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The Observational Method for building pits in soft-soil conditions

A study on measurement-processing and feasibility of the Observational method Ab Initio approach.

W.J. de Wolf
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A study on measurement-processing and feasibility of the Observational method Ab Initio approach.

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Summary

In the Observational Method Ab Initio approach a flexible design and construction plan is established to allow anticipation to observational feedback during the construction phase. This way, the structural design can be optimized to the in-situ conditions which is beneficial from both safety and economic point of view. In the application of building pits in the soft-soil conditions of the Netherlands the method has the additional value to verify SLS criteria and timely detect unforeseen events. So far limited building pits have been executed via this design strategy. The main reasons for this are the lack of a design procedure and the problematic quantification of safety of the flexible retaining wall design.

A strategy for the execution of the Observational Method Ab Initio approach to retaining wall design is described by the CIRIA guideline C760. In this study, the suitability of this 5 step-strategy has been investigated by means of a benchmark. This investigation indicated that, although the CIRIA guideline C760 contains a valuable design strategy, only a qualitative description of safety is provided. Therefore, this study introduces a methodology for real-time measurement-processing with the use of a Bayesian update. The Bayesian update combines the information of the predictive computer model with the information obtained from measurement sets during construction. By describing this information via probability density functions different uncertainties in both the design and construction phase can be weighted in the outcome of the Bayesian update. Consequently, the retaining wall behavior can be re-assessed throughout construction.

This methodology is applied to measurement sets gathered at the construction of two different building pits in the Netherlands. Both case studies showed that with the Bayesian update and consequential calibration new parametric distributions can be found. Those parametric distributions describe the retaining wall behavior from which safety definitions in term of a reliability index can be derived. The performance of this methodology for measurement-processing depends on the accuracy of the calculation model and the measurement interpretation. Especially in the case of unexpectedly high and/or fast progressing retaining wall displacements, measurement interpretation is necessary to select the best strategy to redirect the structure. Although this measurement interpretation is a challenge, it is believed that the Observational Method Ab Initio approach complemented with the Bayesian update is a promising design strategy. Its application to the construction of building pits definitely has economic potential and would be favorable for risk management.

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A diploma contains more than the prove that I read books, derived formulas and passed exams. As this work finalizes my study at the Technical University of Delft, a final thanks goes out to family, friends and whom it may concern. Although each of us have or have had their own struggles, although some of you are plane tickets away, you provided me with joy. Thank you for taking care of me.

The latest story that I know is the one that I'm supposed to go out with – Janis Ian.

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List of symbols

Abbreviations

EC7	Eurocode 7 [9]
OM	Observational Method
PDF	Probability density function
SLS	Serviceability limit state
SSI	Soil-structure interaction
ULS	Ultimate limit state

Latin symbols

c	Cohesion
C_r	Risk consequence
d	Deflection of the sheet pile wall (measured)
EI	Stiffness
k	Modulus of subgrade reaction
n	Number of measurements
OCR	Overconsolidation ratio
P	Surcharge loads
P_f	Probability of failure
P_r	Probability of risk
R	Risk
V	Coefficient of Variation
X	Indication of soil parameter
X_d	Design value of parameters X
$\mathrm{X}_{d,OM}$	Design value in adopting the Observational Method
$\mathrm{X}_{k,5\%}$	Characteristic value of parameter X typically at 5% tail

Greek symbols

α	Parametric influence coefficient
α_{hyp}	Critical value for hypothesis test
β	Reliability index
β_{OM}	Reliability index related to the design based on Observation Method
$\gamma_{5\%}$	Partial factor corresponding to the 5% characteristic value
γ_{sat}	Saturated Volumetric weight
γ_{unsat}	Unsaturated Volumetric weight
δ	Wall friction angle
μ	Average of a PDF or dataset
σ	Standard deviation of a PDF or dataset
μ'	New estimation for μ according to Bayesian update
σ'	New estimation for σ according to Bayesian update
φ	Soil friction angle

1. Introduction

1.1 Uncertainty in Geo-Engineering

A good foundation is the basis of every structure. The foundation transfers load from the structure to the ground to provide stability. Once it is failing problems arise as often known in practice: For instance, structures show too much settlement, or, in worse situations, partial collapse could occur. Such technical losses also result in economic losses and depreciation of the building industry [1]. It is thus important to understand interactions and failure mechanisms.
In a building project, it is the job of Geo-Engineers to predict how ground will behave during the construction and lifetime of the new structure. This requires knowledge of the ground profile and its properties. Additionally, the interaction between the structure, its foundation and the soil needs to be assessed. The overall assessment is associated with the following complexities [2]:

- Geological uncertainty: Due to geological processes, it is hard to predict what stratigraphy and geological conditions are to be encountered on the construction site.
- Heterogeneity: The degree of heterogeneity describes the variability of characteristics within a material [3]. This means that within one soil layer there are weaker and stronger zones present.
- Coupled ground behavior: As soils are a mixed medium (containing air, gas, liquids, minerals and organic content) its behavior is complex to describe. This has its effect on the accuracy of the prediction of ground behavior and the soil-structure interaction (SSI).

These complexities introduce uncertainties to the building project. Generally, two broad types of uncertainties can be identified [4]:

1. *Aleatoric uncertainty* is associated with the *inherent variability of information*: To some degree phenomena are not accurately predictable due to natural randomness. An example is the heterogeneity of ground characteristics. For laboratory experiments, different strength parameters can be found for one soil sample. Another example is rainfall. From year to year, the amount of seasonal rainfall differs.
   Based on many measurements, it is often possible to express the natural randomness by a range of variability. This is usually done with probability density functions (PDF).

2. *Epistemic uncertainty* is associated with the *imperfection in our estimation of the reality*. This is the case with mathematical equations that are used to predict coupled soil behavior or the interaction between soil and structure. The epistemic uncertainties lead to a lack of confidence in predictions.

Because of having these uncertainties, risks are posed to construction works to meet their imposed design requirements. It is up to engineers to indicate and minimize those risks. This thesis is about the Observational Method, which is a strategy developed to deal with the uncertainties that are often problematic in the building industry. The method is an alternative to the commonly applied Level I design approach. The Observation Method (OM) is in the framework of risk reduction as will be clarified in the next section.

1.2 Risk Management in Geo-Engineering

Geo-Risk management aims to deal with the *risk profile* of a building project [1]. This starts with an inventory of the uncertainties encountered in the design phase, followed by the categorization of risks. All the risks together form the risk profile. The consequential quantification of risks is important in the decision how to deal with the profile.

The following distinctions are made in the categorization of risks (Figure 1.1) [5]:

- Foreseeable: The risks made explicit in the risk profile. It is possible to anticipate to the risk by the structural design.
- Unforeseeable: The risks that remain unknown until they occur, which could – but not necessarily need to – be disastrous to a project as the design is not anticipated to it. Unforeseeable risks are often encountered in building projects because of the previous described complexities and changing circumstances on site.

To assess the impact of risks to the geotechnical structure it can be useful to further categorize them as positive or negative.

- Negative: Whenever negative risks occur, negative consequences in terms of costs and/or safety are posed to the structure. An example is delay in the construction process due to encountering unexpected soil conditions (like boulders or polluted soil).
- Positive: Positive risks have a favorable effect to meet the project requirements. An example of a positive consequence is encountering stronger soil behavior than presumed in the design phase, which enhances the structure's stability.

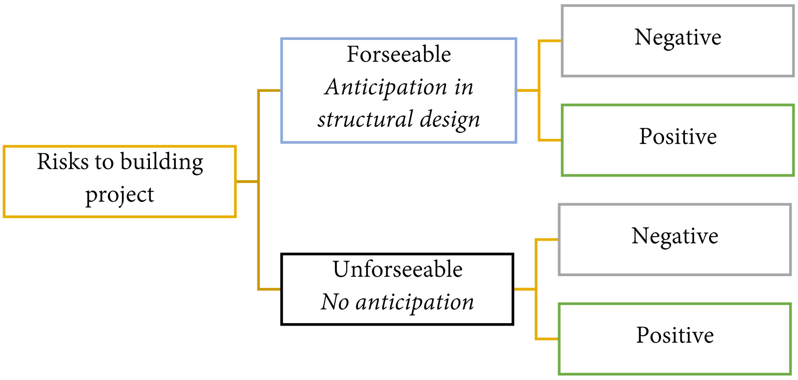


Figure 1.1 Categorization of risks

The chances of the actual occurrence of a risk can be expressed as the probability of occurrence P_r. This probability can be determined by analyzing what conditions accommodate the risk to occur. The impact of a risk to the project is usually expressed in terms of costs. These include costs to fix materialistic damage but can also consist non-materialistic damage like project delay and loss of trust in the contractor. These are included by the risk consequence C_r.
Consequently, the risk R can be expressed in costs as well, according to formula (1) [6]:

$$R = P_r \cdot C_r \qquad (1)$$

As the magnitude and the characteristics of the risk are quantified, it can be decided how to manage it. Three different attitudes can be distinguished in risk remediation [7]:

• Risk reduction	Reducing the impact of the risk by lowering causes and/or effects.
• Risk avoidance	Removal of all risk causes (make the probability of occurrence insignificant).
• Risk retention	The acceptance of a risk as avoidance or reduction is too expensive or impossible.

1.3 The Observational Method in Geo-Engineering

In 1969 the Observational Method has been introduced by Peck as a possible solution to deal with risk profiles in geotechnical engineering [8]. The Observational method (OM) is a *risk effect reduction* method that uses a flexible constructional design. It is based on the fact that ground conditions and behavior remain uncertain until construction works are actually carried-out. Therefore, it would be valuable to evaluate these during the construction period. This can be done by thorough monitoring of for example, pore water pressures, surface settlements and structural displacements. By systematically analyzing the gathered data, the risk profile can be adjusted. Throughout construction the structural design can be modified: If necessary the construction can be enhanced by adding structural elements, or, in a different scenario, elements can be removed to save material costs. The design at the end of construction aims to suit the true site conditions.

Peck defined two different types of application of the OM [8]:

- *Best way out* (in the moment): Primarily, it is not intended to use the monitoring data to adjust the structure. Therefore, a non-flexible design will be constructed. This design is based on rather conservative assumptions. It should be sufficiently safe - regarding the risk profile that is established in the design phase. Monitoring data is gathered once construction starts. However, the data is rather used to act proactive to unforeseen events. If, for some unforeseen event, the structural design is pushed towards a rather unsafe state, the monitoring data can be used to re-assess the situation in the field. That way, a decision can be made on how to adjust the construction works to finish safely.

- *Ab initio* (from the start): In the Ab initio scenario the implemented design (*preliminary design*) is based on *most probable* ground conditions. This is in the contrary of basing a design on rather conservative assumptions, as done in the Best way out approach. By implementing an OM Ab initio design the high degree of conservatism can be avoided [8]. However, the assumption of encountering the most probable ground conditions will need to be verified. This is done as construction progresses by gathering monitoring data. In the design phase different scenarios are thought through that represent possible situations that could be encountered in the field: In case data disproves the assumption of having most probable ground conditions, the flexible constructional design can be enhanced by applying *contingency actions* - structural changes. These contingency actions are also part of the design effort.

1.4 Thesis objective

The CIRIA guideline 185 is the most extensive guide on the Observational method [9]. Despite its potentials, there are few projects executed with the OM Ab initio approach. Geotechnical structures are commonly established following a conservative design approach according to Eurocode 7 [10]. Therefore, research and case studies can still contribute to a shift towards a construction industry in which this method is more widely applied. This research is performed in corporation with the Dutch company *Fugro* under the section Geoconsultancy, with the aim to contribute to innovations in Geo-Risk management.

The focus of this thesis is on the application of the Observational method Ab Initio approach to the design and construction of building pits in soft-soil conditions. Although in the Netherlands there is a lot of experience with building on soft-soils, often there are features in projects that are unexpected or hard to predict. Therefore, the question arises whether the method could be beneficial for these types of projects, leading to the main question of the thesis:

Under what conditions would application of the Observational Method Ab Initio approach be feasible to the construction of building pits in soft-soils?

The following sub-questions are formulated, each of them concerned with the construction of building pits in soft-soils:

1) *How suitable is the prescribed approach of the CIRIA guideline?*

2) *How to ensure safety during the application of the Observational Method?*

3) *What are the benefits and pitfalls of the method?*

1.5 Thesis Outline

To explore the potential benefits of the OM it is at first necessary to understand the methods' principle. This is included by **chapter 2**, which presents the information obtained from the literature study on the method. The chapter aims to present an unambiguous formulation of the method, an understanding of the potential and restrictions that should be considered once actual implemented. From the broad theoretical framework of chapter 2, the focus shifts to retaining walls in the consequential chapters. To prepare for the benchmark and the case studies, **chapter 3** presents features on retaining wall design. This chapter ends with indicating the delimitations.
With regards to sub-question 1, **chapter 4** demonstrates the application of OM according the CIRIA guideline 185 to a theoretical Benchmark. Consequently, **chapter 5** introduces a methodology containing statistical methods to enhance back-analysis of monitoring data with the goal to clarify safety definitions. This is again demonstrated with a Benchmark.
Chapter 6 contains the analysis of two case studies of executed building pits in the Netherlands. This with the aim to further specify on the practicality of both the CIRIA guideline 185 and to test the methodology with statistical methods. Finally, answers are formulated to the sub-questions and main research question in **Chapter 7**, along with recommendations for future research.

2. An overview of the Observational Method

The information presented in this chapter will serve as a basis for the research carried out in this thesis. This chapter introduces the reader to the theoretical framework of the Observational Method Ab Initio approach. At first, the uncertainties as often encountered in construction projects are made explicit. Consequently, as the OM is an alternative to the conventional design method, basics of limit state design are explained. Secondly, the OM principle and potential is presented by the information provided by standards and guidelines. Thirdly, an assessment of project requirements is made according to the methods principle. Finally, complicated aspects that need to be considered in the project organization are stated. The goal of this chapter is to familiarize the reader with the different aspects of the OM Ab Initio approach and to provide insights in the reason why it has not been widely applied yet.

2.1 Uncertainties in Design and Construction of Geotechnical structures

Section 1.1 clarified on the complexities that are associated with construction projects. In this section, the uncertainties are made explicit for both the design and construction phase. They are summarized by Figure 2.1.

Design Phase - Site investigation

The site investigation serves to gather the information on soil conditions. The results will be used as the input for design calculations. They are mainly affected by the following uncertainties [9]:
Geological uncertainty: Between the locations of CPTs, boreholes and/or other performed site investigation tests, it is necessary to interpolate and extrapolate to map the construction site. Ground conditions may vary unexpectedly in between executed boreholes. This phenomenon is labelled as *geological uncertainty* [9]. Due to complex ground formation processes, the layering between boreholes can only be estimated. Also, unexpected natural conditions could be encountered, like boulders or buried channels. Due to past human activities, cables or old structural objects might be present as well.
The degree of geological uncertainty can only be estimated with the information on hand and is therefore an *epistemic uncertainty* (section 1.1). By performing more site investigation tests, the degree of uncertainty can be reduced [13]. A balance between costs and benefits should decide how many site investigation tests should be performed for a specific project. In practice, often information is adopted from previously executed projects in the same area.

Parametric uncertainty: To describe ground behavior, characteristics of the ground are expressed in parameters that can be determined via laboratory test performed on soil samples. The parametric uncertainty is typically *aleatoric*: If multiple tests are performed on one soil sample, different results can be found. For this aspect, it is favorable to perform multiple tests to reduce the variance of a certain parameter [13]. The possible distortion of soil samples, testing errors and scaling effects need to be considered as well.

Design phase		
Site investigation		**Design calculations**
Geological uncertainty: - Exact layering (stratigraphy) - Geological features - Man-made obstructions - Contamination Parametric uncertainty - Degree of heterogeneity	*Modelling input*	Model uncertainty: - Parametric input for materials - Calculation method - Assessments of loads and conditions in/around - Construction environment

Final Design
+
Construction sequence

Construction	
Construction works	
Construction management: - Execution of construction works and/or ground treatment techniques - Following the right construction phasing, (loading) speed. - Storage and transport of construction material	Unforeseen events: - Result of ground treatment techniques - Rapid detoration of construction material - Loads differ from day to day - Circumstances in neighbourhood can differ

Figure 2.1 Overview of uncertainties in design and construction of geotechnical project.

Design phase: Pre-design and final design

Model uncertainty: From the site investigation it follows what in-situ conditions can be encountered. For the design calculations often a computer model is used to assess the soil-structure interaction and ground behavior. This model requires deterministic input of soil parameters (which on itself is uncertain). The output of the calculation is an *epistemic uncertainty* due to the necessity to make simplifying assumptions and to ease up the calculation. Additionally, the selected model may not be able to assess certain failure mechanisms. Sometimes the occurrence of a progressive failure mechanism is not predicted, as they are caused by a series of events. The same holds for failure caused by a rare combination of parallel events. The fact that predictions are based on certain design assumptions, either made by adopting a computer model, or as chosen by Engineers, are the biggest source of uncertainty.

Construction phase

During construction additional risks may arise due to the possibility of unforeseen circumstances and/or poor construction management.

Unforeseen events: As the construction site is a dynamic environment, circumstances are often different than assumed. There is a distinction between *truly unforeseen events* and unforeseen events caused by unthorough risk analysis [14]. The latter is a limitation in knowledge and/or experience of Engineers. However, some changes in the dynamic environment can be regarded as *aleatoric*. The border between truly unforeseen or not is often arguable. For example: When executing ground improvement techniques final quality can vary, which is known in advance. However, the extend of this variation is often hard to quantify.

Construction management: Because this concerns human behavior, the uncertainty caused in the management aspects might get overlooked. However, many past projects showed failure, delay or damage because of poor construction management. The possibility of human errors in the execution of the construction works should be checked [5]. Construction phasing should be followed because they relate to design calculations. Moreover, materials should be stored and handled with care to avoid loss of quality or rapid detoration [9].

2.2 Limit state design

In the quantification of safety of a geotechnical structure it is said that failure happens when (a part of) the structure no longer fulfils one or more of its desired functions. Those functions are not only technical, but also take into account the comfort of users. This definition of safety is translated to so-called limit states, a set of criteria that need to be met by the structural design. The structure fails (and is regarded as unsafe) if limit states are exceeded [15].

In the Eurocode 7 two limit states are distinguished [10]:

1) SLS: The serviceability limit state ensures workability and comfort. For many geotechnical applications, this means that there is a restriction on the maximum value of displacements. This limit value can be derived from both technical and social context, like stakeholder demands. A SLS can be set to the structure itself as well as to components and/or neighboring features. For each of them, the following check is performed:

$$E_k \leq C_k \qquad (2)$$

   C_k is the deterministic limiting value for displacement.
   E_k stands for the deformation that is to be predicted by calculations in the design phase. Its precision and accuracy thus depends on the chosen calculation method, model, parameter input and assumed stratigraphy.

2) ULS: The ultimate limit state is associated with overall and partial collapse. A check is performed with so-called design values of acting forces E and resistant forces R:

$$E_d \leq R_d \qquad (3)$$

The design values are introduced in the ULS to deal with the different uncertainties as introduced in the previous section. The motivation to use design values in the ULS-check is to create enough margin to ensure that, despite uncertainties, there is still a very low chance that collapse will happen (reduction of risk effects). This is done by selecting a rather high deterministic value from the distribution of possible loads. For the resistance, a low value is selected, such that the structures' resisting forces will not get overestimated (Figure 2.2).

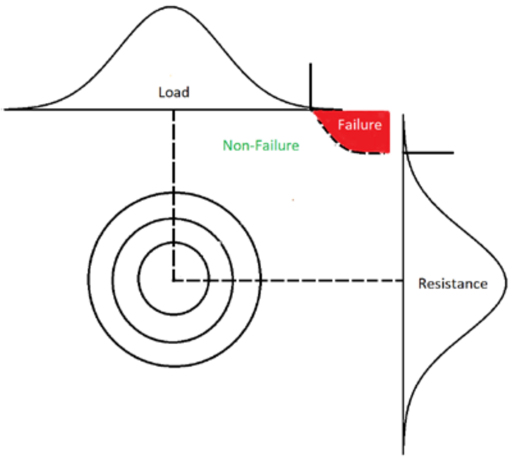


Figure 2.2 Illustration of selection of design values for resistance and loads [18].

Design values for resisting forces can be calculated via formula (4).

$$X_d = \frac{X_k}{\gamma} \tag{4}$$

X_k is the characteristic value at the 5% tail of the distribution that describes the uncertainty in the assessment for quantity X (Figure 2.3). Such characteristic values are also predefined by Eurocode 7.

Based on a student-t distribution, the 5% characteristic can be found:

$$X_k = \mu - k\sigma \tag{5}$$

$$X_{k,5\%} = \mu - 1.645\sigma \tag{6}$$

Via formula 4 the characteristic value is further reduced by the partial factor γ. The partial factors are based on extensive material testing in common conditions of a region. In Europe the partial factors differ from country to country. The partial factor is a deterministic value based on the following considerations [13]:

- The required safety level for the construction of interest. Typically, 3 different safety classes can be assigned to a construction, as in EC7 CC1, CC2 and CC3 with their corresponding reliability index β. The distinction of safety classes comes from the fact that exceedance of the limit states effects some buildings more than others: The impact to a hospital (CC3) is bigger than to a house (CC2) or a shed (CC1).
- The influence of X to the outcome of the overall geotechnical design analysis. This factor is typically assigned as influence coefficient α. For example, in an analysis both the uncertainty in the friction angle and the unit weight should be considered. However, the sensitivity of the friction angle to the outcome is higher than that of the unit weight. Therefore, the friction angle is assigned with a higher influence coefficient.
- The variation of X, e.g. a standard deviation of coefficient of variation will be considered.

Because of the parameter dependency, partial factors will vary for each relevant parameter for the ULS design calculations. Because the reliability is project dependent, also different partial factors will be applied according to the consequence class of the designed structure. After applying the corresponding partial factor, the obtained design value X_d is even smaller than the $X_{k5\%}$ as illustrated by Figure 2.3.

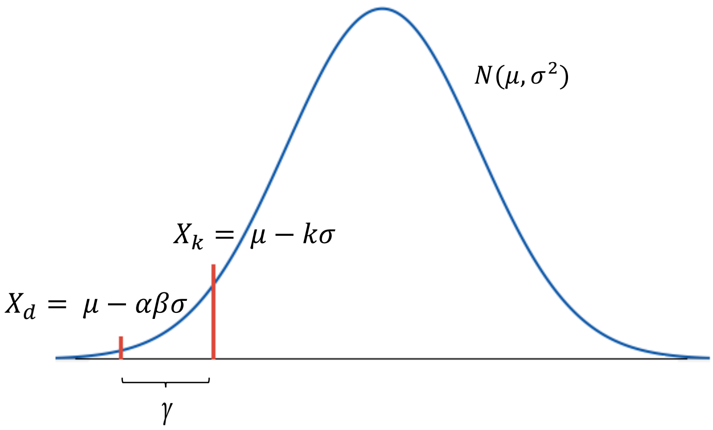


Figure 2.3 Relationship design value and characteristic value.

The design value is then be expressed as:

$$X_d = \mu - \alpha\beta\sigma \tag{7}$$

in which:
α = influence coefficient, parameter dependent.
β = reliability index, project dependent.
σ = standard deviation, parameter dependent

The reliability index β is statistically connected to a probability of failure via:

$$\beta = \Phi^{-1}(P_f) \tag{8}$$

The partial factor that are applied within one consequence class are all based on the same value for β. Theoretically, a structural design that is based on these design values would have probability of failure as stated in Table 2.1.

Consequence class [reference period: 50years]	RC1	RC2	RC3
β	3.3	3.8	4.3
P_f	0.0005	0.0001	0.00001

Table 2.1 Safety coefficient and corresponding probability of failure per consequence class [19].

This limit state design approach as described here is labelled as a Level I method – Semi probabilistic approach. A fully probabilistic approach aims to exactly calculate the P_f[15]. This is done by full evaluation of all the probability functions of different properties X. The methods to do that are labelled as Level III. An example of such a method is Monte Carlo simulations. Execution of such calculations usually go along with a lot of computational effort from numerical integration and parameter sampling. There are also Level II methods that only make use of the μ and σ of the parameters instead of the full probability density functions. An example is FORM – First order reliability method.

2.3 The definition of the Observational Method

Terzaghi recognized 2 types of methods to deal with uncertainties of ground behavior [8]:

1. Adopt an excessive factor of safety.
2. Derive assumptions in accordance with general, average experience.

As in Terzaghi's words: *"The first method is wasteful, the second is dangerous"*. By applying the first method it is feared for too much conservatism along with unnecessary construction costs. The second method might be dangerous to apply as experience is hard to manage. Design could be systemized by methodology, but this requires the generalization at the expense of customization.

Therefore, Terzaghi, and after him other authors, pleaded for a third method which is now known as the Observational Method. Other terms that are used for this method are *"active design"* or the *"learn as you go"* approach [11]. In 1969, Peck further promoted the Ab Initio approach by his Ninth Rankine Lecture. Although Peck was aware of certain technical, economic and social drawbacks he mentioned: "*The method offers many possibilities of spectacular savings of time or money, and it can provide the assurance of safe construction without the financial penalty attached to excessive safety*" [8].

The potential of the OM is in the use of monitoring data as construction takes place: As each construction site is unique, it would be more economic to base a structural design on these unique conditions. However, these conditions cannot be known with certainty just from site investigation (section 2.1). Also, other uncertainties can be encountered during construction. Therefore, it would be valuable to collect data on the site. The gathered information can be used to "finetune" the structural design. By selecting *trigger limits*, data could indicate if certain risks truly affect the structure and in what extend. Consequently, the structural design can be anticipated to those risks by means of a *contingency measure*. Already in the design phase, different scenarios with contingency measures are thought through. Projects suitable for this way of OM application need to have the following 3 characteristics [9]:

1) Flexibility in the design: The possibility to add or remove structural elements.
2) Flexibility in construction: The possibility to change the sequence of construction works in order to gain necessary monitoring data and to adjust the structure.
3) Enough time to learn and adapt the structure before failure happens: Any potential failure mechanism should not rapidly develop. There is time necessary to collect and interpret data. Consequently, if it turns out the construction should be adjusted, enough time is needed for execution and its effect.

In Peck's time, a big limitation was the lack of monitoring equipment and computer models that allow fast design calculations. However, throughout the '90s several authors dedicated their research to further develop the method, especially in the application of tunneling and retaining walls [17]. In the years starting form 2000 strong innovations in the field of Remote Sensing and ICT have changed the view towards the practicality of the method. The current definition of the OM, followed in this thesis, is the one stated by the CIRIA guideline on the Observational method [9]:

"The Observational Method in ground engineering is a continuous, managed, integrated, process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising

safety."

This CIRIA guideline C760 describes the following 5 steps on the execution of the Observational method [12]:

Step 1	*Parameter selection – Identifying the most probable set of parameters and a set of characteristic (5%) parameters based on site investigation.*
Step 2	*Calculations – Prediction of behavior of construction based on the parameter selection.*
Step 3	*Trigger limits and contingency measures – For each construction state trigger limits for the implementation of adjustments to the structure should be established. These are based on the predictions made in step 2. For each trigger limit a contingency measurement should be established as well.*
Step 4	*Monitoring system: Setup a robust monitoring system.*
Step 5	*The construction phase: Interpretation of the monitoring data and evaluation against trigger limits for each construction phase. Decision for contingency measures should be made based on the results of the evaluation.*

There are different approaches for the selection of the parameter set that serves as a basis for the preliminary design. The term *most probable* represents the probabilistic mean of the data (50%). Another parameter set, which is indicated as *moderately conservative*, has been promoted by Powderham [19]. This parameter set represents "a cautious estimate of the value affecting the occurrence of the limit state" [11], typically in between the mean and characteristic value (Figure 2.4). The exact position can be selected by Engineers based on the quality of laboratory tests and/or the quality and practicability of the back-analysis of monitoring data. As a cautious estimate is used, it would also be possible to implement *progressive modifications* to the structure – positive contingency actions.

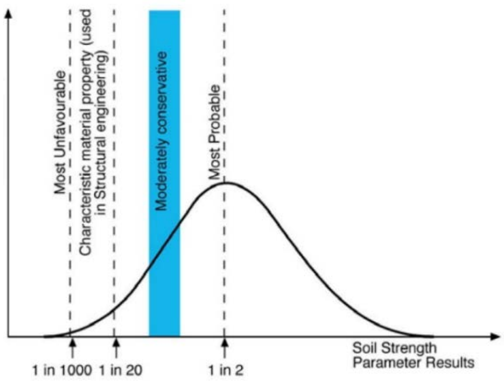


Figure 2.4 Moderately conservative parameter set.

2.4 Potential of the Observational Method Ab Initio

A drawback of the commonly applied Level I approach (section 2.2) is the high degree of conservatism. The following 3 drawbacks can be mentioned of this design strategy [16]:

1) If conditions on the building site are more favorable, savings could have been made in construction costs.
2) If unforeseen events happen, it could be that it affects the building pit such that limit states become a concern. Adjustment of the robust structure or adjacent buildings is then often costly. This is not always easy as well, as first the situation in the building site needs to be re-assessed.
3) This method of limit state design may lead to unthorough risk assessment and management, although this is not intended by the Eurocode.

On the contrary, the OM preliminary design is based on the soil conditions that are most probable to encounter (section 2.3). The potential in terms of money is mainly in the performance of this preliminary design: Because it is less robust, the costs of the preliminary design are less than in the case of limit state design. Once the monitoring shows the necessity to apply a contingency measure, costs are raised for the additional material, execution and the change in construction time. The potential of the OM is visualized by Powderham [19] with the graph in Figure 2.5.

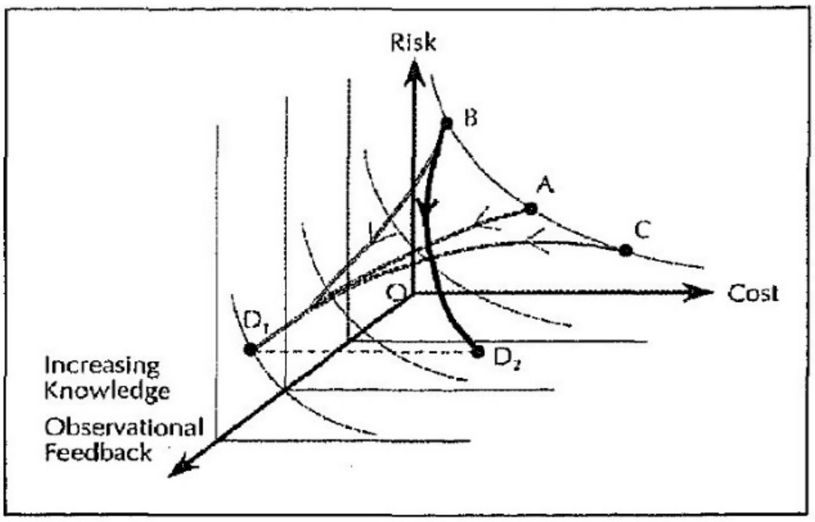


Figure 2.5 The 3D perspective of cost, risk and increasing knowledge [11].

With the OM application a 3rd dimension is added to the 2D Cost & Risk relationship.

<u>Costs & Risk relationship (2D):</u> In conventional design there is only the Costs and Risk perspective. If the risk needs to be mitigated, costs are required to enhance the structural design. This is illustrated with 3 states of a project depicted in the 2D plane: A, B, C. Project state B represents a structure that is under-designed. Once a risk indeed occurs, there is a fair chance that damage is caused to the project. Project state C has a lower risk profile than A and B. Because the risk level is low, the cost component is higher as C would typically have high safety margins.

Cost, Risk & Observational Feedback (3D): By adding a 3rth dimension to the problem it is stated that certain desired risk levels can be maintained not only by costs, but also by the observational feedback. This can be seen by following the line of project C to another project state, D1. Instead of enhancing the structural design to all the risks, a basic design is implemented. By Observational feedback it was shown that risks were not present at all when constructing. D1 thus illustrates a successful OM Ab Initio application.
Project B is in a critical unsafe state. The line between B and D2 illustrates an adjustment to B in which significant costs were still necessary. The risk level of the project has been reduced mainly by the adjustments of the construction that are based on the gain of knowledge of the in-situ behavior. This is an illustration of the "Best way out" approach of the Observational method. The line B – D1 illustrates the case that monitoring data showed that, although having a high-risk level at first, proved to be untrue.

Project D1 shows the biggest possible gain that can be made by adding the Observational feedback as a 3rd dimension. Projects that are in between D1 and D2 are applications of the OM in which some contingency actions turned out to be necessary. It illustrates that although these measures take money, still there is overall gain in construction costs possible. By stating this, it needs to be noted that the assumption is made that the contingency measures have lower costs than an overall adjustment to the construction as done in the "Best way out" approach. It can be understood that the OM preliminary design should not be in such a high-risk state as that of B as an uneconomical "Best way out" approach should be avoided. This requires the necessity to thoroughly assess the risk profile, categorize and quantify risks, as mentioned earlier.

2.5 Suitability of projects

In the framework of section 2.1, the CIRIA guideline mentions the following uncertainties as being especially suitable for the OM Ab Initio approach [9]:

Uncertainty	**Potential of the OM**
Geological uncertainty	*Actual ground conditions can be detected. Structure can be adjusted to appropriate design solution.*
Parameter uncertainty	*Mean and variance of parameter input can be verified. Structure can be adjusted to appropriate design solution.*
Uncertainty in ground treatment	*During treatment verification can be done by monitoring. Modifications can be made.*

Table 2.2 Suitable uncertainties for the OM Ab Initio approach according to CIRIA guideline [9].

The suitability of a project depends on different aspects. An overview of technical aspects is listed below. The list is partially based on a previous performed SWOT analysis [20], supplemented with further information provided from the CIRIA guideline 185 on the Observational method [9]. Other aspects include management and economic considerations. These are stated in Appendix I.

Technical aspects:

1) A **high degree of flexibility in the structural design**: It should be able to add additional support, either temporary or permanently, to the construction.
2) **Detectability of failure mechanisms**: The interpretation of the measurement data should be unambiguous, such that the development of a potential failure mechanism can be derived with certainty.

3) **Enough time to interpret data**: Failure mechanism should not be developing rapidly.
4) **Flexibility in the construction sequence**: Construction should be incremental and/or in stages to allow for data-processing and structural changes.
5) **The monitoring system should be robust**: The system is able to perform under the changing circumstances of the construction site throughout the construction process. Failure or damage to devices should be detectible and a back-up solution should be available.
6) **The monitoring data should be reliable**: The device should have sufficient precision and outliers and/or systematic biases should be detectible.

Example projects

With regard to technical aspects 1 to 4, multistage projects (staged excavation, staged application of load) are preferred. An example project would be ground improvement by preloading [9]. The goal of ground improvement is to reduce the amount of settlement during construction and lifetime. Based on monitoring of surface settlement the stiffness of the ground could be assessed. As a contingency the amount of preloading can be adjusted. Because this concerns a consolidation problem, displacements are of primary interest. These can be directly monitored and interpreted. Another application is the NATM tunneling method [21]. The amount of added support and the rate of advance in the excavating process is controlled by monitoring results.
In the construction of building pits, staged excavation takes place. The occurring sheet pile wall deflections, that can be monitored by inclinometers, depend on the soil strength characteristics. Therefore, the CIRIA guideline 185 also suggests the suitability of the OM for the application of retaining walls.

2.6 Context with Eurocode 7

The OM is mentioned in the Eurocode 7 as formulated in 2004 [10]. In a new formulation of the Eurocode, which is still in draft, more detail on the OM is given in a special Annex [22] The information is adopted from the CIRIA. Appendix I contains an overview of both formulations. The main differences show a renewed interest in the method's potential.

The use of the OM is stated in the original Eurocode 7*"in case of high uncertainty in the ground profile or soil-structure interaction"* [10]. The design standard acknowledges that for such cases the use of standardized partial factors would be incorrect as the factors do not correspond to what is commonly encountered in the subsurface. The Eurocode thus promotes either a different design approach or the use of adjusted partial factors obtained by extensive site investigations. In the renewed formulation the OM is stated as a method that could be used more generally instead of only in case of high uncertainty. By stating this it enlarges the methods' use.
Many authors have raised their concerns about prerequisites [11]. In the new version, arbitrary terms like "acceptable limits of behavior" are avoided. Instead it is referred to limit state design based on most probable soil conditions, following the 5 steps recommended by CIRIA. This gives the engineers a better indication on a design procedure.

2.7 Contracting & Team management considerations

Traditionally, projects consist of at least a client, designer and contractor. The client is the initiator of the project and therefore, responsible for communication with stakeholders and financing the project. The job of the designer is to come up with a technical design, meeting the ULS and SLS requirements. The contractor is responsible for execution of the design.
In the *contracting phase* different parties join to allocate the risks and responsibilities of a project by means of a contract. There are different types of contracts possible, but not all of them are favorable

in case the OM is adopted [1].

In the implementation of the OM, all parties should be on line about the risks and benefits that come along with it.
The flexible OM design leads to variability in the construction costs and possible completion date. All parties should be aware of this. The impact of contingency measures on costs and completion date should not influence the choices made on the structure's safety. The option of using the OM might be interesting for designers and contractors because of the opportunity to learn. However, it is the client that is financing the projects and often carries the responsibility to communicate with the other stakeholders [1]. Therefore, the client is often more interested in a successful result in time. This requires Engineers to take their responsibility to manage the risk level and to actively communicate throughout the construction process.
The integrated approach between design and construction phase constantly requires the need to verify the design assumptions and to analyze whether the in-situ state of the design matches the predictions made in the design phase. Besides the team's appetite for review, construction workers also need to be properly informed of the building method to be comfortable with the application of contingency measures [1]. This emphasizes the need for on-point management in the building pit.

It should be realized that the demands in terms of management and communication should be met by the party that is responsible for construction and monitoring. The party responsible for the design, should take the technical aspects (section 2.5) in consideration. In a Design & Construct contract only one party is responsible for both the design and the construction (and preferably, the monitoring as well). This type of contract is especially favorable, because of the following reasons [12]:

- The possibility for the design and constructing teams to closely corporate. This is favorable for the communication.
- As one party responsible for the risks that come along with implementation of the OM, motivation in both design and construction teams increases.
- As a result of close corporation, the objective is clear to everyone involved. With enhanced communication, reaction time to risks might be minimized. This allows fast anticipation, favorable for safety and construction costs.

2.8 Current status of Observational method

Currently, the Observational Method is commonly applied as a “Best way-out” solution [23].
In this Best way out approach usually high costs are needed to adjust the non-flexible structure established by limit state design. Although “Best way out” solutions can serve as an inspiration for dealing with certain failure mechanisms, such cases do no good to the term “Observational method”.

In 1996, Powderham and Nicholson summarized the following list of required improvements that would be necessary to make this shift [24]:

1) Establish a clear definition of method including objectives, procedures and terms, with a clear emphasis on safety.
2) Increase awareness of the method's potential and benefits, particularly to clients, contractors and regulatory bodies.
3) Remove contractual constraints.
4) Identify potential for wider use.
5) Initiate focused research projects.
6) Improve performance and interpretation of instrumentation systems.
7) Establish extensive database of case histories.

Developments in the recent past renewed the interest in the Ab Initio approach. Especially developments in the ICT resulted in cost-effective monitoring: The automatization of data processing, data sharing and real time monitoring has made it easier to control the construction site.
Some authors, like Spross [16,23,25] have contributed their research to safety definitions (improvement 1). Although he motivated the value of probabilistic methods, his research is still in concept. Additionally, few research is focused on soft-soil areas. A previous research questioned whether the Ab Initio process could be standardized, given the complexities in the interpretation of monitoring data and the prediction of soil behavior [26].
As long as point 1) has not been improved, it is hard to improve the others. However, as seen in the new formulation of the Eurocode 7, awareness of the potential is developing.

3. Retaining walls in soft-soil conditions

In section 2.1 design uncertainties were presented for geotechnical projects in general. Those are also applicable to retaining wall design. As in the next chapters the potential of Observational Method will be investigated for sheet pile wall designs, it is important to first create an understanding of the design procedure. Therefore, some aspects on sheet pile wall design are presented in section 3.1 according to Eurocode 7 [10] and Dutch guideline CUR166 [27]. Next, section 3.2 presents delimitations that will be followed during the research.

3.1 General aspect on sheet pile wall design

Main function

Retaining walls are used to laterally support the soil positioned for slopes that are not naturally kept. This implies temporary or permanently withstanding earth and water pressures. For excavation works in soft-soil conditions a sheet pile wall is used. Its application is to excavation depths less than approximately 15m. For building pits, the sheet pile wall needs to ensure water is kept out and conditions are safe for workers inside the building pit. Outside the building pit deformations and hindrance should be limited. These requirements are formulated in SLS and ULS criteria.

Stability

In the construction of a building pit, the stability of the wall should be checked. The check starts with the inventory of all normative loads, considering the excavation depth and lifetime of the structure, which is usually 50 years. In case of temporary walls, the lifetime is 2 years [27]. The primary stresses exerted against the wall are caused by the soil on the other side of the excavation. This is referred to as active soil pressure. Surcharge loads, indicated by *P*, increase the stress level. The remaining soil on the side of the excavation offers resistance, referred to as passive pressure.

The structural design of the sheet pile wall with its supports – struts or anchors – should be able to withstand the resultant force. Besides ground pressures, hydrological conditions can endanger the stability of the structure. Examples are piping, leakage through the sheet pile wall or burst at the bottom of the building pit [27].

