability that the structure fails during year i and has survived all previous i years. In mathematical terms, the probability of failure in year i is written as follows:

$$P_{f,i+1} = P(F_{\boldsymbol{i+1}} \mid S_{\boldsymbol{1}} \cap S_2 \cap \dots \cap S_i) \quad (2.1)$$

This failure probability is usually called conditional failure probability or the conditional failure rate.
In the simple case where only one failure mode is considered, the annual probability of failure can be expressed as follows:

$$P_{f,i+1} = P[g(\boldsymbol{X}_{i+1}) \leq 0 \mid g(\boldsymbol{X}_1) > 0 \cap g(\boldsymbol{X}_2) > 0 \cap \dots \cap g(\boldsymbol{X}_i) > 0] \quad (2.2)$$

where:
- g is the performance function;
- $\mathbf{X}_i$ is the vector of the uncertain parameters considered in the limit state function.

The vector **X** contains both time-invariant and time-variant elements. Examples of time-variant components are the soil properties, the dead weight, the uncertainties of the structural models. Variable loads on top of the sheet pile wall, the water level, the loss of thickness due to corrosion are examples of time-variant uncertain parameters. Due to the presence of these time-variant parameters in the limit state function, vector **X** is different from year-to-year.
In addition, the failure event F_{i+1} and the survival events S_1, S_2, …, S_i are correlated due to time-variant components of vector **X**. In case of retaining structures the correlation between failure and survival events is generally more significant than for bridges because the reliability of retaining structures is governed by time-independent parameters.

The Z-function of year 2 is correlated with the Z-function of year 1 as it is expected that the failure probability is dominated by the soil parameters. These soil parameters do not change over time. The failure probability in year 2 should therefore account for the fact that there was survival in year 1. In general the failure probability should be updated for every year *i* given that the structure survived the previous years.

2.3 Reliability methods for the assessment of the annual probability of failure

The conditional probability of failure $P_{f,i+1}$ (Eq. (2.2)) can be rewritten as follows, by using the definition of conditional probability:

$$P_{f,i+1} = \frac{P[g(\boldsymbol{X}_{i+1}) \leq 0 \ \cap \ g(\boldsymbol{X}_1) > 0 \cap g(\boldsymbol{X}_2) > 0 \cap \dots \cap g(\boldsymbol{X}_i) > 0]}{P[g(\boldsymbol{X}_1) > 0 \cap g(\boldsymbol{X}_2) > 0 \cap \dots \cap g(\boldsymbol{X}_i) > 0]} \quad (2.3)$$

Both the denominator and the numerator require the solution of one parallel system reliability problem. These problems can be solved by a manifold of reliability methods, e.g. sampling methods or methods based on approximations of the limit state function. Two first-order system reliability methods are used in this investigation because of their computational efficiency. These methods are presented in the following.

2.3.1 First order system reliability method

The first order system reliability method [6] is an extension of FORM method to problems involving multiple performance functions. The numerator of Eq.(2.3) is estimated as:

$$P[g(\boldsymbol{X}_{i+1}) \leq 0 \ \cap \ g(\boldsymbol{X}_1) > 0 \cap g(\boldsymbol{X}_2) > 0 \cap \ldots \cap g(\boldsymbol{X}_i) > 0] = \tag{2.4}$$

$$= P\left[g(\boldsymbol{X}_{i+1}) \leq 0 \cap \bigcap_{j=1}^{i} g(\boldsymbol{X}_j) > 0\right] =$$

$$= P\left[\boldsymbol{\alpha}_{i+1}\boldsymbol{U} + \beta_{i+1} \leq 0 \cap \bigcap_{j=1}^{i} \boldsymbol{\alpha}_j\boldsymbol{U} + \beta_j > 0\right] =$$

$$= P\left[Z_{i+1} \leq -\beta_{i+1} \cap \bigcap_{j=1}^{i} Z_j > \beta_j\right] = \boldsymbol{\Phi}_{i+1}(\boldsymbol{\beta};\boldsymbol{\rho})$$

where:

- **U** is the vector of independent random variables with standard normal distribution;
- α_{i+1} is the vector of the sensitivity factors of the components of vector **X**;
- β_{i+1} is the reliability index estimated using a FORM reliability calculation considering the limit state function $g(\mathbf{X}_{i+1})$;
- Z_{i+1} is a standard normal random variable;
- β is the vector of the reliability indices of the i+2 reliability analyses:

$$\boldsymbol{\beta} = [\beta_1, \beta_2, \ldots, -\beta_{i+1}]^T \tag{2.5}$$

- ρ is the correlation matrix of the limit state functions:

$$\rho_{\boldsymbol{kj}} = \begin{cases} -\boldsymbol{\alpha}_k \cdot \boldsymbol{\alpha}_j & \text{if k} = \text{i} + 1 \\ \boldsymbol{\alpha}_k \cdot \boldsymbol{\alpha}_j & \text{if k} \neq \text{i} + 1 \end{cases} \tag{2.6}$$

- $\Phi_{i+1}(\cdot,\cdot)$ is the CDF function of the multi-variate normal distribution with i+1 components.

The denominator of Eq.(2.3) is estimated using the same approach and it results into the following expression:

$$P[g(\boldsymbol{X}_1) > 0 \cap g(\boldsymbol{X}_2) > 0 \cap \ldots \cap g(\boldsymbol{X}_i) > 0] = \boldsymbol{\Phi}_i(\boldsymbol{\beta};\boldsymbol{\rho}) \tag{2.7}$$

In practice, the first order system reliability method consists of two steps. The first one is the solution of one reliability analysis per year, leading to the reliability index β and the sensitivity factor α. The second step is the evaluation of the probabilities defined in Eq. (2.4) and Eq.(2.7) and the assessment of the annual failure probability by means of Eq. (2.3).

2.3.2 *Equivalent Planes method (Hohenbichler)*

The Equivalent Planes method is a method to compute the failure probability of a system of two correlated elements. The method is based on the work of Hohenbichler and is therefore also called herein the Hohenbichler method. It was developed for a series system and used extensively in Dutch flood defence practise. Although it was developed for a series system it is initially a method for computing the conditional failure probability of two Z-functions, like:

$$P_f = P(Z_2 < 0 | Z_1 < 0) \tag{2.8}$$

where:

- $Z_1 = g(\mathbf{X}_1)$;
- $Z_2 = g(\mathbf{X}_2)$.

A detailed explanation of the method and application can be found in [6] and [7]. A brief description is given in this paragraph. Like described in previous section, in this research we are looking for the conditional failure probability of year i+1 given that the structure survived in the years before (Eq (2.2)). This requires a slight adaptation to the Equivalent Planes method. This will be explained in section 2.3.3.

For the Equivalent Planes method, the individual failure probability of each element of the system and the correlation between these element failures are required. To compute this correlation, both the autocorrelation between the same variables in the individual elements and the influence coefficients of the variables should be known. The method combines the elements sequentially and is limited to linearized forms of the limit state function. The individual failure probabilities are calculated using the FORM method.