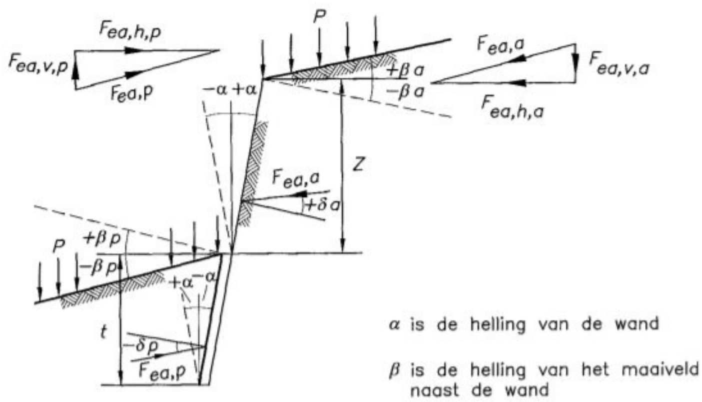


Fig. 3.1. Niveaus en hellingshoeken bij een damwandberekening.

Toelichting termen in figuur 3.1:

F_{ea}	is de door de grond uitgeoefende kracht op de damwand
P	is de belasting op het maaiveld
t	is de inbeddingsdiepte aan de passieve zijde
z	is de kerende hoogte
α	is de hellingshoek van de damwand met de verticaal
β	is de hellingshoek van het maaiveld met de horizontaal
δ	is de wandwrijvingshoek

Indexen

a	is actief
h	is horizontaal
p	is passief
v	is verticaal

Figure 3.1 Illustration of forces sheet pile wall design [24].

According to the EC7, the following main failure mechanisms should be avoided in sheet pile wall design:

- Loss of overall stability
- Exceedance of displacement criteria
- Hydraulic failure – heave due to groundwater pressures.
- Los of stability caused by ductility of steel or exceedance of maximum passive ground resistance.
- Loss of vertical equilibrium: This is in case the vertical anchor load or crane loads cannot be taken by the sheet pile wall.
- Failure of a structural element (anchor, strut), including “Kranz” instability.

The failure mechanisms are summarized by the fault-tree below [27].

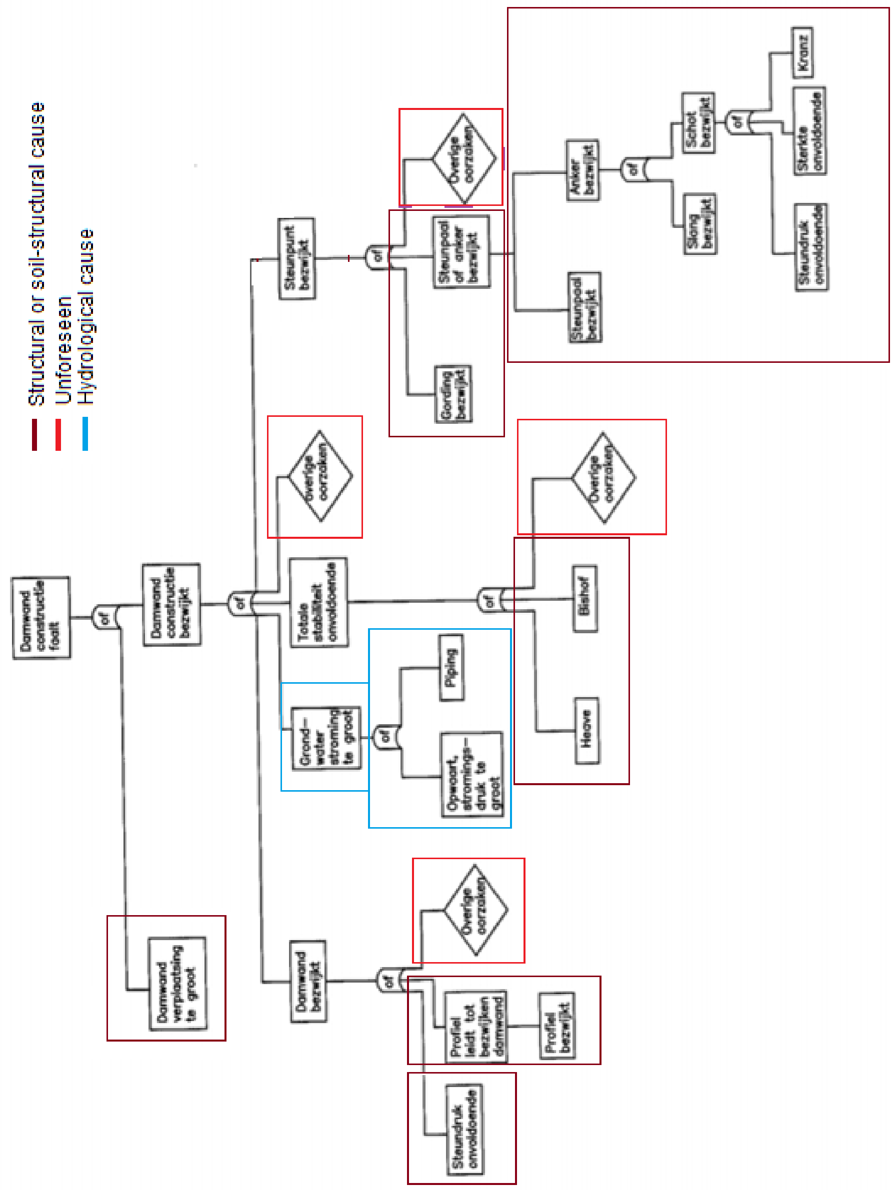
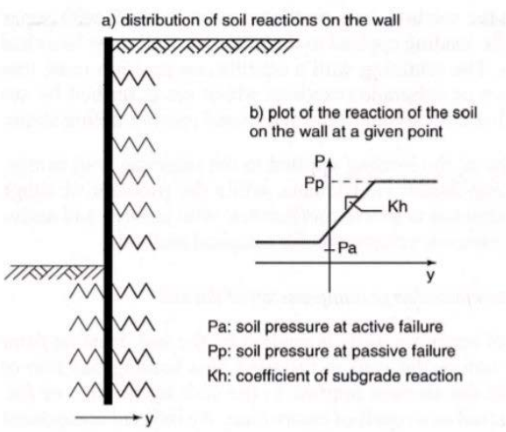


Figure 3.2 Fault tree presented in CUR166 [27].

Calculations

In the method for sheet pile wall design calculations prescribed by the CUR166, the following assumptions are made [27]:

- A sheet pile wall can be modelled as supported by springs (Figure 3.3).
- Long term conditions are normative: In case of excavation, it is assumed that short term soil behavior delivers temporary favorable strength. Therefore, the situation after consolidation - fully drained - is considered as being normative for stability.
- Steel sheet pile walls are labelled as rough.



Active and passive ground force is primarily a function of the friction angle φ, wall friction angle δ and cohesion c. For the determination of ground pressures, a sliding surface of the sheet pile wall should be considered. This can either be assumed as straight or curved, depending on the selected calculation method. This choice has its influence on the angle δ.

Figure 3.3 Spring model [28].

In this thesis it is chosen to work with the 1D DsheetPiling software from Deltares [28]. Among other features, the software checks for the optimal installation depth, global stability, Kranz stability and safety in relation with the Eurocode. The software contains 3 different approaches to assess slip planes [28]:

1. Curved slip surfaces — Kötter — In case of high soil friction angles, the method of curved slip surfaces is recommended to avoid over-estimation of the resistance.
   The following assumptions are made in the model:
   - Horizontal surface layer
   - Only uniform loads can be applied
   - Homogeneous soil with volumetric weight zero
   - Slip planes exist of logarithmic spiral followed by a straight part.

   In this method the wall friction angle is assessed as:

$$\delta \leq \varphi - 2.5° \;\&\; \varphi \leq 27.5° \qquad (9)$$

2.	Straight slip surfaces	Culmann	The application of straight slip surfaces in case of non-horizontal ground levels and/or discontinuous surcharge loads.

Along a slip surface equilibrium calculations are performed. Multiple slip surfaces are considered in DsheetPiling. The slip surface with the maximum active pressure and minimum passive pressure is selected as a normative situation for stability.

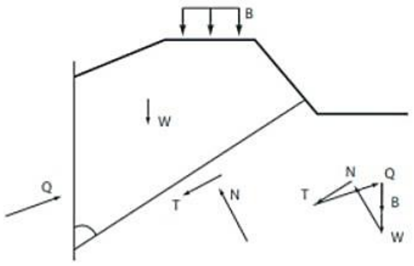


Figure 3.4 Example of straight slip surface calculation according to Culmann method [28].

In this method the wall friction is assessed as: $\delta = 0.667\ \varphi$ (10)

3.	Straight slip surfaces	Müller - Breslau	Friction angles lower than 30º (not to be applied for sands). This model is not considered in this thesis.

Displacements

Besides stability calculations, there are also limitations to displacements. A FE-model (like PLAXIS 2D) is especially useful as it can estimate deformations outside of the building pit [29]. Also, some empirical relationships are derived to estimate surface settlements (Figure 3.5). Displacements often remain a concern in the execution of building pits. Therefore, adjacent buildings and surface settlement are usually strictly monitoring to check if SLS criteria are met. The sheet pile wall displacements are valuable to monitor as they can give direct indication of structural response. Once installed in the field, those can be monitored as commonly done with the use of inclinometers [30]. For the inclinometers a casing is installed along the length of the wall. To take a measurement, a probe is lowered that exist of a cable and two wheels. Wheel A- is perpendicular to the wall from which tilt can be measured (Figure 3.6).

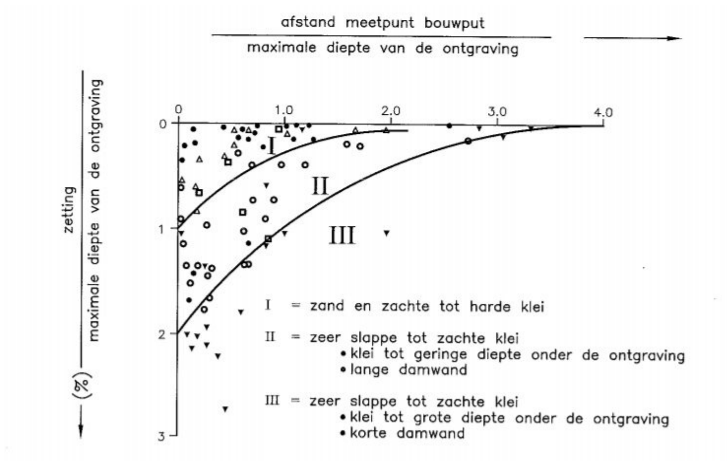


Figure 3.5 Surface settlement at distance from sheet pile wall as a function of excavation depth [27].

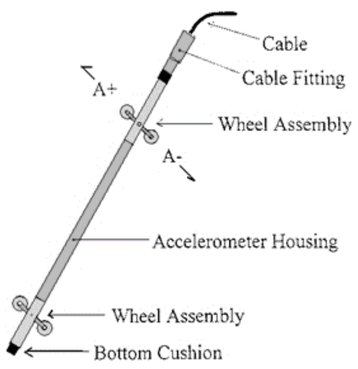


Figure 3.6 Inclinometer [30]

3.2 Delimitations

In this report DsheetPiling software is chosen for design calculations as it is commonly used by Dutch Engineering companies. By adopting this software, the assumptions of the CUR166 (page 28) are valid. Most importantly, this means that *drained calculations* will be performed to assess stability and displacements. This means that uncertainties associated with short-term soil behavior are not considered.

As presented in chapter 2.5, parametric and geological uncertainties are mentioned by the CIRIA guideline as being suitable for the Ab Initio approach. By adopting *most probable* soil conditions, the sheet pile wall's response to forces (Figure 3.1) will change according to the Soil-Structure interaction as predicted by DsheetPiling. This has on itself effect on:

- Sheet pile wall deformations and displacements outside the building pit.
- The optimal installation depth
- The estimation of strut/anchor forces.
- The outcome of stability and safety checks

Monitoring data is gathered to verify the performance of a sheet pile wall. This data can be collected by inclinometers. Although it is important to check other displacements as well, this data could give a direct indication of the true parametric uncertainty. Therefore, the focus in his thesis is on failure mechanisms with purely ground-structural cause (Figure 3.2). The possibility of unforeseen events is considered as well. The failure mechanisms with mainly structural and hydrological causes are disregarded in this research.
Furthermore, it is assumed that construction management was sufficient. Table 3.1 summarizes the delimitations.

Uncertainties in Sheet Pile wall design

Not considered	**Considered**
Parametric + model uncertainty regarding aspects: - Hydrological - Structural	Parametric uncertainty in soil strength parameters Model uncertainty - limitations of DsheetPiling to assess true SSI.
Unthorough construction management - Project execution - Handling material	Unforeseeable events: - Unexpected Geology - Unexpected failure mechanisms

Table 3.1 Overview of sheet pile wall design uncertainties followed in this thesis.

4. Illustration of the CIRIA guideline

In this chapter the 5 steps prescribed in the CIRIA guideline C760 (section 2.3) will be demonstrated by means of a benchmark.

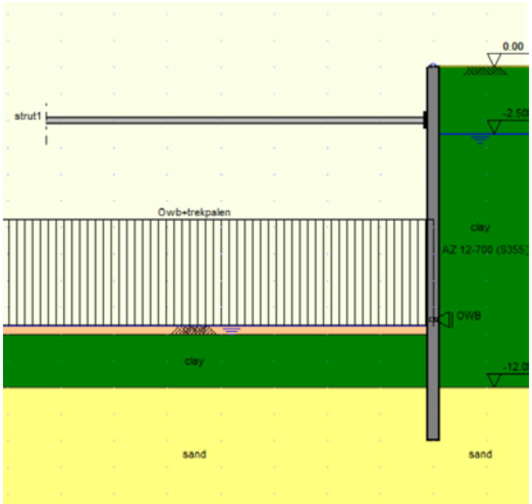


Figure 4.1 benchmark 1 – Design of sheet pile wall

The benchmark concerns a staged excavation with a total depth of -10m NAP. It is chosen to perform wet excavation as this works favorably for the horizontal equilibrium. As the excavation is completed, under water concrete is poured on a thin layer of gravel. Tension piles are used to ensure vertical equilibrium. The sheet pile wall is enhanced by a strut layer on -1.5m NAP.
The considered consequence class is CC1 and the SLS criterion regarding the maximum deflection along the sheet pile wall is 10 cm. There are no further displacement restrictions.

The following construction phases are considered:
Stage 0: Initial, undisturbed situation.
Stage 1: Installation of sheet pile wall to required retaining depth.
Stage 2: Excavation till ground level -2m.
Stage 3: Installation of s strut at level -1.5m.
Stage 4: Wet excavation till -5.5m.
Stage 5: Wet excavation till -8.5m.
Stage 6: Finalizing wet excavation till -10m.
Stage 7: Addition of 0.3m thick gravel layer, followed by pouring 0.5m thick underwater concrete.
Stage 8: Hardening of underwater concrete and dry pumping of excavated pit.

The demonstration starts with the selection of two parameter sets (step 1). As most probable parameters the mean will be used (Figure 2.4 of section 2.3). This is done because in this thesis the interest is in the maximum potential of the OM. After the parameter sets are established, calculations can be performed to establish a preliminary sheet pile wall that is meeting SLS and ULS requirements (step 2). The design of contingency measures is also part of this exercise. In step 3 it is demonstrated how trigger limits can be used to verify the performance of the preliminary design. Because this is a fictive case, no measurements are available. This means that steps 3 to 5 of the CIRIA guideline will only be discussed theoretically.

N.B. The previously stated delimitations of section 3.2 are applicable. Supporting documents are presented in Appendix II on CUR verification steps, partial factors and calculation results.

4.1 Step 1: Parameter selection

The first step of the execution of an OM design is to define 2 sets of parameters:

- *Most probable*, for which in this report the <u>mean values</u> are assumed.
- *Worst probable*, typically 5% characteristic values.

For the worst probable parameter set, the data from the standard NEN-9997-1 [31] are followed as stated in Table 4.1. This Table contains the 5% characteristic values of different parameters per soil type, indicated as $X_{k,5\%}$. These values can be directly adopted for the worst probable set. To determine the most probable parameters, the coefficient of variation V is used. They are also stated Table 4.1. This coefficient represents the spread of a parameter via:

$$\frac{\sigma}{\mu} = V \tag{11}$$

Substituting this expression in equation (6) (section 2.2), it holds that:

$$\frac{X_{k,5\%}}{\mu} = 1 - 1.645V \tag{12}$$

Next, the mean value can be calculated as:

$$\mu = \frac{X_{k,5\%}}{1 - 1.645V} \tag{13}$$

The coefficient of variation thus plays a big role in the determination of the most probable parameter set. The coefficients as stated in Table 4.1 are established from the collection of many laboratory tests. For each test the mean and standard deviation of a certain soil parameter were found. From the collection of tests, a student t-distribution is derived that described the parametric variability. Therefore, the values stated in Table 4.1 should be representative for what is commonly encountered in Dutch soils. The use of this student-t distribution is conservative and therefore, the coefficient of variation is a conservative value. Moreover, many researches share the conclusion that the coefficient of variation as stated in Table 3 fluctuates within close borders [13].
For this Benchmark, based on equation (13) the mean values of the friction angle φ and cohesion c become as presented in Table 4.2. The wall friction angle δ is derived for the Kötter model (section 3.1, equation (9), page 24).

Characteristic values for the modulus of subgrade reaction k_{char} that are prescribed by the CUR166 can be found in Table 3.3 of Appendix II. The CUR166 mentions the following relationships between the mean modulus of subgrade reaction $\bar{k}$ and the characteristic value:

$$k_{d,low} = \frac{k_{char.}}{1.3} = \frac{\bar{k}}{2} \tag{14}$$

Which leads to:

$$1.5\, k_{char.} \approx \bar{k} \tag{15}$$

The consequential mean and characteristic values for the modulus of subgrade reaction are included in Table 4.2 as well.

Grondsoort			Karakteristieke waarde van de grondeigenschappen van het laaggemiddelde						
Hoofd-naam	Bijmengsel	Consis-tentie 1)	γ 2) kN/m³	γ_{sat} kN/m³	q_c 3) 6) MPa	E_{100} 6) 7) MPa	φ' °	c' kPa	c_u kPa
grind	zwak siltig	los	17	19	15	45	32,5	-	-
		matig	18	20	25	75	35,0	-	-
		vast	19 20	21 22	30	90 105	37,5 40,0	-	-
	sterk siltig	los	18	20	10	30	30,0	-	-
		matig	19	21	15	45	32,5	-	-
		vast	20 21	22 22,5	25	75 110	35,0 40,0	-	-
zand	Schoon	los	17	19	5	15	30,0	-	-
		matig	18	20	15	45	32,5	-	-
		vast	19 20	21 22	25	75 110	35,0 40,0	-	-
	zwak siltig kleiig		18 19	20 21	12	35 50	27,0 32,5	-	-
	sterk siltig kleiig		18 19	20 21	8	15 30	25,0 30,0		
leem 4)	zwak zandig	slap	19	19	1	2	27,5 30,0	0	50
		matig	20	20	2	3	27,5 32,5	1	100
		vast	21 23	21 22	3	5 7	27,5 35,0	2,5 3,8	200 300
	sterk zandig	-	19 20	19 20	2	3 5	27,5 35,0	0 1	50 100
klei	schoon	slap	14	14	0,5	1	17,5	0	25
		matig	17	17	1,0	2	17,5	5	50
		vast	19 20	19 20	2,0	4 10	17.5 25,0	13 15	100 200
	zwak zandig	slap	15	15	0,7	1,5	22,5	0	40
		matig	18	18	1,5	3	22,5	5	80
		vast	20 21	20 21	2,5	5 10	22.5 27,5	13 15	120 170
	sterk zandig	-	18 20	18 20	1,0	2 5	27,5 32,5	0 1	0 10
	organisch	slap	13	13	0,2	0,5	15,0	0 1	10
		matig	15 16	15 16	0,5	1,0 2,0	15,0	0 1	25 30
veen	niet voorbelast	slap	10 12	10 12	0,1	0,2 0,5	15,0	1 2,5	10 20
	matig voorbelast	matig	12 13	12 13	0,2	0,5 1,0	15,0	2,5 5,0	20 30
variatiecoëfficiënt			0,05		-	0,25	0,10	0,20	

Table 4.1 Characteristic values per soil type according to NEN-9997-1 [31].

	Sand		Clay	
	Most probable 50%	Worst probable 5%	Most probable 50%	Worst probable 5%
γsat [kN/m³]	20	20	17	17
γunsat [kN/m³]	18	18	17	17
φ [°]	38.9	32.5	21.5	18
δ [°] Kötter	27.5*	27.50	11.9	9.5
c [kPa]	-	-	7.5	5
Modulus of subgrade reaction				
50%	3.00E+04	2.00E+04	6.00E+03	4.00E+03
80%	1.50E+04	1.00E+04	3.00E+03	2.00E+03
100%	7.50E+03	5.00E+03	1.20E+03	8.00E+02

Table 4.2 Input variables for OM and Characteristic design scenario.
** Based on the Kötter model the δ cannot be bigger than 27.50°* [28].

4.2 Step 2: Calculations

Eurocode 7 states a demand for the OM to assess “ranges of possible behavior” [10]. The CIRIA guideline meets this requirement by the calculation of two scenarios. An overview of the required calculations is stated in scheme 4.1. The CUR166 design procedure is followed as recommended by the Dutch National Annex of EN1997 [31]. The calculations result in 4 outcomes: 2 structural designs and 2 corresponding set of displacements.

<u>Structural design:</u>
1. *The characteristic design* - A sheet pile wall design for “worst probable conditions”.
2. *The OM design* - A sheet pile wall design for “most probable conditions”.
Both design need to fulfill ULS and SLS requirements.

Displacement results:
For each construction phase, the expected displacements of each design should be calculated with partial factor of unity, leading to a collection of:
3. *Characteristic movements*
4. *Most probable movements*

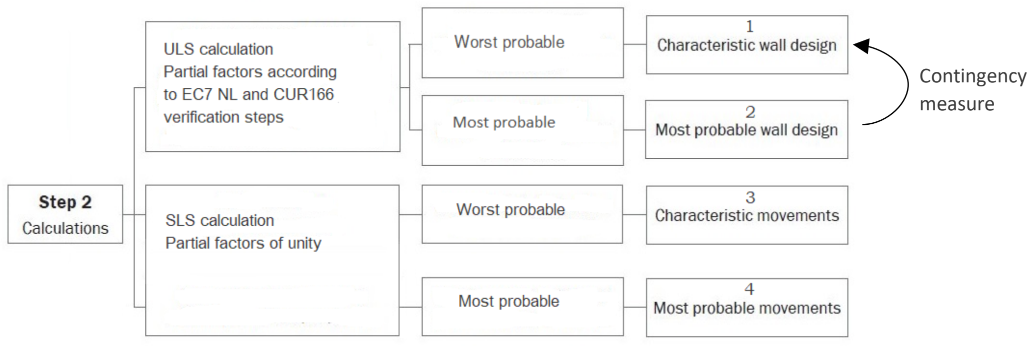


Figure 4.2 Calculations step 2.

Structural design
The structural designs of (1) and (2) should not be independent from each other as during construction it should be able to switch from one scenario to another. Therefore, they should be related to each other by a contingency measure (Figure 4.2). However, some components of the structure cannot be changed anymore once installed. Because of this it is necessary to adopt some structural features of one design into the other:

- One type of sheet pile wall.
- The same installation depth.
- First construction phases should be the same for comparison: In this benchmark this implies that the same strut should be installed in stage 3.

Because the OM design is the starting point, the mean soil conditions are leading. This means that a thinner wall profile will be selected as the assumed strength parameters are higher. In characteristic soil conditions this sheet pile wall is rather weak and could become unstable. This instability is seen from the first calculation results of calculation step 1, presented below.

N.B. In the following presented calculations the sheet pile wall profile AZ 13-700 has been selected as a result of different try-outs. At first, the AZ 17-700 was selected, which turned out to be uneconomical, because it was rather robust in mean soil conditions. The AZ 12-700 was sufficient but could not be sufficiently enhanced for the case of characteristic soil parameters.

Calculation step 1: Differences in structural stability
At first a trial-calculation is presented to illustrate the actual differences between the two sets of soil conditions. In both calculations the same struts are applied. The results are presented in Table 4.3.

In case of the OM the selected wall is stable and fulfilling the SLS requirement. For the calculation based on characteristic parameters an unstable situation occurred as excavation continued to -8.5m (phase 5). The loss of stability may be noted from the obtained differences in the calculated results: It can be seen that both the strut force and the maximum moment are significantly bigger than those of the OM. Hence, the unity check for the characteristic wall design is < 1, indicating the potential ULS failure. Also, there is a clear difference in calculated maximum displacements. Moreover, the installation depth of 13.5m was allowed for the OM scenario, whilst 14m is required for the

characteristic.
Based on this results it would be necessary to enhance the AZ13-700 in case characteristic soil conditions are applicable.

Sheet pile: AZ 13-700 S355 Struts: 40cm diameter

	Most probable wall design	Characteristic wall design
Sheet piling length [m]	13.5	14
Strut force [phase 5] [kN/m]	152.2	238
Moment [phase 5] [kNm]	238.9	493.6
Max displacement [phase 5] [mm]	26.3	82.4
Unity check		
Sheet pile wall (Elastic moment)	1.10	0.92

Table 4.3 Results first trial calculations benchmark 1, sheet pile wall profile AZ 13-700.

Calculation step 2: OM design + Contingency measure
As seen in Table 4.1, the installation depth of 14m was required to ensure stability in the characteristic design. For the OM, an installation depth of 13.5m would have been sufficient. However, a length of 14m is also in the OM design more favorable for the distribution of forces. Given this installation depth, stability of the AZ 13-700 under characteristic soil conditions can be guaranteed by a second layer of struts. This second layer of struts is thus the contingency measure, that should prevent potential failure in phase 5. Therefore, the extra struts layer should be installed at the end of phase 4, after the excavation depth of -5.5m has been reached (Figure 4.3).

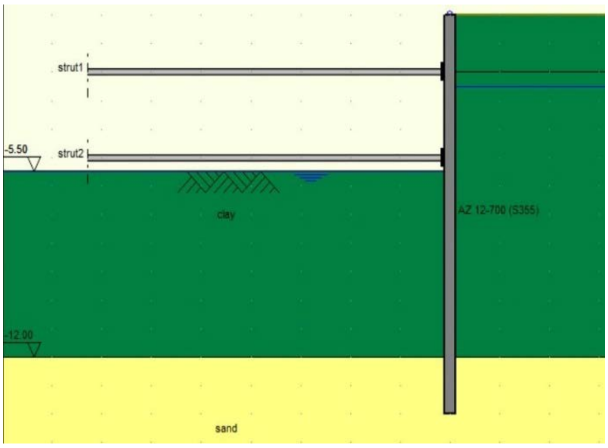


Figure 4.3 Installation of the second strut to enhance stability in conventional design.

As wet excavation is performed, the water should first be removed for the installation of this second layer of struts. Therefore, the construction sequence is changed. After the installation, the water can be pumped back into the building pit to continue the excavation process. This results in a new construction phasing:

Stage 0: Initial, undisturbed situation.
Stage 1: Excavation till ground level -2m.
Stage 2: Installation of strut at level -1.5m.
Stage 3: Wet excavation till -5.5m.

Stage 4: Dry pumping till current excavation depth.
Stage 5: Characteristic design: Installation of second strut at -5m.
In OM design: This stage is skipped.
Stage 6: Reset water level till -2m. Wet excavation till -8.5m.
Stage 7: Finalizing wet excavation till -10m.
Stage 8: Addition of 0.3m thick gravel layer, followed by pouring 0.5m thick underwater concrete.
Stage 9: Hardening of underwater concrete and dry pumping of excavated pit.

Following this new construction sequence allows to from most probable to characteristic wall design. Finally, the results of both design calculations are stated in Table 4.4. Because of the second strut, the unity check > 1.

AZ13-700 S355	Most probable	Characteristic
Sheet piling length [m]	14	14
Strut 1 max. force [kN/m]	211.0	216.8
Strut 2 max. force [kN/m]	-	126.4
Max moment [kNm]	409.5	440.9
Max displacement [mm]	61.6	81.8
Unity check		
Sheet pile wall (Elastic moment)	1.10	1.05

Table 4.4 Results of calculations preliminary design and .

Displacement results
To conclude step 2, the characteristic and most probable movements (3) and (4) are presented by Figure 4.4. As can be seen from these results, a big difference in displacements could be observed during the dry pumping – phase 4. Thus, during this activity, monitoring data could show the need for installing the second strut. The decision-making process for the contingency measure will be further elaborated in step 3 to 5.

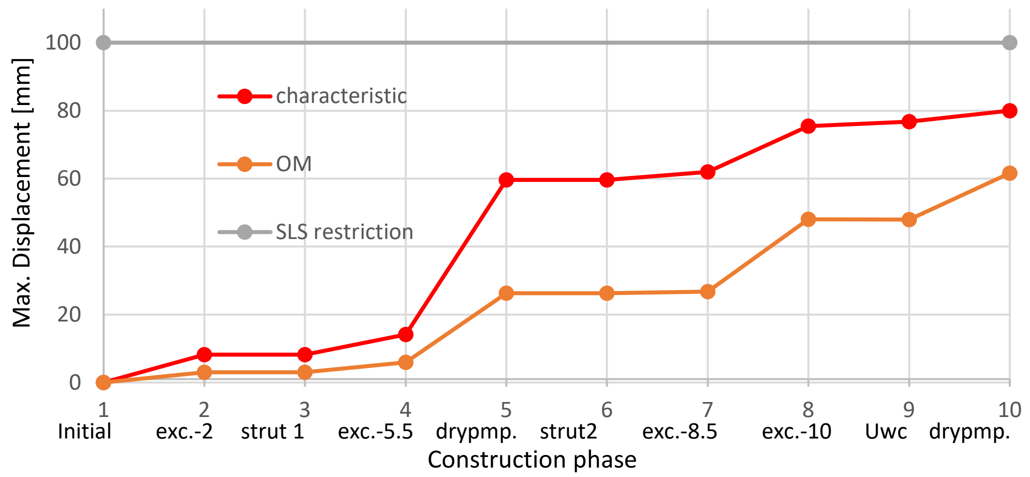


Figure 4.4 Chart for benchmark 1 containing displacement predictions for both OM and characteristic parameter sets.

4.3 Step 3: Trigger limits and contingency actions

Once actual construction takes please, measurements are gathered on sheet pile wall displacements. The CIRIA guideline distinguishes 3 categories for those measurements: Red, orange and green. This distinction should guide designers in how to use the new data to decide for a contingency action.

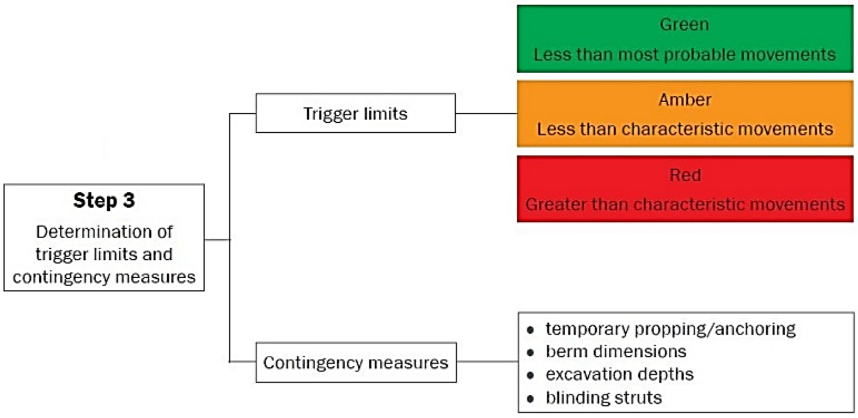


Figure 4.5 Outline of Step 3 according to CIRIA guideline: Traffic light system.

According to this traffic light system, the calculated SLS displacements of both the OM design and characteristic design serve as trigger limits. Those trigger limits warn that the measurements give rise to other (soil) conditions than first assumed: SLS and possibly ULS requirements might not be met if continued. For the Benchmark, the 3 different measurement zones become as depicted in Figure 4.6:

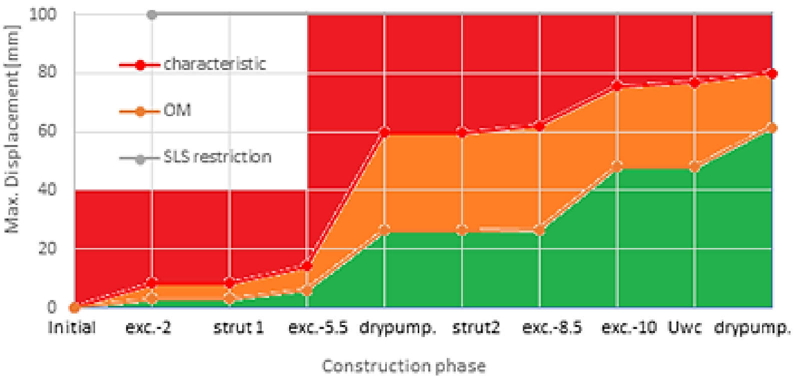


Figure 4.6 Traffic light system applied to Figure 4.3.

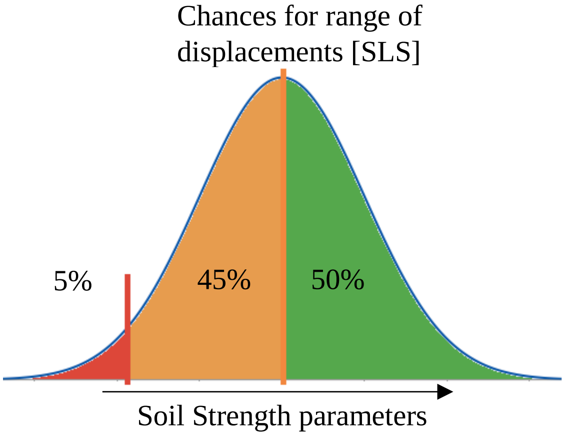


Figure 4.7 Chances of ending up in red, green and orange zone according to soil strength

The zones have the following characteristics:

(1) The green zone: If displacements occur that are less than predicted for the most probable soil conditions, the preliminary design is structurally stable and SLS will be easily met.
(2) The orange zone: For displacements more than predicted, it is feared that, if construction is continued, the preliminary design becomes unstable and SLS requirements will not be met for consequential construction stages. The assumption of having most probable soil conditions is probably not valid and the sheet pile wall should be enhanced.
(3) The red zone: The design based on the characteristic parameter set will need to be enhanced as it is feared that ULS and SLS requirements will not be met as actions turned out to be higher than foreseen. The observational method "best way out" approach should be applied.

As SLS calculations are directly based on soil parameters, the chances of ending up in the red zone should theoretically be less than 5%.The chance to encounter displacements in the green zone should be 50% and thus, for the orange zone, 45%. This distribution of chances is illustrated by Figure 4.7.

Applying a contingency measure normally negatively effects the construction sequence. However, in an OM design the engineers anticipate to this possibility, reducing these effects. In the benchmark it is expected that this situation might occur during the dry pumping (phase 4). This phase is thus the first important phase to check and decide on the performance of the preliminary design. To illustrate the use of this Traffic light system, 3 scenarios of measurement sets are considered as presented in Figure 4.8.

Scenario 1: Measurements fall within the green zone such that the preliminary design seems to be sufficient throughout the full construction. Phase 5, the installation of the second layer of struts can be skipped.
Scenario 2: During the dry pumping (phase 4), the measurements indicate the need for a contingency action. Therefore, the second row of struts needs to be installed in phase 5, before further excavation continues.
Scenario 3: At first, it seemed like the conclusion of scenario 1 could be drawn. However, as excavation continued in phase 6 and 7 sheet pile wall displacements increased more than expected. Further construction works should be stopped to install the second layer of struts. This action requires care as wet excavation is performed and thus first dry pumping should be applied again. This could lead to even more displacements. Therefore, new calculations would be necessary to decide on the implementation of the contingency measure.

For scenarios 1 and 3 a high measurement interval could timely detect the need for a contingency measure. Scenario 3 could be seen as an unforeseen event. In general, it needs to be realized that unforeseen events can arise throughout every construction phase. It is recommended to have struts ready for installation, even if a first crucial phase (phase 5) has been completed with favorable behavior of the preliminary design. In this Benchmark, the choice to perform wet excavation is actually not preferable, as it does not allow an easy and quick installation of struts.

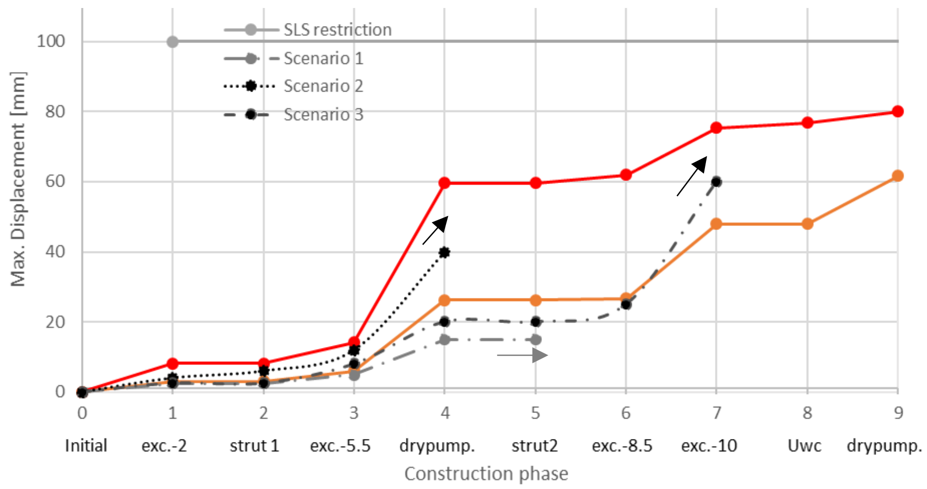


Figure 4.8 Different measurement scenarios.

4.4 Step 4: Monitoring

A monitoring system should be made that enables Engineers to proactively verify the performance of the preliminary design. Many different indicators are available to deliver this information. In this thesis it is chosen to only look at sheet pile wall deformations (section 3.2). However, as seen in the ULS results of step 2, failure is not only detectible by displacements, but (even better) by strut forces and the quantification of bending moments. Therefore, it is recommended to consult monitoring experts to establish an extensive monitoring plan. By making this plan, it should also be considered that stakeholders often have interest in certain data [1]. An example would be the settlement of a neighboring structure. Another consideration is that certain monitoring data could be useful for research.

The CIRIA guideline recommends distinguishing primary monitoring systems – that deliver the data of most interest – and secondary systems that would serve as a back up to ensure redundancy. The type of monitoring system depends on the following:

- Required interval of measurements: This depends on the planned speed of the construction works and the expected rate of change in loads.
- Desired accuracy and precision of measurements.
- Total time that system should be able to operate.
- Operation: Automatic or manual data processing.

Based on the requirements devices and positions could be selected in consultation with monitoring specialists and specified in Table 4.5.

Monitoring device	Specified for	Measurement interval
Inclinometer	Retaining wall – displacements alongside wall	Daily, more frequently depending on construction stage.
Piezometers	Water levels adjacent to building pit	Dependent per position on expected influence and/or stakeholder demands
Strain gauge	Struts	Depending on construction stage
Total station	Adjacent buildings	Dependent per position on expected influence and/or stakeholder demands
…	…	…

Table 4.5 Example of monitoring plan.

4.5 Step 5: Construction phase

With this step the CIRIA guideline discusses the integral role of the designers as construction actually takes place. An example of such an approach is illustrated by the flow chart in Figure 4.9. Such flowcharts help to systemize control and communication.
The CIRIA guideline recommends the evaluation of monitoring data at each construction stage. From each evaluation, the decision for an action should be made. This decision is based on the Traffic light system. The guideline emphasizes the importance of a proactive attitude of the whole project team: Reporting to each other and reviewing with each other. As already stated in section 2.7 communication can be better systemized if monitoring, design and construction is done by one party.

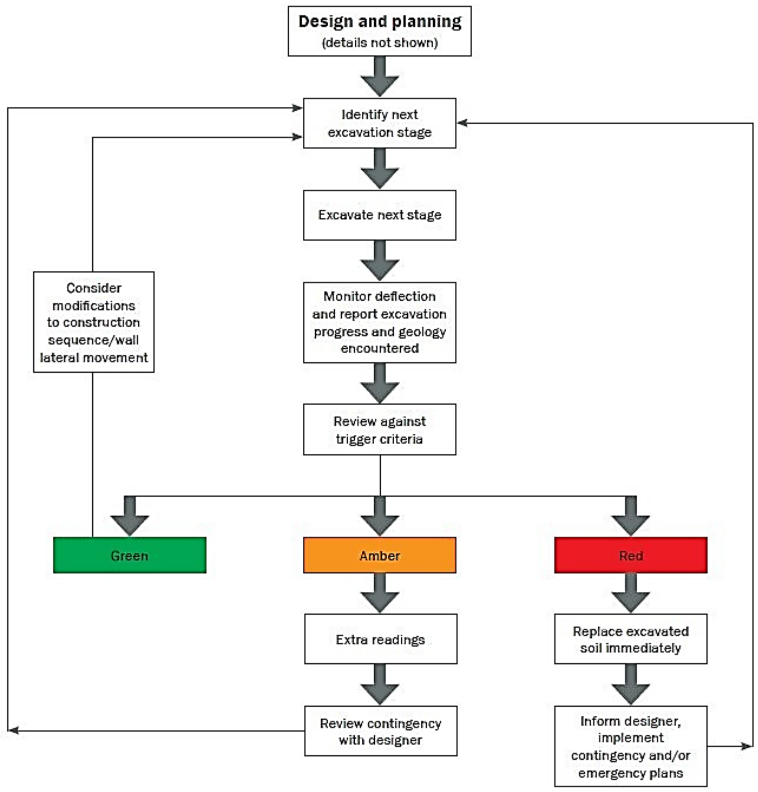


Figure 4.9 Flow chart for step 5 CIRIA guideline.

4.6 A critical note on the Traffic light system

According to the guideline, a decision can be made each phase based on measurement processing standardized by the Traffic light system. However, a few side-nodes are applicable if this way of measurement processing is adopted:

1) Dependency on the model accuracy
The red, orange and green marked areas in the SLS charts (Figure 4.6) are indicated by Trigger limits. Those Trigger limits follow from the most probable and characteristic movements established in step 2. The predicted movements are influenced by the design and model uncertainties as presented in section 2.1. The accuracy of DsheetPiling is questionable, while accuracy in the Trigger limits is important to correctly establish the green, orange and red zones. Therefore, the correct use of the SLS design chart depends on the accuracy of the computer model to predict movements. In practice, inaccuracy of the Trigger Limits would lead to more "unexpected" scenario's, like scenario 3 presented in section 4.3.

2) Time margin
The Trigger limit is a limiting value for the max. displacement of the sheet pile wall that should ensure there is enough time to apply a contingency measure before actual failure happens. This requires a certain time margin, T_{margin} that consists of the following components:

$$T_{margin} = T_{detection} + T_{decision} + T_{act} + T_{effect} \qquad (16)$$

In which $T_{detection}$ and $T_{decision}$ represent the possible time in between taking a next measurement, interpretation and actually plan on the implementation of a contingency measure. T_{act} and T_{effect} are associated with the installation of the contingency measure. For anchors and struts there is a time required for installation, but there is also some time needed before actually an effect can be noticed.

The above time components are hard to estimate, let alone the margin in terms of time should be expressed in a representative value in terms of displacements. Based on the CIRIA Guideline, it is not clear what to do once the first measurement in the orange zone has been obtained. According to step 5 extra readings are allowed, however, the guideline leaves some space for engineering judgement. In the decision making it is recommended to consider the following 2 features that are not explicitly mentioned in the guideline:

1. The rate of change in between measurements: If displacements strongly increase in between consequential measurements, the time margin might get endangered. It is then recommended to act fast and responsibly, as it cannot be waited for extra readings or extensive back-analysis.
2. Magnitude of the measurement: This comes back in the determination of the margin between the suspected ULS displacement. Once a measurement falls within the orange – or even the red zone, but its magnitude is largely below the SLS, extra readings could be taken if 1. is not applicable.

Feature 1 emphasizes the need to establish a sufficient measurement interval, as also seen in scenario 3 of section 4.3. The statement of step 5 of the CIRIA guideline to evaluate each construction phase is contradictive. As each construction phase differs and unforeseen events might occur at any time, this statement might be a bit too generalized. Instead, the value of automatic, real time measurement-processing should perhaps get more attention.

SLS restriction

In the Traffic light system, the SLS restriction plays an important role in ensuring safety, as movements in any scenario should be kept below this restriction. Once the structure is in a critical state of failure, typically the movements that go along with this state should be well above the SLS restriction. This is however not explicitly mentioned in the CIRIA.

For illustration, the SLS restriction of 10cm is checked for the benchmark in Figure 4.10. Failure happens in phase 7. From the figure it can be seen that in phase 5 and 6 the predicted ULS displacements, according to CUR verification step 4, fall well above the SLS restriction.

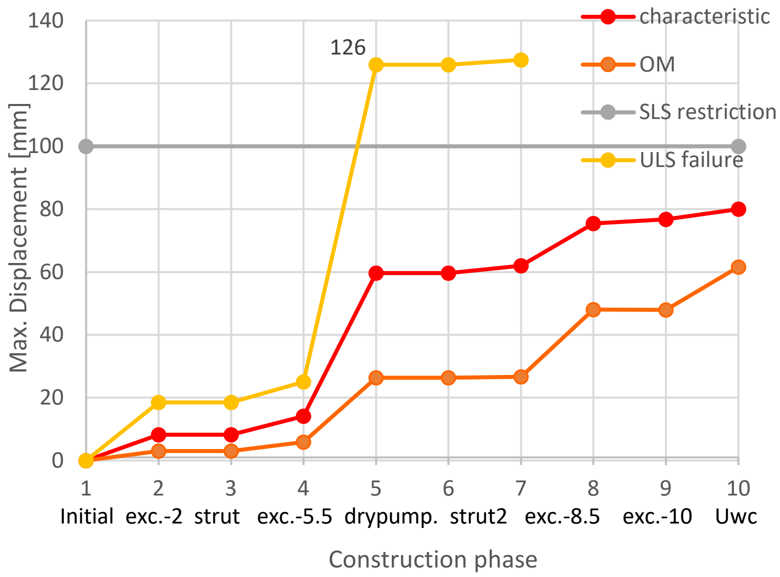


Figure 4.10 The SLS restriction evaluated by predicted displacements of ULS failure.

4.7 Conclusions

The aim of this chapter was to:

(1) Quantify differences between retaining walls designed based on most probable or characteristic parameter sets.

(2) Illustrate the OM design procedure according to the 5 steps of the CIRIA guideline on retaining walls.

The following conclusions are made regarding these aims.

Aim (1): By executing the first two steps, 2 design scenarios were established. The obtained differences in terms of acting moments and forces are represented in Table 4.3. These differences led to 2 structural designs that are related to each other by a contingency measure. In this case, a second layer of struts can be installed to enhance the OM design if necessary.
Based on the 2 structural designs, this benchmark shows that there is potential to save costs, with a change of 50% to end up in the green zone.

The actual feasibility of the Observational method depends on the difference in construction costs between Limit state design and the OM design. Feasibility could then be quantified by the potential of making profit. The total costs for the OM design also depend on the costs of the extensive monitoring and contingency action. These different costs aspects are summarized by the decision tree in Figure 4.11 [25].

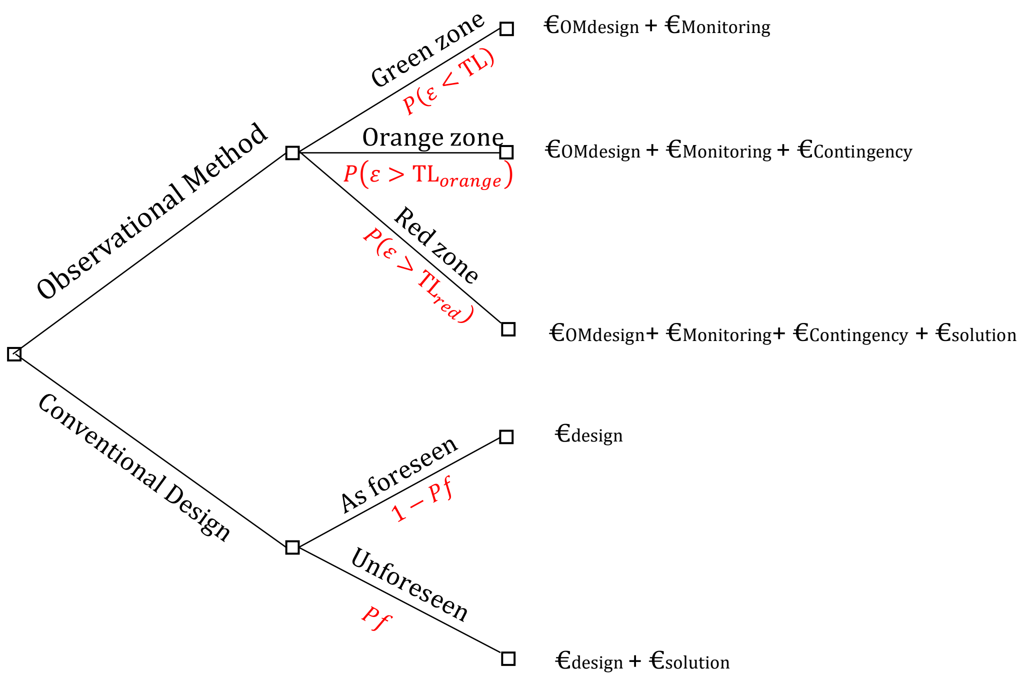


Figure 4.11 Decision tree based on cost-perspective.

From the decision-tree it can be derived that the Observational Method is feasible once the costs for the conventional design outweigh the costs for the OM design, monitoring process and the most expensive contingency measure [25]:

$$€\text{design} \geq €\text{OMdesign} + €\text{Monitoring} + €\text{Contingency} \qquad (17)$$

For the conventional design costs are linked with the Probability of failure Pf. Based on Limit state design, this probability should always be lower than 1:1000. As mentioned in section 2.2, the true Pf often remains unknown [15]. A customized Pf can be estimated by Level II or Level III approaches. For the Observational method, construction costs are linked with the chance displacements exceed a Trigger limit in a certain phase. The Trigger Limits follow from deterministic results of a forward DsheetPiling model. It is thus important that DsheetPiling has a certain accuracy in the displacement prediction. Regarding these statements, statistical methods could perhaps give more insights on the true feasibility of the Observational Method.

Aim(2): The five steps as prescribed by the CIRIA guideline have been illustrated. For actual implementation there might be lack of detail on some crucial parts:

- It is not well-defined how to act once the first measurement in the orange zone is obtained: Should a contingency measure be applied immediately? Are extra readings allowed? The guideline leaves this open. This might be a pitfall of the guideline as it calls for Engineering judgement, while not specifying certain important features as discussed in section 4.6.

- Besides the traffic light system, it is not further specified how measurement data should be post-processed in order to actually learn from conditions in the field. Once structural displacements come in the orange or red zone it can be questioned whether that is related to weak soil conditions. The influence of model inaccuracy, measurement errors or the potential to encounter unforeseen events is not highlighted in the guideline. However, to some extent the sheet pile wall behavior should be explainable in order to decide whether a certain contingency measure is truly necessary and/or effective.

- In case of an unforeseen event the guideline mentions the back-analysis strategy without the presentation of an efficient way to do this. As safety is the most important aspect, information would be useful on how to prove certain uncertainties can be decreased with the monitoring data.

All in all, the following questions are formulated that summarize the concerns that are left after following the CIRIA guideline procedure:

1) How to post-process displacement data in order to specify structures' safety?

2) How to deal with different types of uncertainties in the OM?

The next chapter presents a methodology for question 1).

5. Statistical methods in OM

In chapter 4 the application of the Ab Initio approach has been presented by means of a Benchmark. In the conclusion it is stated that some additional insights would be desired. In this chapter a methodology is formulated to process measurement data, learn about the true uncertainty in ground conditions, with the goal to support the decision-making process.

5.1 Low safety margins in the OM application

As described in chapter 3, in conventional design methods the safety of the structure is ensured by a margin between loads and resistances, provided by partial factors. These partial factors are pre-set and applied to the 5% characteristic value of property X. The partial factor is described as the relationship between the design and characteristic value via formula (18):

$$\gamma_{5\%} = \frac{X_{k,5\%}}{X_d} = \frac{\mu - 1.645\sigma}{\mu - \alpha\beta\sigma} \tag{18}$$

In the OM the *most probable* parameter (μ) is selected instead of the $X_{k,5\%}$. A partial factor $\gamma_{50\%}$ should then be applied resulting in the same value for the X_d. To avoid the presumed high degree of conservatism, the CIRIA guideline prescribes to apply the $\gamma_{5\%}$ instead of the $\gamma_{50\%}$. This results in a X_{dOM} that is bigger than X_d (Figure 5.1).

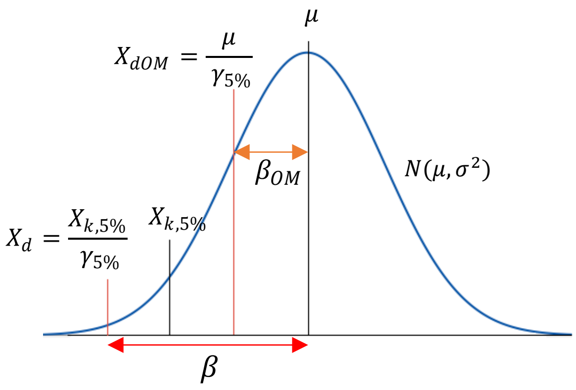


Figure 5.1 Calculation of OM design value and relationship with conventional design value.

By doing this, the correct Level I approach is rejected. This raises concerns about low safety margins in an OM design. As mentioned in the conclusion of previous chapter, the CIRIA guideline does not specify safety levels. This is justified as in the EC7 it is not explicitly mentioned that it is required to calculate exact safety levels. However, the lower reliability indexes are one of the biggest concerns for actual implementation of the method [11].
By the Level I approach the safety factor are applied such that for Consequence Class 1 β = 3.3. As the X_{dOM} is more optimistic than the characteristic design, it is feared that this β is never reached by an OM design. This can be shown by first deriving an expression for X_{dOM}:

$$X_{dOM} = \frac{\mu}{\gamma_{5\%}} = \left(\frac{\frac{\mu}{\mu - k\sigma}}{\mu - \alpha\beta\sigma}\right) \tag{19}$$

$$X_{dOM} = \frac{\mu(\mu - \alpha\beta\sigma)}{\mu - k\sigma} = \frac{\mu^2 - \alpha\beta\sigma\mu}{\mu - k\sigma} \tag{20}$$

Diving the right-hand term by μ leads to:

$$X_{dOM} = \frac{\mu - \alpha\beta\sigma}{1 - kV} \tag{21}$$

Substituting formula (7) $X_d = \mu - \alpha\beta\sigma$ this leads to:

$$X_{dOM} = \frac{X_d}{1 - kV} \tag{22}$$

Rewriting formula (7) the reliability index can be found by:

$$\beta = \frac{X_d - \mu}{-\alpha\sigma} \tag{23}$$

As seen in equation (22) it thus holds that $X_{dOM} \neq X_d$. Therefore, for the same $N(\mu, \sigma^2)$ and α a different reliability index is found for an OM-based design:

$$\beta_{OM} = \frac{X_{dOM} - \mu}{-\alpha\sigma} \tag{24}$$

With typically $\beta_{OM} < \beta$ as illustrated in Figure 5.1 as well. This validates the concern in safety.

5.2 Purpose of measurement-processing

Based on NEN9997-1 [28] the initially assumed distribution of soil parameter X can be described with $N\left(\mu_x, {\sigma_x}^2\right)$. By adopting the variables as presented in Table 4.1 (section 4.1) a rather conservative estimation of the variability is adopted. In the OM, the goal would be to learn more about the in-situ conditions from the gathered monitoring data to narrow down this parametric variability. For the soil strength parameters, measurement processing could lead to a new estimation of the parametric distribution as indicated by μ_x' and σ_x'. According to formula (24) a change in deterministic values μ_x' and σ_x' lead to a different reliability index ${\beta_{OM}}'$ that can be determined per parameter. If either $\mu_x' > \mu_x$ and/or $\sigma_x' < \sigma_x$ a smaller normal distribution can be obtained, meaning that there is less variability of the parameter X. Correspondingly, ${\beta_{OM}}'$ goes up as illustrated in Figure 5.2 in which the standard deviation is reduced and $\mu_x' = \mu_x$. Ideally, the coefficient of variation V of the dominant soil parameters change such that at the end of construction it can be shown that $\beta_{OM} \approx \beta$.

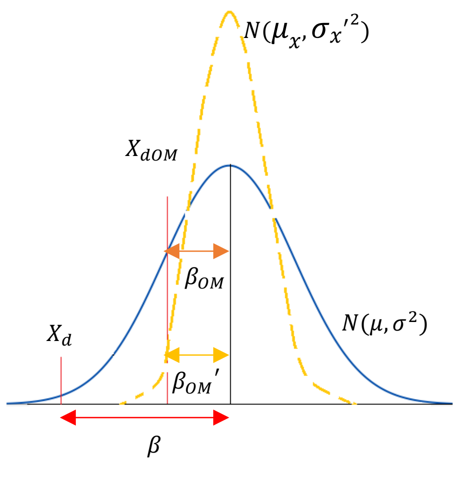


Figure 5.2 Illustration of $\sigma_x' < \sigma_x$ and its effect on the reliability index.

In order to find new estimations of soil parameters, it is needed to apply a *back-analysis* with the gathered monitoring data. *Back-analysis* aims to derive input parameters from a given output. Unfortunately, most Geo-Engineering guidelines and standards do not give a standardized description for this. The reason for this is that back-analysis is associated with time consuming and complicated algorithms, that are highly problem dependent and could return questionable results. As conditions differ from site to site and design strategies differ from project to project, it is hard to standardize a procedure. However, some fast and relatively simple statistical methods could already provide a lot for the Observational Method [8]. One of these methods that will be adopted in this thesis is *Bayesian inference*.

During the construction phase, a decision needs to be made on the performance of the preliminary design based on two available sources of information:

1. *Prior information:* Displacement predictions, based on site- and soil investigations, engineering judgement and computer modelling (DsheetPiling).
2. *Measurement data:* Monitoring data on sheet pile wall displacements collected during construction.

Once a large amount of data is collected, the measurements represent the truth. Unfortunately, in real-life projects it is not possible to gather large datasets. Therefore, it is not convenient to base decisions only on (2). On the other hand, it might be undesired to base decisions only on the prior information (1) as design assumptions might be wrong and modelling errors are always present. Therefore, a combination of the two might be more reliable.
As an alternative to the Traffic light system, next section describes a methodology that aims to perform Bayesian inference in real-time measurement processing. The main interest is in limiting the displacements that occur during the different construction phases. In this thesis, the focus is on the maximum deflection anywhere along the sheet pile wall. By combining the two sources of information, a new prediction could be made for the deflections that would occur in a consequential construction phase. This prediction carries an uncertainty, that can be expressed in terms of a probability density function. Via a calibration procedure, presented in the next section, the corresponding distributions in soil parameters can be found, from which safety definitions can be derived as previously presented.

5.3 Bayesian update

The methodology consists of 5 steps. Step 1 and step 2 of provide the *prior information* for the Bayesian update. Consequently, a Bayesian update (step 3) returns a new displacement prediction that consists of both the prior information and the measurements. From this "updated" prediction new sets of input parameters can be estimated (step 4). Based on the level of safety and the updated predictions it could be decided if contingency measures are necessary. This can be done, for instance by applying a Hypothesis test (step 5).

1.Sensitivity analysis

A sensitivity analysis aims to assess what parameters have the most influence on a model result. In a sensitivity analysis each input parameter is varied on its own by taking its 5% and 95% tail values. As these values are filled in, the model output is recorded according to Figure 5.3. That way, the influence of the parameter on itself can be presented as an absolute change in output results. In case of the OM design it is of interest to see what influence each parameter variation would have on the output. For strength parameters, low values (5% tail value) would typically lead to more displacements. The influence of each parameter depends on the stratigraphy and differs for each construction stage.

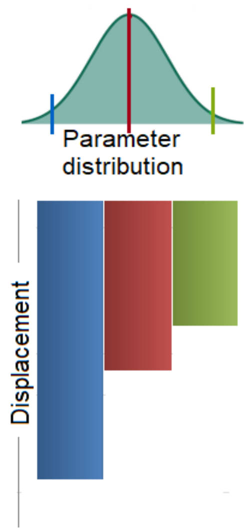


Figure 5.3 Sensitivity analysis

2.Monte Carlo simulation

To produce a PDF of the displacement predictions made with DsheetPiling, a Monte Carlo simulation is performed. This is a simulation that uses the full range of input distributions from different parameters X. That way, a complete range of possible sheet pile wall displacements is generated (Figure 5.4).

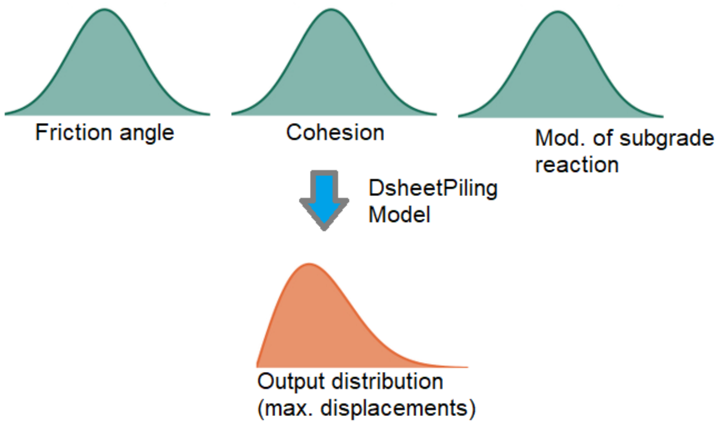


Figure 5.4 Monte Carlo simulation principle.

The input is typically normally distributed, adopting the assumptions of NEN9997-1 [31]. As will be seen in the next section, the output distribution of sheet pile wall displacements is typically Lognormal. The outcome of a Monte Carlo simulation is thus, theoretically, based on the full range of possible input parameter combinations. To speed up the simulation only the parameters could be used that had a relevant score in a sensitivity analysis [32].

A reasonable combination of input parameters is preferable. After all, parameter combinations that are actually unrealistic should be rejected. For example, within one clay layer it is reasonable to assume that, if the friction angle is very low, the modulus of subgrade reaction is on the high end of its distribution. Therefore, correlations between parameters can be specified. The determination of correlations is a controversial aspect in soil characterization [13]. Therefore, this should be done with care. In this thesis the following assumptions are made on these correlations:

a. Soil strength parameters are independent from layer to layer.
b. Within one layer a correlation of +0.5 is assumed between cohesion and friction angle.
c. In all soil layers the friction angle is directly correlated to the wall friction angle, such that equations (9) and (10) hold.

3.Bayesian Inference

Bayesian inference is a method that aims to estimate a new parameter in case different sources of information are available [4]. In the context of the Observational method the measured quantity is displacements, indicated by the subscript d.

A lognormal distribution of the <u>prior information</u> (believe of displacements) can be described as [4]:

$$f_x(x_d) = \frac{1}{\sqrt{2\pi}\zeta x_d} \exp\left[-\frac{1}{2}\left(\frac{\ln x_d - \lambda}{\zeta}\right)^2\right] \tag{25}$$

In which:

λ = mean deflection of the lognormal distribution

ζ = standard deviation of the lognormal distribution

Typically, the uncertainty in measurements can be described by a normal distribution. The normal distribution can be combined with a lognormal distribution: Given that $X \sim N(\mu_d, \sigma_d{}^2)$, then $\exp(X) \sim LogN(\mu_d, \sigma_d{}^2)$. Therefore, it holds that

$$\mu_d = \exp\left(\lambda + \frac{1}{2}\zeta^2\right) \tag{26}$$

The variance of the lognormal distribution is related to the coefficient of variation via:

$$\zeta^2 = \ln\left[1 + \left(\frac{\sigma_d}{\mu_d}\right)^2\right] \tag{27}$$

Rewriting the equation leads to:

$$V = \sqrt{e^{\zeta^2} - 1} \tag{28}$$

If the coefficient of variation is $\leq$ 0.30 it can be noticed that $\zeta \approx \left(\frac{\sigma_d}{\mu_d}\right)$. The shape of the lognormal distribution is than similar to that of a Gaussian distribution [4].

In the Bayesian update the following formulations are used:
The information of the prior distribution is translated of the lognormal mean $E(\lambda) = \mu_d$ and the variation $Var(\lambda) = \sigma_d{}^2$. A new prediction, which is a result of formulas (30) and (31) is also a property of the lognormal distribution and indicated by the superscript $\mu_d{}'$. This new prediction is referred to as the *posterior* or simply as *the Bayesian update*.

If the measurement data set $(x_1 \ldots x_n)$ contains *n* measurements, then $\bar{x}$ is the mean of the measurements. The variance ζ^2 concerns the *natural logarithm of the measurement's variance*. Additionally, the error of the measurement device $\sigma^2_{m.e.}$ can be included via [23]:

$$\zeta^2 = LN\left[\sigma^2_{inh.} + \sigma^2_{m.e.}\right] \tag{29}$$

Consequently, the formula to determine posterior statistics via Bayesian updating are:

$$\mu_d{}' = \frac{\mu_d\left(\frac{\zeta^2}{n}\right) + \sigma_d{}^2 \ \ln\bar{x}}{\left(\frac{\zeta^2}{n}\right) + \sigma_d{}^2} \tag{30}$$

$$\sigma_d{}' = \sqrt{\frac{\sigma_d{}^2 \ \left(\frac{\zeta^2}{n}\right)}{\left(\frac{\zeta^2}{n}\right) + \sigma_d{}^2}} \tag{31}$$

4.Calibration
With the Bayesian update a new posterior distribution is formed on expected displacements. A corresponding parameter set can be found by looking at what input parameters of DsheetPiling correspond to the displacement output. This process is known as calibration, the opposite procedure of forward modelling (Figure 5.5).

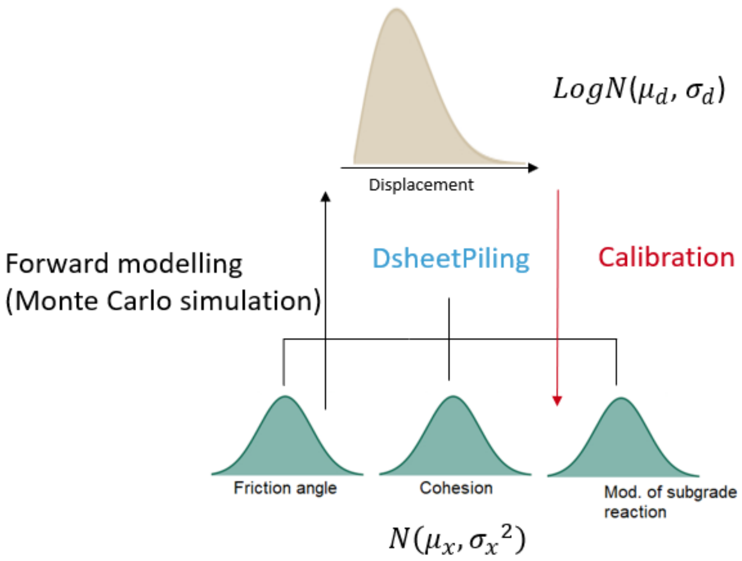


Figure 5.5 Principle of Calibration.

As for the prior distribution a Monte Carlo simulation was performed, it is possible to analyse the input variations of the MC that correspond to the posterior mean and 5% tails (Figure 5.6). For soil parameter X the new set of input parameters is indicated as $N\left(\mu_x^{'}, \sigma_x^{'\,2}\right)$. Correspondingly, a new $\beta_{OM}{}'$ could be determined.

Often multiple different realizations could lead to the similar displacement results. This thesis adopts for each parameter the average of the corresponding realizations. Based on the calibration at displacement $\mu_d{}'$ mean input parameters can be found. At the posterior tails, input parameters are found from which parametric variance can be determined. From the calibrated μ_x' and $\sigma_x'^{\,2}$ the coefficient of variation can be derived that gives us the valuable information described in section 5.3.

The outcome of this way of calibration strongly depends on the input of the Monte Carlo simulation. Most information can be found on parameters with a high sensitivity score. The following factors also influence calibration results:

- The assumed correlation between parameters.
- The number of Monte Carlo realizations.

This way of calibration is not in justice with the complexity that should actually be assumed in inverse analysis problems. However, it is quick and easy and therefore adopted as a first trial. The results can be verified by the use of more complex algorithms.

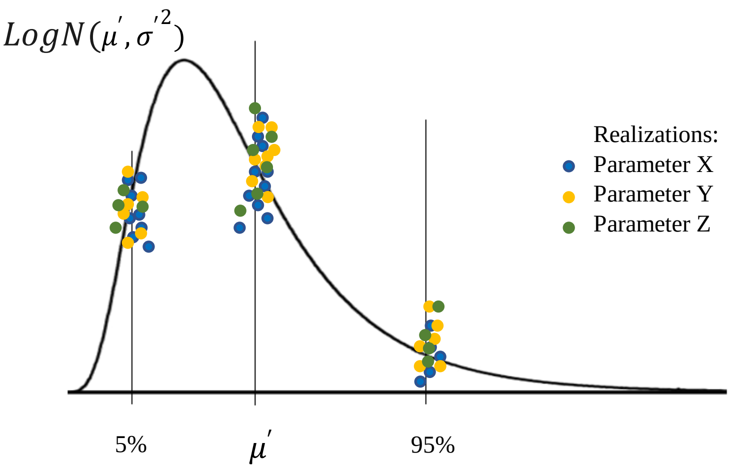


Figure 5.6 Calibration points on tails of the Posterior distribution.

5.Decision making

Following from the calibration, the new set of parameters can be adopted to make a displacement prediction for the next construction phase. This can be achieved by performing a new Monte Carlo simulation for the sequential construction phase, adopting the new input characteristics. By consistency, this prediction should be closer to field measurements. This can again be verified once the construction phase is completed and new measurements are gathered. Again, a Bayesian update can be performed, now with the prior distribution being the updated prediction. This repetitive way of applying Bayesian updating, each phase, would ideally lead to a posterior distribution that is representative for the situation in the field, considering all the information on hand. In the end, the distribution could help Engineers to decide on the structural safety.

Hypothesis testing

In the last phase that a contingency measure could be installed, all the measurement data should be considered to make a decision. In the benchmark this corresponds to phase 5, in which the water level in the building pit is temporary lowered. The performance of the sheet pile wall during this phase and the previous phases, should then be evaluated. This can be done via the Bayesian update procedure as just described.

Via hypothesis testing two representing displacements scenarios H0 and Ha can be tested against each other for the final decision making [4]:

H0: Null hypothesis: The Updated prediction, based on calibrated soil parameters $N\left(\mu_x', \sigma_x'^{\,2}\right)$.

Ha: Alternative hypothesis: The prior distribution, based on soil parameters with characteristics $N\left(\mu_x, \sigma_x^{\,2}\right)$ as defined in design standards.

If the null hypothesis is rejected, the alternative hypothesis will be the prior distribution, to represent that the full range of possible parameters is still to be encountered. In other words, the information obtained from the posterior distribution is not sufficient for decision making. By introducing a so-called level of significance α_{hyp} it can be quantified when there is enough confidence in the null hypothesis (Figure 5.7)

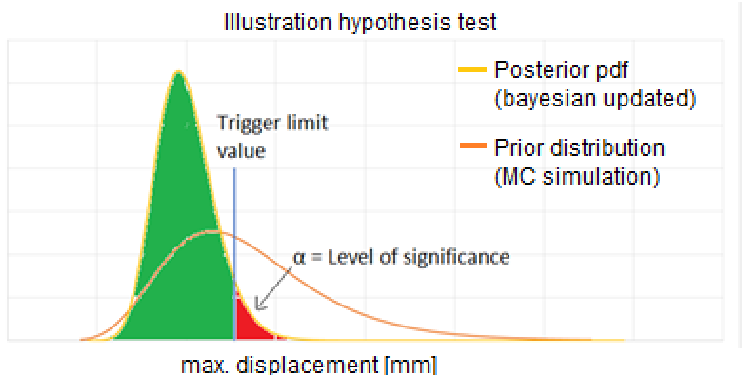


Figure 5.7 Illustration of hypothesis test.

If α_{hyp} = 10%, there is a 90% chance (green area of Figure 5.7) that displacements in the next construction phase will be below the Trigger limit as well – provided that the next construction phase is consistently predicted by the computer model and no unforeseen events occur. There is a 10% chance of making a type I error: This error means that H0 is rejected while it was actually true. The type I error primarily effects the construction costs, as rejection of the posterior would mean that a contingency action should be executed.

Finally, an outline of the statistical methods forms a methodology for back-analysis as summarized by Figure 5.8. In the next sections this methodology will be further illustrated by means of the benchmark.

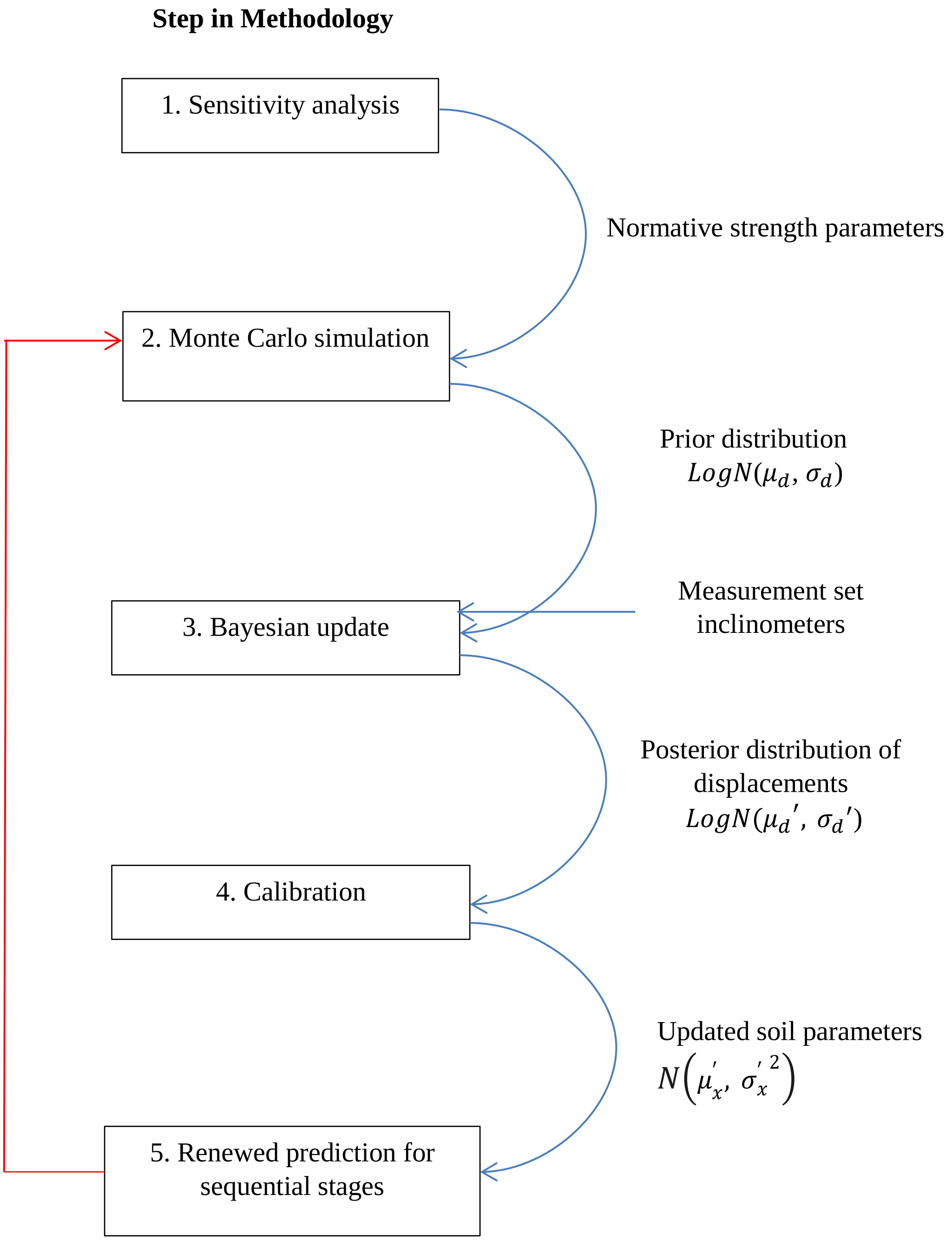


Figure 5.8 Scheme summarizing 5 steps of the Methodology.

5.5 Demonstration of Bayesian update

As in the fictive benchmark no real measurements exist, a dataset will be assumed to section is to demonstrate the Bayesian update.

1. Sensitivity analysis

First a sensitivity analysis is performed by varying each strength parameter of the clay and sand layer. The φ and δ are related to each other via formula 9 of the Kotter model (section 3.1). The results for several construction phases are presented by Figure 5.9.

It can be seen that the friction angle φ, cohesion c and modulus of subgrade reaction k of the clay layer have the highest scores. This makes sense given the stratigraphy of the Benchmark. As excavation progresses the score of the modulus of subgrade reaction reduces. This is linked with the stress-displacement tree that is used in DsheetPiling to assess displacements (Figure 3.1, Chapter 3). As an active stress state is reached, the magnitude of both the φ and c determine the magnitude of the maximum sheet pile wall displacement.

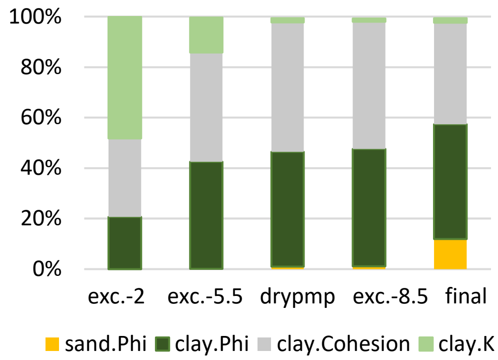


Figure 5.9 Results of Sensitivity analysis benchmark 1.

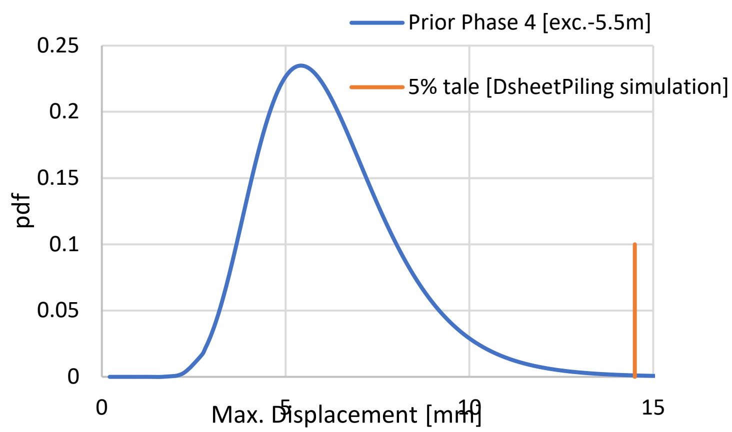


Figure 5.10 MC simulation for phase 4, used as prior information.

2. Monte Carlo simulation

A Monte Carlo (MC) simulation has been performed for phase 4 – excavation till -5.5 m. Following the results of the sensitivity analysis, only the clay φ and c were used as input variables. The input was based on the parameter distribution according to NEN9997 and stated in Table 4.1 (section 4.1). The output of the MC simulation is depicted in Figure 5.10. The output distribution is lognormal, with a mean of 6.1 mm and coefficient of variation V of 30%. This distribution was established by taking n=1000 samples. This is not at all the number of samples that is in justice with an accurate simulation of the complete output distribution. Therefore, it can be seen in Figure 5.10 that the tails are not accurate in the pdf. However, as the Bayesian update uses a combination of the prior distribution and the measurement data, this is not necessary. It is important that the mean is well simulated. The somewhat inaccurate covariance of the lognormal displacement distribution can be corrected. A quickly performed Monte Carlo simulation is therefore preferred.

3.Bayesian update: Application to phase 4

In benchmark 1 dry pumping is applied in phase 5 with the goal to check if a second layer of struts needs to be implemented. The Bayesian update of phase 4 can give a new estimation of displacements for phase 5.

The lognormal output distribution of the Monte Carlo simulation (step 2) serves as the prior distribution for the Bayesian update. A fictive dataset is assumed (dataset 1) as stated in Table 5.1. This dataset represents measurements of four inclinometers taken at the end of construction phase 4. With the input stated in Table 5.1, the Bayesian update can be performed according to formula (30) and (31). This results in the distribution as depicted in Figure 5.11. It can be seen that, although the prior distribution has a slightly lower coefficient of variation, the Bayesian update converges towards the mean of the measurement dataset, because there are n = 4 measurements.

	Prior information (phase 4)[lognormal]	Dataset 1, n=4 [normal]	Posterior distribution [lognormal]
		2.3	
		3.6	
		5.5	
		5.2	
μ [mm]	6.2	4.15	4.7
σ[mm]	1.86	1.47	1.18
V	30%	35%	17%

Table 5.1 Input for Bayesian update.

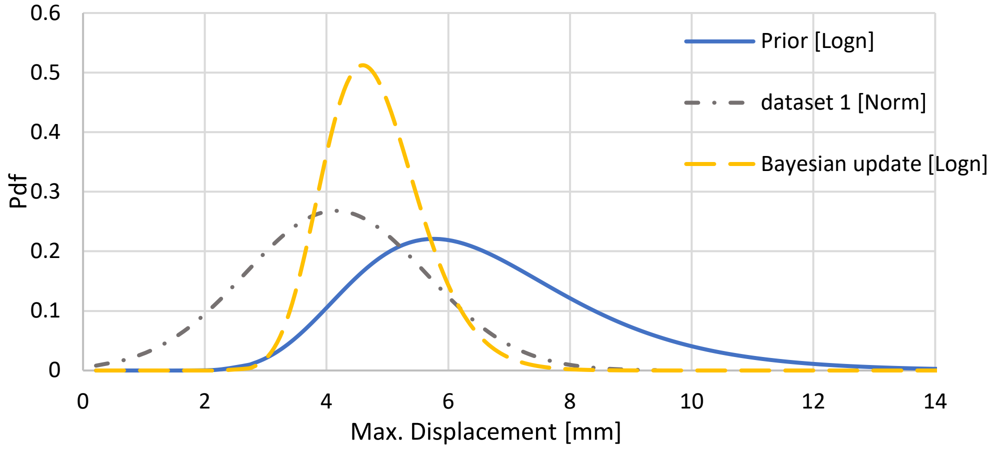


Figure 5.11 Bayesian update with dataset 1.

In phase 4, the measurement set of phase 5 is still unknown. Only a DsheetPiling prediction can be made for phase 5. However, as seen in phase 4, the measurement set and the prior estimation by DsheetPiling did not coincide. The Bayesian update of phase 4 uses the two sources of information for an enhanced estimation of the output distribution. This new estimation can be used for a renewed DsheetPiling prediction for displacements in phase 5. In order to perform another DsheetPiling simulation, the input parameters are needed that corresponds to the Bayesian update of phase 4. This is provided by step 4, calibration.

4.Calibration

Next, calibration is performed based on the mean and the tails of the Bayesian update (Figure 5.12). Calibration would lead to new estimation for the clay φ and c. This is done by analyzing the samples that were filled in the Monte Carlo simulation of step 2.

The results of the calibration are stated in the Table 5.2 and visualized by the Figure 5.13. The calibrated parameters both have a higher mean that originally assumed. This is in correspondence with the Bayesian update showing less displacement than the prior distribution. For both parameters the coefficient of variation has reduced as well.