The correlation between 2 elements in the system is calculated using both the influence coefficients α and autocorrelation ρ_{ac}:

$$\rho(Z_i, Z_j) = \sum_{k=1}^{n} \alpha_{ik} \cdot \alpha_{jk} \cdot \rho_{ac,ijk} \tag{2.9}$$

where $\rho_{ac,ijk}$ is the correlation coefficient of the k^{th} random variable between the Z-functions Z_1 and Z_2. This correlation coefficient is equal to 0 for random variables that are assumed independent in subsequent years (e.g. maxima of loads) and it is equal to 1 for the others (e.g. soil properties).
The limit state functions Z_i and Z_j can be expressed as follows, where u_i and u_j are standard normally distributed variables:

$$\begin{aligned} Z_i &= \beta_i - u_i \\ Z_j &= \beta_j - u_j \end{aligned} \tag{2.10}$$

Where $u_{(i,j)}$ is actually defined as:

$$u_{(i,j)} = -\left(\alpha_{(i,j)1}u_{(i,j)1} + \ldots + \alpha_{(i,j)n}u_{(i,j)n}\right) \tag{3.6}$$

The correlation between Z_i and Z_j is the same as the correlation between u_i and u_j as the reliability index is constant. Therefore we can write u_2 as a function of u_i and an independent standard normally distributed variable u_j^*. Next the condition $Z_i < 0$ is used as through Eq (2.10) this is equivalent to $u_i > \beta_i$. Therefore the variable u_i can be replaced by a truncated normally distributed u_i' variable capturing only the tail of the distribution of u_i. The expression for Z_j then becomes:

$$\begin{aligned} Z_j &= \beta_j - \rho u_i - \sqrt{1-\rho^2}u_j^* \\ Z_j' &= \beta_j - \rho u_i' - \sqrt{1-\rho^2}u_j^* \end{aligned} \tag{2.11}$$

Now the conditional failure probability of Z_j given $Z_i < 0$ $P(Z_j' < 0)$, or equivalent failure probability $P(Z^e < 0)$, can easily be computed through familiar reliability methods e.g. FORM or Monte Carlo simulations.

The next element can now be considered in a similar way, where in the explanation above Z_1 can be replaced by the previously calculated conditional failure probability and Z_2 is the next element. Like in previous calculation, the influence coefficients for both elements should be known. However the influence coefficients for the individual and original variables in Z_1 which is now the two-element system are yet unknown. Therefore the equivalent influence coefficients should be calculated. An influence coefficient is defined as the partial derivative of the reliability index with respect to a variable. Therefore a numerical procedure is used that estimates $\delta\beta^e / \delta u_k$ through the difference in the computed system reliability index β^e for a small deviation in each individual variable. The equivalent influence coefficients should be normalized so the sum of their squared values equals one.

When both Z-functions are not fully correlated, the variable u is not the same in element i as in element j. They are correlated, so we can write one as the function of the other, with a correlated and an uncorrelated part. The partial derivative with respect to β^e should be derived separately with respect to the correlated and uncorrelated part, after which they can again be combined according to:

$$\alpha_k^e = \sqrt{\left(\frac{\partial\beta^e}{\partial u_{k,corr}}\right)^2 + \left(\frac{\partial\beta^e}{\partial u_{k,uncorr}}\right)^2} \tag{2.12}$$

Probabilistic tools that include the Equivalent Planes method exist in Dutch water defense practice e.g. Matlab toolbox in OpenEarth software of Deltares or in the probabilistics toolbox Hydra-Ring. These toolboxes were used in this research, but had to be adjusted in order to calculate the failure probability conditional on survival in previous years.

2.3.3 Equivalent Planes method for considering survival years

In the application of survival of the sheet pile walls in the years before year *i*, the conditional failure probability is defined by Eq.(2.3). This is similar to Eq.(3.3) which is calculated by the Equivalent plane method, however the failure probability is conditional on the survival in year *i -1* (or $Z_1 > 0$) instead of failure in year *i -1* (or $Z_1 < 0$). Therefore the two limit state functions in the calculation are now:

$$Z_i * = -(\beta_i - u_i) \qquad (2.13)$$
$$Z_j = \beta_j - u_j$$

Using a similar approach as in paragraph 2.3.1, the conditional failure probability of element j can now be evaluated using Eq.(3.7) using the condition $Z_i^* < 0$, or $u_i < \beta_i$. The equivalent influence coefficients can be computed accordingly.

The approach for multiple elements is the same as described before. However, one should be aware that the failure probability of the equivalent plane ($Z_e < 0$) is calculated in the previous step and the survival probability is input for the next step in the calculation. Therefore the equivalent influence factors should be altered accordingly ($\alpha_k^{e*} = - \alpha_k^e$).

The Equivalent Planes method is an approximation method for calculating the system failure probability. Therefore the accuracy of the method was addressed considering a simple system with two random variables per year (resistance R and load S). The results can be found in the Appendix. It was found that the method was accurate enough for cases with similar reliability levels as the sheet pile wall considered in this research.

2.4 Reliability updating

Information gathered from the structure can be used to update the estimated reliability of the structure after year i. Two sources of information are considered in the following: outcomes of inspections in terms of the residual thickness of the sheet pile wall and loads tests. The approaches used for updating the reliability are explained in the following.

2.4.1 Reliability updating with survival years

Using the reliability calculated for the individual years for the reliability over the lifetime of the structure would be an overestimation of the failure probability. The Z-function of year 2 is correlated with the Z-function of year 1 as it is expected that the failure probability is dominated by the soil parameters. These soil parameters do not change over time. The failure probability in year 2 should therefore account for the fact that there was survival in year 1. In general the failure probability should be updated for every year i given that the structure survived the previous years. Therefore the following conditional probability should be computed:

$$P_{f_{yr,i}} = P(Z_i < 0 | Z_{1 \ldots i-1} > 0) \qquad (2.14)$$

2.4.2 Reliability updating based on measurements of the residual thickness

The scatter of the loss of thickness due to corrosion reported in literature is very high, as shown in Figure 2.1, where the black curve is the mean and the grey area represents the scatter of the investigated structures.

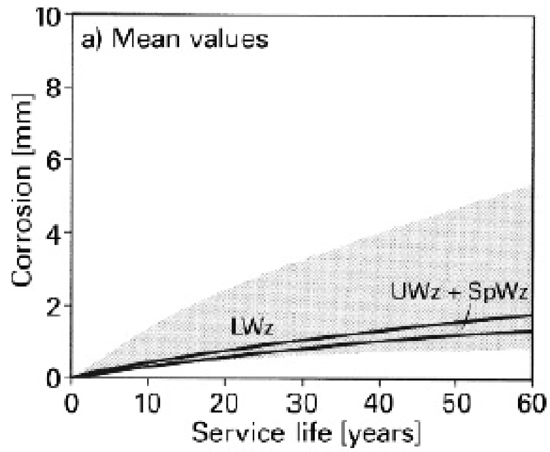


Figure 2.1: Thickness loss induced by corrosion in fresh water [7].

A prior probabilistic model of the thickness loss can be determined either:

- from an extensive database of inspection results, when available;
- by assuming the average loss of thickness (or the corrosion rate) from the design standards and an appropriate coefficient of variation of the distribution (very often the lognormal distribution is suggested in literature)

Due to the large uncertainty of the factors affecting the loss of thickness (environmental conditions, chemical and physical properties of the steel), prior models of the loss of thickness might be very conservative for the specific structure to be assessed. Therefore, inspection outcomes can be used to update the probabilistic distribution of the loss of thickness and the reliability of the structure.

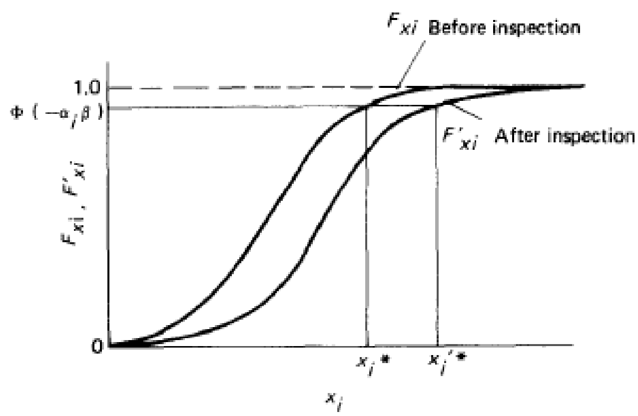


Figure 6 Cumulative distribution functions before and after inspection

Figure 2.2: Prior and updated cumulative distribution functions based on inspection.

Measurements of the residual thickness are considered as equality type of observations and they are formulated by the following limit state function:

$$h_{insp}(\boldsymbol{X}) = t(\boldsymbol{X}) - t_{meas} + \varepsilon_{meas} \tag{2.14}$$

where:
- t(X) is the residual thickness at the time of the inspection;
- t_{meas} is the measured value of the thickness;
- ε_{meas} is the measurement error.