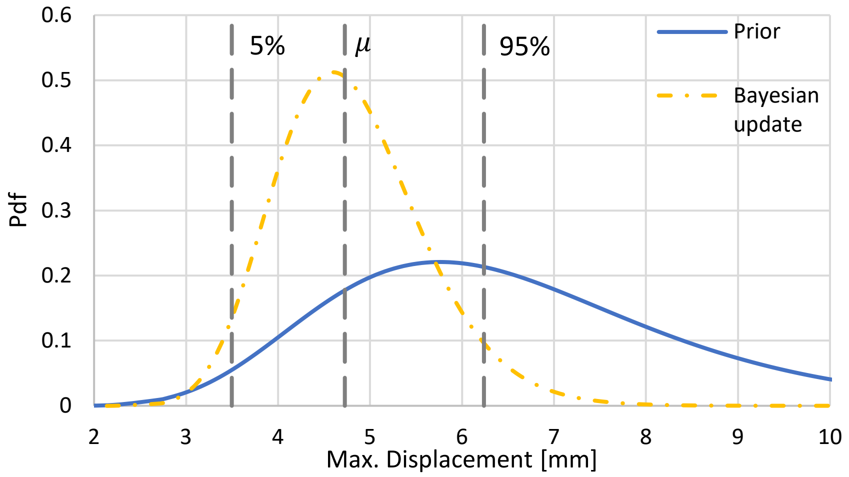


Figure 5.12 Mean and 5 and 95 tails of Bayesian update used for calibration.

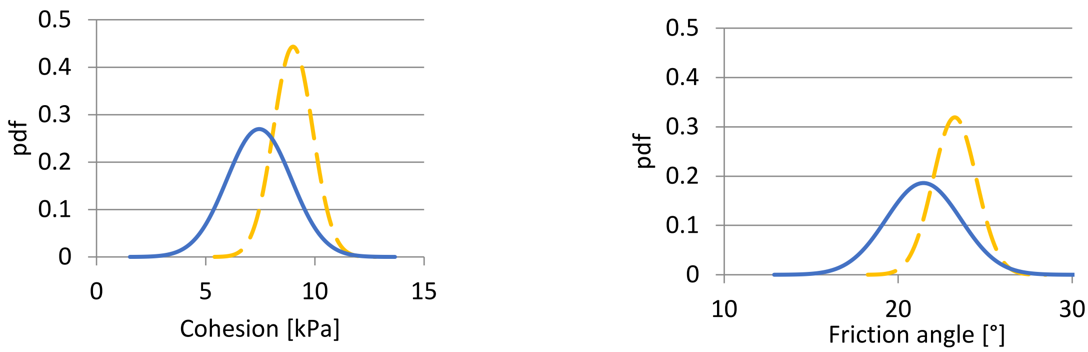


Figure 5.13 Calibrated results for cohesion (left) and friction angle of Clay layers (right).

		Cohesion	Friction angle
Original	μ	7.45 kPa	21.5°
	V	20%	10%
Calibrated	μ'	8.7 kPa	23.3°
	V'	10%	5%

Table 5.2 Results of calibration of strength properties of Clay layer.

The calibrated properties are favorable in terms of the safety. Figure 5.14 demonstrates the influence of the change in μ and V of the cohesion on the reliability index β. With the original parameters β=1, which is low compared to the required β of Consequence Class 1, which is 3.3. For the calibrated μ', it can be seen that the reliability index already increases. The consequential change in V' results in an updated β' = 3.9.

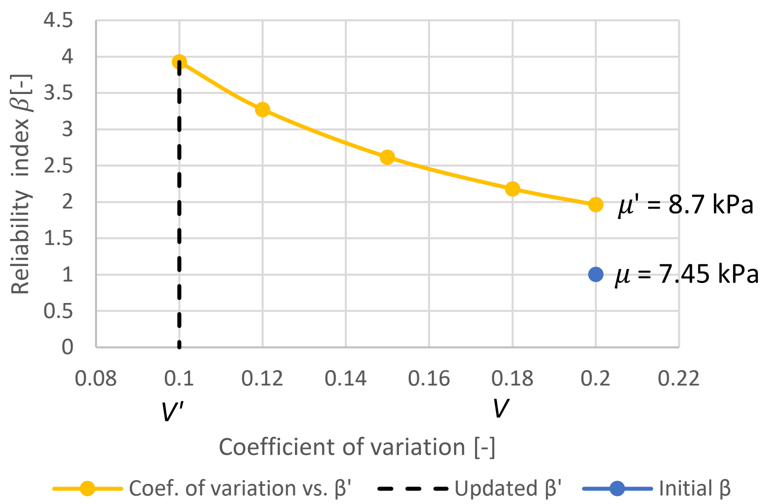


Figure 5.14 Relationship Reliability index and change in mean and coef. of variation.

New predictions based on calibration results
A new forward prediction for phase 5 can be produced via a Monte Carlo simulation with the updated parameters as input. This MC simulation results in the yellow distribution depicted in Figure 5.15. It can be seen that if no Bayesian update was performed, the prior distribution based on the original parameters would have been significantly wider. This illustrates how the information of the measurements taken at phase 4 can be used to reduce the range of possible outcomes for the next phases.
If at the end of phase 5 a decision should be made for a contingency measure the Bayesian update indicates that, based on the measurements taken at phase 4, there is a confidence of about 90% that the deterministic Trigger limit is not exceeded.
Once phase 5 has been completed, its updated prediction can again be compared with a new measurement set. Again, a Bayesian update could be performed in phase 5. This repetitive way of performing the update aims to take into account all the measurements throughout the different construction phases. This repetitive way is demonstrated by the case studies in chapter 6.

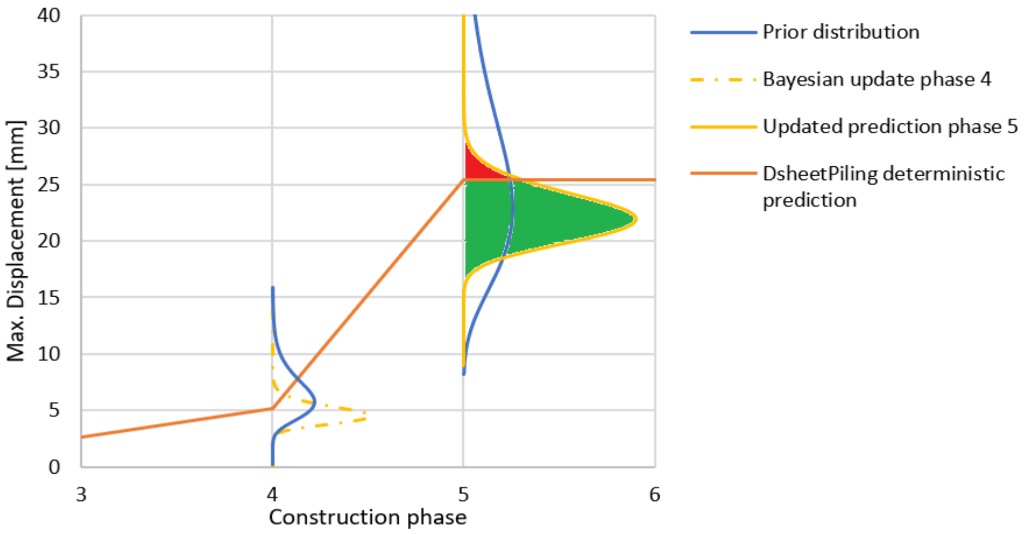


Figure 5.15 New prediction based on results of Bayesian update phase 4.

5.6 Introducing errors to Bayesian update

As mentioned throughout the report, the effect of uncertainties to retaining wall design should be considered. In Bayesian inference, additional uncertainties can be introduced via the standard deviation of both the prior and the measurement distribution. To see the effect of these uncertainties on the Bayesian update, the following 3 errors are introduced that are relevant for a Bayesian update

1. Model error: Taking into account an absolute DsheetPiling model error of 10%.
This 10% is based on a statement made in the CUR166 [13] on the accuracy of the Kotter model. Introducing this error leads to a slightly wider prior distribution (Figure 5.16). This results in a Bayesian update with a mean shifting more towards the measurements. The corresponding coefficient of variation changes to 18% (+2%).

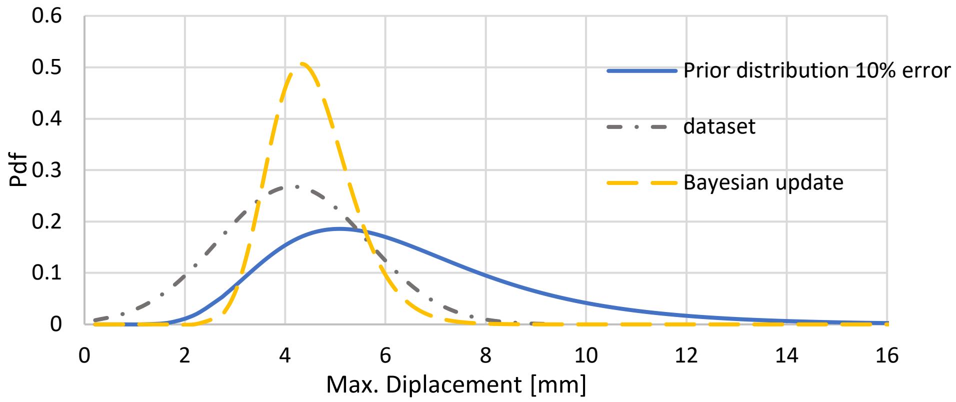


Figure 5.16 Influence of applying a 10% model error to Prior distribution.

2. Measurement error: As mentioned in section 3.2, the inclinometers that are used to monitor sheet pile wall deflections have a measurement error of 6mm per 25m sheet pile wall length. For the length of the wall in this benchmark, 13.5m, this error has a magnitude of 3.3mm. According to the 3σ rule in a Gauss-curve 99.73% of the data lies within 3σ. This inclinometer error can be considered by adding $\sigma_{m.e.} = 1.1$mm to the standard deviation of the measurements (Formula 29). Therefore, even negative displacements could be measured (Figure 5.17). The Bayesian update shift more towards the prior distribution. However, this shift is not that large, as n=4 and the variance of the measurements are reduced by this factor n, according to formula (30).

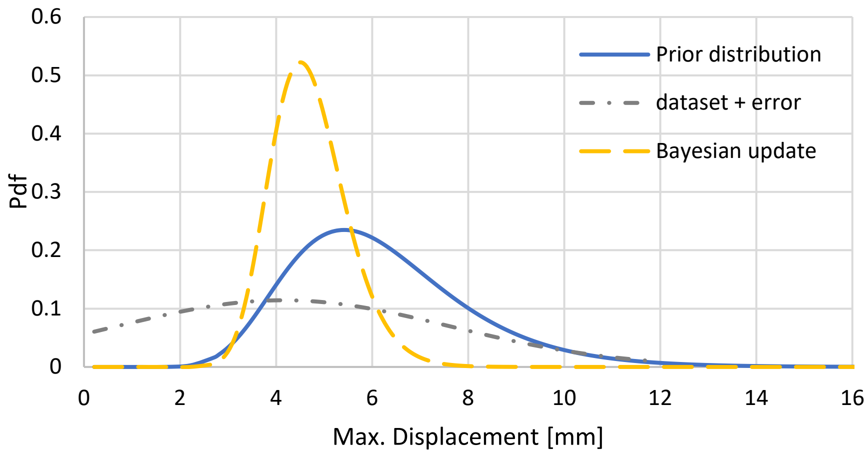


Figure 5.17 Influence of adding inclinometer error to measurement distribution.

3. It is not always practical to install multiple inclinometers. Therefore, the results in Table 5.3 and Figure 5.18 illustrate the effect of having a different number of inclinometers. With a lower n, the prior distribution gets more weight in the Bayesian update. It can be seen that the bigger n, the less it has its effect on reducing the V and the shift in mean.

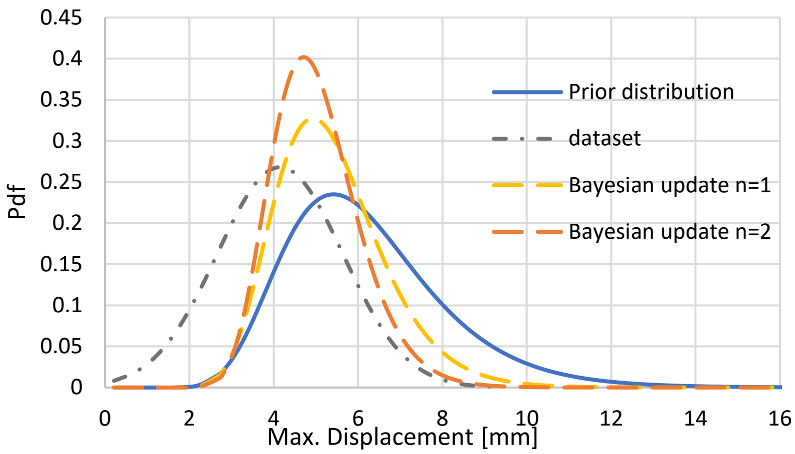


Figure 5.18 Output distributions for different number of measurements.

	n=1	n=2	n=3
Mean [mm]	5.2	4.9	4.85
V	24%	20.5%	18%

Table 5.3 Results of Bayesian update output distribution as a function of number of measurements n.

Influence of errors on decision making

Adopting error 1 and 2, again a new prediction is made for phase 5 as presented in Figure 5.19. The shape of the Bayesian output distribution has widened leading to a confidence of 65% while at first this was 90% (Figure 5.15). Based on these results, a contingency measure should be applied. It would have been valuable to have an additional measurement set, for instance during the dry pumping, in order to have a second verification for this decision.

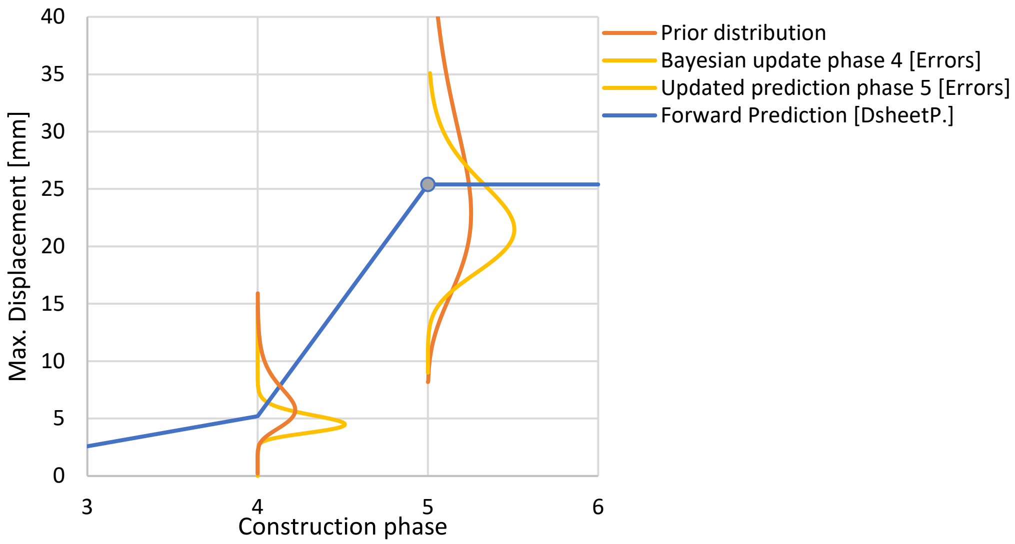


Figure 5.19 Forward prediction for phase 5, based on Bayesian update of phase 4, including modelling error.

5.7 Conclusions

The methodology has been introduced and tested with a fictive dataset for the rather simplified benchmark. Calibration could be done for the cohesion and friction angle of clay. It showed that with the fictive dataset, the coefficient of variation of both parameters could be decreased. With the updated soil strength parameters it was shown that the reliability index increased. A renewed prediction could be made for displacements in the next phase.

Consequently, the impact of errors to the results of the Bayesian update were demonstrated. In order to investigate the potential of the method to real construction works, the methodology is tested in actual case studies in the next chapter. These case studies contain extensive stratigraphy and actual datasets. Analyzing real datasets gives the additional opportunity to investigate how to deal another uncertainty that has not been considered yet – the possibility to encounter unforeseen events.

6. Case studies

In this chapter two case studies are presented of building pits in the Netherlands. The retaining wall design was made according to conventional design methods. However, as both projects dealt with complexities, monitoring was done to check the construction process. The inclinometer data will be analyzed in order to test the methodology presented in chapter 5. Also, for each case study an OM design is proposed. Both analysis aims to conclude on the question whether or not an OM Ab Initio approach could have been feasible. To summarize, the focus is on answering the following questions:

Does the Methodology as presented in chapter 5 work for these case studies?
Could significant savings have been made if the OM Ab Initio approach had been used?
How to deal with unforeseen events?

This chapter is structured as follows: For each case study, first it is presented how the project was executed. Secondly, the inclinometer data is processed with the Bayesian update. This leads to new estimation of parameters that suits the measured structural behavior. Thirdly, an OM design is proposed based on mean soil parameters to indicate the structural savings that could have been made. Each case study ends by concluding on the above questions.

6.1 Case study 1: Merckt, Groningen

The first case study presents the analysis of monitoring data collected by the construction of a deep building pit in the city center of Groningen, located in the north of the Netherlands. The geology in this area is characterized by stronger soils (sand, silt layers, over-consolidated properties). These conditions are in terms of bearing capacity more favorable than the soft-soil conditions typically found in the western part of the Netherlands. However, the analysis is interesting because of the presence of a thick loamy clay layer.

Project description

The project concerns the construction of an apartment building with a 2-floor basement. A building pit is constructed as depicted in Figure 6.1. Features of the building pit are:

- A strongly asymmetric shape divided by 4 cross-sections.
- Total excavation depth: 9.6m
- Along cross-section 2 the adjacent buildings have monument status. This results in strict SLS requirements.
- Strong heterogeneity and varying surface levels along the different cross-sections of the building pit.
- Presence of boulders in the subsurface with strongly varying sizes.
- Consequence class: CC2

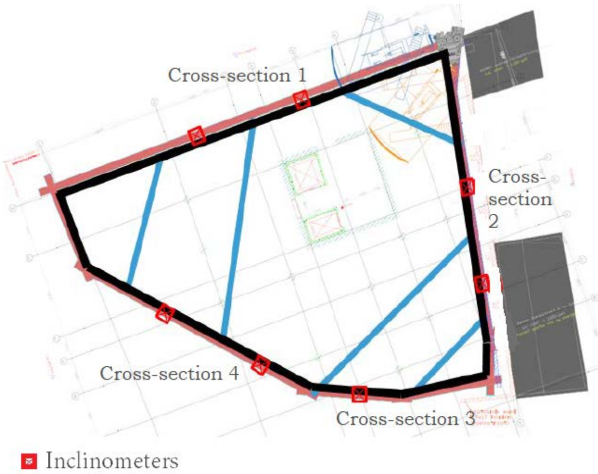


Figure 6.1 Top view of building pit plan, Groningen.

Stratigraphy
Several CPTs were performed during the site investigation. The surface level on site varied from +7.60m to +6.88m NAP. The layering strongly varies and is at some locations complemented with an additional strong sand/gravel layer. Not all CPTs reached to the required depth due to the presence of boulders of varying size.

To give an impression of the variability on construction site, the stratigraphy of cross-section 1 and cross-section 4 are presented by Figure 6.2, respectively Figure 6.3. Different soil layers are described by Table 6.1. The mean and soil properties of the layers in cross-section 4 are presented by Table 6.2.

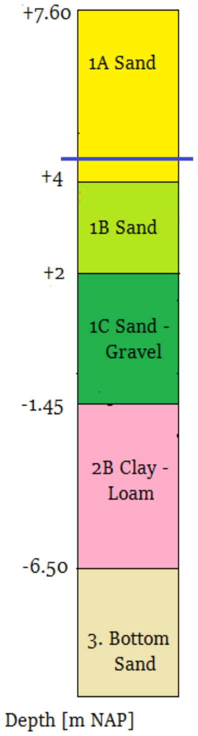


Figure 6.2 Stratigraphy Cross-section 1

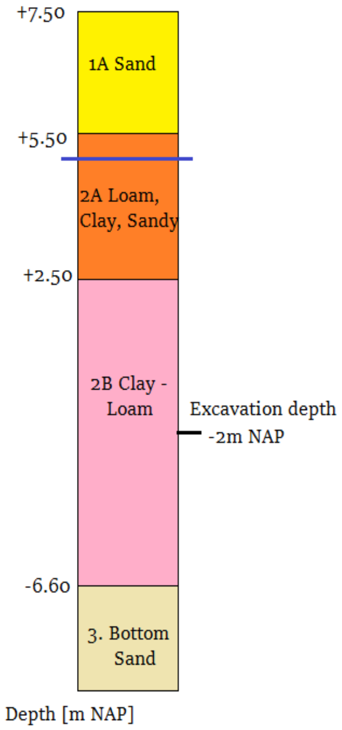


Figure 6.3 Stratigraphy Cross-section 4

Soil layer	Description
1A. Sand	Top layer, loosely packed sand
1B. Sand	Loose packed sand complemented with gravel
1C. Sand-Gravel layer	Sand, moderately packed, complemented with gravel. Presence of boulders Cone resistance > 100MPa
2A Loam, Clay, sandy	Loamy clay with a slight degree of sand.
2B. Clay – Loam layer	Presence of boulders Low permeable Over consolidated Thickness strongly varying
3. Bottom Sand layer	Serves as foundation for the retaining wall

Table 6.1 Description of soil layers.

	1A. Sand		2A. Clay		2B. Loam/clay	
	50%	5%	50%	5%	50%	5%
γsat [kN/m^3]	19	19	19	19	21.5	21.5
γunsat [kN/m^3]	17	17	19	19	21.5	21.5
φ [°]	35.9	30	26.9	22.5	33.5	28
δ [°]	35.9	30	26.9	22.5	33.5	28
c [kPa]	-	-	-	-	3.7	2.5
OCR [-]	-	-	-	-	4	3
Modulus of subgrade reaction						
50%	1.80E+04	1.20E+04	6.00E+03	4.00E+03	9.00E+03	6.00E+03
80%	9.00E+03	6.00E+03	3.00E+03	2.00E+03	6.00E+03	4.00E+03
100%	4.50E+03	3.00E+03	1.20E+03	8.00E+02	3.00E+03	2.00E+03

Table 6.2 Soil strength properties of layer 1A, 2A and 2B of cross-section 4.

Building pit execution

Because of the depth of the bottom sand layer a long retaining wall needed to be constructed, from +7.60m NAP till an installation depth of -8m NAP. Because of the presence of boulders, a regular sheet pile wall turned out to be unsuitable. Instead it was chosen to perform a Mixed in Place (MIP) wall. This wall is a mixture of soil and concrete fabricated with the use of a triple-axis auger and enhanced with steel H-profiles (Figure 6.4) [33]. Although such a wall is costly, the installation technique is able to deal with the presence of boulders till a diameter of 30cm. The strength of the wall depends a bit on the quality of the ground mixture but is mostly determined by the steel frame. A concern of the MIP wall would be the quality of the mixture to establish a fully impermeable wall. In the project this did not turn out to be a problem.

To limit deformations of the MIP wall, 3 rows of struts are installed with a framework as depicted in Figure 6.5. This construction has been modelled in DsheetPiling according to Figure 6.6.

At each side of the building pit inclinometers were installed in accordance with Figure 6.1. Table 6.3 summarizes the construction phasing.

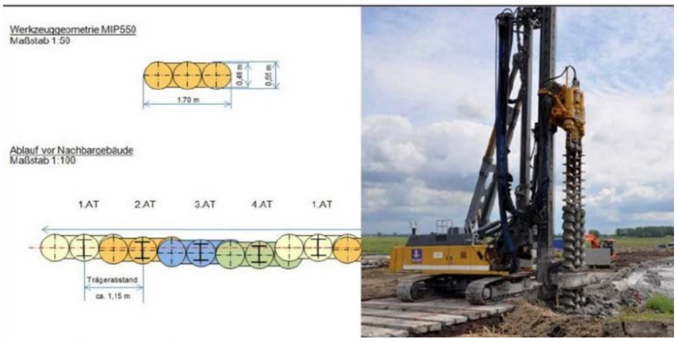


Figure 6.4 MIP-wall [33].

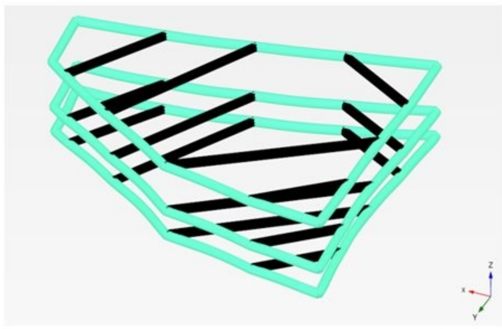


Figure 6.5 3D view of strut plan

Phase	Description	Limiting max. displacements along wall [mm]
1	Installation of MIP wall	
2	Excavation +6.1m	5
	Install 1st layer of struts	
3	Excavation +3.52m	15
	Install 2nd layer of struts	
4	Excavation +0.6m	25
	Install 3rd layer of struts	
5	Excavation – final depth at -2m NAP.	30
6	Installation of concrete floor	
7	Removal of strut 3	30
8	Removal of strut 2	30
9	Removal of strut 1	30

Table 6.3 Construction phasing and measurement intervals

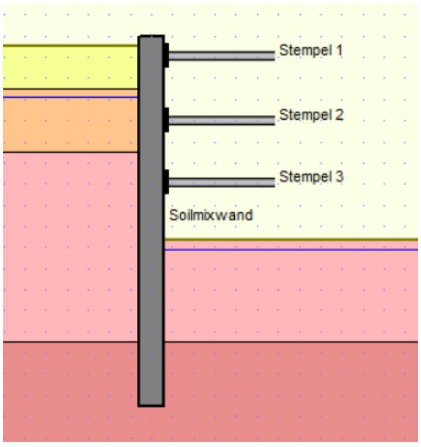


Figure 6.6 2D view of MIP wall and struts

6.2 Analysis

For the analysis of the deformations of the sheet pile wall data of the inclinometers HMB1 and HMB2 is used. These were installed at cross-section 4. This cross-section has been chosen because of the presence of layer 2B (loam/clay), which has the biggest uncertainty in strength parameters. Therefore, the goal is to find the representative soil strength parameters of this layer.
With the model geometry as depicted in Figure 6.6 and the parameter input of Table 6.2 DsheetPiling calculations were performed. This resulted in the displacement predictions as presented in Figure 6.7. The average of the 2 inclinometers is used as the measurement set. It can be seen in Figure 6.7 that the measurements kept below a 5mm maximum displacement. This is well below both predictions of the DsheetPiling model.

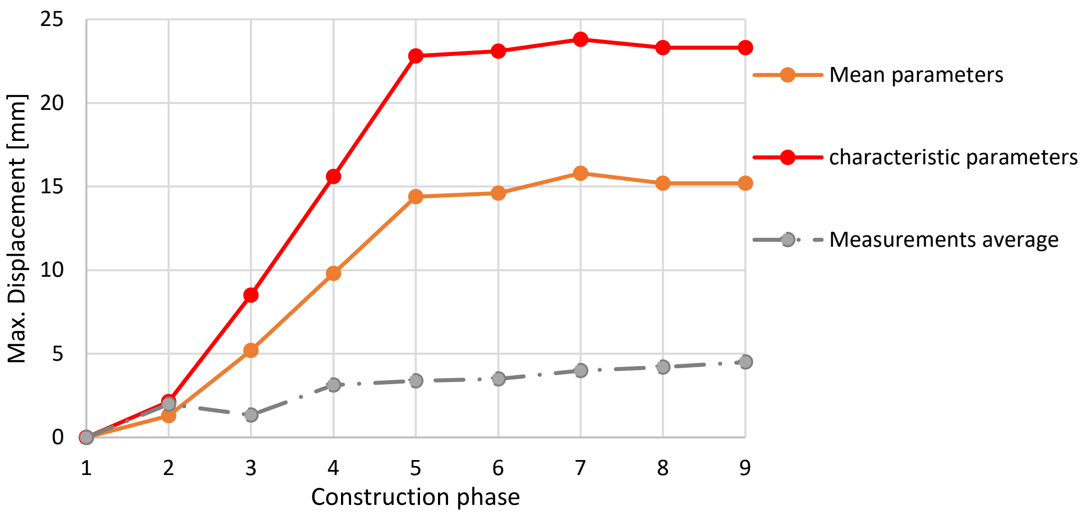


Figure 6.7 Case Groningen: DsheetPiling predictions versus measurements.

Modelling the structural components

The asymmetric shape of the building pit (Figure 6.1 and 6.5) has a distribution of forces among the different strut layers that cannot be taken into account by the simplified 2D DsheetPiling model geometry. This has been confirmed by comparing the DsheetPiling predicted strut forces N with the strut forces found in a PLAXIS 3D model (Table 6.4) [35]. In the PLAXIS 3D model, the strut forces per meter turned out to be significantly higher. Consequently, the possibly incorrect assessment of strut forces by DsheetPiling might lead to an overestimation of the displacements. This could be a reason for the lower displacements shown by the measurement set in Figure 6.7.
To correct for this, it is chosen to adopt an overall uncertainty of 25% in actual stiffness (*EI*) of the wall. This is reasoned as follows: The support of the wall is provided by (1) the soil and (2) the 3 struts [34], as demonstrated in Figure 6.8. The total measured wall displacement can therefore be subdivided in the 2 components. The deflection is a function of the wall length l, wall stiffness *EI* equivalent force. As for (2) the total strut force N is underestimated, the deflection d_2 will be overestimated.

Predicted strut forces [kN]						
	PLAXIS 3D			DsheetPiling		
	Strut 1	Strut 2	Strut 3	Strut 1	Strut 2	Strut 3
Phase 3	64			47		
Phase 4	68	223		6	193	
Phase 5	58	285	280	10	128	266

Table 6.4 Strut forces according to DsheetPiling and PLAXIS 3D model.

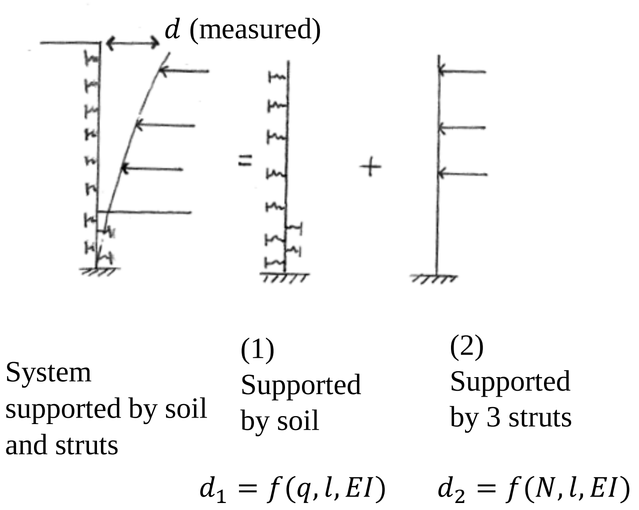


Figure 6.8 Behaviour of the sheet pile wall subdivided in (1) supported by soil and (2) supported by struts.

The strut force N is an output variable of DsheetPiling and can therefore not be changed. To correct for this error the input parameter EI can be made stochastic just like the soil strength parameters. By making EI a variable, it needs to be noted that this will also influence d_1.
A bigger EI leads to a structure that behaves stiffer. The originally assumed stiffness is 87500 kNm^2/m, which is a somewhat conservative value for the strength of the MIP wall. In the following analysis a variability of 25% is assumed.

Step 1: Sensitivity analysis
During the excavation process (construction phase 2 to 5) most wall displacements are expected. Therefore, these phases are of primary interest of the Bayesian update.
The outcome of the sensitivity analysis is presented in Figure 6.9. Because of the soil mixture, the friction angle φ equals the wall friction angle δ. Therefore, the parameters are fully correlated. The excavation in phase 2 takes place in the top sand layer (1A). As the depth is limited, it can be seen that the EI of the wall is not yet significant. Instead, the friction angle of the top layer has the highest part in total deformations, followed by the strength properties of 2B.
The properties of layer 2B score higher in the consequential excavation stages. As excavation continues, the value for the sheet pile wall stiffness EI becomes more significant as well. The influence of the properties of the top layers (1A and 2A) decrease with excavation depth. For that reason, it is chosen to not consider their modulus of subgrade reaction k in the consequential analysis steps: 1A.k and 2A.k will not be calibrated.

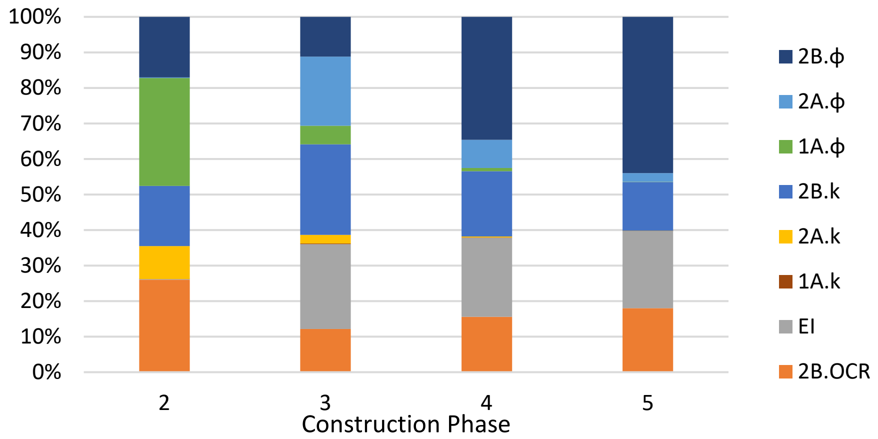


Figure 6.9 Sensitivity scores in % for excavation phases 2 – 5.

Step 2/3: Bayesian update
Next, for the excavation stages (phase 2 to 5) the Bayesian update is performed.
For the prior information of the first update, phase 2, a Monte Carlo simulation has been performed adopting the mean soil parameters as stated in Table 6.2. This resulted in a lognormal distribution with a mean of 1.27 mm and a variance of 18% (Table 6.5).
In the contrary of what is found in the other construction phases, the measurements of HMB1 and HMB2 showed bigger displacements than predicted by the DsheetPiling model. However, the measurement set carries a large uncertainty as an absolute inclinometer error of 1.36mm needs to be considered. This error leads to a coefficient of variation of 92%. The wide measurement distribution of phase 2 is illustrated by Figure 6.10.
In the Bayesian update this measurement set is added as a very uncertain source of information. This results in an updated prediction of displacements that is actually more uncertain than that of the prior distribution (Table 6.5 and Figure 6.11).

Phase 2	Prior	Measurements	Bayesian update
Mean [mm]	1.27	2.0	1.44
V	18.2%	92%	36%

Table 6.5 Results of Bayesian update, phase 2.

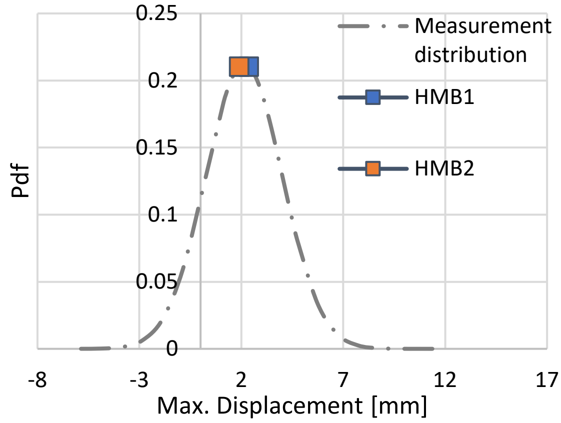


Figure 6.10 Measurement distribution Phase 2.

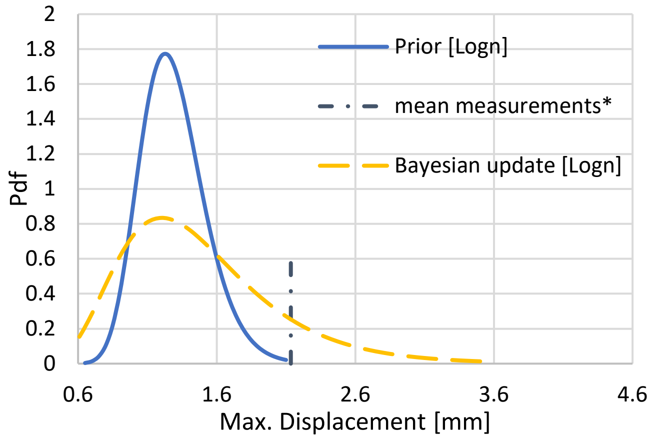


Figure 6.11 Bayesian update Phase 2.
**Measurement distribution is not included because of shape.*

The high variability in the measurement distribution of phase 2 is questionable. It can be seen in Figure 6.10 that the measurements of HMB1 and HMB2 are actually very close to each other. As also the result of the Bayesian update increases the uncertainty, it is decided to not adopt the update and stay with the prior distribution.
The absolute inclinometer error of 1.36mm is very large compared to the magnitude of the measurements and leads to unsatisfactory results. To avoid these results in the following Bayesian updates it is decided to decrease the inclinometer error $\sigma^2_{m.e.}$ per phase. This is done with the reason that throughout excavation more measurements are obtained with the same magnitude of +- 2mm (Figure 6.9) which gives confidence in the performance of the inclinometer device. Consequently, formula (32) is used for the variance of the measurement set:

$$\zeta^2 = LN\left[\sigma^2_{inh.} + \frac{\sigma^2_{m.e.}}{n_{phase} - 1}\right] \tag{32}$$

in which n_{phase} represents the construction phase.

By applying this reduced inclinometer error, the Bayesian update of phases 3 to 5 slowly converges towards the measurement set as can be seen in Figure 6.15. For phase 3, the variation of the measurements is still 80%. However, as the number of inclinometers is 2, the Bayesian update falls in the middle the prior and measurement set (Figure 6.12). In Figure 6.13 it can be seen that the Bayesian update of phase 4 shift more towards the mean of the measurements. At the end of excavation (phase 5) the coefficient of variation of the Bayesian update is about 5% with a mean that coincides with the mean of the measurements HMB1 and HMB2 of phase 5.

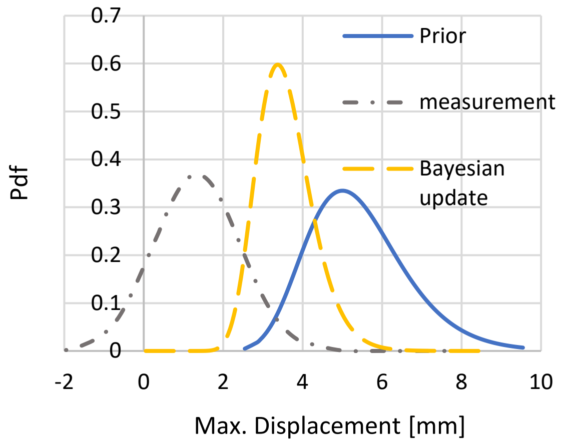


Figure 6.12 Bayesian update phase 3

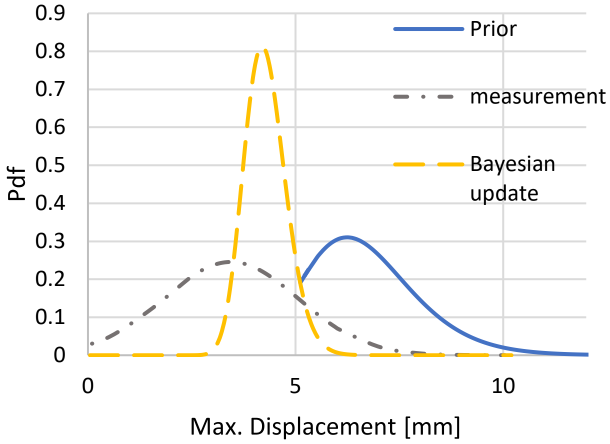


Figure 6.13 Bayesian update phase 4.

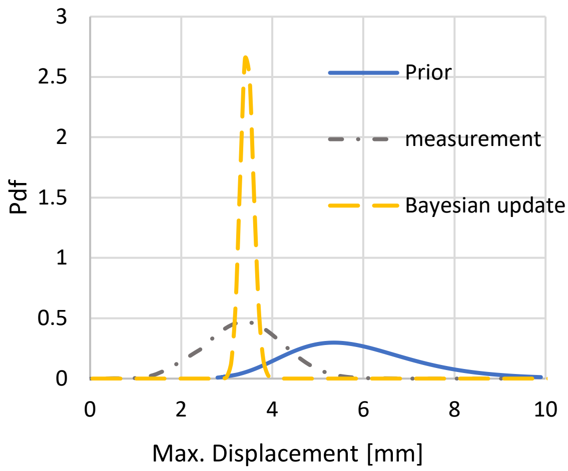


Figure 6.14 Bayesian update phase 5

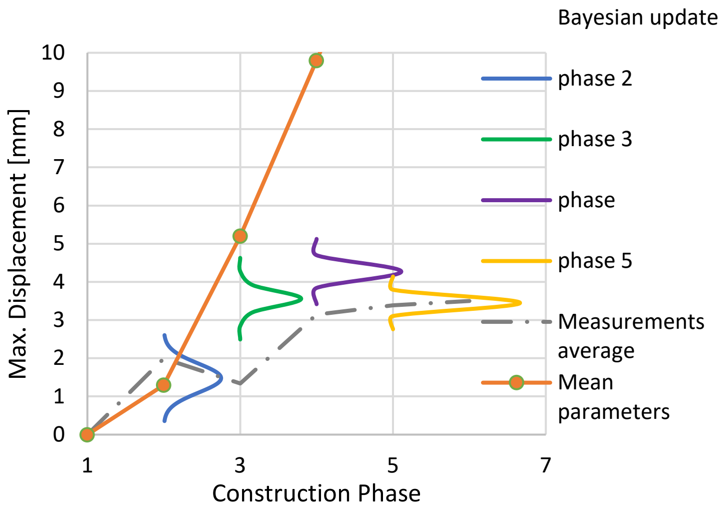


Figure 6.15 Overview of different Bayesian updates and measurement sets.

4.Calibration

Based on the sensitivity analysis the parameters 1A.k and 2A.k have been rejected for the analysis because of their low sensitivity scores. As previously mentioned, the special interest is in the soil parameters of layer 2B.

It was decided to reject the Bayesian update of phase 2. This decision is contributed as from calibration of phase 2 no significant results were found for the strength parameters other than 1A.φ (Table 6.6). Originally, the 5% characteristic value of the friction angle of 1A was assumed to be 30°. The calibrated friction angle corresponds to its lower tail as the prediction of DsheetPiling was less than measured by the inclinometers (Figure 6.16a). This result is in contrary with the other calibration results.

Calibration of phase 3 and 4 resulted in clear convergence towards higher strength properties of layers 2A and 2B. This was expected as the Bayesian update converged towards the measurement set – representing more favorable displacements than assumed by the DsheetPiling predictions.
Most of the information on parameter 2A is obtained in the calibration of phase 3. This is in accordance with its score in the sensitivity analysis. Because of the lower sensitivity score for phases 4 and 5, the parameter is not strongly changing anymore after phase 3 (Figure 6.16 b).
The calibration of the friction angle of 2B ends up closely to its 95% tail. The results for the OCR are somewhat inconclusive, fluctuating around the mean. The reason for this could be that although in each phase the OCR scored on the sensitivity analysis, other parameters were more prominent.
Numerical calibration results are presented in Table 6.7.

	Calibrated Phase 2			
Parameter	1A φ	EI	2B φ	2B OCR
μ	28.8	88026	33.2	3.5
V	4%	25%	11%	9%

Table 6.6 Calibration results phase 2.

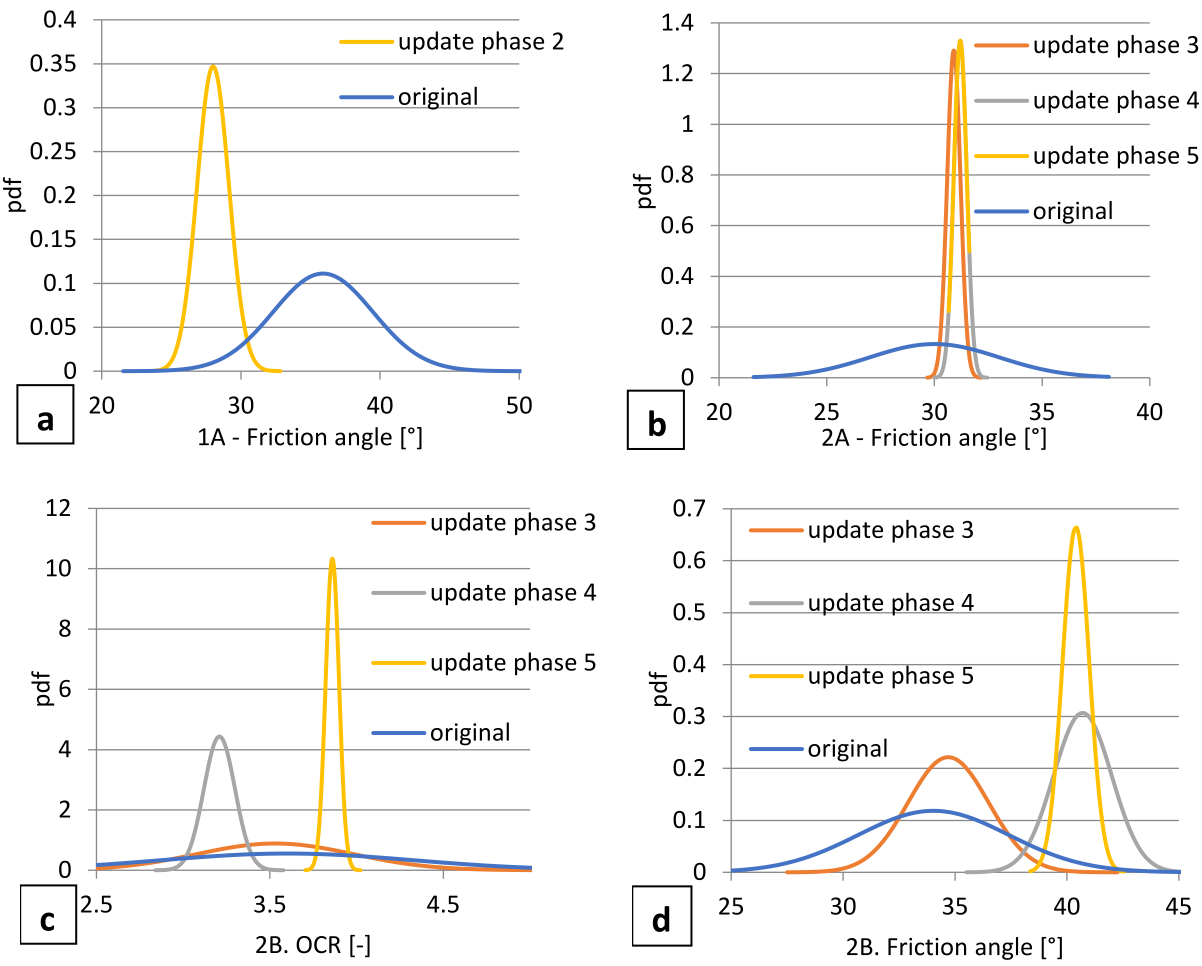


Figure 6.16 Distributions of calibrated soil strength parameters.

Original distributions					
Parameter	2B φ	2A φ	EI	2B OCR	2B k
μ	33.5	26.9	87500	4.0	9.00E3
V	10%	10%	25%	20%	20%
Calibrated Phase 3					
μ	34.7	30.9	110228	3.2	14.0E3
V	5%	1%	12%	3%	13%
Calibrated Phase 4					
μ	40.7	31.2	173817	3.9	24.5E3
V	3%	<1%	7%	<1%	10%
Calibrated Phase 5					
μ	40.4	31.2	191906	3.6	50.5E3
V	1%	<1%	2%	1%	5%

Table 6.7 Results of parameter calibration.

Relationship EI and 2B strength parameters

Calibration in phase 3 already indicated a possible correlation between the *EI* and the strength modulus of subgrade reaction of layer 2B. This was also found in phase 4. The blue dots in Figure 6.17 represent all the realizations that correspond to the same value for the maximum displacement (with an absolute error of 0.1 mm). This result is not surprising given the soil-structure interaction of the system as presented in Figure 6.8. Formula (33) describes the trendline of Figure 6.17.
Because the variability of *EI* has been restricted to 25% of its original value, the *EI* stagnates around 19E4 kNm^2/m, its upper tail in the calibration of phase 5. As a results, the modulus of subgrade reaction 2B.k is further increased. The calibrated value of 50E3 kN/m^3 is about 5 times higher than originally assumed (Table 6.7).

$$EI = -4 * 2B.\mathrm{k} + 2.1\mathrm{E}^5 \quad (33)$$

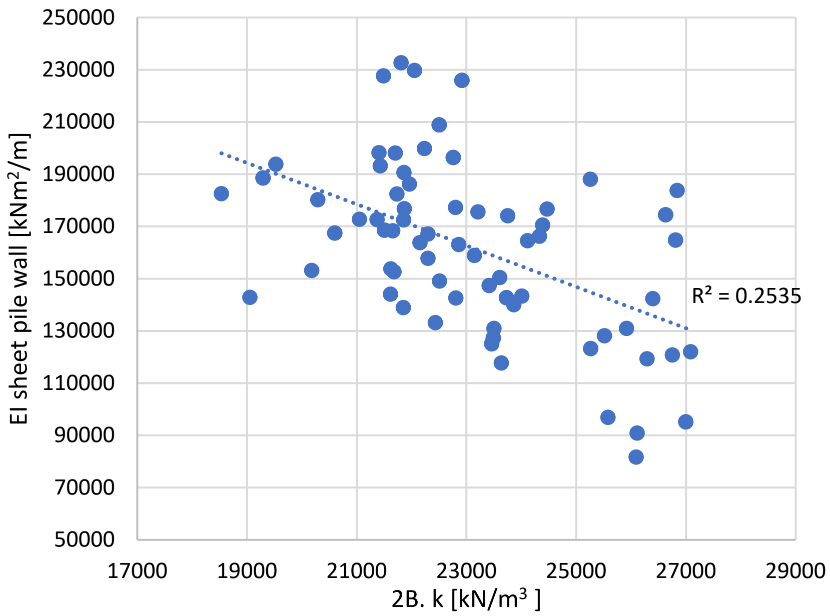


Figure 6.17 Realizations of EI and 2B.k1 for calibration to Bayesian update phase 4.

5.Forward prediction

In phase 5 the Bayesian update converged towards its measurement set. The calibrated parameters of this Bayesian update should thus be the solution to the complete measurement set. A forward model prediction with the calibrated parameters of phase 5 indeed shows a fit as presented by the green line in Figure 6.18.

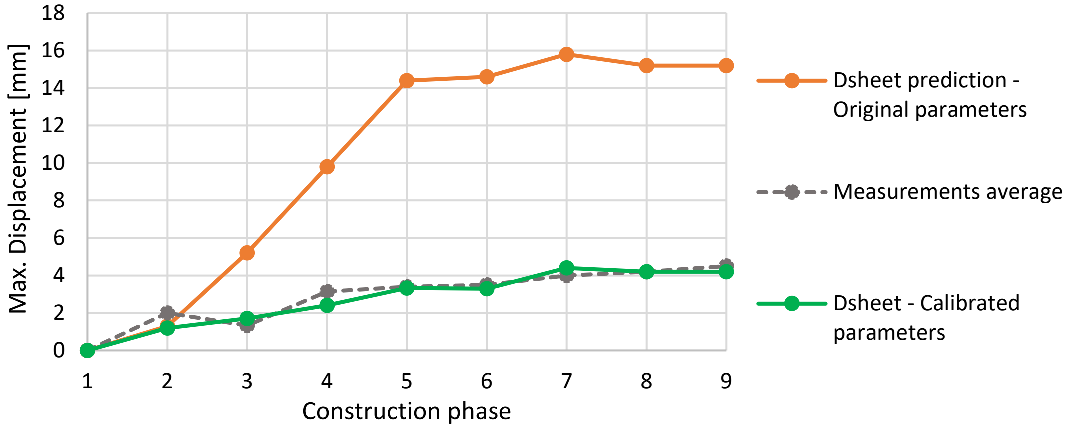


Figure 6.18 Forward model with calibrated parameters [phase5].

Another fit is presented by the purple line in Figure 6.19. From the trendline of *EI* and 2B.k by Formula (33), a solution could be found if *EI* was held constant to its original value. This means however a drastic increase of the modulus of subgrade reaction to 2.6E5 kN/m^3, about 29 times higher than its original value. This could indicate that the sustained reaction pressure of the soil is much higher for MIP walls. This might be a result of the soil mixing: Loads from the wall might be better distributed to the soil skeleton.

Overall, the fact that the wall displacements kept below 5mm could be explained by:

a) A more favorable distribution of loads to the soil skeleton by the MIP wall.
b) A more favorable distribution of loads within the structure that cannot be modelled accurately in DsheetPiling.

To verify b) the calibrated parameters of phase 5 were used as input for a new PLAXIS 3D simulation (Figure 6.19). The calibrated parameters did not represent a solution for this model. The differences between DsheetPiling and PLAXIS 3D should be noticed: PLAXIS 3D does not define the modulus of subgrade reaction as a parameter. Also, in PLAXIS 3D the MIP wall is modelled as a plate structure that might not capture additional beneficial load distributions from the wall to the soil. Hence, the above explanation of the measured wall displacements stays a hypothesis for now.

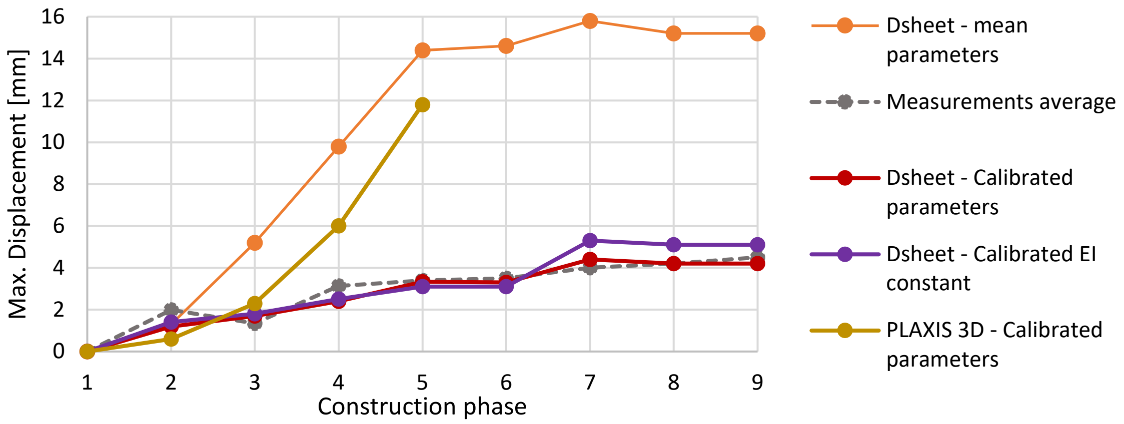


Figure 6.19 Fit of forwards model with calibrated parameters [phase 5] and solution if EI is no variable.

6.3 OM design

To investigate the cost potential of the OM Ab Initio approach, an OM design is proposed for this case study. The OM design is established in accordance with steps 1 and 2 of the CIRIA guideline. Again, the displacement restrictions are valid as stated in Table 6.3.
At first a choice is made on the Sheet pile wall profile. In the executed project the choice for the Mixed in Place wall was led by the concern to encounter boulders during installation. The selection of this wall was not a function of different soil parameters. For this reason, the same wall is adopted for the OM design.
Regarding the planned basement floors, the locations of the struts and the installation depth could not be adjusted. This leaves that the only adjustment could be made for the number of struts. The original, characteristic design contained 3 struts (Figure 6.6). A calculation based on most probable (mean) soil conditions indicates that the complete second layer of struts could have been saved. Therefore, Figure 6.20 presents the OM design.

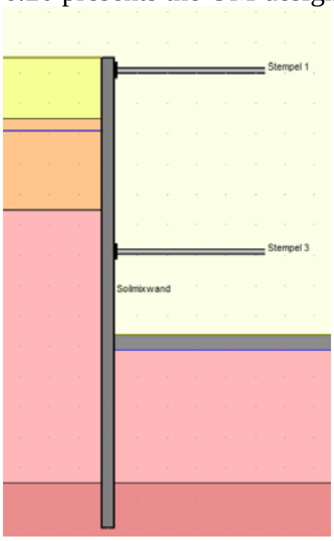


Figure 6.20 OM design with 2 layers of struts.

If this OM design is applied the 25mm restriction will just be met. However, in case characteristic soil conditions are applicable the restriction gets violated in phase 7 – removal of the third strut layer. This can be seen from the red displacement prediction in Figure 6.21. Therefore, during the excavation stages, the monitoring data should be processed via de Bayesian update to indicate whether the second layer of struts is still needed. Given the

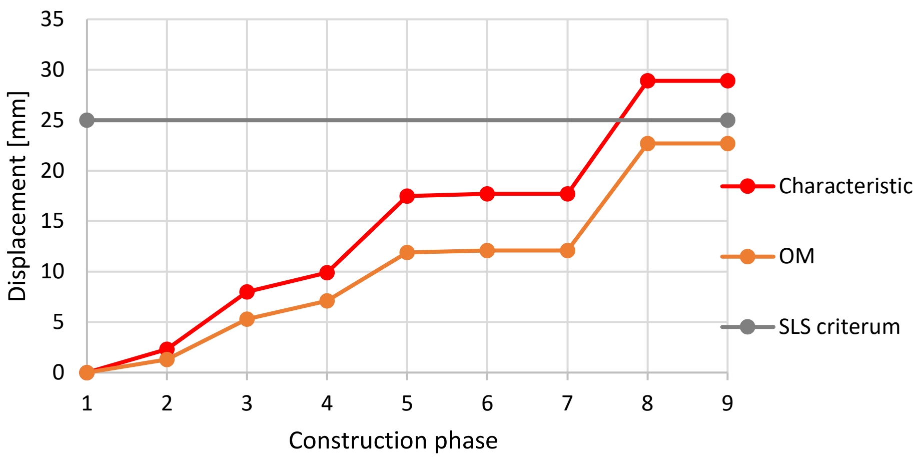


Figure 6.21 SLS design chart for prosed OM design.

6.4 Conclusions on Case study 1

From the results described in the previous sections, the following answers are formulated on the questions of interest:

Does the Methodology as presented in Chapter 5 work for this case study?

The Methodology showed a successful application of the Bayesian update. The following statements are valid on the performance of the Bayesian update:

Statement 1: As seen in phase 2, the DsheetPiling displacement prediction was less than that of the measurement set. This is contradictive with what is found for the consequential phases, leading to the suspicion that this deviation is caused by either a modelling or a measurement mistake.
An explanation for the deviating measurement can be explained as follows: Typically, measurements on wall movements start once excavation starts. However, it needs to be realized that with installation, soil gets disturbed. Therefore, the way the initial stress situation is modelled by DsheetPiling is inaccurate, leading to underestimated soil displacements. Past project showed that due to installation of a sheet pile wall (and other structural components, such as tension piles) already a significant part of total surface movements can take place [29]. Installation effects should therefore be considered. Especially for the MIP wall these installation effects are still somewhat unknown.
By applying a Bayesian update, this suspected inaccuracy of the model is not a big problem: A rather large measurement error is considered as this was the first measurement. Therefore, the first Bayesian update was actually rejected. However, it needs to be realized that, according to the above hypothesis, it is actually the model error instead of a measurement error.

Statement 2: In the calibration of phase 2 the lower friction angle found for 1A is incorrect as it is based on a false Bayesian update. This is a motive for the statement that <u>the results for the calibration are only as good as the model</u>. With this it is meant that wrong results will be derived if a Bayesian update is based on a model that is not able to accurately predict the state at a certain construction phase.

Statement 3: It was chosen to reduce the inclinometer error per construction phase. If it was not chosen to reduce this, convergence might not have been reached. The analysis of the measurement as done in section 6.2 is biased in the sense that now that construction is completed, the course of the measurements is known.
In the moment of construction, it might have taken more than 2 measurement sets in order to realize that maybe the inclinometer error is too big. <u>This emphasizes the importance to use case studies with real measurement datasets to establish an unambiguous way of when to apply certain errors in the Bayesian update.</u>

Statement 4: Although a representative parameter set could be found, the wall displacements could not be explained with evidence. It is unfortunately unknown what the true strut-forces were as they were not monitored. This emphasizes the need to approach the OM in an integral way and to maybe use more extensive computer models and algorithms to fully understand the wall behavior.

Statement 5: The traffic light system would also have been suitable. This is simply because the measurements showed very low displacements in all phases (except for phase 2). The additional value of the back-analysis is primarily in the ability to determine a set of soil parameters from which safety status can be determined. It needs to be emphasized that the calibrated results are not truly representative soil parameters: The results can be used to derive safety definitions as they find a new solution in DsheetPiling.

Regarding the potential of the OM Ab Initio approach the following statements are made:
Could significant savings been made if the OM Ab Initio approach had been used?

The OM design has been presented in section 6.3. As it turns out for the concerned project, the stratigraphy did not allow for savings in the type of sheet pile wall. Also, as locations for the struts were fixed because of the planned installation of the basement floors, only the second layer of struts could have been left out. These costs savings are still significant. Regarding the measurements as they actually turned out in the project, even more savings could have been made.

Has the OM Ab Initio approach additional value once unforeseen events occur?

Although it could not be explained, the unexpectedly low deflections of the wall could be seen as a positive unforeseen event. As from the above results, the OM Ab Initio approach would have had additional value as a layer of struts could be saved. Even more savings could have been made by applying progressive modifications, for example by speeding up the excavation process. It needs to be emphasized that this conclusion is only true for cross-section 4.

6.5 Case study 2: Amsterdam

This chapter presents the analysis of monitoring data, collected by the construction of a 2-layered basement in the city of Amsterdam. This case was selected because of its challenging project circumstances: The building pit was located very close to the neighboring houses. The houses have old wooden pile foundations which made them vulnerable to the construction works. The Engineers had the responsibility to fully control the displacements in and around the building pit. They needed to communicate actively with the stakeholders and timely intervene when limits were about to get exceeded. An extensive monitoring system was setup to facilitate their job.

Project description

The location of the building pit is marked in Figure 6.23. The soil stratigraphy is depicted in Figure 6.24. As typical for Amsterdam, a thin first layer of sand is encountered, followed by a soft layer of clay and/or peat. Deeper, the sand layer is located that serves as the foundation of the 2-floor basement.

The project has the following characteristics:

- Building pit:
  Area: 20x28.2m^2
  Bottom of the basement floor: -7.25m NAP.
  1st floor of basement: -4.1m NAP.
  The desired basement is almost as big as the building pit, limited space is available for storage.
- Total excavation depth: 10m.
- Distance neighboring house on the left: 1m
- Distance neighboring house on the right: 6.5m

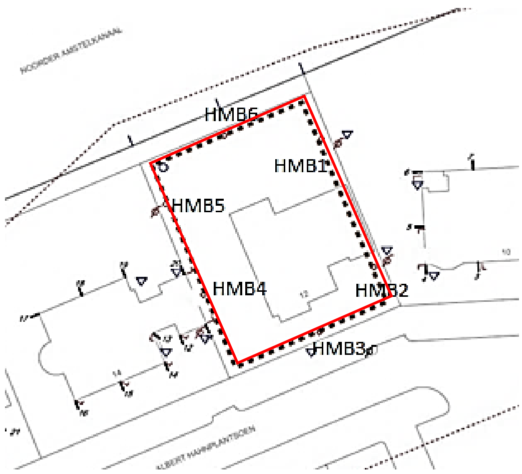
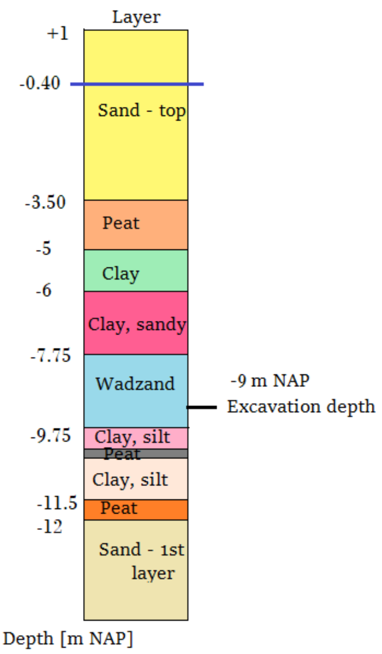


Figure 6.23 Top view of planned building pit



#	Soil layer	Description
1	Sand – top layer	Top layer
2	Peat	Cohesive
3	Clay	Cohesive
4	Clay, sandy	Predominantly strong
5	Wadzand	Serves as draining layer, moderately strong
6	Clay, silty	Moderately strong, slightly cohesive
7	Peat	Slightly preloaded
8	Clay, silty	Moderately strong, slightly cohesive
9	Peat	Slightly preloaded, strong cohesive
10	Sand – first layer	Serves as foundation

Table 6.8 Description of the soil layers.

Figure 6.24 Stratigraphy

Stratigraphy
The initial soil investigation has been performed by the contractor. The investigation exists of 1 standpipe, 1 boring by hand till a depth of 3 meters and 6 CPT's till a depth of 30 meters. The soil parameters were determined conform NEN 9997-1:2016 [31] and are stated in Table 6.9.

Layer	(1) Sand, Top layer		(2) Peat		(3) Clay	
	50%	5%	50%	5%	50%	5%
γsat [kN/m^3]	20	20	11	11	14	14
γ_{unsat} [kN/m^3]	18	18	11	11	14	14
φ [°]	35.9	30.0	19.5	15.0	21.0	17.5
δ [°] Kötter	27.5	27.5	0*	0*	18.5	15.0
c [kPa]	-	-	7.5	5.0	7.5	5.0
Modulus of subgrade reaction						
50%	9.00E+03	6.00E+03	1.50E+03	1.00E+03	2.25E+03	1.50E+03
80%	4.50E+03	3.00E+03	7.50E+02	5.00E+02	1.13E+03	7.50E+02
100%	2.25E+03	1.50E+03	3.25E+02	2.50E+02	5.63E+02	3.75E+02

Layer	(4) Clay, sandy		(5) Wadzand		(6) Peat/clay	
	50%	5%	50%	5%	50%	5%
γsat [kN/m^3]	16	16	20	20	13.5	13.5
γ_{unsat} [kN/m^3]	16	16	18	18	13.5	13.5
φ [°]	29.9	25.0	35.9	30.0	19.7	16.5
δ [°] Kötter	27.4	22.5	27.5	27.5	17.2	14.0
c [kPa]	-	-	-	-	6.0	4.0
Modulus of subgrade reaction						
50%	3.00E+03	2.00E+03	4.50E+03	3.00E+03	1.88E+03	1.25E+03
80%	1.50E+03	1.00E+03	2.63E+03	1.75E+03	9.38E+02	6.25E+02
100%	7.50E+02	5.00E+02	1.13E+03	7.50E+02	4.69E+02	3.13E+02

Table 6.9 Parameter input.

Limit values for sheet pile wall deformations		
	Max. displacement along wall [mm]	Max. top displacement [mm]
Phase 3 – Full excavation	21	5
Phase 6 – Dry pumping	28	5
Phase 7 - End	30	48

Table 6.10 Requirements on sheet pile wall deformations

Building pit execution

According to strict demands for the sheet pile wall displacements stated in Table 6.10 the sheet pile wall profile 37-700 has been selected (Figure 6.25). It was chosen to maintain a high water level in the building pit during the excavation phase as this is favorable for the distribution of forces. Table 6.11 described the construction phasing with wet excavation.

During preparations for the construction works, neighbors strongly experienced noise and hinder from vibrations. This caused a negative view towards the whole project. It was therefore crucial for the Engineers to limit negative effects to neighboring features and to communicate actively. Especially the management of settlements has been a difficult task because of the small distance between the building pit and house on the left side, which is 1m. For that reason, monitoring has been actively used to verify deformations. The strict sheet pile wall displacements in Table 6.10 were formulated in order to act proactive to avoid damage. 6 inclinometers (indicated with " HMB") were installed along the 4 different sides of the building pit as depicted in Figure 6.23. A main concern in the project was the top displacement that would occur after the strut is removed in construction phase 7.

#	Phase	Date in between previous measurement
0	Installation sheet pile wall and tension piles	-
1	Excavation (1/3) of total depth (wet excavation)	
2	Excavation (2/3) of total depth (wet excavation)	7 days
3	Excavation till final depth	14 days
4	Pouring under water concrete (UWC) + hardening	>2 months
5	Half pumped	>4 months
6	Fully pumped	7 days
7	Construction 1st layer of basement, removal of strut.	> 7 months

Table 6.11 Construction phasing.

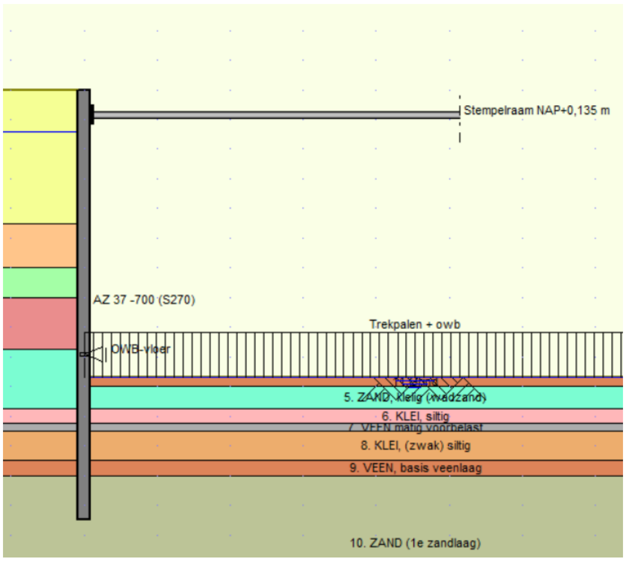


Figure 6.25 Implemented sheet pile wall design.

6.6 Analysis – Interpretation of measurement set

Before starting with the Bayesian update, a choice will be made on the measurement set. The data from the inclinometers will be compared with DsheetPiling model predictions. These predictions may be very inaccurate once surface loads need to be taken into account. Therefore the 3 inclinometers are selected at those locations no surface loads were present: HMB1, HMB2 and HMB5 (Figure 6.23).

This results in a measurement set with the following side nodes:

- The average of the 3 datasets will be the mean of the measured displacement distribution.
- Heterogeneity between HMB1, HMB2 and HMB5 is present and the effects are in the variation between the measurements, that will be taken into account as inherent standard deviation of the measurements.

The stratigraphy requires more computational effort than needed for Case study 1 simply because there are more soil layers. To minimize computational effort, the problem is analyzed with the use of a sensitivity analysis to see if stratigraphy could be somewhat simplified. It results of this sensitivity analysis showed that the cohesion of the clay and peat layers did not have a significance contribution to displacements. Below layer 5 thin layers of clay and peat alternate. In the sensitivity analysis, the influence of the parameters of each layer was lower than 5%. Therefore, they are accommodated in a substituted layer: Layer 6.Clay/peat, with parameters as stated in Table 6.9. This results in the stratigraphy depicted in Figure 6.26.
This new model with simplified stratigraphy is compared with the original model. The maximum error between the two is less than 2.5%. Also, the error between strut forces differed with only 5kN/m, which is regarded as being neglectable.

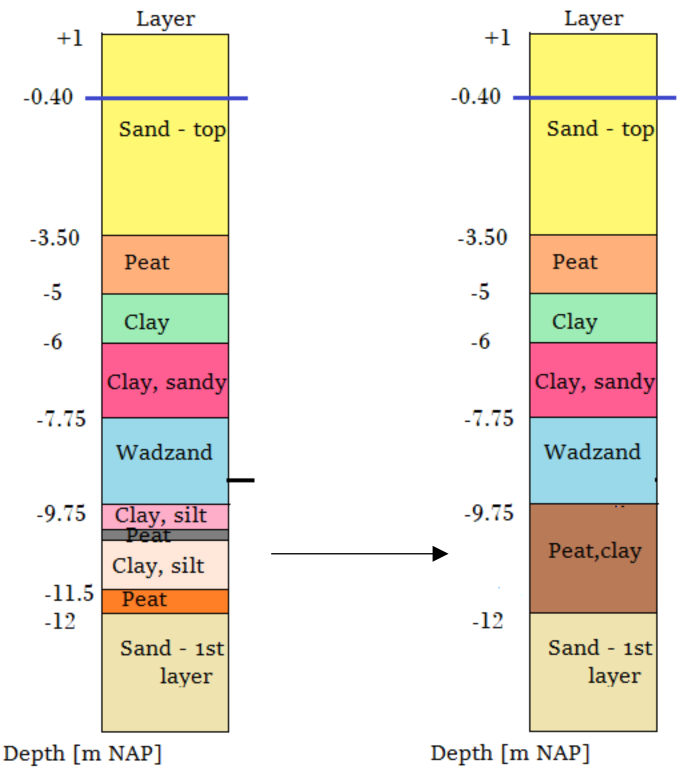


Figure 6.26 Simplification of soil-stratigraphy

Uncertainties: Installation effects

In phase 0 the sheet pile wall was installed, followed by the tension piles. It was then noticed that on every side of the building pit the sheet pile walls gave a "negative" deflection - a movement outwards the building pit. The hypothesis is that the installation of the piles caused significant radial pressures on the sheet pile. This was a highly unexpected event that already happened from the start, even before the first inclinometer measurement was taken in phase 1. In this case the Engineers were supported by the monitoring data collected on surface settlements (behind the sheet pile wall) and movements of the adjacent building. In the project execution this unexpected event gave the necessity to take preventive measures to the adjacent building on the west side. It was chosen to adjust the foundation of the adjacent building on the west side.

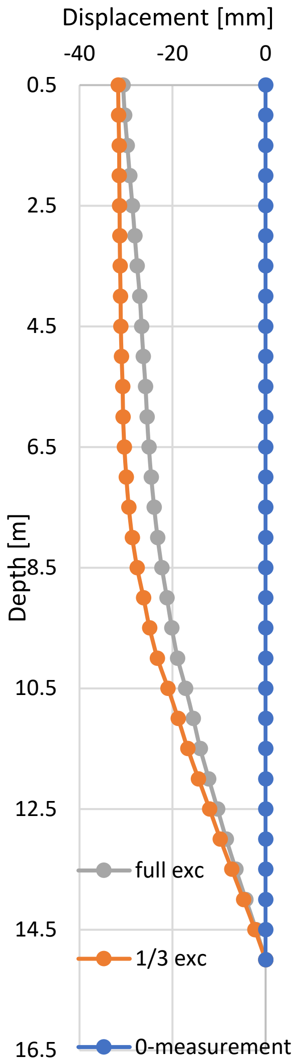


Figure 6.27 Measurement data of inclinometer 1 for excavation

Although it had its effect on the position of the sheet pile wall and the positioning of the strut, it can be seen that once excavation continued the profile deflected back towards the building pit (Figure 6.27). The positive side of the unforeseen event is that further sheet pile wall displacements inwards would probably not affect adjacent buildings anymore. However, it is still necessary to check this as soil stresses and the sheet pile wall position are different after the event. Regarding the investigation in this thesis, the unforeseen event gives the opportunity to investigate if the methodology still works after such an event.

Correction to the unforeseen event

The consequences of the unforeseen event are hard to quantify. It was a rare case: In the field it could not be assessed why it occurred or how it could be prevented. Therefore, no attempt has (yet) been made to model this event. In order to investigate whether or not the methodology still works, it is necessary to look beyond the unforeseen event. Because further measurement showed inward movements again, a new calibration point is chosen: Instead of the 0-measurement, the data will be corrected to the first measurement (taken at 1/3 of the total excavation depth, phase 1). By this correction deformations towards the inside of the building pit are further analyzed.

The consequential inward displacements are plotted in Figure 6.28. It can be seen that the measurement set somewhat fluctuates among the 2 different DsheetPiling predictions. In the next sub-section some explanations are given for this.

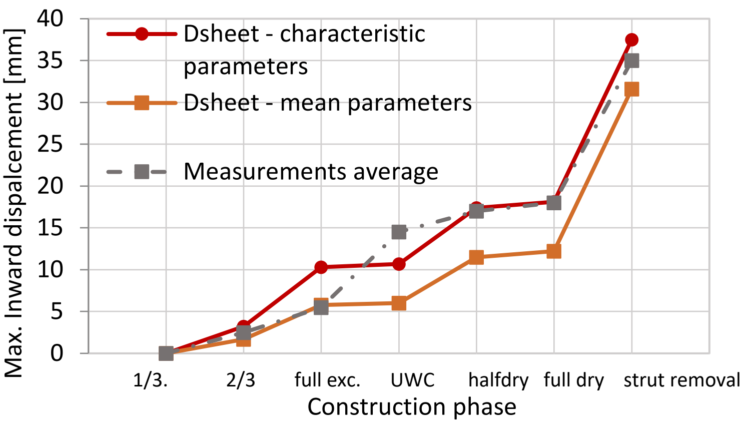


Figure 6.28 Forward DsheetPiling models on the movement inwards building pit.

Undrained behavior

From Figure 6.28 it can be seen that for the excavation stages the measurement coincide with the DsheetPiling prediction based on most probable soil conditions (orange line). In the phase that Under water concrete (UWC) is poured, deflections strongly increase. No model predicted this as the added load is limited for the 1.2m thick concrete. Therefore, it is investigated if the measurements for the excavation stages are influenced by undrained behavior. This could be the case if measurements are performed as soon as the phase has been completed and fully drained conditions are not yet reached. Also, the stratigraphy does not allow fast drainage of water. The effect of time-dependency is related with the speed of the excavation works which is not exactly known. Also, it is unknown what the exact time was in between finishing excavation and the moment the measurement is taken. Only an indication could be derived from the time in between two consequential measurement. These estimations are stated in Table 6.11.

With the information on hand both drained and undrained PLAXIS 2D simulations are performed, from which the results are presented in Figure 6.29. In both PLAXIS models the characteristic 5% parameters are used. The drained PLAXIS simulation (the light blue line) should coincide with the characteristic DsheetPiling simulation. This is true until the last 2 phases.

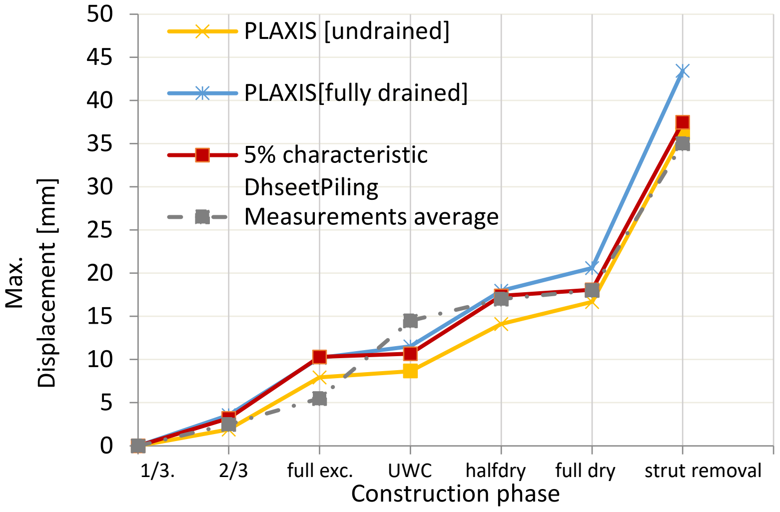


Figure 6.29 Results of drained and undrained PLAXIS simulations

Although exact data on excavation speed is unknown, the estimation of undrained behavior by the PLAXIS model indeed gives an indication of a time effect. Additionally, it should be considered that the structural response of an oblique sheet pile wall is different and that there is possibly an error in the modelled stress-state of the soil.

One important additional bias is that there is a possibility that measurements are wrongly normalized: The magnitude of the other measurements are determined by the magnitude of the measurement taken at 1/3th of excavation depth. If this measurement is also actually belonging to undrained behavior, it influences the true magnitude at drained behavior. This could be an additional reason why a relatively high measurement is obtained at the UWC-phase. However, the measurement set cannot be adjusted.

Water level uncertainty
Another source of uncertainty is in the water level that is maintained during wet excavation. This is in DsheetPiling set to +1m NAP, as high as possible. However, it is likely that this level was actually lower as the top of the sheet pile wall was at the same level. Naturally, water levels fluctuate during execution, but no data is available. A variable water level is considered in ULS calculations, however, it also effects the SLS to some extent. For a water level 30cm lower this effect is +- 2mm as shown in Figure 6.30. This might be small but equals 15-10% when looking at measurements 4 (UWC) till 6 (end of dry pumping).

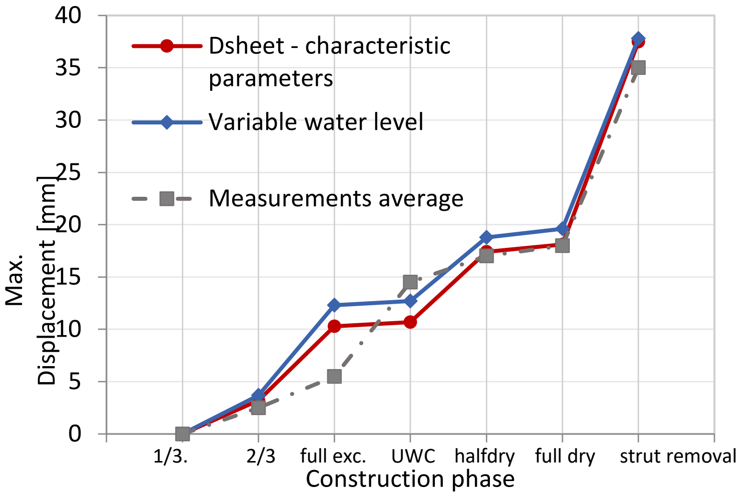


Figure 6.30 Influence of a variable water level on predicted displacements.

6.7 Analysis – Bayesian update

The above-mentioned uncertainties will be considered in the Bayesian update as follows:

1) The measurements 1 to 3 – the excavation stages - are susceptibly influenced by undrained behavior. Therefore, no Bayesian update is performed as calibration would lead to misleading results. This means that the Bayesian update will start at measurement number 4, the UWC (although this measurement is believed to be biased too).

2) The error for water level uncertainty during wet excavation will be included in the 10% overall DsheetPiling error. It will not be explicitly included, as there is no evidence for the true water level. Its influence however should be realized as being a possible explanation for the rather high UWC measurement.

With these assumptions a Bayesian update is performed for the measurement numbers 4-6 with the goal to predict the displacements for phase 7 – removal of the struts.

1.Sensitivity analysis
Results of the sensitivity analysis for measurement number 4 till 7 are summarized in Figure 6.32. For further interpretation the sensitivity analysis of the UWC phase is split into 2 components in Figure 6.31. It can be seen that the friction angle takes the biggest part of the total contribution to displacements. This is also the case in the other phases, as can be seen in Figure 6.32. The higher scores for the friction angles are linked with the stress-displacement tree that is used in DsheetPiling (section 3.1). For instance, already in the UWC phase, an active stress state is reached throughout layer 4 Wadzand. This causes a 0% score for the modulus of subgrade reaction of layer 4. The higher score of the friction angle φ is also in the correlation with the wall friction angle δ.
For the UWC and dry pumping, the parameters of the lower layers – layers 4 and 5 – score higher.

Only in phase 7, which concerns the removal of the strut, the strength parameters of the upper layers become more important.
Overall, the cohesion scored low. Only in phase 7 the cohesion of layer 2 scores about 5%.