The residual thickness is a function of components of the random vector **X**, e.g. the initial thickness and the loss of thickness at the time of inspection. The cumulative distribution functions of the random variables X that affect the thickness t are defined as conditional distribution functions given the outcome of the inspection:

$$F_{X|insp} = P\big(X \leq x \mid h_{insp}(\boldsymbol{X}) = 0\big) = \frac{P\big[X \leq x \cap h_{insp}(\boldsymbol{X}) = 0\big]}{P\big[h_{insp}(\boldsymbol{X}) = 0\big]} \tag{2.15}$$

However, the denominator of Eq.(2.15) is equal to zero.

Two approach to overcome the problem are considered in the following. The first approach [8] consists of reformulating the equality information into a likelihood function which can be expressed as equivalent inequality information in the space of an altered set of random variables $\boldsymbol{X}_{+}$.
The likelihood function associated with $h_{insp}(\boldsymbol{X}) = 0$ is defined as:

$$L(\boldsymbol{X}) = f_{\varepsilon}[t_{meas} - t(\boldsymbol{X})] \tag{2.16}$$

where $f_{\varepsilon}[\cdot]$ is the probability density function of the measurement error ε_{meas}. This error is assumed to have a normal distribution with zero mean and standard deviation σ_{ε}, as often done in literature.
In case of multiple measurements, the likelihood function can be written as the product of the likelihood functions of the individual measurements:

$$L(\boldsymbol{X}) = \prod_{i=1}^{n} L_i(\boldsymbol{X}) \tag{2.17}$$

under the assumption that the measurement errors of the individual measurements are independent.
In order to perform the updating of Eq.(2.15), the likelihood function of Eq.(2.16) is rewritten as:

$$L(\boldsymbol{X}) = \frac{1}{c} P\{U - \Phi^{-1}[cL(\boldsymbol{X})] \ \leq 0\} \tag{2.18}$$

where:
- c is a positive constant to ensure that $0 \leq cL(\boldsymbol{X}) \leq 1$;
- U is a standard normal random variable;
- $\Phi^{-1}[\cdot]$ is the inverse of the cumulative distribution function of U.

By means of Eq.(2.18), the limit state function $h_{insp,eq}(\boldsymbol{X})$ that is equivalent to $h_{insp}(\boldsymbol{X})$ can be expressed as:

$$h_{insp,eq}(\boldsymbol{X}, U) = U - \Phi^{-1}[cL(\boldsymbol{X})] \tag{2.19}$$

and the updating reliability problem can be reformulated as:

$$F_{X|insp} = P\big(X \le x \mid h_{insp}(\boldsymbol{X}) = 0\big) = \frac{P\big[X \le x \cap h_{insp,eq}(\boldsymbol{X}) \le 0\big]}{P\big[h_{insp,eq}(\boldsymbol{X}) \le 0\big]} \tag{2.20}$$

The updating of the probability of failure in year i+1 is performed as follows:

$$\begin{aligned} P_{f,i+1} &= P\big[F_{i+1} \cap S_1 \cap S_2 \cap \ldots \cap S_i \mid h_{insp}(\boldsymbol{X}) = 0\big] = \\ &= P\big[F_{i+1} \cap S_1 \cap S_2 \cap \ldots \cap S_i \mid h_{insp,eq}(\boldsymbol{X}) \le 0\big] = \\ &= \frac{P\big[F_{i+1} \cap S_1 \cap S_2 \cap \ldots \cap S_i \mid h_{insp,eq}(\boldsymbol{X}) \le 0\big]}{P\big[h_{insp,eq}(\boldsymbol{X}) \le 0\big]} \end{aligned} \tag{2.21}$$

The second approach [9], consist of solving the following reliability problem:

$$\begin{aligned} P_{f,i+1} &= P\big(F_{i+1} \cap S_1 \cap S_2 \cap \ldots \cap S_i \mid h_{insp}(\boldsymbol{X}) = 0\big) = \\ &= \frac{\partial P\big[X \le x \cap h_{insp}(\boldsymbol{X}) - \delta \le 0\big]_{\delta=0}}{\partial P\big[h_{insp}(\boldsymbol{X}) - \delta \le 0\big]_{\delta=0}} \end{aligned} \tag{2.22}$$

where δ is a dummy parameter. This approach requires the partial derivatives of the probabilities with respect to δ at the numerator and denominator of Eq.(2.22). The partial derivatives can be obtained by using the finite difference method. A sensitivity analysis with respect to the parameter δ should always be carried out to verify the robustness of the outcome.

2.4.3 Reliability updating based on load tests

Load tests on structures provide useful information for updating the distribution of the load bearing capacity of the structure. If the structure has survived a load test with load intensity q_{test}, this information leads to the truncation the distribution of the structural resistance R resistance at q_{test}, as shown in Figure 2.3.

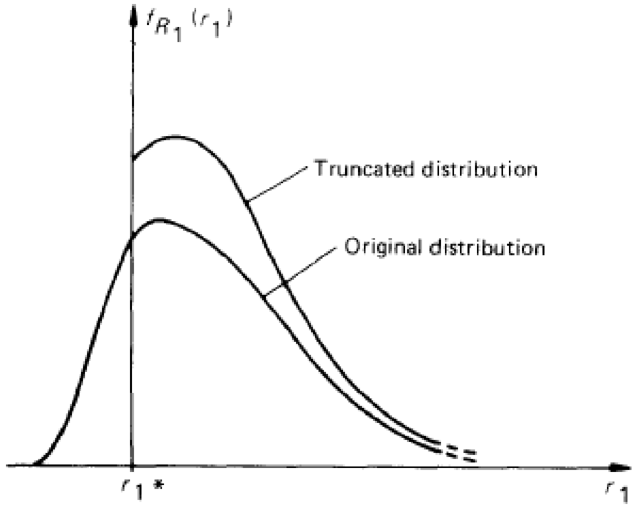


Figure 4 Truncation of distribution function of resistance variable R_1

Figure 2.3: Prior and updated distribution of the resistance.

The left truncation of the distribution excludes the values of R lower than q_{test}, because the structure has survived the load test.
Often the performance function is not written in terms of external loads and structural resistance, like in the case of the ULS condition of yielding of the sheet pile profile. Therefore, the updating of the distributions is performed for the random variables contained in vector **X**:

$$F_{X|LT} = P[X \leq x \mid g_{LT}(\boldsymbol{X}_{LT}) > 0] = \frac{P[X \leq x \cap g_{LT}(\boldsymbol{X}_{LT}) > 0]}{P[g_{LT}(\boldsymbol{X}_{LT}) > 0]} \tag{2.23}$$

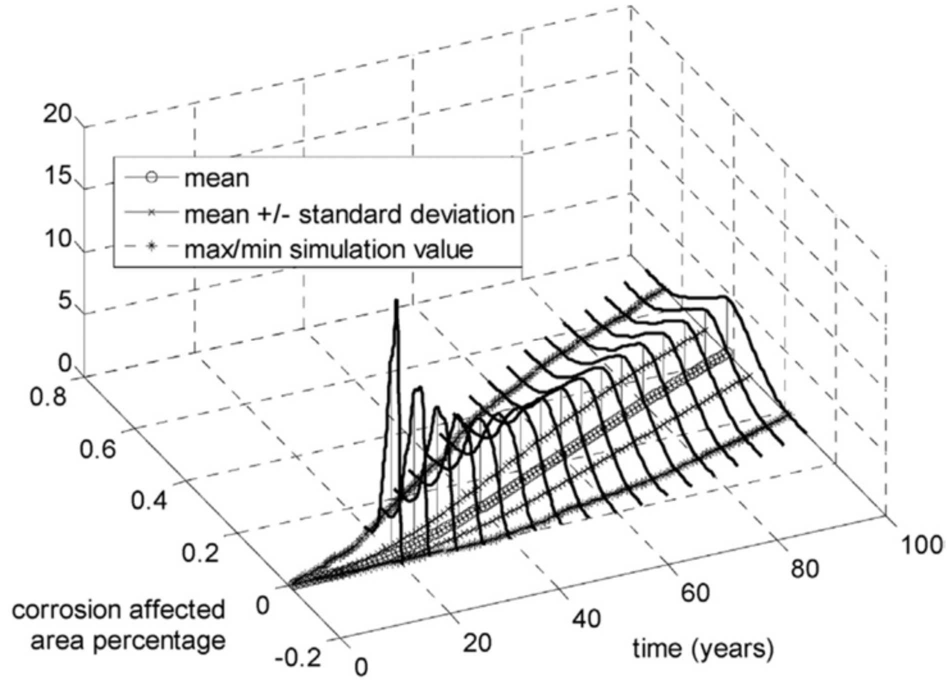
where:
- $g_{LT}(\mathbf{X}_{LT})$ is the performance function of the load test;
- $\mathbf{X}_{LT}$ is a subset of random vector **X** and it contains the random parameter that are not under controlled during the load test.