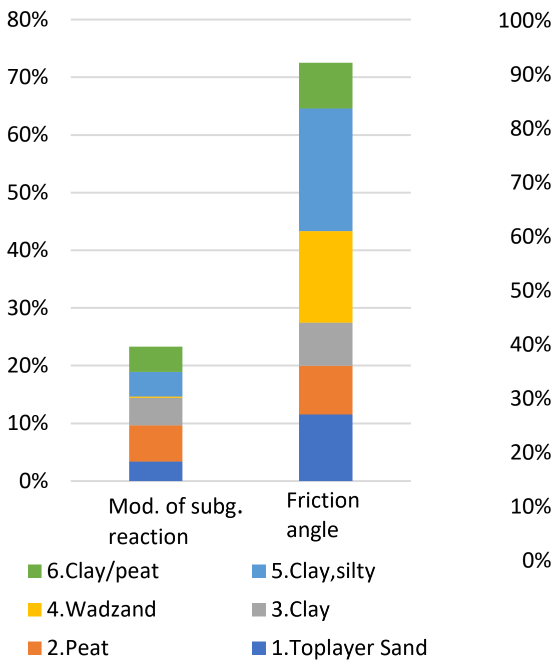


Figure 6.31 Sensitivity analysis for phase 4 - UWC.

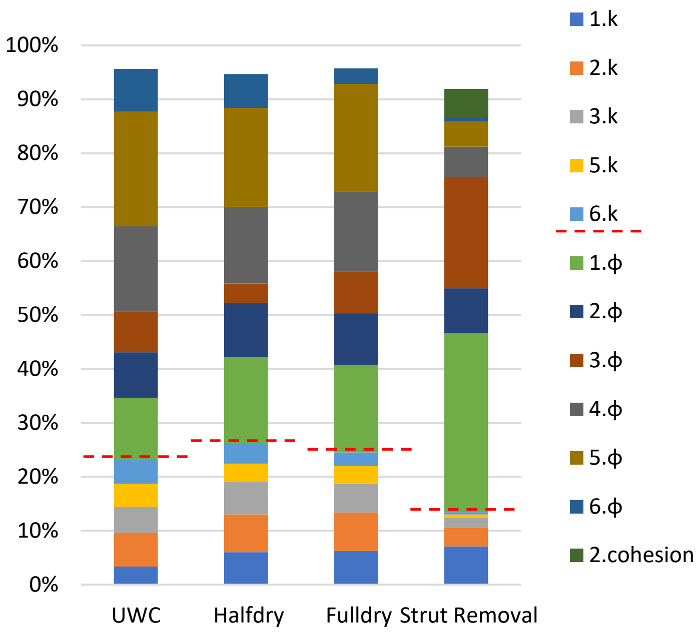


Figure 6.32 Sensitivity analysis phase 4-7.

To predict displacements for phase 7 more insights should be gained in the friction angle of layer 1 and layer 3. However, it can be seen that the max. sheet pile wall displacements of construction phases 4 to 6 are primarily determined by strength parameters of layers – 4 and 5. Due to the extensive layering a lot of different parameters each contribute a little to the DsheetPiling outcome. With many small contributions of parameters not much information can be gained. Also, given Figure 6.28 of the previous section, it seems that during the dry pumping and strut removal the retaining wall displacement is close to the DsheetPiling prediction based on 5% characteristic soil parameters. Therefore, the following assumptions are made to ease up the problem:

- Based on the sensitivity analysis, all the moduli of subgrade reaction k together score less than 15%. Therefore, it is chosen to fix the moduli of subgrade reaction to their characteristic value.
- Based on their relatively low scores, 2.φ, 2.c and 6.φ are restricted to their 5% values as well. Based on their scores in the phases UWC – Full dry, valuable information could be gained on 5.φ, 4.φ and 1.φ
- 3.φ had a high sensitivity score for phase 7. Although its lower score in the phases UWC – Full dry it is hoped that calibration in those phases can give some information on this friction angle as well.

This narrows down the “search space” by the 4 parameters of layers 5.Clay/silt φ, 4.Wadzand φ, 3.Clay φ and 1.Sand, top layer φ

Bayesian update
At first the Bayesian update for phase 4 – UWC is presented by Table 6.12 and Figure 6.33. To be consistent with the procedure the prior distribution of phase 4 is based on mean soil parameters.

Phase 4: UWC	Prior	Measurements n=3	Bayesian update
Mean [mm]	5.8	14.5	14.0
V	30%	20% + 1.16 mm = 27.5%	15%

Table 6.12 Bayesian update phase 4.

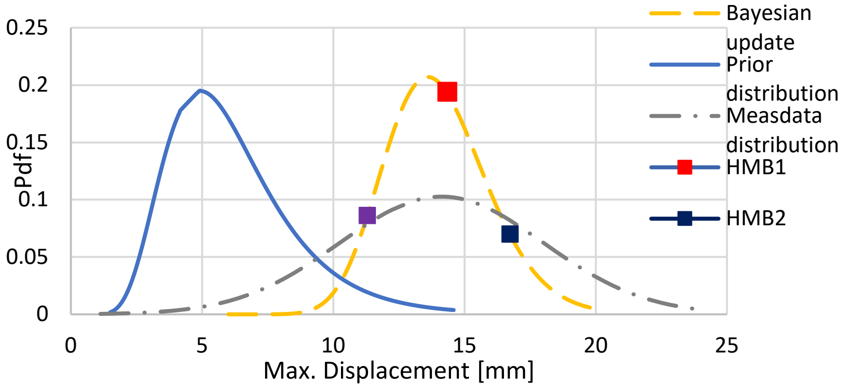


Figure 6.33 Bayesian update for phase 4: Under water concrete

Although it is suspected that in the DsheetPiling model is based on an incorrect water level, this suspicion cannot be verified. Therefore, the coefficient of variation of the prior distribution only contains the additional model error of 10%.
The coefficient of variation of the measurement set consists of the variability of the three inclinometers, which is 20%, with an additional inclinometer error of 7.5%. From Figure 6.33 it can be seen that the measurement and prior distributions strongly differ. Because there are *n*=3 inclinometers, the Bayesian update shift towards the measurements as it is basically 1 model prediction versus 3 measurements.

The consequential calibration results of phase 4 are stated in Table 6.13. The table shows that the calibrated friction angles are even lower than their 5% characteristic value. This is because the Bayesian update converged towards the measurement set that shows sheet pile wall displacements beyond the prediction with 5% characteristic soil parameters. This combination of parameters is not likely. These results are an indication that probably model errors are present.

Calibration – Phase 4 UWC				
layer	1. φ	3. φ	4. φ	5. φ
5% Characteristic value [°]	30.0	17.5	25.0	30.0
μ [°]	26.1	16.4	21.8	25.9
V	6%	3%	4%	6%

Table 6.13 Calibration results phase 4 UWC.

By adopting the calibrated values, a new prediction can be made for phase 5. In the Bayesian update of phase 5 this new prediction is adopted as the prior distribution. This prior distribution is depicted in Figure 6.34. The Bayesian update of phase 5 again converges towards the measurement data as this prior distribution overestimated the displacements.

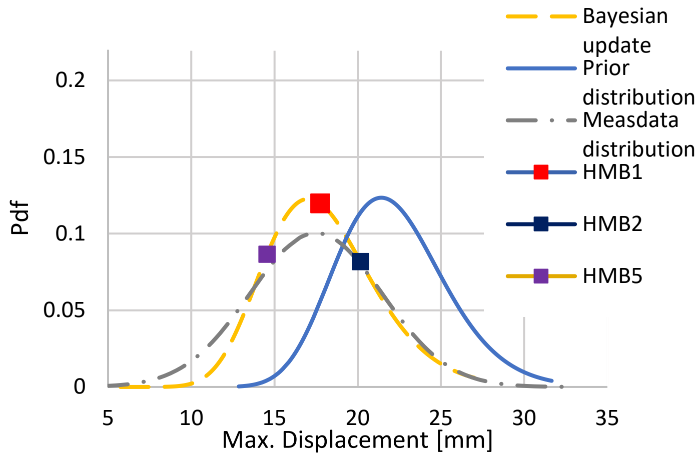


Figure 6.34 Bayesian update phase 5 – Dry pumped halfway

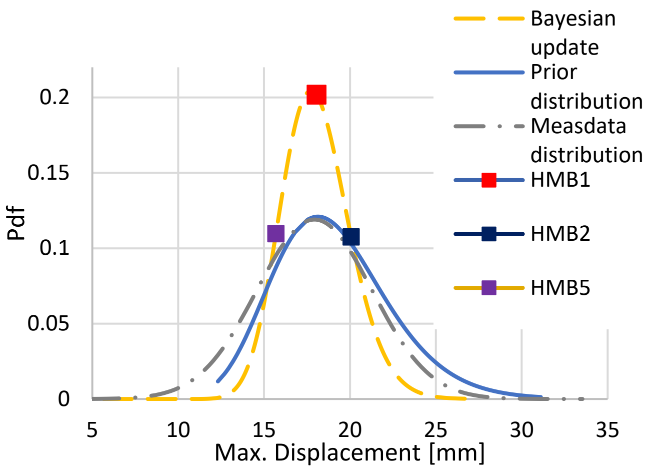


Figure 6.35 Bayesian update phase 6 – Fully dry pumped.

Finally, in the Bayesian update of phase 6 its prior and measurement distribution coincide. This is favorable to decrease the coefficient of variation of the Bayesian update from 19% in phase 5 to 11% in phase 6.

4.Calibration

Table 6.14 states the calibrated friction angles for the phases 5 and 6. The results are visualized by the graphs a to d of Figure 6.37. Compared to the earlier presented calibration results in Table 6.13 the friction angles end up closer to their 5% characteristic value in phase 6. However, the variances of 1.φ and 5.φ are still big.

It can be seen, once calibration is done for phase 5, that the spread of the friction angle of layer 3 and layer 4 reduce. This is not the case with the friction angle of the top layer 1 and layer 5. Both layers are sand, starting with the same characteristic value. Also, the layers have similar scores on the sensitivity analysis. This results in a large set of mutual realizations possible for the solution of the Bayesian update as shown by Figure 6.36.

A Bayesian update with consequential calibration can specify those parameters any better. However, a decrease in the V of friction angle layer 3 is found which is desirable to perform a forward prediction for phase 7.

Phase 5 – Half dry pumping				
layer	1. φ	3. φ	4. φ	5. φ
5% Characteristic value [°]	30.0	17.5	25.0	30.0
μ [°]	27.7	17.0	23.5	29.2
V	13%	5%	1%	13%
Phase 6 – Dry pumping finished				
	1. φ	3. φ	4. φ	5. φ
μ [°]	29.3	17.3	23.5	29.2
V	11%	>1%	>1%	9%

Table 6.14 Calibration results phase 5 and 6 – reducing amount of variables.

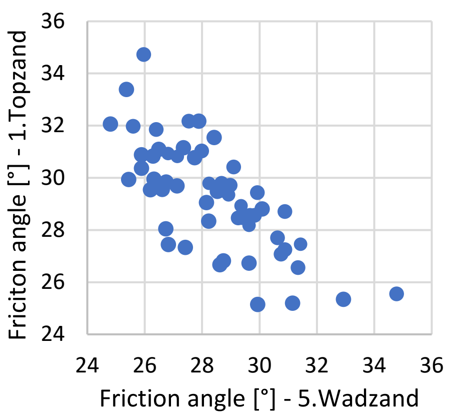


Figure 6.36 Mutual realizations found in the calibration to the mean of phase 6.

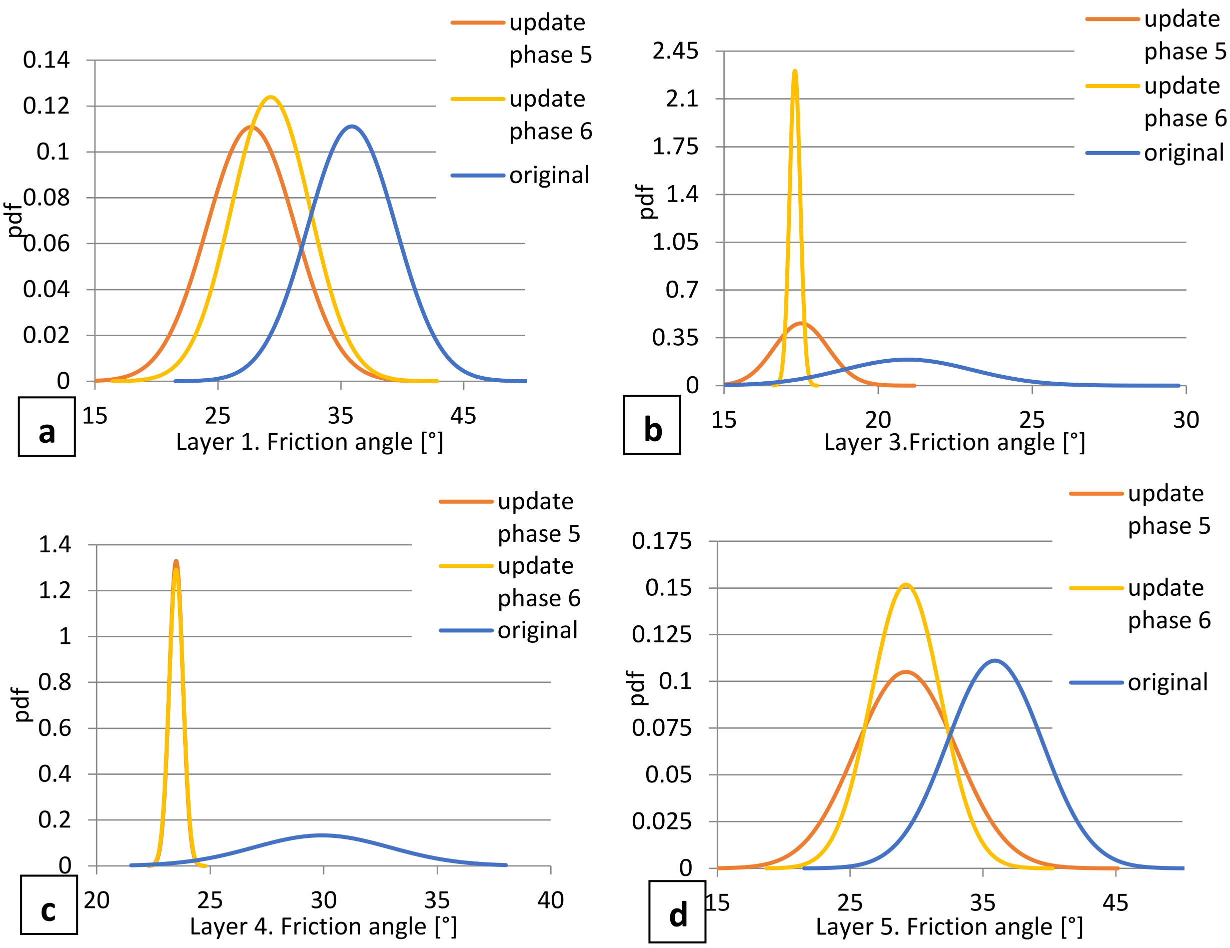


Figure 6.37 Calibrated Parameter distributions found in Bayesian update phases 5 and 6 – Dry pumping.

5.Forward predictions

Based on the results of the Bayesian updates performed in phases 4-6 (Figure 6.38), a new prediction is made on the displacements in phase 7. The coefficient of variation of the prediction equals 6.5%. This is not as high as the parametric variability found for 1. φ and 5. φ – which was about 10%. This is because of the low variation in the friction angle of layer 3, which has a high contribution to the output according to the sensitivity score in phase 7.
The prediction for phase 7 is presented by Figure 6.39 in which it can be compared with the inclinometer measurements, the DsheetPiling predictions and the PLAXIS prediction. It can be seen that the Drained PLAXIS 2D predictions is at the other side of the prediction, which can be a result of the inherent differences between this FE model and DsheetPiling.
The mean of the Bayesian update falls in between the two DsheetPiling predictions, indicating the "orange zone" to speak in terms of the Traffic light system. Not all the measurements however fall within this range: Inclinometer HMB5 stays under the mean prediction – green zone. Between the 3 inclinometer measurements that is a rather big spread. This might indicate that the effects of removing the strut is harder to predict by computer models: It concerns a new distribution of loads in between the soil and structure. As this is a different loading situation than that of dry pumping, on which the calibration results are based, the predictive power of the Bayesian update is limited. However, the Bayesian update can somewhat reduce the variability in prediction which is preferable.

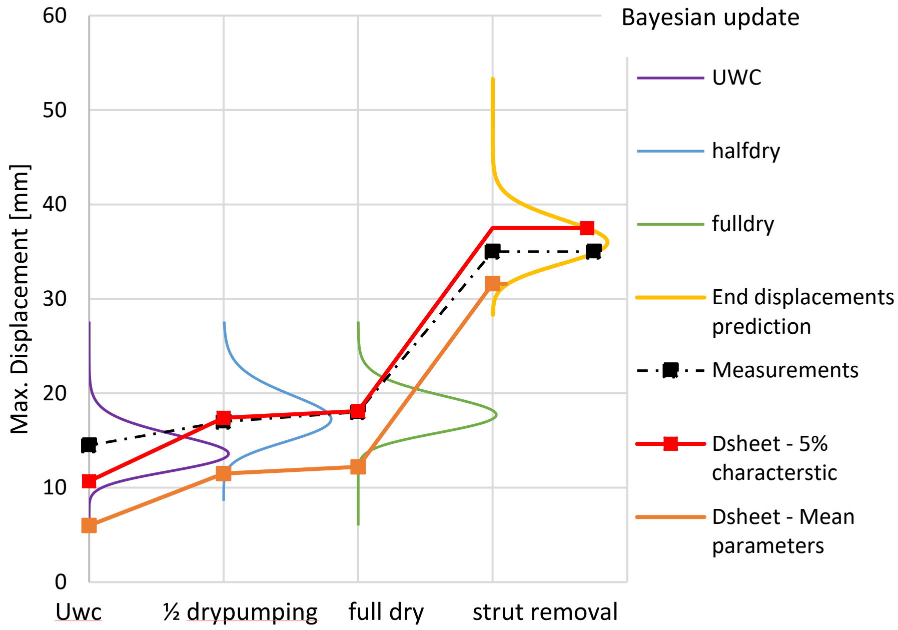


Figure 6.38 Bayesian update Phase 4-7 along with DsheetPiling predictions and measurements.

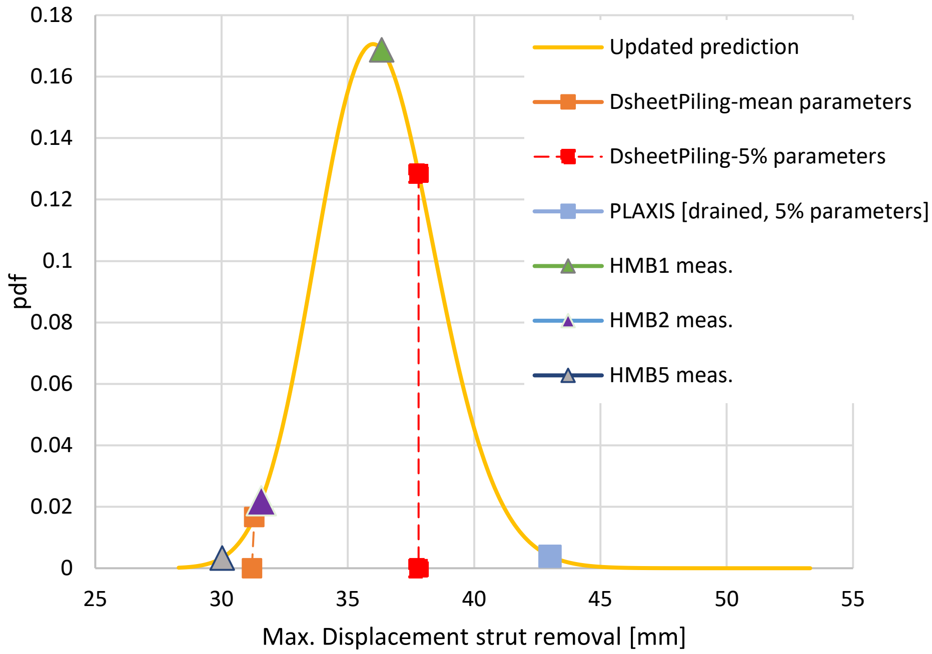


Figure 6.39 Predictions and measurements of the max. displacement of sheet pile wall in strut removal – phase 7.

6.8 OM design

In case the OM was applied from the start, a design would have been made based on both characteristic and most probable soil conditions, both meeting the requirement of 48mm and 5mm top displacement.

To follow step 1 and 2 of the CIRIA guideline an economical sheet pile wall will be selected based on the most probable soil conditions. The mean soil parameters are presented in Table 6.9.
Based on this parameter input the AZ 26-700 could be used enhanced with a strut on the same level as in the original design (Figure 6.40). The strut ensures that the top displacement is limited to 5mm in the first excavation step.

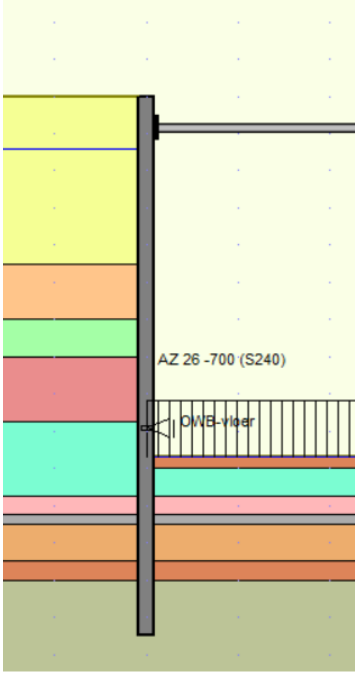


Figure 6.40 OM design enhanced with 2 strut rows.

By applying such a strict SLS criteria, the OM design meets the ULS requirements in both soil parameter sets. Just like in the actual project the main concern is in the displacements that would occur in the last phase. As can be seen from the displacement prediction in Figure 6.41 the AZ 26-700 just meets the SLS required under mean soil conditions. If 5% characteristic parameters are applicable the SLS is most likely to get violated. Therefore, a contingency measure should be designed only for this last construction phase. This is not straightforwardly done as in this last phase supports should be removed for the workability in the building pit. Perhaps some extra temporary support could be provided by purlins or other framework that can be attached to the top of the sheet pile walls. A decision for these measures should be made during the dry pumping phase.

From what was seen in the previous section, the model prediction of the last phase might not be that accurate. Therefore, the predictive power of the Bayesian update is only limited. However, it would be possible to decrease the variability in the prediction. Additionally, more research could be done to improve the model accuracy on the last phase. For example, more case studies on the same problems could be analyzed to estimate the probability that DsheetPiling would underestimate those movements. By taking care of the model accuracy and measurement-processing, there is definitely the potential to save structural costs. The applicability of the OM design depends on the appetite of the project team take the risk to implement this AZ 26-700.

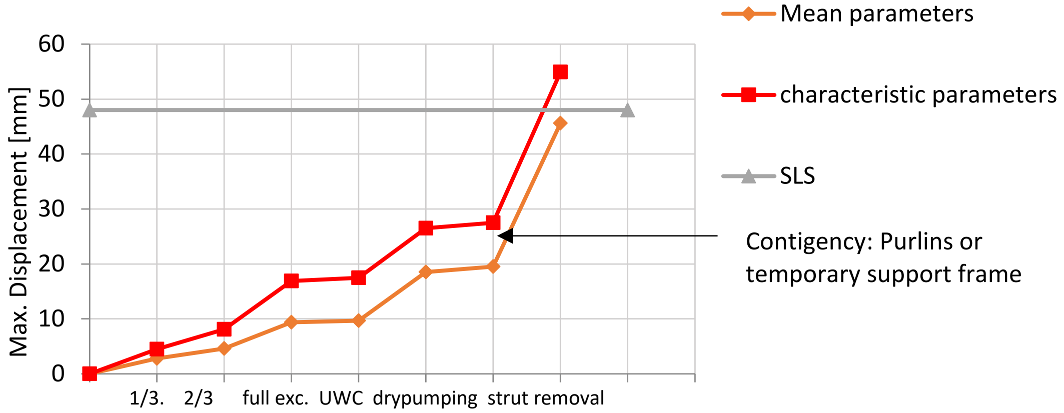


Figure 6.41 OM design with AZ 26-700 and strut +0.135m NAP.

6.9 Conclusions on Case study 2

To formulate conclusions on case study 2 again it is started with answering the question:

Does the Methodology as presented in Chapter 5 work for this case study?

As extensively discussed, the measurement data set was hard to interpret. A Bayesian update could be applied, however, with a reduced measurement set. For this aim, the extensive stratigraphy could be reduced to find parameters, suiting the structural behavior during dry pumping. Consequently, for phase 7 a prediction could be made. 1 inclinometer suited the prediction. The other 2 inclinometers fell on the lower tail of the predicted displacement distribution. Regarding the method, it can be stated that it has added value to reduce the variability of the prediction. However, care should be taken as probably the DsheetPiling model could not accurately predict phase 7.

The following statements complement statements 1 to 5 of the previous case study:

Statement 6: A more extensive stratigraphy requires more measurements from different construction phases in order to get conclusive results. In this case study an inconclusive relationship was found between 1. φ and 5. φ. This, because their properties and sensitivity scores are similar. Different sensitivity scores are then necessary to find a solution. This could have been done if the measurements of the excavation phases (stage 1 to 3) were not rejected.

Regarding the interpretation of the measurement set:
Statement 7: The outward deflections that happened because of – presumably – the installation of the tension piles was unexpected for the Engineers. However, it is not uncommon in soft-soils. So far it is still difficult to quantify installation effects and to predict under when problems can be encountered. Future research, performed by Fugro at the moment, might give a better indication on this.

Statement 8: The performance of a Bayesian update strongly depends on the interpretation of the measurements. As undrained behavior is expected in soft-soil conditions, it should be detected beforehand to avoid false conclusions: Incorrect results will come from comparing undrained measurements with drained predictions. Therefore, it would be valuable other computer models as well. The PLAXIS model offers the possibility to perform undrained calculations. In order to model as accurately as possible, attention should be paid to construction time and execution.

Statement 9: In both case studies, monitoring data was actively used to verify the situation in the field. However, in order to use the monitoring data to derive the safety status of the build structure, the monitoring plan could be enhanced with regards to statement 7:

- If drained conditions are applicable, sudden increases in displacements (like in the UWC phase) are an indication that something is wrong: Either something unforeseen is happening in the building pit, the model input has been wrong, or measurement(s) are biased. Timely intervention is then necessary to assess the cause. This emphasizes the value of automatization real-time measurement processing.
- A more precise documentation on the site conditions is desirable in order to retrieve a possible bias in measurement data: As would have been valuable in this case study: Water levels and duration of construction works.

The next questions regard the feasibility of an OM Ab Initio approach:

Could significant savings have been made if the OM Ab Initio approach had been used?

An OM design was proposed in section 6.8. Based on this cross-section, savings could have been possible by the selection of the sheet pile profile AZ 26-700.
Like in case study 1, the design freedom for contingency measures was limited by practical project demands: The support of the sheet pile wall should not hinder construction works in the building pit. Also, structural savings are affected by the SLS demands. The 5mm restriction on top displacements during the first excavation phases made a strut on the top of the sheet pile wall necessary. However, the practicality of such a strict SLS restriction can be questioned given that a strut on a lower position is more favorable to limit displacements in the rest of the construction phasing.

Has the OM Ab Initio approach additional value once unforeseen events occur?

Regarding the events as encountered on the construction site it can be stated that the implemented sheet pile wall, the AZ 37-700 had not been overdesigned. In fact if a less robust wall was selected like the AZ 26-700 outward deflection could have had bigger impacts. Regardless of the sheet pile wall, the best solution was to perform the "Best way out" intervention as the Engineers had done in practice. This meant enhancing the old wooden pile foundation of the neighboring house.
Because of the extensive monitoring plan, the event could be timely detected. With this kind of unforeseen events, the OM Ab Initio approach did not have extra value. However, this does not influence the conclusion that with the OM structural savings could have been made.

7. Conclusions

Based on the benchmark and the case studies, a conclusion is formulated to answer the main question of this report. Three sub-questions were formulated to guide to an answer and will therefore be answered first.
N.B. The conclusions as presented here are valid for the research carried out with the delimitations as presented in Chapter 3.

Sub-question 1: *How suitable is the prescribed approach of the CIRIA guideline?*

The CIRIA guideline is suitable if complemented with the Bayesian update: As seen from the Benchmark the CIRIA guideline gives a clear procedure of how an OM design could be established (step 1 and 2). It also specifies many aspects to be considered in the project organization. Therefore, it is a valuable guideline. However, measurement-processing via the Traffic light system cannot be used to further **quantify** safety. The influence of errors in both the measurements and the model are not explicitly mentioned by the guideline. Regarding the conventional safety definitions and the need for real-time measurement processing, the use of the Bayesian update as proposed in this thesis is preferred over the Traffic light system.

Sub-question 2: *How to ensure safety during the application of the Observational Method?*

Safety definitions as known from Limit state design can be derived by the methodology presented in chapter 5: Throughout construction the Bayesian update can be used to create a new sheet pile wall displacement prediction based on measurements and model predictions. For both sources of information uncertainties can be considered via the standard deviation of their distributions. Calibration of the new displacement prediction allows to find the corresponding input parameters of the model, from which safety definitions can be derived. In both case studies this methodology proved its value. The outcome of the methodology depends on the used model for calibration and the quality and the amount of measurements.

Sub-question 3: *What are the benefits and pitfalls of the Observational method Ab Initio approach in the application of building pits?*

The following benefits are formulated:

a) The cost potential: There is potential for structural savings as the difference between mean and characteristic soil conditions is significant. Even in case study 1, in which project requirements limited the design freedom, the cost savings for leaving out a whole layer of struts is already significant.
b) Improved safety control: In cases where SLS criteria are difficult to meet the OM has the special value to timely detect and economically adjust the structure. Also, the need to explain the measured sheet pile wall behavior results in a thorough assessment of the conditions on the construction site and their influence on the structural behavior.
c) The active learning approach: The OM raises the awareness to model errors and emphasizes the need to understand interactions on the construction site. The aim of the Observational method to study those features can improve model predictions and is helpful for similar projects.
d) Non-economic consequences of the Observational Method: The benefit mentioned under b) results in social trust in the project. Additionally, the implementation of a more economic

sheet pile wall design and the ability to speed up the construction process potentially result in an increased appreciation of the project. For example, the installation of a lighter sheet pile wall profile is associated with less hindrance (vibrations) to neighboring structures.

In general, an accurate model and the right measurement interpretation is necessary to establish the most economical contingency measure. The main pitfall is that it is not always possible to: a) interpret the measurement data and b) accurately model site conditions. Therefore, the occurrence of unforeseen events is especially problematic as it can be hard to explain why such an event occurred. Also, the consequents of unforeseen events are hard to quantify by computer models.
Regarding this pitfall recommendations are formulated in the next section.

As a result, an answer to the main question if formulated:
Under what conditions would application of the Observational Method Ab Initio approach be feasible to the construction of building pits in soft-soils?

The methodology for measurement-processing as presented in this thesis is a powerful concept that complements the CIRIA guideline. It is therefore recommended to apply this way of measurement-processing to projects. As a result, the following conditions are associated with feasibility of the OM Ab Initio approach:

1) As sheet pile wall deflections are linked to important SLS criteria and indicate structural failure, most building pit projects are suitable for the OM Ab Initio approach.
2) Sufficient design freedom as flexibility in both the structure and the construction sequence are necessary. With “sufficient” it is indicated that the costs for the Observational Method design and construction should outweigh the costs for conventional limit state design.
3) Experience, expertise and motivation in the project’s team to optimally use the observational data to establish a safe and economic design, suiting the conditions on site.
4) Adjacent buildings or surface loads that increase the degree of complexity: The thorough monitoring, real time measurement processing and need for interpretation strongly improve the risk management.
5) Also, for less complex building pits it is believed that the OM is feasible, regarding conditions 1) and 2).

Recommendations

Practical recommendations
For future implementation of the OM a few practical recommendations are formulated:

1) Laboratory testing on soil samples: The verification as done with the Bayesian update aimed to decrease the variability in soil parameters as originally assumed by the conservative NEN9997. This original variability could already be decreased at the start of the project by laboratory testing on strength parameters. Additionally, the results these tests could be used to justify the selection of *most probable* parameters: In this rapport the mean has been assumed, but also a moderately conservative value could be selected if indicated in the lab.
2) Continue the implementation of real-time monitoring and automatization of the Bayesian update as presented in this thesis. This way, potentially more case studies could be executed to strengthen the conclusions found in this thesis.
3) Make use of experience, expertise and corporation in the measurement interpretation: Especially in case of extensive projects, there are many sources that can contribute to the monitored solution. Experience is then a valuable source of information. Expertise and corporation are necessary as the behavior of the retaining wall can be explained in both a structural and soil-structural component. Therefore, an integral approach is necessary.

4) Get an overview of all sensitive parameters to the model: As seen in the case studies not only soil parameters influence the model result, but also other components like water levels, structural parameters and geometry. It is therefore recommended to create an overview of all project features that can have an effect on the monitored quantity. This indication beforehand allows control during the construction process – for example to maintain the right water levels.
5) The use of mathematical and statistical algorithms to analyze the reliability of model results and to update them: As seen in this thesis, some rather simplified statistical methods could already provide a lot of insights. Therefore, it is recommended to look out for the potential of algorithms to other type of projects as well.
6) A review on SLS criteria: As seen from the case studies some design freedom, which is desired in the Observational method, it taken away by restrictions on sheet pile wall displacements that might be too strict. For instance, the impact of 5mm top displacements could be questioned – even if adjacent buildings are located closely to the building pit.

Recommendations for further research
The following recommendations can further shape and/or strengthen the conclusions made in this thesis.

1) A study in the context with undrained soil behavior: This thesis adopted design calculations based on fully drained conditions. As seen in the case studies the measured displacements are affected by time-dependent soil-behavior, which is favorable in terms of soil stiffness. This effect should be recognized in order to draw the correct conclusions on the long-term structural safety. Therefore, it is important to predict undrained behavior and to investigate the reliability of these models. Additionally, it is recommended to investigate the potential to make use of the temporarily stiffer soil-behavior [24].

2) The performance of the measurement-processing as proposed in this thesis is limited by the computer model and the amount and quality of measurements. Throughout the thesis DsheetPiling has been used which has its limitations in, for example, estimated surface settlements. In future work DsheetPiling could be substituted by PLAXIS to see if the methodology can be extended by combining sheet pile wall displacements with other measured quantities, like strut forces and surface settlements.

3) In design nowadays, the most complex cross-section is leading, which is often determined by its adjacent structures. Case studies could conclude better on the true feasibility of the OM design, if these normative cross-sections would be considered. These normative cross-sections are also motivation to select PLAXIS for the methodology.

4) The additional value of using *moderate soil conditions*. This includes the study on the cost potential when applying *progressive contingency measures*.

5) A better quantification of installation affects: Based on the case studies the effects of the installation of sheet pile walls and pile-driving might get underestimated in practice.

6) Analyzing more measurement sets by means of case studies could give a better indication on how errors should be taken into account in the Bayesian update. Especially the reduction of the inclinometer error needs special attention as seen in case study 1.

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9. Appendix

Appendix I - Guidelines

Appendix I on Guidelines contains extra documentation on the Eurocode 7 (2004 and 2018 draft version) and the CIRIA guideline 185 on the Observational method.

Page 1: Comparison Observational method as defined in Eurocode 7 2004 and Draft version.
Page 2: List of management and economical aspects (to complement section 2.5).
Page 3: Annex H Eurocode 7 [draft]
Page 8: Paragraph 7.4 of CIRIA guideline 185.

Appendix II – Retaining wall design

Appendix II contains background information on CUR166 and supporting documents on the sheet pile wall design calculations performed for benchmark 1 with DsheetPiling.

Page 1: Representative values for modulus of Subgrade grade reaction and wall friction angle.
Page 2: DsheetPiling rapport for benchmark 1.

Appendix III – Case studies

Appendix III contains supporting documents on the sheet pile wall design calculations performed for the case studies with DsheetPiling.

Page 1: DsheetPiling rapport for Case study 1.
Page 11: DsheetPiling rapport for Case study 2.

Appendix I - Guidelines

Comparison of Eurocode formulations		
	New version 2018 [draft]	Old version - 2004
Application	Limit states involving geotechnical structures may be verified using the Observational Method.	When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as "the observational method", in which the design is reviewed during construction.
Requirement	Different sets of assumed behaviour of the geotechnical structure should be established, covering all foreseeable ground responses and ground-structure interactions. For each set of behaviour, a design variant shall be established and verified. The sets of assumed behaviour shall cover the whole range of foreseeable geotechnical behaviour. For each design variant, alarm values shall be specified. The design variants shall be sufficiently similar to each other to allow rapid replacement of a design variant by one matching the observed behaviour.	(2)P The following requirements shall be met before construction is started: - acceptable limits of behaviour shall be established; - the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits; - a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. Tile monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully; - the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system; a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.
	The monitoring, observation, and testing program shall be planned so that the frequency of measurements, observations, and tests – as well as the procedures for reading and analysing the results – allow for rapid detection of and reaction to changes from the assumed behaviour of the current design variant. The results of monitoring, observation, and testing shall be assessed at regular intervals and the design variant matching the actual geotechnical behaviour shall be put into operation immediately if the alarm values for the current design variant are exceeded.	The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded.

Table A: Overview of different formulations of the Eurocode 7. Left: Draft as proposed in 2018. Right: Eurocode 2004.

Management aspects (section 2.5):

1) The organization should be **willing to learn and adjust** as construction progresses, also implying the **willingness to take up risks** associated with the implementation of the Ab initio approach.
2) **Active attitude**. Engineers should actively follow the development of the construction process and be able to make the call for the implementation of a contingency measure. At the time of the call, sufficient construction workers will need to be on-site for the implementation and know what to do.
3) **Ability to store material for contingency measures.** At the construction site, extra space is needed for the storage of additional construction material. This extra material could be present on the site for a long period without loss of quality. It should be on-hand when necessary, but not hinder construction workers in their daily works.
4) The extensive monitoring data can be used as a **communication tool** to stakeholders. By processing the monitoring data, the organization is given a learning opportunity as well. The data could also serve to update geotechnical models for other projects.
5) Workers on site should be **informed** about the application procedure, for instance to raise awareness of the importance of well-working monitoring devices.
6) Design, construction and monitoring are integrated, meaning that for a **contractual** point of view it would be beneficial if one party would take up the responsibility or good agreements are made on communications and risk allocation.

Economic aspects:

7) The **non-use of the additional material** for contingency measures should still be economic: If there is no other application for the extra material stored on site, there is still some loss in costs.
8) The costs of the extensive design process, monitoring and contingency measures should **outweigh** the costs of a conventional design approach.

Annex H
(Informative)

Observational method

<Drafting note: This Annex is new. It is not essential for Part 1 but may be required at several locations in Part 3 and is therefore added to Part 1 at this stage of development of EN 1997. PT6 will make the decision whether to include or cancel this Annex.

H.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in 4.8, on the Observational Method.

NOTE The way in which this Informative Annex can be used in a Country is given in the National Annex. If the National Annex is silent on the use of this informative annex, it can be used.

H.2 Scope and field of application

(1) This Annex covers aspects of the application of the Observational Method in the design of geotechnical structures.

H.3 General

(1) <PER> The design of geotechnical structures may be undertaken using the Observational Method as described in 4.8.

(2) The Observational Method can be applied *ab initio* (before construction starts as a planned design procedure) or *ipso tempore* (applied to a structure during construction that has been designed by another method, usually by calculation).

- The Observational Method offers many potential advantages:
- More economic design in terms of materials and programme;
- Improved safety control;
- Control of design uncertainties;
- Stronger connection between designers and constructor;
- Improved construction control and management;
- Greater motivation for the project team;
- Providing databases for benchmarking.

NOTE See Nicholson (1999) for a fuller description of the advantages of the Observational Method.

(3) The Observational Method requires more design effort to consider a range of possible behaviours, improved characterisation of the ground and ground water conditions, enhanced instrumentation and monitoring systems and greater involvement of the designer during the construction period.

(4) The extra costs associated with these enhancements is typically adequately offset by economies in construction.

H.4 Analytical approach

(5) When adopting the Observational Method, the design is required to consider a range of possible behaviours including:

- the ground (and groundwater);
- the structure;
- the surrounding area (for example, the construction site).

(6) To implement the Observation Method, for each set of assumed behaviour:

(7) An appropriate analysis and construction sequence should be fully developed, satisfying ULS and SLS requirements;

- A set of appropriate measurements or observations should be set to enable the designer to determine which set of assumptions are realised during construction (for example, movements, pore water pressure, or forces).
- An instrumentation and monitoring scheme is required that will provide the required observations required above in a timely, robust and reliable fashion. Redundancy in the instrumentation and monitoring scheme should also be considered.

H.5 Framework

(1) A range of approaches and definitions have been applied to the use of the Observational Method since its formalisation in 1969. Four possible approaches are summarised in Table H.1.

NOTE 1 This framework is based on CIRIA C760 (Gaba et al, 2017).

NOTE 2 The Observational Method was originally formalized by Peck (1969).

(2) Alternative assumptions about ground behaviour can also be considered.

NOTE For example, "more probable" parameters, as defined by Powerdham (1994).