When the ULS limit states of a component are investigated, the performance function $g_{LT}(\mathbf{X}_{LT})$ corresponds to the performance function of the component.

The updating of the probability of failure in year i+1 is performed as follows:

$$P_{f,i+1} = P[F_{i+1} \cap S_1 \cap S_2 \cap \ldots \cap S_i \mid g_{LT}(\boldsymbol{X}_{LT}) > 0] = \tag{2.24}$$

$$= \frac{P[F_{i+1} \cap S_1 \cap S_2 \cap \ldots \cap S_i \mid g_{LT}(\boldsymbol{X}_{LT}) > 0]}{1 - P[g_{LT}(\boldsymbol{X}_{LT}) \leq 0]}$$


mean
mean +/- standard deviation
max/min simulation value
20
15
10
5
0
0.8
0.6
0.4
0.2
0
-0.2
corrosion affected
area percentage
0
20
40
60
80
100
time (years)

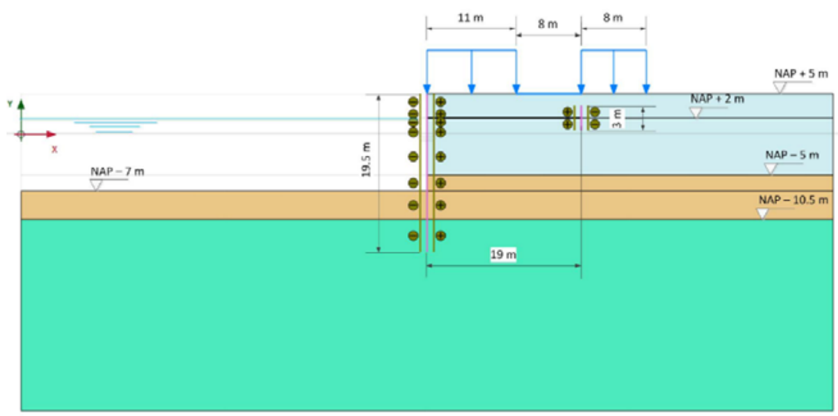
3 Case study: retaining wall

3.1 General

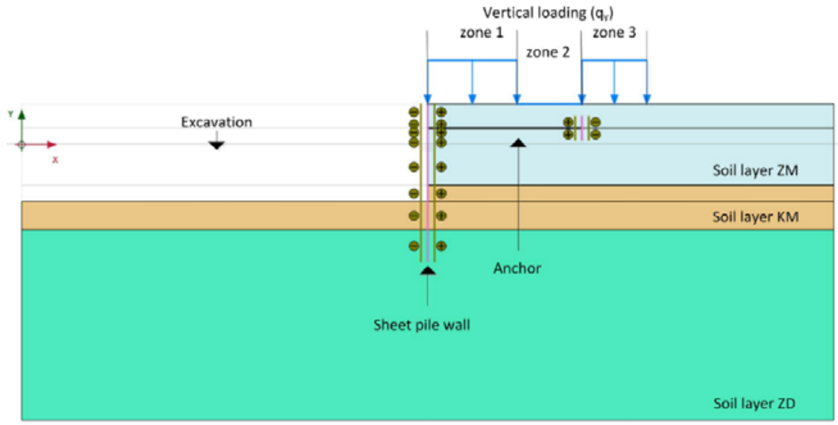
This research considered one case study. This case study is similar as in previous research [1], where a detailed description of the case study was given. In this chapter only a brief overview of the case study and the stochastic parameter for the reliability analysis is given.

The case study is representative for a wall of a lock chamber in fresh water. Specific for this application are the high fluctuations of the water level, causing significant corrosion during the service life. The input parameters for the case study are based on [2] and adjusted for certain aspects. The dimensions and elements of the case study retaining wall are presented in Figure 3.1.

In this research only limit state of buckling of the sheet pile wall is considered, aiming at the evaluation the effect the effect of thickness measurement data and load tests on the reliability of the wall. Failure of the anchor or soil structure are therefore not considered.



(a) Dimensions



(b) Elements

Figure 3.1: Schematisation of the case study retaining wall

3.2 Soil characterization

For the soil characterization the Mohr-Coulomb soil model was considered. The parameters for the three different soil layers are presented in Table 3.1.

Table 3.1: Soil parameters for the Mohr-Coulomb soil model (average values per layer)

Soil layer	Material	γ [kN/m^3]	γ_{sat} [kN/m^3]	c_a' [kN/m^2]	φ_a [°]
1: ZM	Medium dense sand	18.5	20.7	1.0	37.0
2: KM	Firm clay	γ_{sat} - 2.0	17.4	14.8	25.8
3: ZD	Dense sand	γ_{sat} - 2.0	21.8	1.0	39.8

3.3 Sheet pile wall

The sheet pile wall consists of AZ26 profiles, with properties according to Table 3.2. Only elastic behaviour of the wall is considered. Failure of the sheet pile is defined as exceedance of the yield strength.

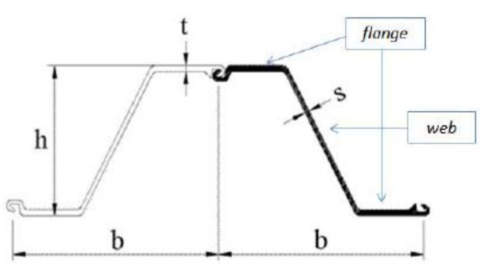


Figure 3.2: Sheet pile AZ26 profile [http://ds.arcelormittal.com]

Table 3.2: Sheet pile properties AZ26 profile

Parameter	Value
Width Z-element *b*	630 mm
Height *h*	427 mm
Thickness flange *t*	13 mm
Elastic section modulus W_{el}	2600 cm^3/m

3.4 Corrosion

A uniform corrosion process is assumed and modelled by applying a reduced thickness to the webs and flanges of the profile. Different thickness reduction values are considered for the different zones over the height and side of the sheet pile wall are considered (contact with soil, water, air or both). The zones are indicated in Figure 3.3. For structures in a lock chamber zone C reaches up to the bottom. Therefore, zone D is for the case study not relevant. The mean yearly corrosion rates were derived from [3] for the water side and from [4] for the sides in contact with only soil. Clean and untouched soil was assumed. The total values are calculated by the sum of the loss of thickness on the water side and on the soil side. In both documents only the total corrosion after 5, 25, 50, 75 and 100 years is given. Therefore, the yearly corrosion rates are calculated by means of linear interpolation between these points. The corrosion rates on the side of the water in a lock chamber are constant in time as these are caused by an eroding environment. However, the corrosion rate at the side of the soil is not constant in time. This leads to the mean corrosion rates in Table 3.3.

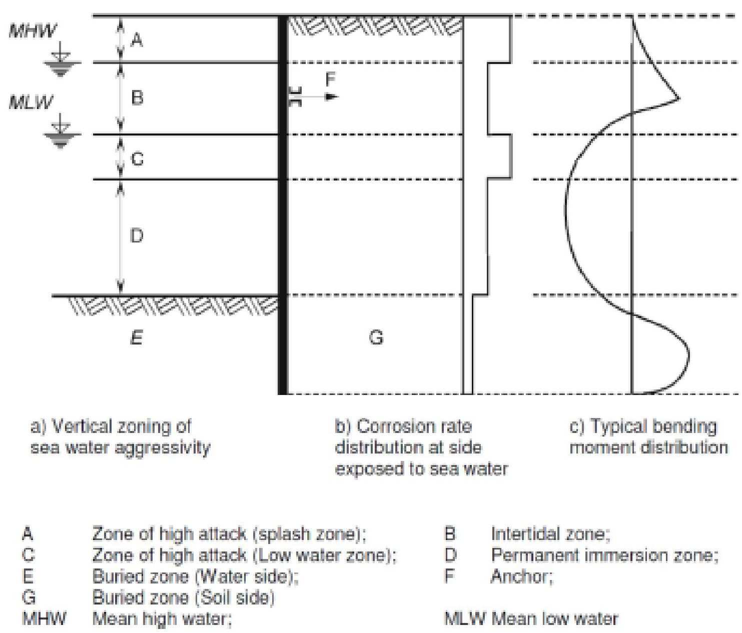


Figure 3.3: Definition of corrosion zones

Zone	Description	Location
A	Above highest lock level	+5.0 to +3.0 m NAP

B	Between highest and lowest lock level	+3.0 to -0.5 m NAP
C	Between lowest lock level and excavation depth	-0.5 to - 7.0 m NAP
E	Below excavation depth	below -7.0 m NAP

Table 3.3: Mean corrosion rate *Δt* [mm/yr] during the lifetime of the sheet pile wall

Zone / time	0 – 5 yrs	5 – 25 yrs	25 – 50 yrs	50 – 75 yrs	75 – 100 yrs
A	0.05	0.065	0.062	0.062	0.062
B	0.02	0.035	0.032	0.032	0.032
C	0.05	0.065	0.062	0.062	0.062
E	0	0.03	0.024	0.024	0.024

To calculate the stresses in the sheet pile wall the section modulus should be updated with the reduced thickness. This is performed using Eq(3.1).

$$W_{corr} = W_0 \frac{(h - t_{corr})t_{corr}}{(h - t_0)t_0} \tag{3.1}$$

where:

- W_0 is the initial section modulus based on the nominal geometrical dimensions of the profile;
- h is the height of the cross section;
- t_0 is the initial, nominal thickness;
- t_{corr} is the loss of thickness induced by corrosion.

3.5 Stochastic properties

The stochastic properties of the parameters considered as random variables are given in Table 3.4. The other parameters relevant for the reliability calculation are considered deterministic.

Table 3.4: Stochastic properties for the reliability calculation

Parameter	Description	Distribution	Parameters
γ_{sat}	Saturated soil weight	Normal	CoV = 0.05
c_a'	Soil parameter	Lognormal	CoV = 0.2
φ_a	Soil parameter	Truncated normal [0,60]	CoV = 0.1
w_l	Water level [+ m NAP]	Gumbel (minima)	Mean = - 0.5685, Std = 0.1664
q_y	Load [kPa]	Gumbel (maxima)	Mean = 19.75, Std = 2.78
z_{exc}	Excavation depth [m]	Normal	μ = 7.0, σ = 0.15
f_y	Yield strength steel [N/mm^2]	Lognormal	μ = 409.1, σ = 28.6
$\Delta t_{(A,B,C,E)}$	Thickness reduction	Normal	CoV = 0.2

The parameters are uncorrelated apart from some of the soil parameters within one layer. The correlation for these parameters is given in the correlation matrix in Table 3.5.

Table 3.5: Correlation matrix of random soil parameters

Material	Material Parameter	Sand (ZM) γ_{sat}	Sand (ZM) φ_a	Clay (KM) c_a'	Clay (KM) φ_a
Sand (ZM)	γ_{sat}	1	0.5	0	
	φ_a	0.5	1		
Clay (KM)	c_a'	0		1	-0.65
	φ_a			-0.65	1

3.6 Limit state condition

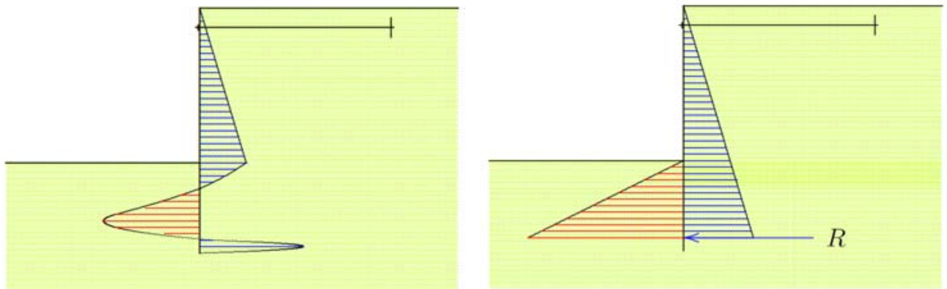
The reliability of the sheet pile wall in the individual years is calculated by using FORM method. The failure mode considered is structural failure of the sheet pile wall which is assumed to fail when the stresses exceed the yield stress of the steel material. The corrosion is different in the four zones of the sheet pile wall. Therefore the highest stresses do not naturally occur at the location of the maximum load effect, but every zone should be checked individually. The limit state function used, is therefore given by:

$$g = min[g_{zone}(A, B, C, E)] \tag{3.1}$$

where the performance function for each zone is defined as:

$$g_{zone} = \sigma_y - \left(\frac{M_{max}}{W_{corr}} + \frac{N_{max}}{A_{corr}}\right) \tag{3.2}$$

To calculate the stresses in the sheet pile wall a numerical method was utilized that is not based on a finite element model. This to reduce the computation time of the calculation. For this purpose the method proposed by H. Blum in the 1950’s to analyse the deformation and bending of a sheet pile wall, was used in this research. This method assumes that the toe of the wall will act as a clamped edge. The idea behind this assumption is that the length of the wall is usually taken somewhat larger than necessary to ensure equilibrium. Therefore at this extra length extra pressures can build up which ensure this clamping behaviour (see Figure 3.4 for the visualisation of this behaviour). The force *R* in the right figure is the resultant force of these extra pressures and will result in a shear force introduced at the toe of the wall. The clamping is supposed to be so strong that the displacement, as well as the rotation, are zero at this end of the wall. Therefore the second derivative is zero, which results in zero bending moment. The method of Blum is an iterative procedure which uses the conditions of equilibrium to determine the length of the wall and the resulting load effects. Inputs for the calculation are the horizontal stresses from the different soil layers and water levels and the anchor depth. For the detailed explanation of the methodology reference is made to [5].



(a) Total pressures against the wall (b) Schematization

Figure 3.4: Blum’s schematization of pressures against a sheet pile wall [5]

However, Blum’s method does not address the normal forces in the sheet pile wall due to friction between the wall and the soil. It was checked if these normal forces were relevant for the calculation through the Plaxis model used in [1]. For the mean values of the parameter values the resultant load effects in the Plaxis model were checked. The full results of this step are presented in Appendix B. It was found that the stresses due to the normal force are negligible with respect to the stresses resulting from the bending moment. Therefore the normal force was not considered in the reliability calculation.

In previous research [REF Diego] it was found that Blum’s method overestimates the bending moments in the sheet pile wall. Therefore a reduction factor could be applied, which was found to be 0.7. The maximum bending moments per zone are therefore multiplied with this reduction value before they are used in the limit state function (Eq (3.2)).

4 Case study results

4.1 Introduction

In this chapter the annual reliability of a sheet pile wall with respect to the limit state of yielding of the steel wall is investigated for a period of 100 years. The objective of this chapter is to present the benefit of using structure-specific information in terms of the probability of failure. Three types of information are considered: survival in previous years, measurements of residual thickness and load tests.

The assessment of the annual probability of failure and the reliability updating are performed using the formulations presented to in Chapter 2. The Equivalent Planes method and the First Order System Reliability method are used to calculate the annual probability of failure of the sheet pile wall.

4.2 Annual probability of failure

As explained in Section 2.3, the Equivalent Planes method and the First Order System Reliability method require that a reliability analysis is performed in advance for each year of the 100 year period considered herein. The reliability index obtained from each individual analysis is plotted in Figure 4.1. The plotted reliability index does not take into account any information about the structure.