Table H.1 — Possible approaches for applying the Observational Method

Approach	**A**	**B**	**C**	**D**
	Optimistically pro-active	**Cautiously pro-active**	**Pro-active modifications**	**Reactive corrections**
Timing	***Ab initio*** **(from the start)**		***Ipso tempore*** **(in the moment)**	
When implemented	Observational Method is planned from project inception		Starts with conventional design with no explicit intention of applying the Observational Method	
Back analysis requirements	Necessary before construction starts from available reliable and relevant case history data	Preferable, but not essential	Necessary – from assessment of initial construction stages	
Analysis assumptions for design of the geotechnical structure	Design and construction sequence in accordance with "design by calculation" method			

Approach	**A**	**B**	**C**	**D**
	Optimistically pro-active	**Cautiously pro-active**	**Pro-active modifications**	**Reactive corrections**
Timing	***Ab initio*** **(from the start)**		***Ipso tempore*** **(in the moment)**	
Parameters adopted for initial design	Most probable	Characteristic	Characteristic	
Implementation	Most probable design and associated construction sequence implemented on site. Alternative construction sequence fully developed in accordance with "design by calculation" adopting characteristic parameters for use as contingency, depending upon the actual performance of the structure	Characteristic design and associated construction sequence implemented on site Alternative construction sequence fully developed in accordance with "design by calculation" method adopting "most probable" parameters for use on site, depending on actual performance of the structure	Monitoring, observations and back analysis during construction show structure performing better than anticipated. Ground, material and structural parameters and ground and analytical models recalibrated on this basis. Construction sequence modified and fully developed in accordance with "design by calculation" method	Monitoring and observations during construction show structure not performing in accordance with design predictions Additional measures put in place to prevent breach of a limit state e.g. damage to nearby structures or to prevent catastrophic collapse
Parameters adopted for revised design	Characteristic	Most probable	Recalibrated	-
Advantages and possible savings	Maximum potential for savings in materials and construction programme duration	Savings in construction programme duration but no material savings, although some savings in materials may be possible due to reduced requirements	Possible savings in during execution by modifying construction sequence requirements. Only likely to be feasible on large projects with long construction duration	Provides a systematic approach to implementation of remedial contingency measures/actions

H.6 Application of the *ab initio* approaches (A and B)

H.6.1 Parameter selection

(1) A number of assumed behaviours are determined based on the available ground investigation data. These can be represented by adopting "characteristic" or "most probable" parameters for design.

(2) In addition to the choice of design values for soil strength and stiffness, the behaviour of the ground (for example, drained or undrained) could be considered as well as ground water level and structural behaviour.

H.6.2 Calculation

(1) The design and analysis of the geotechnical structures are undertaken for each of the sets of assumed behaviour.

(2) Structures that cannot easily be changed once constructed have to be considered in all analyses and only the construction sequence is modified.

H.6.3 Trigger limits and contingency measures

(1) Based on the serviceability analyses for each of the sets of assumed behaviour, a set of trigger values can be set against which the performance of the geotechnical structure is compared.

(2) The choice of parameters that are monitored depends on the type of structure.

H.6.4 Construction phase

(1) It is imperative that the designer is involved during the construction phase and is integral to the review of monitoring data and the decision-making process on the implementation of planned contingencies or modifications.

(2) At each construction stage, the monitoring data is compared to the trigger limits and an appraisal made of the performance of the structure. Opportunities to consider future construction stages and whether contingencies or modifications will or will not be needed should be taken when possible.

H.7 Application of the *ipso tempore* approaches (C and D)

H.7.1 Evaluation of the structure's performance and calibration of parameters

(1) Following initiation of the Observation Method ipso tempore, an audit of the geotechnical structures performance should be undertaken.

(2) Monitoring data and observations from the construction site should be used to back analyse the structure to obtain recalibrated parameters and assumptions.

H.7.2 Calculation

(1) The recalibrated parameters and assumptions should be used to forward predict the remaining construction stages and redefine the design or construction sequence.

H.7.3 Trigger limits and contingency measures

(1) The same procedure defined for *ab initio* (see H.6.3) should be used to define trigger limits based on the recalibrated parameters and assumptions.

H.7.4 Monitoring system

(1) The monitoring system installed at the start of construction may require augmentation following initiation of the Observational Method ipso tempore.

H.7.5 Remaining construction stages

(1) For the remaining construction stages, the performance of the geotechnical structure should be compared to the trigger limits defined in H.7.3 to ensure the contingency or modification are having the intended effect.

(2) Further recalibration may be required during the remaining construction stages.

reinforcement to the depth required for lateral stability, rather than the full length. If this approach is adopted, the designer should check the likely flexure at the depth of the reinforcement curtailment and the effect that the formation of a crack in the wall at this depth will have on its ability to carry the applied load satisfactorily. This is a particularly pertinent consideration for piles constructed using the CFA technique where the maximum cage length is limited.

7.4 GEOTECHNICAL DESIGN OF THE WALL BY USING THE OBSERVATIONAL METHOD

EC7 (EC7-1 Clause 2.7) is the first design standard in the UK that explicitly includes provision for using the OM.

The OM offers potential savings in construction programme and costs as well as a rigorous and clear allocation and treatment of construction risk. It allows the designer the opportunity to modify the design of any structure during construction using back analysis of reliable observations and a feedback loop to make refinements or to implement contingencies. Due to the increased use of monitoring in a robust and planned way there should also be an improvement in the safety of the construction works.

An illustration of some of the potential advantages to the OM is shown in **Figure 7.6**.

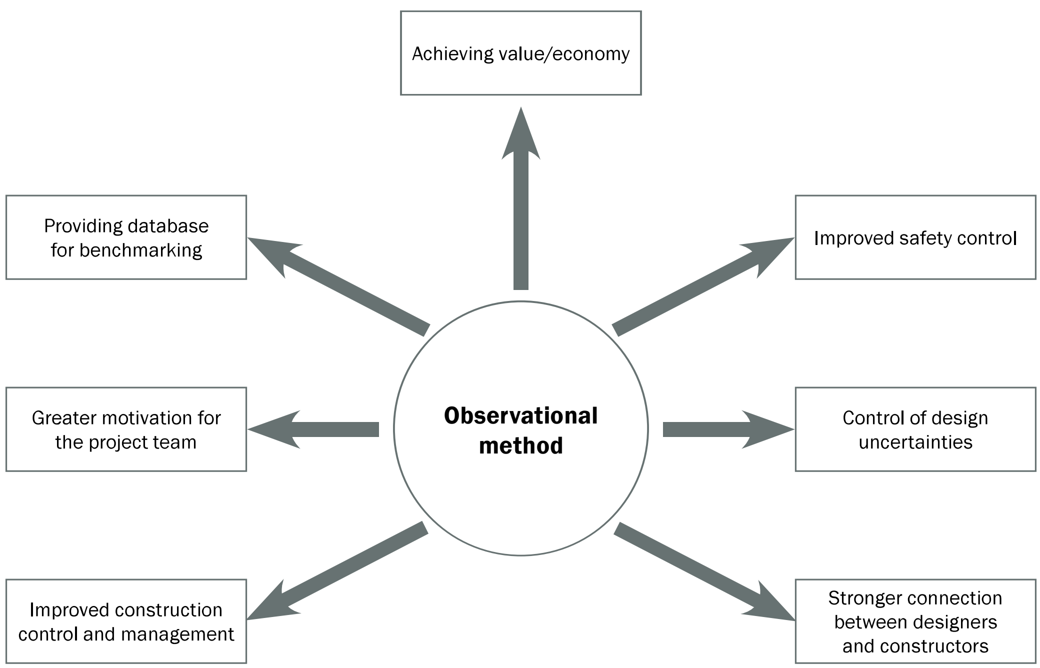


Figure 7.6 Illustration of potential advantages by using the OM (after Nicholson et al, 1999)

The OM has been used for many centuries, but was first formalised for geotechnical design by Peck (1969). A thorough treatment of the method is given in Nicholson *et al* (1999) and will not be repeated here, however, key elements of the design and analysis procedure that are applicable to embedded retaining walls are provided in this section.

There are many uncertainties associated with the design and construction of structures. However, when the OM is applied to embedded retaining walls, the principal uncertainties are considered to be:

- conditions during wall installation and subsequent construction (eg control of adjacent surcharge loading, excavation levels and impact of dewatering and nearby ground improvement processes such as grouting)

- structural behaviour of the retaining wall and support system (eg material properties, the behaviour of connections between the wall and props and connections between individual wall elements such as diaphragm wall panels)
- geology (eg unmapped scour hollows)
- parameter selection (eg variability and reliability in determining characteristic, 'most probable' and 'recalibrated' values)
- assumed ground behaviour, particularly in relation to speed of construction (eg undrained versus drained behaviour and isotropic versus anisotropic behaviour).

So adaptation of the design concentrates on the refinement of these assumptions.

7.4.1 Definition of the OM

Since the formalisation of the OM, a range of approaches and definitions have been applied to its use. For clarity and consistency, the different approaches presented in this publication have been divided into two categories:

1. *Ab initio* (from the start) where the use of the OM is planned from the start of the project.
2. *Ipso tempore* (in the moment) where the design of the embedded retaining wall is reassessed during construction.

Ab initio

For the design of embedded retaining walls, the OM is implemented from the start of the design process and requires a range of possible ground behaviours to be considered at an early stage. An important decision has to be made by the designer regarding the approach to be used when undertaking the structural design of the wall and choosing the embedment depth. Once the wall has been installed, this cannot be changed. In view of this, the designer can assume a range of ground behaviours, although these are typically bounded by the following two approaches:

1. An approach that optimistically adopts a construction sequence assuming 'most probable' ground and structural behaviour, with a fully developed alternative construction sequence (which will require additional support provision) for rapid implementation should the actual behaviour tend towards characteristic.
2. An approach that cautiously adopts a construction sequence assuming characteristic ground and structural behaviour, with a fully developed alternative construction sequence which assumes 'most probable' behaviour resulting in fewer support requirements for the wall.

It should be noted that for any embedded retaining wall design (be it undertaken by calculation or by the OM) there are scenarios where the adoption of 'cautious' design parameters may result in a locally less onerous design case. An example could be the bending moment at the bottom of a retaining wall embedded in a competent rock where more rotational restraint would result in a larger induced bending moment. To address this, it is important to note that EC7 requires the designer to 'consider a range of possible behaviours' to complete the design of the wall.

The choice between these approaches (or any intermediate approach and associated set of assumptions between these two) will depend on the designer's awareness and knowledge of construction activities, methods and sequences actually adopted (or to be adopted) on site, confidence in the actual (or predicted) performance of the wall structure and its support system, familiarity with the particular ground and groundwater conditions at the site (including reliable and relevant well documented case studies) and the project team's appetite for risk. If the approach adopted by the designer is more optimistic, the potential for savings in materials will be maximised (due to the minimised wall depth and thickness, reduced support requirements and reduced reinforcement provision in the case of a reinforced concrete wall). A more cautious approach will only typically offer potential savings relating to a reduction in construction programme and support requirements, which can still be a significant advantage to a project.

Ipso tempore

The application of the *ipso tempore* method differs from the *ab initio* approach in that it is not planned from the beginning of the project, but rather it is adopted during construction after the wall has been installed.

The design of the wall is undertaken by the calculation method set out in Section 7.3. However during construction, a decision is made by the project team to apply the OM. Two scenarios are possible:

1 The wall is performing better than anticipated by the designer and a proactive decision is made by the project team to modify the construction sequence to achieve programme savings. As with the cautious *ab initio* approach there is no opportunity to make savings on wall materials, as the wall will have already been installed. Savings can be made in respect of reduced construction duration due to the reduced wall support requirements and associated potential savings in material costs arising from this.

2 The actual wall performance is of concern to the designer and reactive intervention is required to prevent a SLS or ULS occurring. This approach is analogous to the 'best way out' method proposed by Peck (1969).

The possible approaches to the application of the OM to the design of embedded retaining walls are summarised in Table 7.2.

Table 7.2 Summary of OM approaches to the design of embedded retaining walls

Approach	***Ab initio* (from the start)**		***Ipso tempore* (in the moment)**	
	A* *Optimistically proactive	***B* *Cautiously proactive***	***C* *Proactive to make modifications***	***D* *Reactive to make corrections***
When implemented	The OM is planned from project inception		Starts with conventional design with no explicit intention of applying the OM	
Back analysis requirements	Necessary before construction starts from available reliable and relevant case study data	Preferable, but not essential	Necessary – from assessment of initial construction stages	
Analysis assumptions for design of the wall and its support system	Wall embedment, design and construction sequence in accordance with 'design by calculation' method (Section 7.3) adopting 'most probable' parameters	Wall embedment depth, design and construction sequence in accordance with 'design by calculation' method (Section 7.3) adopting characteristic parameters	Wall embedment, design and construction sequence in accordance with 'design by calculation' method (Section 7.3) adopting characteristic parameters	
Implementation	Most probable wall design and associated construction sequence implemented on site Alternative construction sequence fully developed in accordance with 'design by calculation' (Section 7.3) adopting characteristic parameters for use as contingency, depending upon the actual performance of the wall and its support system	Characteristic wall design and associated construction sequence implemented on site Alternative construction sequence fully developed in accordance with 'design by calculation' method (Section 7.3) adopting 'most probable' parameters for use on site, depending on actual performance of the wall and its support system	Monitoring, observations and back analysis during construction show wall performing better than anticipated. Ground, material and structural parameters and ground and analytical models recalibrated on this basis. Construction sequence modified and fully developed in accordance with 'design by calculation' method (Section 7.3) adopting 'recalibrated' parameters	Monitoring and observations during construction show wall not performing in accordance with design predictions Additional measures put in place to prevent breach of a limit state, eg damage to nearby structures or to prevent catastrophic collapse
Advantages and possible savings	Maximum potential for savings in materials and construction programme duration	Savings in construction programme duration but no wall material savings, although some savings in materials may be possible due to reduced wall support requirements.	Possible savings in wall support system during excavation in front of the wall by modifying construction sequence and support system requirements. Only likely to be feasible on large projects with long construction duration	Provides a systematic approach to implementation of remedial contingency measures/actions

7.4.2 EC7 requirements

EC7-1 Clause 2.7(2)P requires that the following conditions shall be met before construction starts:

- *"Acceptable limits of behaviour shall be established"*

  This requires appropriate monitoring of the wall and its support system to measure wall deflection and profile, ground movements (due to wall installation and deflection and other effects such as dewatering), prop/anchor loads, SLS and ULS limitations on deflections/movements relating to the tolerance of nearby structures/utilities to accommodate such movements etc.

- *"The range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits"*

  This requires the designer to consider ground behaviour particularly in relation to speed of construction (eg undrained behaviour versus drained behaviour), ground anisotropy, structural behaviour of the wall and its support system (eg material properties and the behaviour of connections – between the wall and props/anchors and between individual wall elements), parameter selection (eg variability and reliability in determining characteristic, most probable, recalibrated values).

- *"A plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully"*

  This requires an early verification phase to confirm that actual behaviour is within acceptable limits.

- *"The response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system"*

  This requires an efficient process that reviews/assesses/back analyses the monitoring data to enable the instrumentation and monitoring system to be adapted/developed further to facilitate appropriate decisions to be made and implemented in a timely manner.

- *"A plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits."*

7.4.3 Application of the OM to embedded retaining walls

When applied to the design of embedded retaining walls, the OM can be implemented in two ways:

1. When the information from the completed construction at one location informs the design of the same or similar structures at a nearby location. This can be particularly useful for long linear retaining structures used, for example, in road or railway construction.
2. When the knowledge gained through observations at the early stage of an excavation at a location can be used to modify the excavation sequence and implementation of the temporary support system at that location in later stages.

In the first case, savings can be made at an early stage because the design of further structures will benefit from the experience and monitored performance of the original construction and back analysis of data to derive more reliable parameters. The design of the wall could adopt either the *ab initio* or the *ipso tempore* approach.

In the second case, savings could be made through modification of the design as construction progresses, providing actual movements are monitored to be within acceptable limits. Conversely, contingency methods can be employed to ensure that movements do not exceed acceptable limits.

To be applicable, it is imperative that time is available during construction for observations to be reported, processed and fed back into the design and for decisions to be made on how the excavation should proceed.

In this context, it is imperative that any brittle failure mechanisms, which would not give adequate warning of their occurrence, are avoided. These may include pull-out failure of ground anchors or prop loss due to accidental removal or structural buckling.

Application of the *ab initio* approach

The principal considerations and key steps in the design of an embedded retaining wall using the *ab initio* OM are illustrated in **Figure 7.7** and discussed in detail here.

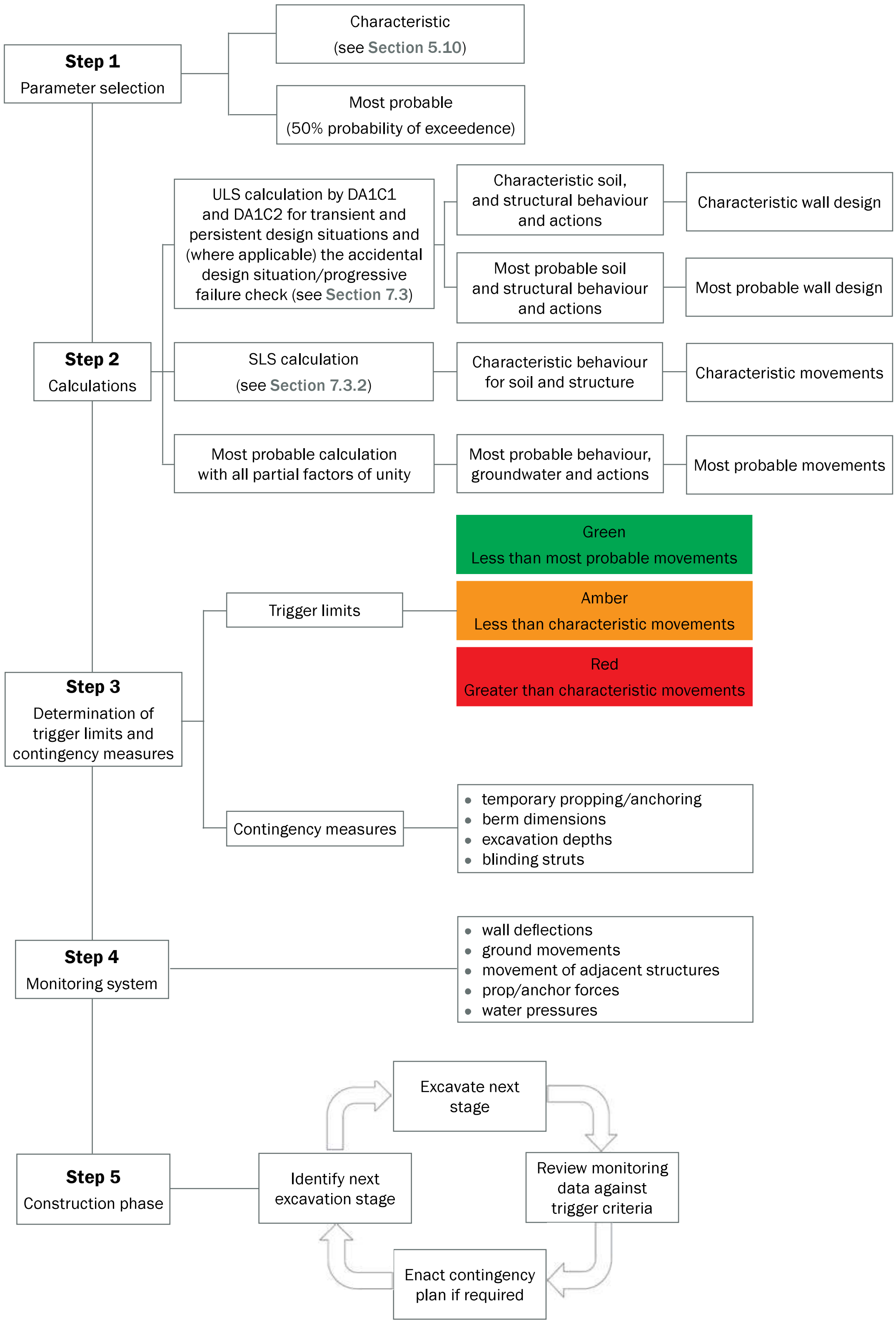


Figure 7.7 ***Ab initio OM applied to embedded retaining walls***

Step 1: Parameter selection

The design of embedded retaining walls requires the determination of characteristic values for soil and rock parameters (see Section 5.10), actions and structural behaviour for the wall and propping system. When partial factors are applied to these and the effects of actions arising from associated calculations in accordance with the requirements of EC7 DA1, the outcome is a retaining wall design that is considered to be sufficiently remote from the ULS and should not require modification during construction. If monitoring of the structure is to be done, it is only to confirm that the structure is performing as anticipated.

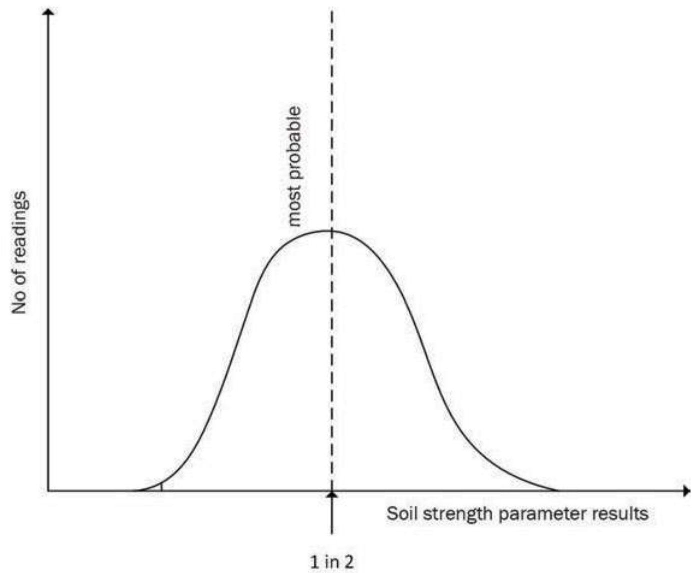


Figure 7.8 ***Definition of most probable parameters (after Nicholson et al, 1999)***

When using the OM, the designer is required to establish trigger values relating to wall movements to be expected during construction. For this purpose, in addition to characteristic parameters, the designer will also be required to identify a set of most probable parameters.

Most probable is defined in Nicholson *et al* (1999) as *"a set of parameters that represents the probabilistic mean of all possible sets of conditions…in general terms, the design condition most likely to occur in practice"*.

The most probable parameter is illustrated in Figure 7.8, which assumes a normal or Gaussian distribution.

To determine the groundwater levels for the design cases considered, an approach based on that outlined in Figure 7.4 should be adopted.

Step 2: Calculations

The observations that feedback into the design of an embedded retaining wall using the OM are principally the deflection of the retaining wall, the measured forces in props/anchors and possibly the settlement of the ground or structures behind the wall. The prediction of these elements is central to the successful use of this method. As the limit equilibrium method provides no information on movements, application of the OM is limited to SSI analyses (typically numerical analyses).

The designer should undertake the following for each of the *ab initio* OM approaches:

1. **Approach A:** when using the optimistic *ab initio* approach, the design of the embedded retaining wall should first be undertaken by calculation using DA1C1 and DA1C2 as set out in Section 7.3.1. The most probable parameters derived in Step 1 should be adopted and, where applicable, the accidental design situation/progressive failure check as set out in Section 7.3.3 also adopting most probable parameters. The outcome of this will be the most probable wall embedment depth and the most probable design effects of actions (wall bending moments, shear forces and prop/anchor forces) for the assumed construction sequence and wall support system (propping/anchoring arrangement).

   Once the wall embedment depth has been determined, the designer should undertake a further calculation adopting most probable parameters, groundwater assumptions and actions with partial factors of unity applied throughout. The output from this additional calculation will be most probable movements of the wall and associated most probable ground movements.

   The designer should structurally design the wall and its support system based on these calculations.

For the wall geometry (embedment depth) and structural capacity/strength (including reinforcement in the case of reinforced concrete walls) derived from these calculations using most probable parameters, the calculations should be repeated adopting characteristic parameters, ie calculations should be undertaken using DA1C1 and DA1C2 (**Section 7.3.1**) and, where applicable, the accidental design situation/progressive failure check (**Section 7.3.3**) and SLS (**Section 7.3.2**) using characteristic parameters. To ensure that the ULS is satisfied with the characteristic soil and rock strength parameters, it is inevitable that extra temporary support to the wall will be required either in the form of more levels of props/anchors, more closely spaced props/anchors or changes to the geometry of the excavation (such as the use of berms or excavation in bays). The output from the SLS calculation will be SLS characteristic movements of the wall and associated SLS characteristic ground movements.

It is important to ensure that the wall embedment and its structural capacity/strength are adequate and appropriate to avoid the ULS for the respective construction sequences associated with the use of 'most probable' and characteristic parameters. This should result in a characteristic wall design that is buildable.

It is also important that the initial stages of the most probable and the characteristic construction sequences are the same – these verification stages will allow the designer to understand the actual performance of the wall and its support system relative to the design predictions under most probable and characteristic assumptions. The outcome of this evaluation will inform the decision as to whether to continue with the most probable approach or to adopt the alternative characteristic approach as a contingency.

2 **Approach B:** when using the cautious *ab initio* approach, the ULS design of the embedded retaining wall should first be undertaken using DA1C1 and DA1C2 (**Section 7.3.1**) adopting characteristic parameters and, where applicable, the accidental design situation/progressive failure check (**Section 7.3.3**) adopting characteristic parameters. The outcome of this will be the characteristic wall design and associated construction sequence, propping/anchoring arrangement, wall embedment depth and characteristic design effects of actions (wall bending moments, shear forces and prop/anchor forces). A SLS calculation should also be undertaken (**Section 7.3.2**) adopting characteristic parameters. The output from this calculation will be the SLS characteristic movements of the wall and associated SLS characteristic ground movements.

It is an obvious but important consideration when applying the cautious approach to the *ab initio* OM to the design of embedded retaining walls, that once the wall is constructed, its embedment and structural capacity/strength cannot be changed and the designer can only modify the excavation sequence and the support system to the wall. For this reason, the designer should consider where there is flexibility in the design of temporary support to the wall, ie varying levels of props/anchors, spacing of props/anchors or changes to the geometry of any berms used.

For the characteristic wall embedment depth determined from these calculations, the designer should derive and fully develop a modified construction sequence and wall support arrangement. These should confirm the adequacy of the wall embedment depth and its structural capacity/strength from the results of DA1C1 and DA1C2 calculations (**Section 7.3.1**) and, where applicable, the accidental design situation/progressive failure check (**Section 7.3.3**) using most probable parameters.

The designer should also undertake an additional calculation using most probable parameters, groundwater assumptions and actions with partial factors of unity applied throughout. The output from this additional calculation will be most probable wall movements and associated ground movements.

As with Approach A, the initial stages of the characteristic and most probable construction sequences should be the same – these verification stages will allow the designer to understand the actual performance of the wall and its support system relative to their design predictions under characteristic and most probable assumptions. The outcome of this evaluation will inform the decision as to whether to continue with the characteristic approach or to adopt the alternative most probable approach as a modification.

3 **Possible intermediate approaches:** between the optimistic and cautious approaches, there are a myriad of intermediate situations that could be considered by the designer depending on the project's particular circumstances and the project team's appetite for risk. An example could be the more probable design proposed by Powderham (1994), which uses parameters that are less conservative than characteristic, but more conservative than most probable. The principles that apply to these intermediate situations are the same as those discussed here and so will not be discussed further.

Step 3: Trigger limits and contingency measures

The construction and excavation of an embedded retaining wall is typically a multi-stage process. Trigger limits should be set at the start of the project for each construction stage. Typically, a traffic light system is used for this purpose. Table 7.3 gives the identification of trigger limits for movements:

Table 7.3 Identification of trigger limits at each construction stage – ab initio OM (Approach A and B)

Trigger	Value chosen for limit δ	Action
Green	$\delta \leq$ predefined proportion of predicted most probable movements (see Figure 7.9)	If Approach A is adopted: continue with most probable sequence and associated assumptions. If Approach B is adopted: change to alternative most probable approach may be possible.
Amber	Predefined proportion of predicted most probable movements (see Figure 7.9) $\leq \delta \leq$ SLS characteristic movements	If Approach A is adopted contingency (in the form of reverting to the characteristic design construction sequence) may be required. If Approach B is adopted minor modifications to characteristic construction sequence may be possible.
Red	$\delta >$ SLS characteristic movements	Approach D additional measures required.

The application of the OM to a multi-stage excavation is illustrated in Figure 7.9 with three possible scenarios:

1. Where the actual monitored movements at a particular stage are less than those predicted adopting most probable parameters. Under these circumstances the construction sequence based on the most probable design can be implemented.
2. Where the actual monitored movements at a particular stage are within the 'amber' zone. Under these circumstances the construction should be completed according to the characteristic design construction sequence.
3. Where the actual monitored movements at a particular stage are more than those predicted adopting 'characteristic' parameters. Under these circumstances Approach D additional measures will be necessary to prevent breach of a limit state.

There are many eventualities that need to be carefully considered by the designer to have a full understanding of how the embedded retaining wall is performing compared to the range of behaviours considered during the planning stages.

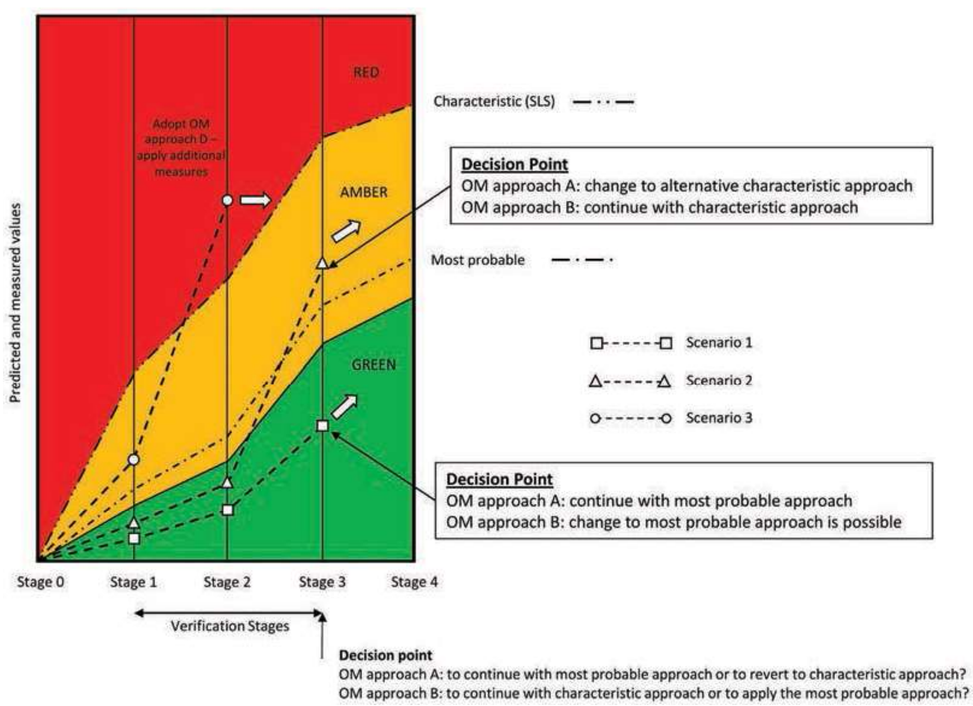


Figure 7.9 Trigger limits for multi-staged excavation – ab initio OM (Approaches A and B)

Step 4: Monitoring system

The parameters that most readily provide the designer with feedback on the behaviour and performance of an embedded retaining wall during construction are the deflection of the wall, the forces generated in the props/anchors and the movement of the retained ground and nearby structures. The monitoring methods for measuring these parameters are summarised in **Table 7.4**. It is important in the specification that the hierarchy for the various monitoring systems in place is created. Trigger limits should be applied to the 'primary' monitoring systems with the other systems introduced as back-ups to ensure there is redundancy in the system.

Table 7.4 Summary of most commonly adopted monitoring systems

Parameter to be measured	Instrumentation examples	Notes
Retaining wall movement	Inclinometer	Inclinometers can be: • manual • in-place • ShapeAccelArray (SAA). Global movement of the wall should be measured by either extending the inclinometer below the toe of the wall or surveying the capping beam.
Prop forces	Strain gauges attached to prop	Vibrating wire strain gauges are the most commonly used and should be used at four locations around the prop and corrected for temperature effects.
Ground and building movement	Surveying points	Movements can be measured by precise levelling studs, levelling pins, prisms or reflective targets.
Groundwater	Piezometers	Piezometers can comprise traditional standpipes or vibrating wire.

The designer of the monitoring system should choose between an automated monitoring system where readings are taken at regular short intervals, or a manual system that requires visits from a surveying team. The choice will depend on a number of considerations including:

- accuracy of the system required – manual systems tend to achieve better accuracy
- rate of change of movements that is expected – automated systems will be able to provide more frequent data
- number of visits required and duration of the monitoring – automated systems have higher capital costs, but lower operational costs compared to manual systems
- accessibility – during a complex construction process it may not be possible for a survey team to safely access monitoring points. Under these circumstances an automated system may be more appropriate.

It is now common for the monitoring data to be available to the entire project team through an internet-based portal so all involved parties can access and interpret the data.

Instrumentation and monitoring is discussed further in **Chapter 9**.

Step 5: The construction phase

During the construction phase, it is important that the designer of an embedded retaining wall remains integral to the construction process. At each stage of construction, the monitoring data should be interpreted and evaluated against the trigger limits that are based on the characteristic and most probable assumptions. At each construction stage, the designer should assess the monitoring data relative to these predictions, and from this recommend what, if any, contingency measures should be implemented. An example of the typical decision-making process used (with a traffic light system) is shown in **Figure 7.10**.

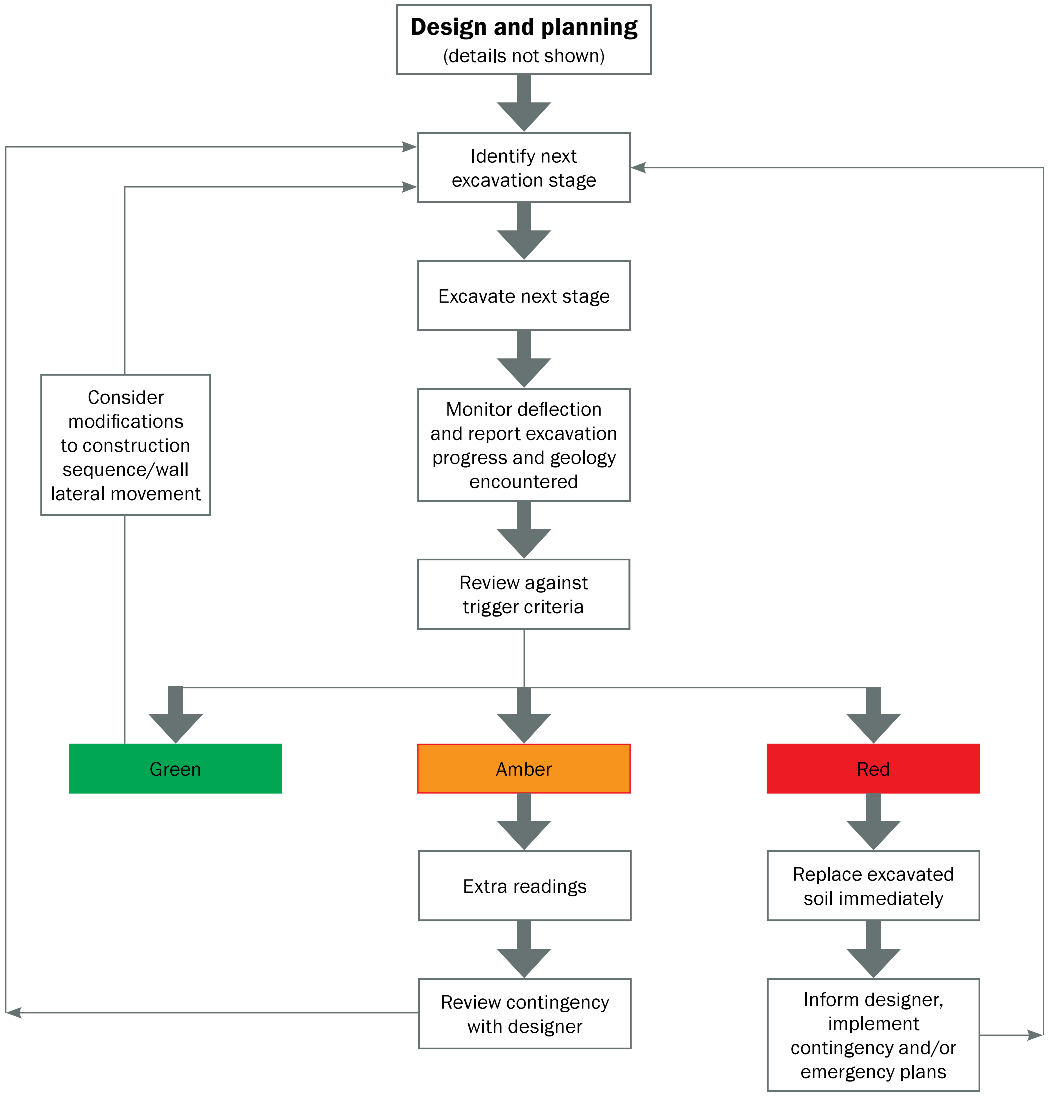


Figure 7.10 Example of a traffic light system for a multi-staged excavation (after Nicholson et al, 1999)

At each construction stage, the designer should take the opportunity to carry out back analysis of the current construction stage using the SSI analysis (typically numerical analysis) that was set up at the design stage. From this calibration exercise, the designer should be able to gauge whether the behaviour of the ground and the wall are relative to the design assumptions.

Any trends that are developing at this stage should be assessed taking the opportunity to consider whether possible modifications or contingencies are likely. The earlier these changes are brought to the attention of the project team, the more effectively they can be implemented to the overall benefit of the project.

Application of the *ipso tempore* approach

When the *ipso tempore* approach to the OM is applied to the design of an embedded retaining wall, it is initiated after the installation of the wall and the completion of some initial excavation stages. Up until this point, the design of the wall has been completed in accordance with the design by calculation method described in Section 7.3 (where calculations using DA1C1 and DA1C2 and SLS and, where applicable, accidental design situation/progressive failure check calculations have all been undertaken using characteristic parameters) without the intention of applying the OM. The wall has been designed and installed to the characteristic wall embedment depth with a structural capacity/strength based on the design effects of actions (wall bending moments, shear and prop/anchor forces) resulting from calculations that adopt characteristic parameters.

Appendix II – Retaining wall design

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

Figure A. Table 3.3 of CUR166 Part 1: Representative values for the modulus of subgrade reactions in retaining wall design.

Wandoppervlak	Ruwheid	δ_a respectievelijk δ_p	
		recht glijvlak	gekromd glijvlak
getand	$> 10\ d_{50}$	$0{,}67\ \varphi'$	$\leq \varphi'$
ruw	$0{,}5 - 10\ d_{50}$	$0{,}67\ \varphi'$	$\leq \varphi' - 2{,}5^0$ en $\leq 27{,}5^0$
half ruw	$0{,}1 - 0{,}5\ d_{50}$	$0{,}33\ \varphi'$	$0{,}5\ \varphi'$
glad	$< 0{,}1\ d_{50}$	0	0

*) Conform NEN 6740 moet in veen gerekend worden met $\delta = 0$. Voor het teken van de hellingshoek δ wordt verwezen naar figuur 3.1. Bij een uitwendige verticale belasting op de damwand kan de wandwrijving van richting omdraaien, zie hoofdstuk 5. Dit is van grote invloed op de grootte van de gronddruk op de wand.

Figure B. Table 3.2 of CUR166 Part 1: Representative values for the wall friction angle in retaining wall design.