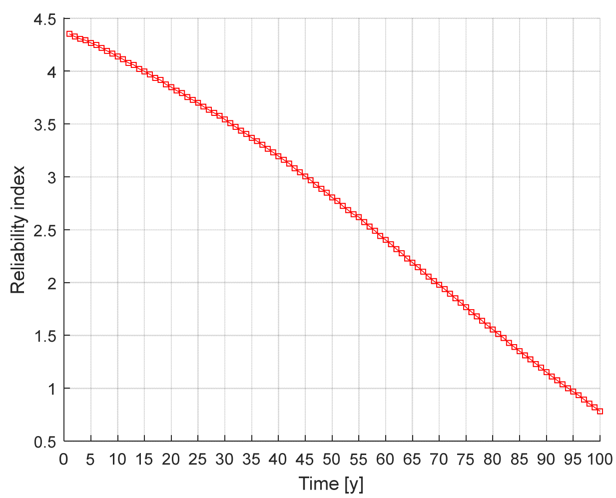


Figure 4.1: Reliability index of the individual years.

The squared sensitivity factors α of the individual random variables obtained from the reliability calculation at year 1, 50 and 100 are plotted in the Figure 4.2. The sensitivity factor of a random variable is a measure of the impact of the uncertainty of the random variable on the reliability index and the sum of the squared sensitivity factors is equal to 1.

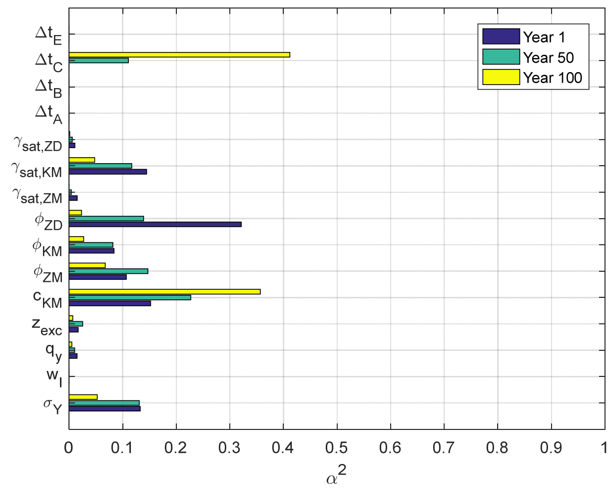


Figure 4.2: Squared sensitivity factors.

The graph above shows that the sensitivity factors change over the 100 year period. After construction, the reliability is mostly influenced by the variability of the soil parameters and marginally by the scatter of the yield stress of steel.
The loss of thickness in the immersion zone Δt_C is the only random variable related to corrosion affecting the reliability, because the maximum stresses in the sheet pile wall are in this zone. As expected, the effect of the uncertainty of the loss of thickness increases during the considered period. The squared sensitivity factor of Δt_C is about 0.1 at year 50 and it is equal to 0.47 at year 100. This results from assuming an increasing mean and a constant coefficient of variation for Δt_C, which reflects the increasing uncertainty of the loss of thickness with time as shown in Figure 2.1.

As explained in Section 2.1, the annual probability of failure for an existing structure can be defined as a conditional probability of failure given that the structure has survived a certain number of years.

In Figure 4.3 the results for the annual failure probability have been presented for 100 year lifetime of the sheet pile wall given that the structure has survived the construction phase. The failure probability is calculated using the Equivalent Plane method. The green curve is the annual probability of failure considering the survivals in previous years (Eq. (2.1)), while the red curve corresponds to neglecting the survivals:

$$P_{f,i} = P(F_i) \tag{4.1}$$

Where the $P(F_i)$ is estimated from a FORM analysis.

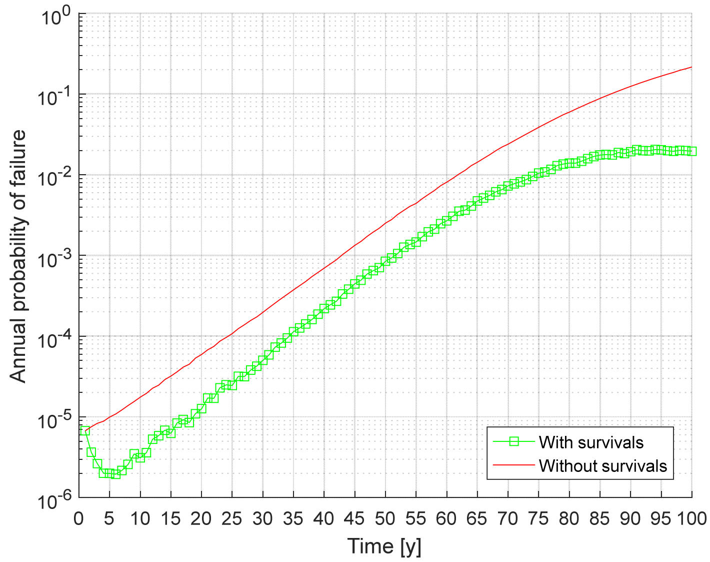


Figure 4.3: Annual failure probability for the lifetime of the sheet pile wall.

The effect of survivals can be observed in particular in the first 5 years of the lifetime, where the correlation between failure and survival events is caused by the soil parameters. After year 5, the probability of failure increases due to the effect of corrosion.

4.3 Reliability updating based on measurements of the residual thickness

The effect of measurements of the residual thickness on the annual failure probability is investigated in the following. It is assumed that the an inspection has been performed at year 75 and the residual thickness has been measured at various depths. The plot of the squared sensitivity factors of the random parameters of Figure 4.2, show that only variation of thickness in the immersion zone (zone C) are relevant for the reliability of the structure. Therefore, only measurements performed in zone C are used for the reliability updating.
The limit state function related to the inspection (Eq. 2.14) is defined as:

$$h_{insp}(\boldsymbol{X}) = t(\boldsymbol{X}) - t_{meas} + \varepsilon_{meas} \tag{4.1}$$

It is assumed that the measured residual thickness t_{meas} is equal to 10.5 mm, Which corresponds to a loss of thickness of 3.5 mm. This value is below the average loss of thickness after 75 years of exposure (see Table 3.3). Therefore, a increase of the reliability is expected.
The measurement technique is affected by a measurement error ε_{meas} modelled by a normal distribution with zero mean and standard deviation equal to 0.5 mm. The accuracy of ultrasound measurement techniques is about ±0.1 mm in ideal conditions [10]. In the immersion zone, it can be expected that the measurement accuracy is higher due to the operational conditions.
Given this information, the objective is the estimation of the annual probability of failure knowing that the structure has survived 75 years and that the expected value

of the residual thickness is 10.5 mm. The probability of failure for the period between year 75 and year 100 is shown in Figure 4.4.

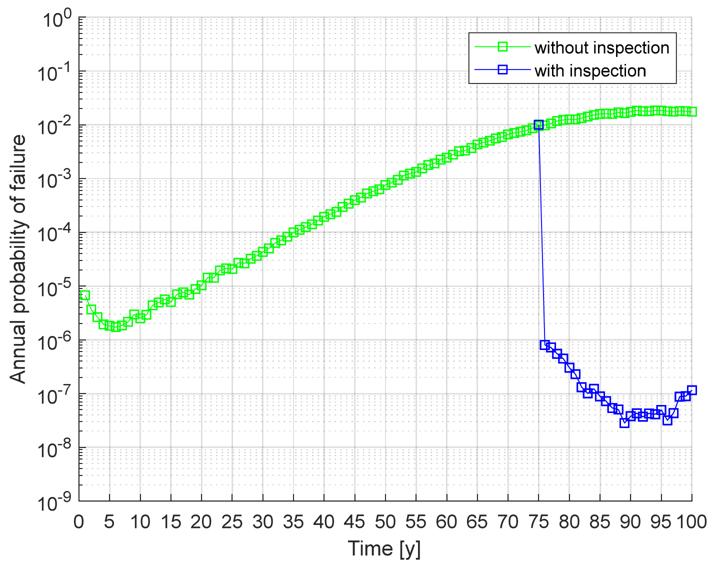


Figure 4.4: Annual failure probability of the sheet pile wall considering an inspection at year 75.

The red curve represents the probability of failure considering only that the structure has survived the first 50 years of the lifetime. The blue curve accounts also for the outcome of the inspection. The blue curve deviates significantly from the red curve for two reasons. The first one is that the loss of thickness has a great effect on the reliability of the structure, as shown in Figure 4.2. The second reason is that the actual loss of thickness is lower than what expected by the a-priori probabilistic model of Table 3.3.

4.4 Reliability updating based on load tests

In the following, it is assumed that a proof load test is performed at year 75. The proof load test is performed by applying a uniformly distributed load on the soil side. The intensity of the load is 25 kN. which corresponds to the 95% fractile of the distribution of the variable load (see Table 3.4).
The limit state function related to the inspection (Eq. 3.2) is defined as:

$$g_{LT} = \sigma_y - \left(\frac{M_{max}}{W_{corr,75}} + \frac{N_{max}}{A_{corr,75}}\right) \tag{4.2}$$

where $W_{corr,75}$ and $A_{corr,75}$ are the section modulus of the cross-section of the steel profile at year 75. These values of the geometrical properties may be derived from the measured residual thickness. In order to evaluate only the effect of proof loading, it is assumed that the actual loss of thickness is not known, but follows the probabilistic model of Table 3.3.
The goal is to estimate the annual probability of failure knowing that the structure has survived 75 years and that the proof load test has been successful. The

probability of failure for the period between year 75 and year 100 is shown in Figure 4.5.

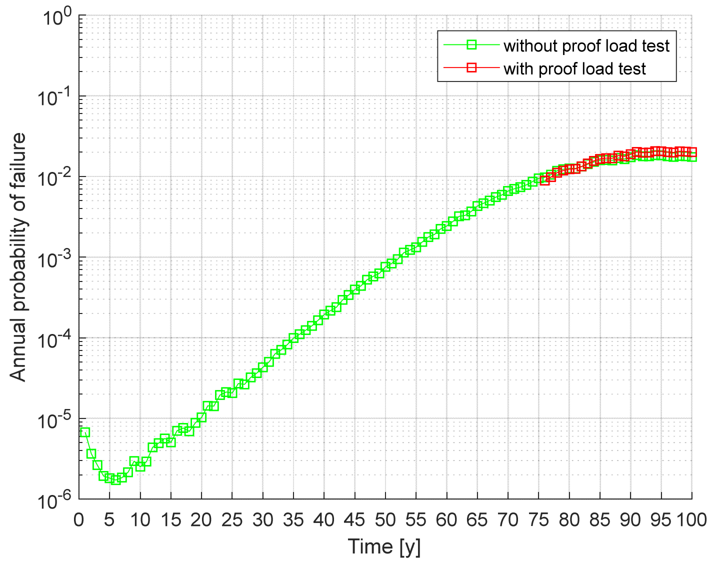


Figure 4.5: Annual failure probability of the sheet pile wall considering a successful proof load test at year 75.

The red curve represents the probability of failure considering only that the structure has survived the first 75 years of the lifetime. The blue curve accounts also for the success of the proof load test. It can be observed that proof load test has a significant impact on the reliability of the structure. Since the proof load test is successful (g_{LT} is positive), it means that a range of values of the maximum stresses caused by the bending moment and the axial force are not associated with zero probability.

5 Conclusions and recommendations

The structural reliability of existing sheet pile walls has been investigated by means of the annual probability of failure. Considering a reference period of 1 year allows to update the reliability of the structure in a rigorous way by accounting for available structure-specific information and the point in time where the information is gathered.

The annual probability of failure has been estimated using the First Order System Reliability method and the Equivalent Plane method in combination with simplified models of the behaviour of the soil-structure system. The investigation is focused on the ultimate limit state of yielding of the steel profile.

Measurements of the residual thickness and outcomes of proof load tests are the information used to update the reliability of the structure. For this purpose, specific performance functions have been formulated. In both cases, it has been shown that there is a reliability gain. However, this conclusion cannot be generalized. It is suggested to perform a sensitivity analysis of the reliability gain with respect to the outcomes of inspections and load tests in the form of what-if-scenario analysis.
The outcomes of such analysis could be used for which information and at which point in time should be gathered from the structure for an optimal assessment of the structural reliability

6 References

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A Comparison with Plaxis results

Input values:

Parameter	Value
γ_{sat}	According to Table 3.1
c_a'	According to Table 3.1
φ_a	According to Table 3.1
w_l	- 4.75 m NAP (excavation side) - 4.00 m NAP (ground side)
q_y	19.75 kN/m
z_{exc}	- 7.0 m NAP
$\Delta t_{(A,B,C,E)}$	0 mm

Figure A.1: Axial force along the sheet pile wall (Plaxis calculation).

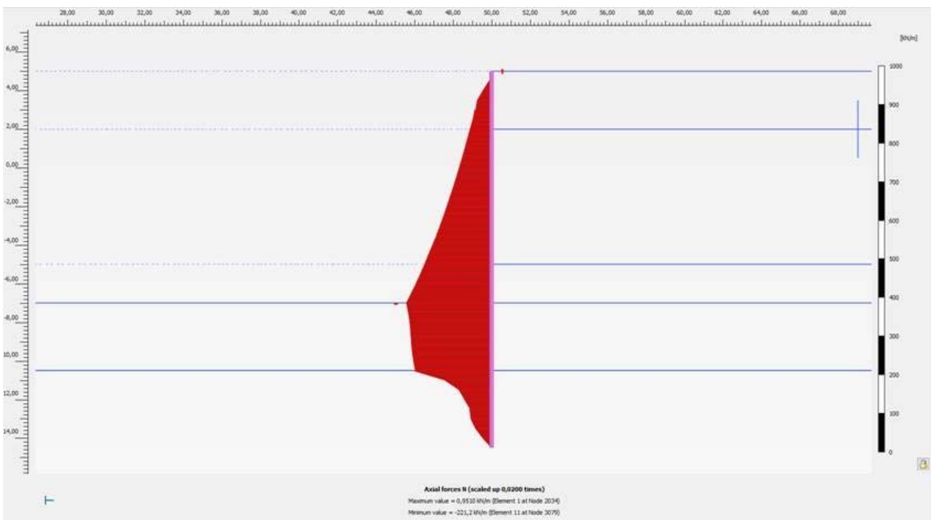
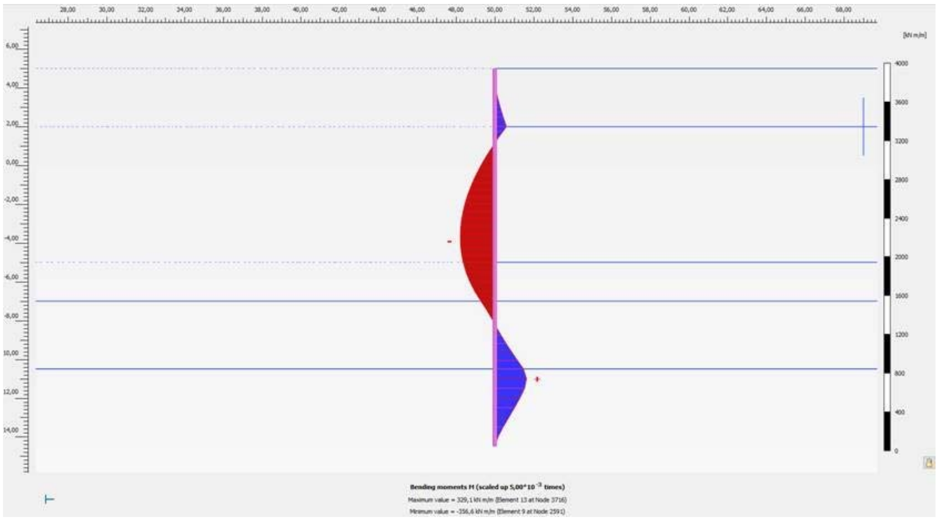


Figure A.2: Bending moment along the sheet pile wall (Plaxis calculation).



The maximum bending moment and normal force lead to the stresses:

σ_M = 356.6 [kNm/m] /2600 [cm3] = 137.2 N/mm2
σ_N = 221.2 [kN/m] /198 [cm2] = 11.2 N/mm2 → 8% of σ_M

The 8% value is considered to be small enough to be able to neglect the contribution to sigma by the normal force.

The maximum bending moment resulting from the Blum's method was also compared to the maximum bending moment from the Plaxis calculation using similar input values. Based on previous research, it was found that the Blum's method overestimated the bending moment and a reduction factor of 0.7 should be applied.
The maximum of the absolute bending moment is 365.5 kNm/m which is close to the result in Plaxis of 356.6 kNm/m. The resulting bending moment is presented in Figure A.3.

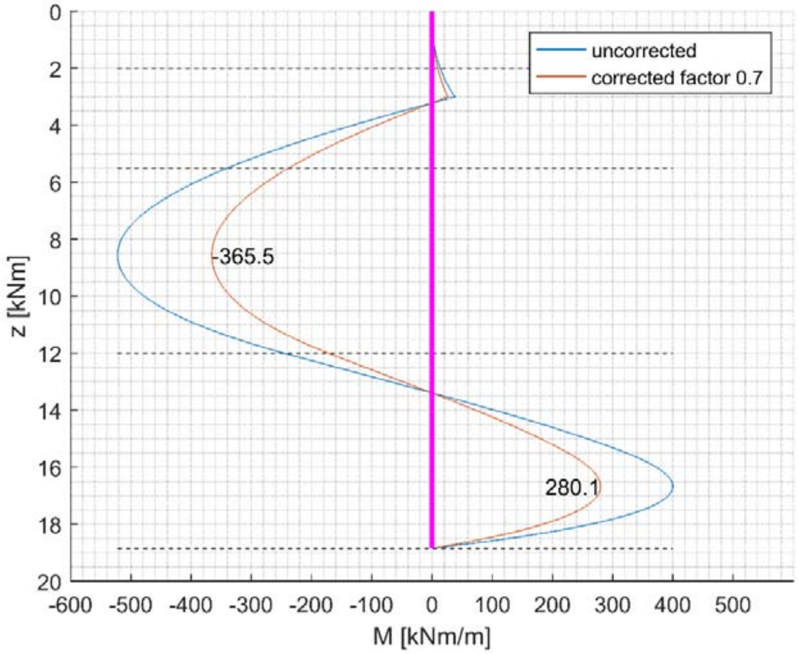


Figure A.3: Bending moment distribution from Blum's method with similar input values as the Plaxis calculation with and without application of the reduction factor of 0.7

B Accuracy of Equivalent Planes method

Two example cases were used to evaluate the accuracy of the Equivalent Planes method and to test the algorithms used. Results for different methods are compared; the Equivalent Planes method, crude Monte Carlo simulations and using the First Order System Reliability Method (FOSRM). The conditional probability of failure evaluated by Equivalent Planes method is calculated using both FORM as crude Monte Carlo simulations.

The first case is a two-variable case with a relatively high probability of failure so few Monte Carlo samples are necessary for an accurate evaluation of the probability of failure. The definition of the two random variables being; resistance R ~ N(10,2) and load S ~ N(9,1). The yearly failure probability is calculated using FORM. The load is independent every year. The resistance however, is fully dependent. Therefore the yearly failure probability is calculated using the information of known survival in previous years. The three methods described in previous paragraph are used, using our own algorithms. The results are presented in the Figures B.1 and B.2. The different methods give similar results for this basic case and are therefore found to be equally accurate.

This algorithm was however developed for probabilities of failure and not probabilities of survival. Therefore this algorithm was also checked against the FOSRM results. These results can be found in Figures B.3. It was found that the results start to deviate from the other methods after a few years. This is due to a difference in influence coefficient calculation. Therefore in this stage of the research the own developed algorithms are used.

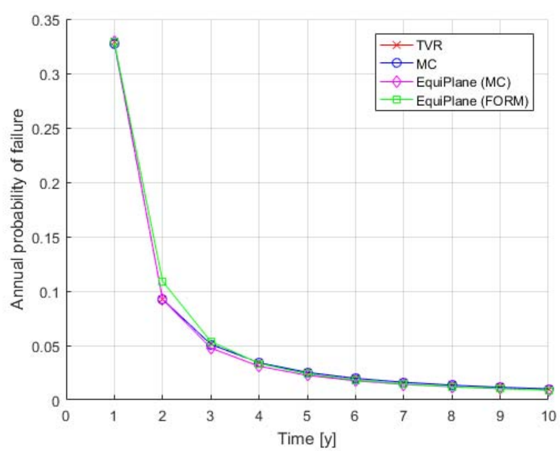


Figure B.1: Annual probability of failure for example case 1 using the First Order System Reliability Method (FOSRM), Monte Carlo simulations (MC) and the Equivalent Planes method (EquiPlane) with either FORM or Monte Carlo.

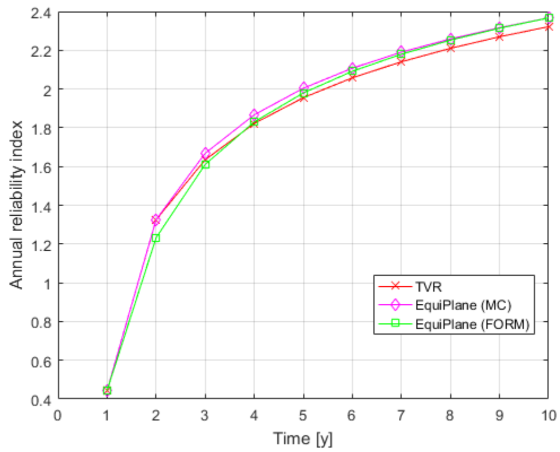


Figure B.4: Annual reliability index for example case 1 using the First Order System Reliability Method (FOSRM), Monte Carlo simulations (MC) and the Equivalent Planes method (EquiPlane) with either FORM or Monte Carlo.

The second example case is a two-variable case with a yearly failure probability as the sheetpile wall considered in this research. The definition of the two random variables being; resistance R ~ N(10,2) and load S ~ N(1,1). The yearly failure probability is calculated using FORM. The load is independent every year. The resistance however, is fully dependent. Therefore the yearly failure probability is calculated using the information of known survival in previous years. The three methods described in previous paragraph are used, using our own developed algorithms. The results are presented in Figure B.3.

The calculated yearly probabilities of failure deviate a little more as with the previous example. For the Monte Carlo simulations more samples should be used in order to get better (and smoother) results. The order of magnitude of all the calculated results is similar. The method and algorithm for the Equivalent Planes method therefore seems sufficiently accurate.

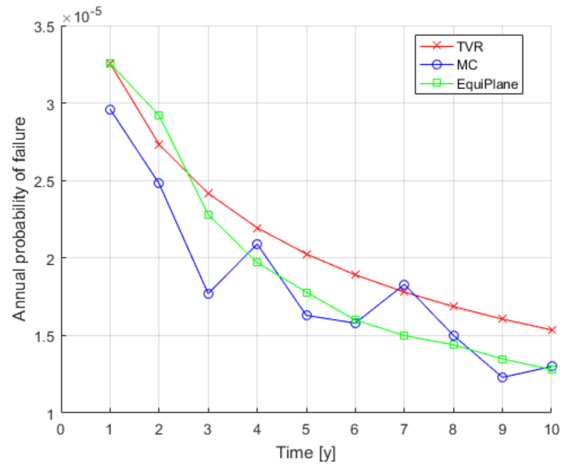


Figure B.3: Annual failure probability for example case 2 using the First Order System Reliability Method (FOSRM), Monte Carlo simulations (MC) and the Equivalent Planes method (EquiPlane) with FORM.