Report for D-Sheet Piling 18.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Date of report:	5/12/2019
Time of report:	8:33:43 PM
Report with version:	18.2.1.20477
Date of calculation:	5/11/2019
Time of calculation:	7:48:23 PM
Calculated with version:	18.2.1.20477
File name:	C:\..\Dsheetmodel_Benchmark1\Clay\Definitief\OM_model_clay_strut
Project identification:	Tutorial 1 for d sheetpiling excavation using ka,k0,kp

Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overview per Stage and Test

Stage nr.	Verification	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	EC7(NL)-Step 6.1		-0.19	0.22	0.0	16.5	---
1	EC7(NL)-Step 6.2		-0.10	0.19	0.0	16.5	---
1	EC7(NL)-Step 6.3		0.31	-0.37	0.0	16.7	---
1	EC7(NL)-Step 6.4		0.17	-0.32	0.0	16.7	---
1	EC7(NL)-Step 6.5	0.0	0.00	0.00	0.0	12.3	---
1	EC7(NL)-Step 6.5 * 1.20		0.00	0.00			
2	EC7(NL)-Step 6.1		-11.34	18.12	0.0	21.6	---
2	EC7(NL)-Step 6.2		8.59	15.23	0.0	21.6	---
2	EC7(NL)-Step 6.3		-11.60	18.40	0.0	21.8	---
2	EC7(NL)-Step 6.4		8.63	15.46	0.0	21.8	---
2	EC7(NL)-Step 6.5	-3.0	-8.25	15.16	0.0	15.4	---
2	EC7(NL)-Step 6.5 * 1.20		-9.90	18.19			
3	EC7(NL)-Step 6.1		10.70	16.54	16.9	20.8	---
3	EC7(NL)-Step 6.2		9.23	13.89	16.9	20.8	---
3	EC7(NL)-Step 6.3		-10.78	16.94	17.2	21.1	---
3	EC7(NL)-Step 6.4		9.10	14.23	17.1	21.1	---
3	EC7(NL)-Step 6.5	-3.0	-8.25	15.16	12.1	15.4	---
3	EC7(NL)-Step 6.5 * 1.20		-9.90	18.19			
4	EC7(NL)-Step 6.1		-49.98	-37.03	22.5	26.5	---
4	EC7(NL)-Step 6.2		-42.14	-33.39	22.5	26.9	---
4	EC7(NL)-Step 6.3		-58.20	-42.14	23.3	27.5	---
4	EC7(NL)-Step 6.4		-49.06	-38.09	23.3	27.9	---
4	EC7(NL)-Step 6.5	-5.9	-26.28	-21.43	15.4	18.6	---
4	EC7(NL)-Step 6.5 * 1.20		-31.53	-25.71			
5	EC7(NL)-Step 6.1		-225.07	-110.83	54.3	60.6	---
5	EC7(NL)-Step 6.2		-201.02	-106.19	59.4	65.6	---
5	EC7(NL)-Step 6.3		-238.85	-114.99	56.4	62.6	---
5	EC7(NL)-Step 6.4		-213.92	-110.43	62.1	68.0	---
5	EC7(NL)-Step 6.5	-26.3	-118.68	-69.68	32.2	39.2	---
5	EC7(NL)-Step 6.5 * 1.20		-142.42	-83.61			
6	EC7(NL)-Step 6.1		-225.07	-110.83	54.3	60.6	---
6	EC7(NL)-Step 6.2		-201.02	-106.19	59.4	65.6	---
6	EC7(NL)-Step 6.3		-238.85	-114.99	56.4	62.6	---
6	EC7(NL)-Step 6.4		-213.92	-110.43	62.1	68.0	---
6	EC7(NL)-Step 6.5	-26.3	-118.68	-69.68	32.2	39.2	---
6	EC7(NL)-Step 6.5 * 1.20		-142.42	-83.61			
7	EC7(NL)-Step 6.1		-213.81	-101.66	46.6	50.1	---
7	EC7(NL)-Step 6.2		-186.77	-101.43	52.6	56.2	---
7	EC7(NL)-Step 6.3		-229.75	-106.90	48.8	52.3	---
7	EC7(NL)-Step 6.4		-202.20	-106.08	55.4	59.0	---
7	EC7(NL)-Step 6.5	-26.7	-109.60	-61.49	24.9	28.3	---
7	EC7(NL)-Step 6.5 * 1.20		-131.53	-73.79			
8	EC7(NL)-Step 6.1		-292.74	-103.85	68.8	71.0	---
8	EC7(NL)-Step 6.2		-278.20	-103.67	78.9	80.5	---
8	EC7(NL)-Step 6.3		-323.69	-114.24	72.0	74.0	---
8	EC7(NL)-Step 6.4		-309.82	-114.20	**80.8**	**82.3**	---
8	EC7(NL)-Step 6.5	-48.0	-168.27	70.44	41.4	44.2	---
8	EC7(NL)-Step 6.5 * 1.20		-201.92	84.53			
9	EC7(NL)-Step 6.1		-294.31	-104.27	38.5	39.3	---
9	EC7(NL)-Step 6.2		-279.39	-103.89	43.7	44.1	---
9	EC7(NL)-Step 6.3		-325.23	-114.66	40.1	40.8	---
9	EC7(NL)-Step 6.4		-311.00	-114.41	44.7	45.0	---
9	EC7(NL)-Step 6.5	-47.9	-170.12	75.31	24.2	25.5	---
9	EC7(NL)-Step 6.5 * 1.20		-204.14	90.38			
10	EC7(NL)-Step 6.1		-386.59	238.44	0.0	47.4	---
10	EC7(NL)-Step 6.2		-368.32	239.64	0.0	51.8	---
10	EC7(NL)-Step 6.3		**-409.53**	232.93	0.0	49.4	---

Stage nr.	Verification	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
10	EC7(NL)-Step 6.4		-392.11	233.97	0.0	53.1	---
10	EC7(NL)-Step 6.5	**-61.6**	-277.87	241.96	0.0	26.4	---
10	EC7(NL)-Step 6.5 * 1.20		-333.44	**290.35**			
Max		**-61.6**	**-409.53**	**290.35**	**80.8**	**82.3**	---

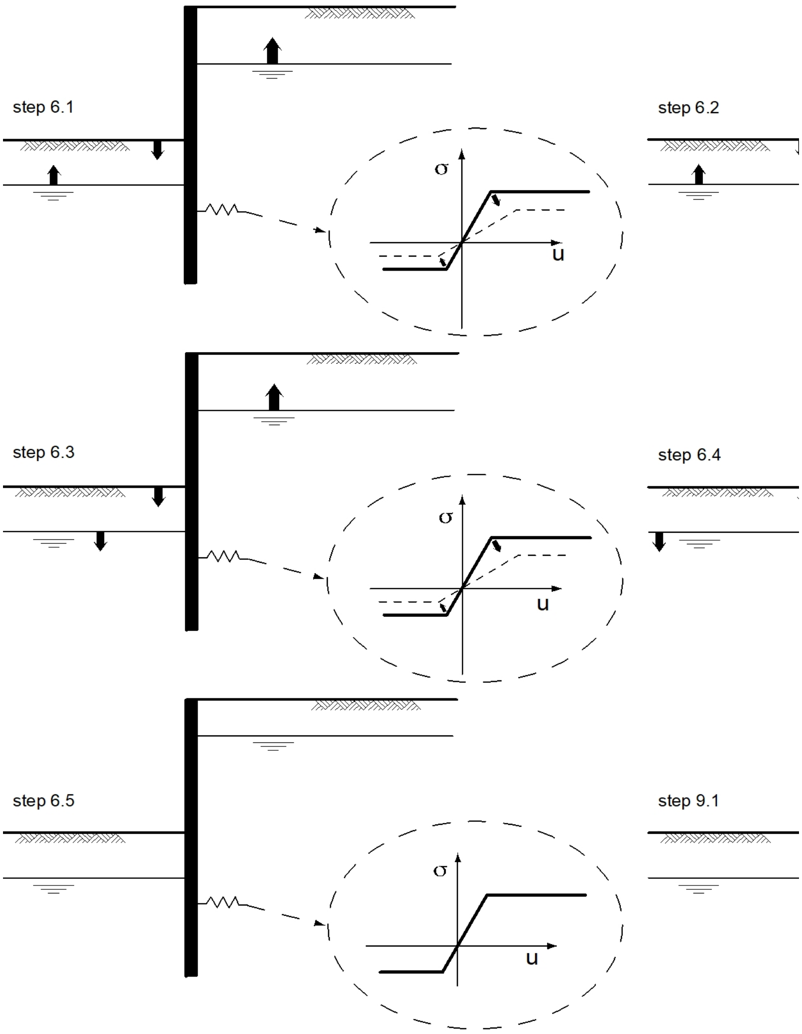
1.2 Overall Stability per Stage

Stage name	Stability factor [-]
Initial	10000.00
Excavation_2m	5.83
Excavation + anchor	5.83
Excavation	3.10
Excavation_end	1.82
Grindlayer	1.82
Excavation -8.5	2.07
Excavation end	1.73
OWB	2.28
drypumping	1.56

1.3 Warnings

Stage	Warning
5	Uplift might occur
6	Uplift might occur

1.4 CUR Verification Steps

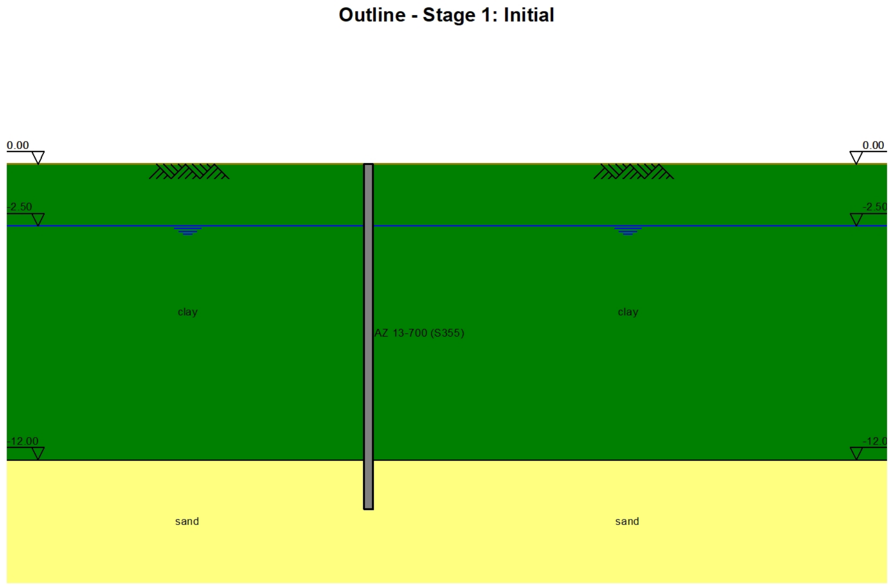


2 Input Data for all Stages

2.1 Calculation Options

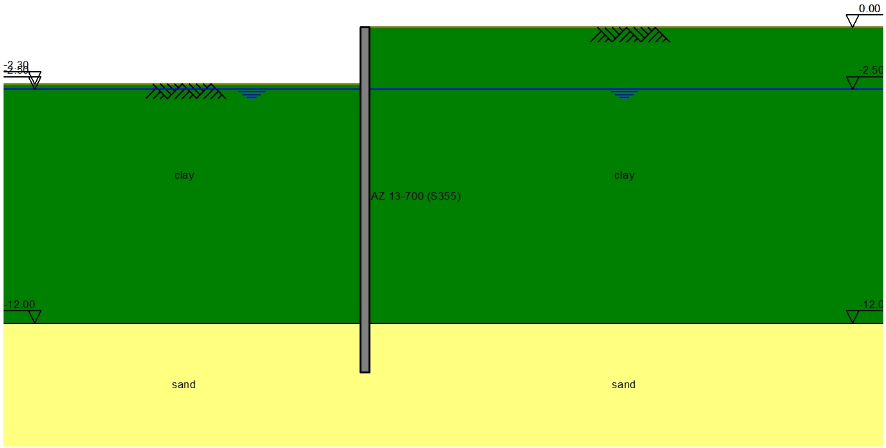
First stage represents initial situation	No
Calculation refinement	Coarse
Reduce delta(s) according to CUR	Yes
Verification	EC7 NA NL - method A: Partial factors (design values) in all sta Eurocode 7 using the factors as described in the National Annex of the Netherlands. It is basically design approach III.
Multiplication factor for anchor stiffness	1.000
Used partial factor set	RC 1
Factors on loads	
- Permanent load, unfavourable	1.00
- Permanent load, favourable	1.00
- Variable load, unfavourable	1.00
- Variable load, favourable	0.00
Factors on representative values	
- Partial factor on M, D and Pmax	1.20
Material factors	
- Cohesion	1.15
- Tangent phi	1.15
- Delta (wall friction angle)	1.15
- Modulus of low representative subgrade reaction	1.30
Geometry modification	
- Increase retaining height	10.00 %
- Maximum increase retaining height	0.50 m
- Reduction in phreatic line on passive side	0.20 m
- Raise in phreatic line on passive side	0.20 m
- Raise in phreatic line on active side	0.05 m
Overall stability factors	
- Cohesion	1.30
- Tangent phi	1.20
- Factor on unit weight soil	1.00

3 Outline Stage 1: Initial



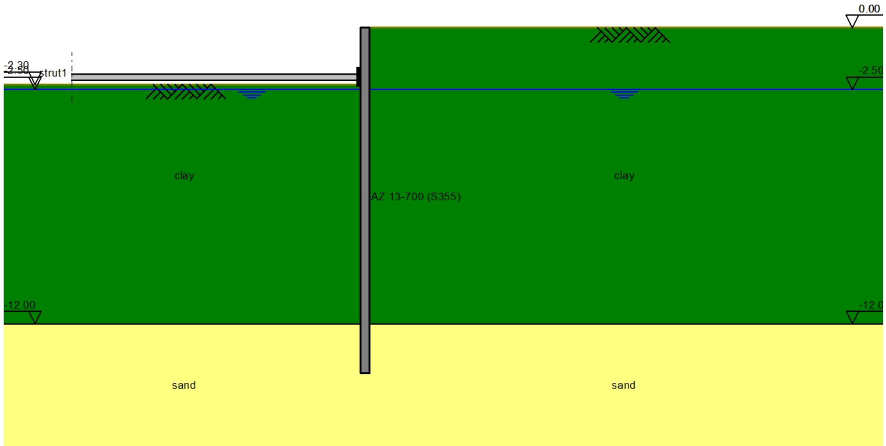
4 Outline Stage 2: Excavation_2m

Outline - Stage 2: Excavation_2m

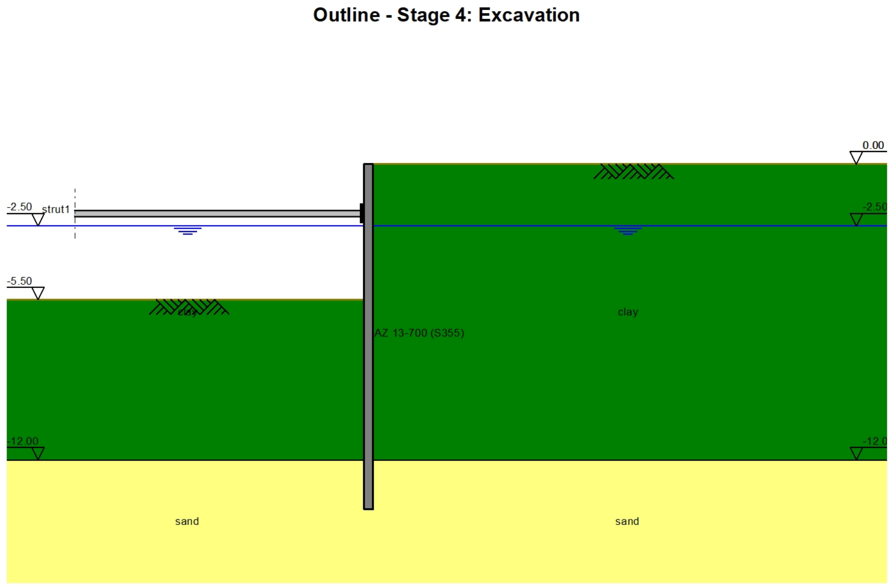


5 Outline Stage 3: Excavation + anchor

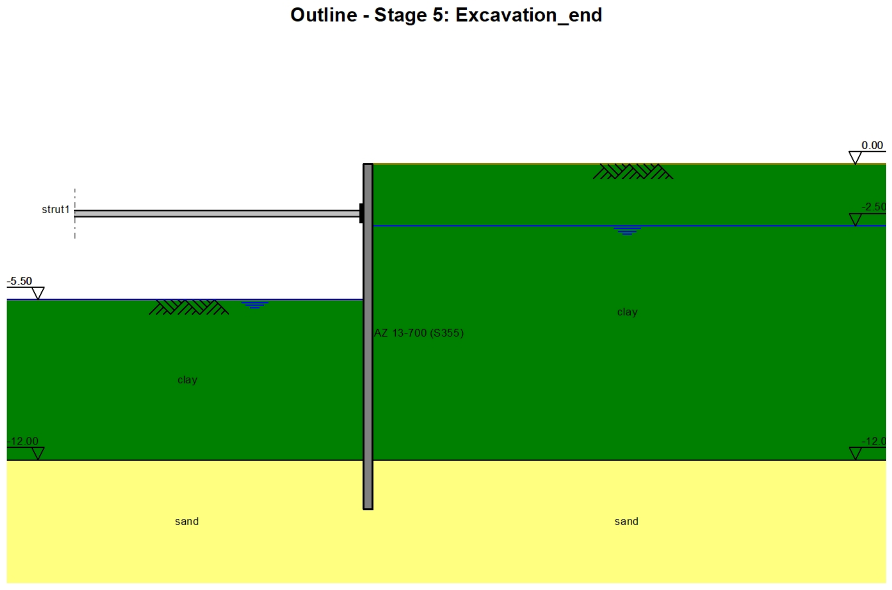
Outline - Stage 3: Excavation + anchor



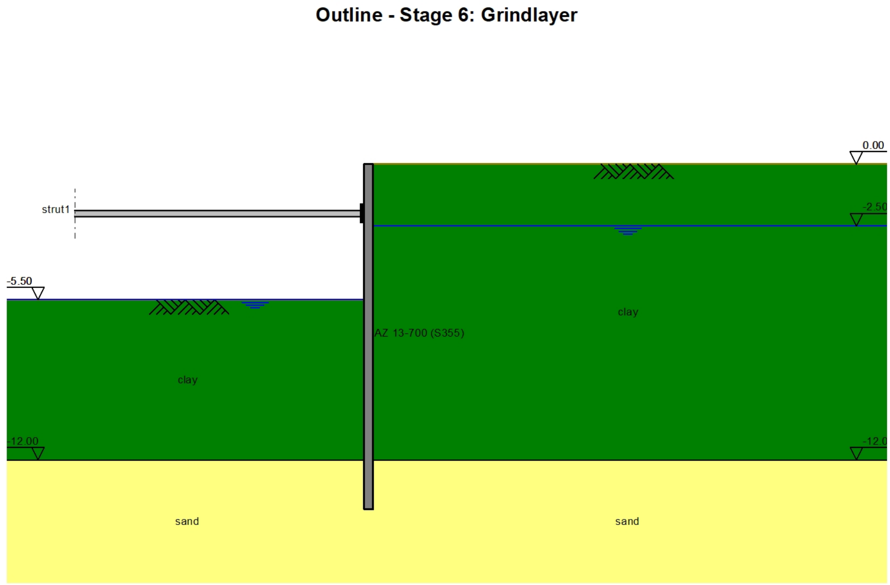
6 Outline Stage 4: Excavation



7 Outline Stage 5: Excavation_end

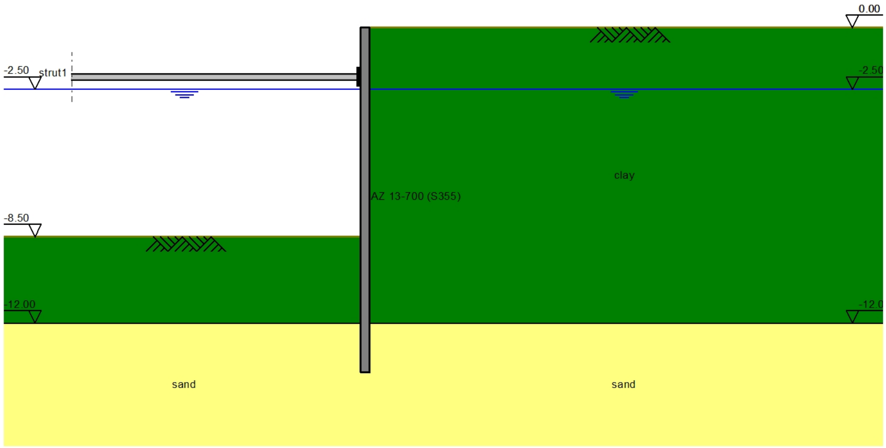


8 Outline Stage 6: Grindlayer

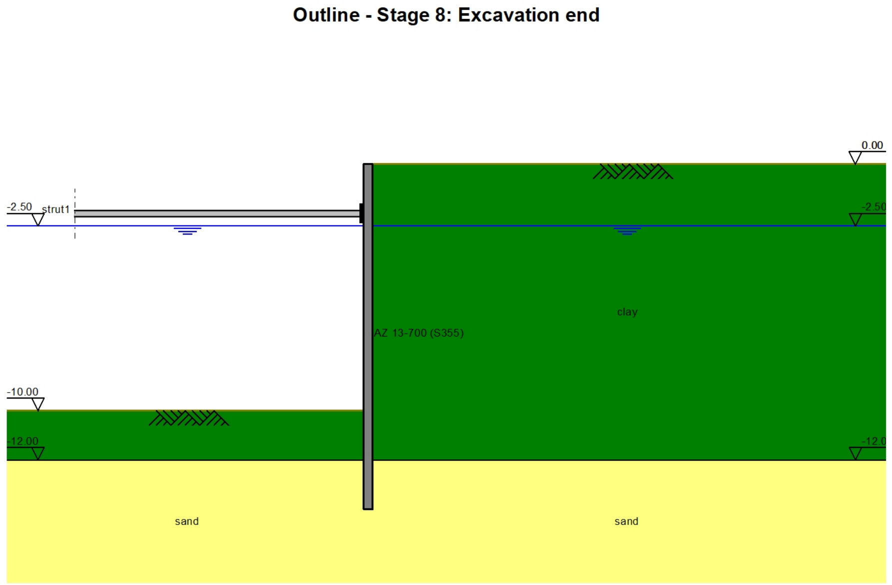


9 Outline Stage 7: Excavation -8.5

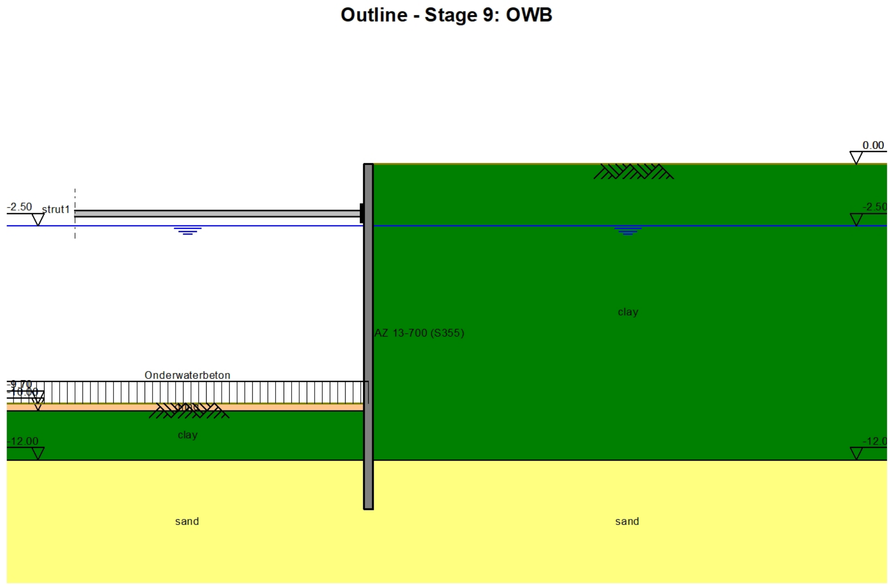
Outline - Stage 7: Excavation -8.5



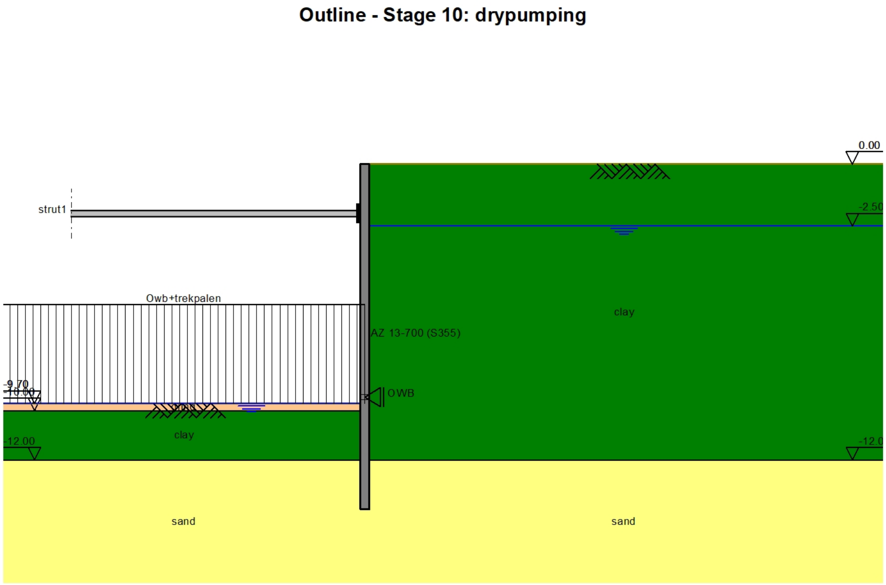
10 Outline Stage 8: Excavation end



11 Outline Stage 9: OWB



12 Outline Stage 10: drypumping



End of Report

Report for D-Sheet Piling 18.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Date of report:	5/12/2019
Time of report:	8:50:37 PM
Report with version:	18.2.1.20477
Date of calculation:	4/25/2019
Time of calculation:	1:59:59 PM
Calculated with version:	18.2.1.20477
File name:	C:\..\Documents\Merckt, Groningen\DsheetPiling\DSP01_Adjusted_OM
Project identification:	Merckt te Groningen Doorsnede 4_DSP01 (westzijde)

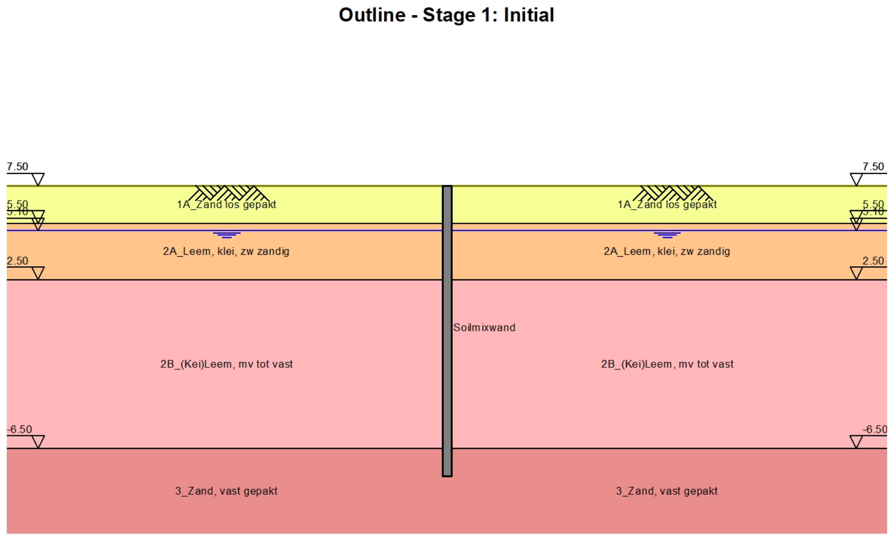
Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overall Stability per Stage

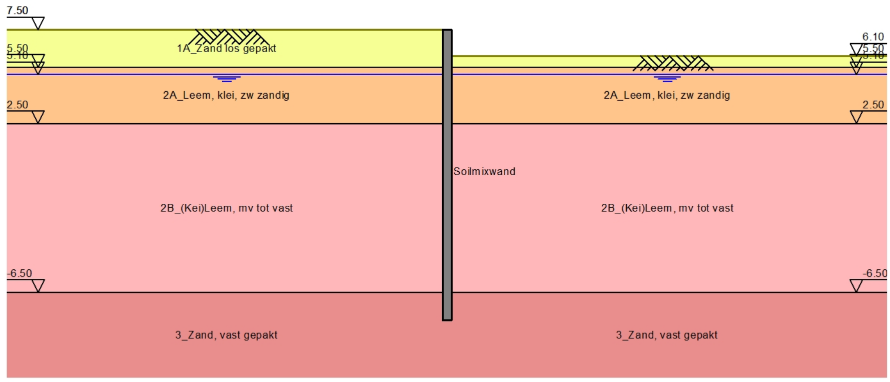
Stage name	Stability factor [-]
Initial	10000.00
ontgraven NAP +6.1	21.73
Ontgraven NAP +3.52	6.55
Ontgraven NAP 0,60	3.00
Ontgraven NAP -1,64	1.83
OWB+grondv	1.79
Verwijderen 3e stempellaag	1.79
Verwijderen 2e stempellaag	1.79
Verwijderen 1e stempellaag	1.79

2 Outline Stage 1: Initial



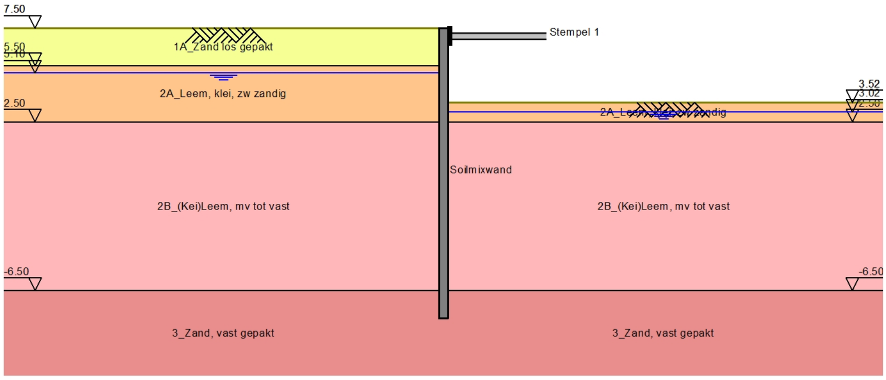
3 Outline Stage 2: ontgraven NAP +6.1

Outline - Stage 2: ontgraven NAP +6.1

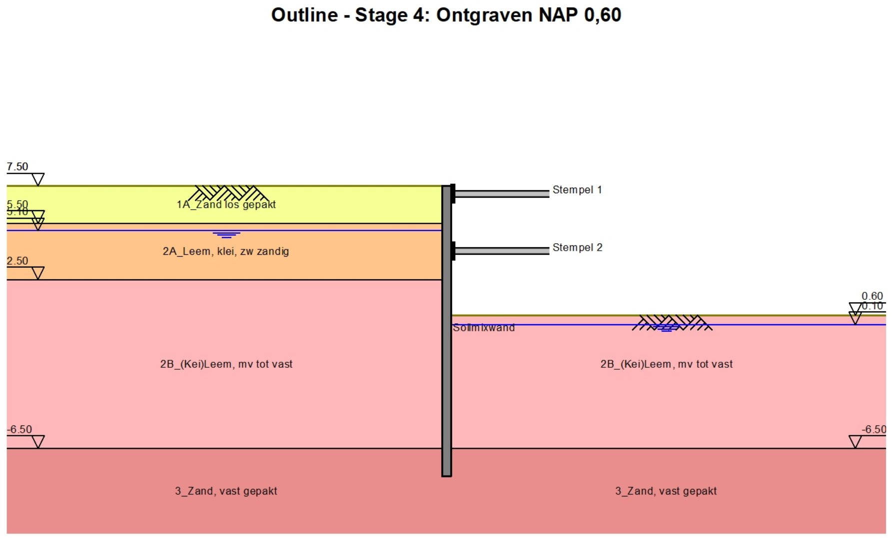


4 Outline Stage 3: Ontgraven NAP +3.52

Outline - Stage 3: Ontgraven NAP +3.52

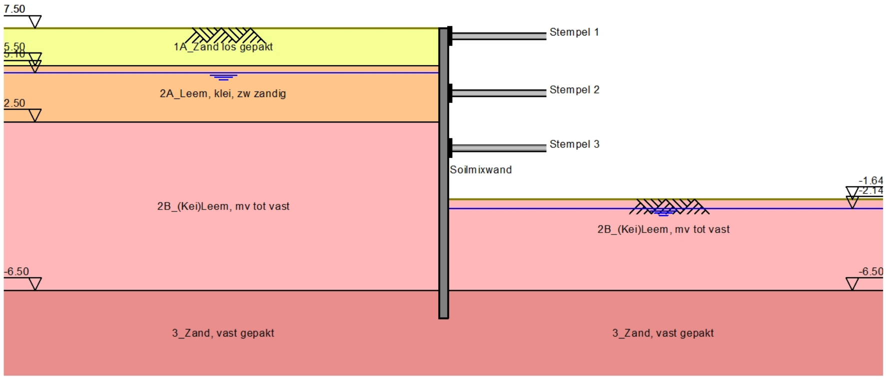


5 Outline Stage 4: Ontgraven NAP 0,60



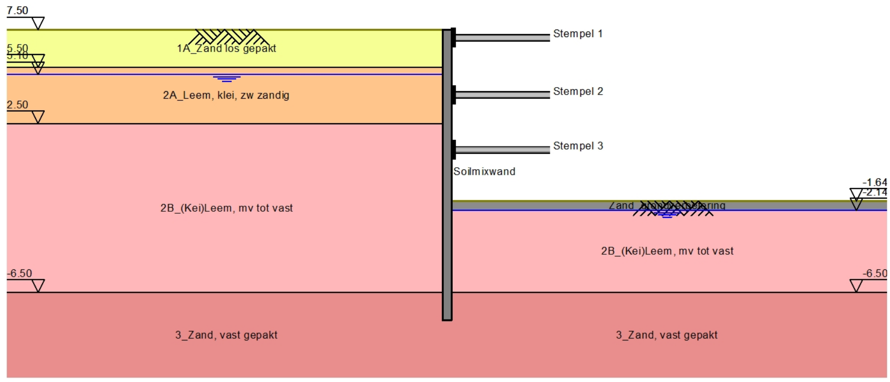
6 Outline Stage 5: Ontgraven NAP -1,64

Outline - Stage 5: Ontgraven NAP -1,64



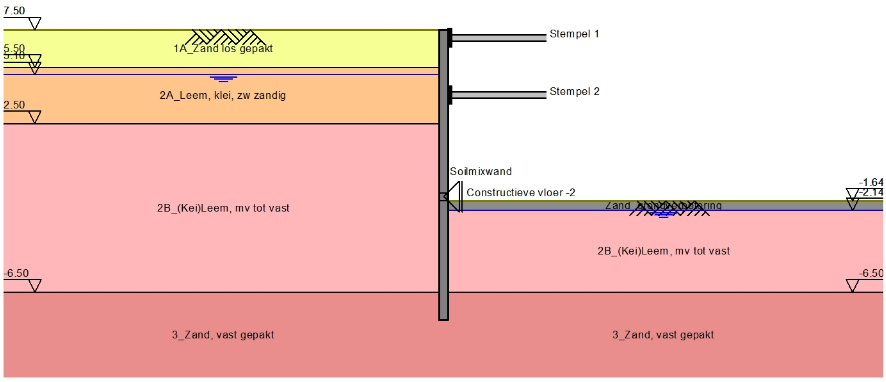
7 Outline Stage 6: OWB+grondv

Outline - Stage 6: OWB+grondv



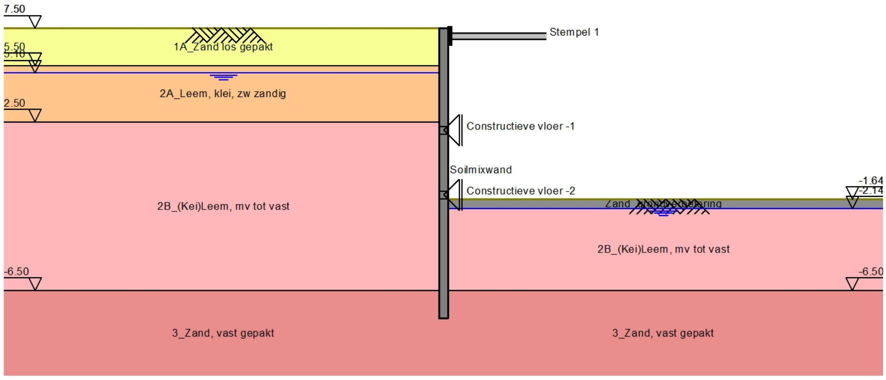
8 Outline Stage 7: Verwijderen 3e stempellaag

Outline - Stage 7: Verwijderen 3e stempellaag

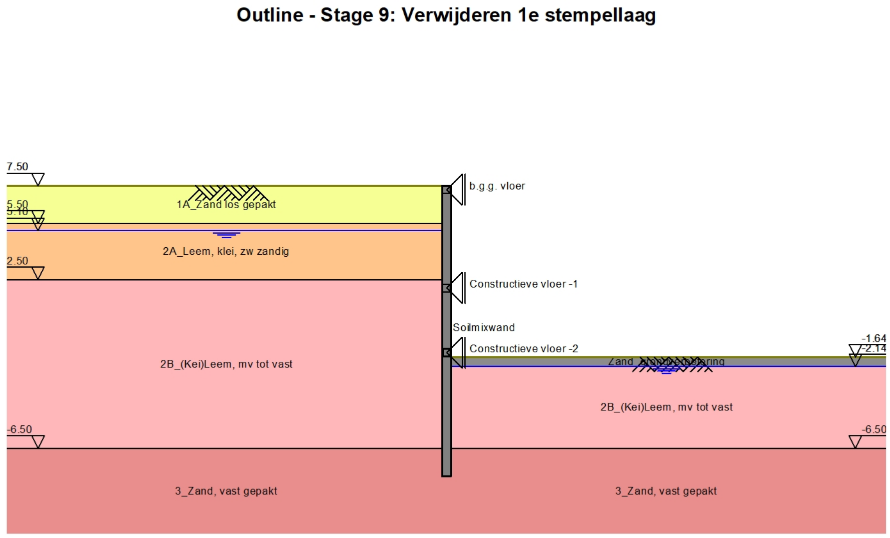


9 Outline Stage 8: Verwijderen 2e stempellaag

Outline - Stage 8: Verwijderen 2e stempellaag



10 Outline Stage 9: Verwijderen 1e stempellaag



End of Report

Report for D-Sheet Piling 18.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Date of report:	5/12/2019
Time of report:	8:46:00 PM
Report with version:	18.2.1.20477
Date of calculation:	5/6/2019
Time of calculation:	1:40:18 PM
Calculated with version:	18.2.1.20477
File name:	C:\..\MC analysis\Amsterdam_OM_Model_50%_Adjustedstratigraphy

Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overview per Stage and Test

Stage nr.	Verification	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	EC7(NL)-Step 6.1		-1.22	0.67	0.0	13.6	---
1	EC7(NL)-Step 6.2		-0.49	0.49	0.0	13.6	---
1	EC7(NL)-Step 6.3		2.65	-1.57	0.0	13.7	---
1	EC7(NL)-Step 6.4		1.14	-1.17	0.0	13.7	---
1	EC7(NL)-Step 6.5	0.0	0.00	0.00	0.0	9.8	---
1	EC7(NL)-Step 6.5 * 1.20		0.00	0.00			
2	EC7(NL)-Step 6.1		33.67	-28.81	0.0	19.6	---
2	EC7(NL)-Step 6.2		20.61	-23.82	0.0	19.5	---
2	EC7(NL)-Step 6.3		37.06	-31.00	0.0	19.9	---
2	EC7(NL)-Step 6.4		22.15	-25.32	0.0	19.9	---
2	EC7(NL)-Step 6.5	3.1	31.60	-24.91	0.0	13.7	---
2	EC7(NL)-Step 6.5 * 1.20		37.92	-29.89			
3	EC7(NL)-Step 6.1		39.28	-27.99	17.7	18.7	---
3	EC7(NL)-Step 6.2		25.43	-24.56	17.7	18.8	---
3	EC7(NL)-Step 6.3		39.61	-27.80	17.7	18.8	---
3	EC7(NL)-Step 6.4		25.90	-24.35	17.8	18.9	---
3	EC7(NL)-Step 6.5	2.5	29.52	-23.42	12.2	13.0	---
3	EC7(NL)-Step 6.5 * 1.20		35.43	-28.10			
4	EC7(NL)-Step 6.1		73.28	-32.24	20.3	21.5	---
4	EC7(NL)-Step 6.2		47.54	-26.17	20.2	21.5	---
4	EC7(NL)-Step 6.3		73.61	-32.29	20.3	21.5	---
4	EC7(NL)-Step 6.4		47.96	-26.19	20.2	21.5	---
4	EC7(NL)-Step 6.5	4.0	56.62	-26.59	13.6	14.4	---
4	EC7(NL)-Step 6.5 * 1.20		67.95	-31.91			
5	EC7(NL)-Step 6.1		121.06	-70.42	**30.8**	31.6	---
5	EC7(NL)-Step 6.2		77.43	-52.80	**30.8**	**31.9**	---
5	EC7(NL)-Step 6.3		121.24	-70.46	**30.8**	31.6	---
5	EC7(NL)-Step 6.4		77.56	-52.86	**30.8**	**31.9**	---
5	EC7(NL)-Step 6.5	7.7	74.99	-51.36	19.5	20.3	---
5	EC7(NL)-Step 6.5 * 1.20		89.99	-61.64			
6	EC7(NL)-Step 6.1		140.66	-44.90	0.0	21.7	---
6	EC7(NL)-Step 6.2		89.31	-30.46	0.0	21.7	---
6	EC7(NL)-Step 6.3		140.83	-44.94	0.0	21.7	---
6	EC7(NL)-Step 6.4		89.44	-30.45	0.0	21.7	---
6	EC7(NL)-Step 6.5	8.0	87.70	-34.94	0.0	14.5	---
6	EC7(NL)-Step 6.5 * 1.20		105.24	-41.92			
7	EC7(NL)-Step 6.1		140.66	-44.90	0.0	21.7	---
7	EC7(NL)-Step 6.2		89.31	-30.46	0.0	21.7	---
7	EC7(NL)-Step 6.3		140.83	-44.94	0.0	21.7	---
7	EC7(NL)-Step 6.4		89.44	-30.45	0.0	21.7	---
7	EC7(NL)-Step 6.5	8.0	87.70	-34.94	0.0	14.5	---
7	EC7(NL)-Step 6.5 * 1.20		105.24	-41.92			
8	EC7(NL)-Step 6.1		299.98	-217.88	0.0	24.9	---
8	EC7(NL)-Step 6.2		268.64	-225.65	0.0	24.5	---
8	EC7(NL)-Step 6.3		303.39	-224.66	0.0	25.2	---
8	EC7(NL)-Step 6.4		272.06	-232.38	0.0	24.8	---
8	EC7(NL)-Step 6.5	13.8	254.30	-209.55	0.0	15.8	---
8	EC7(NL)-Step 6.5 * 1.20		305.17	-251.46			
9	EC7(NL)-Step 6.1		318.07	-289.58	0.0	24.0	---
9	EC7(NL)-Step 6.2		286.84	-297.16	0.0	23.7	---
9	EC7(NL)-Step 6.3		317.00	-289.83	0.0	24.3	---
9	EC7(NL)-Step 6.4		286.13	-297.34	0.0	24.0	---
9	EC7(NL)-Step 6.5	14.4	272.94	-280.96	0.0	15.2	---
9	EC7(NL)-Step 6.5 * 1.20		**327.53**	**-337.15**			
10	EC7(NL)-Step 6.1		-172.68	181.46	0.0	24.7	---
10	EC7(NL)-Step 6.2		-172.73	190.24	0.0	24.6	---
10	EC7(NL)-Step 6.3		-172.68	184.59	0.0	25.0	---

Stage nr.	Verification	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
10	EC7(NL)-Step 6.4		-172.73	192.84	0.0	24.9	---
10	EC7(NL)-Step 6.5	**31.7**	-156.65	165.84	0.0	15.9	---
10	EC7(NL)-Step 6.5 * 1.20		-187.98	199.01			
Max		**31.7**	**327.53**	**-337.15**	**30.8**	**31.9**	---

1.2 Overall Stability per Stage

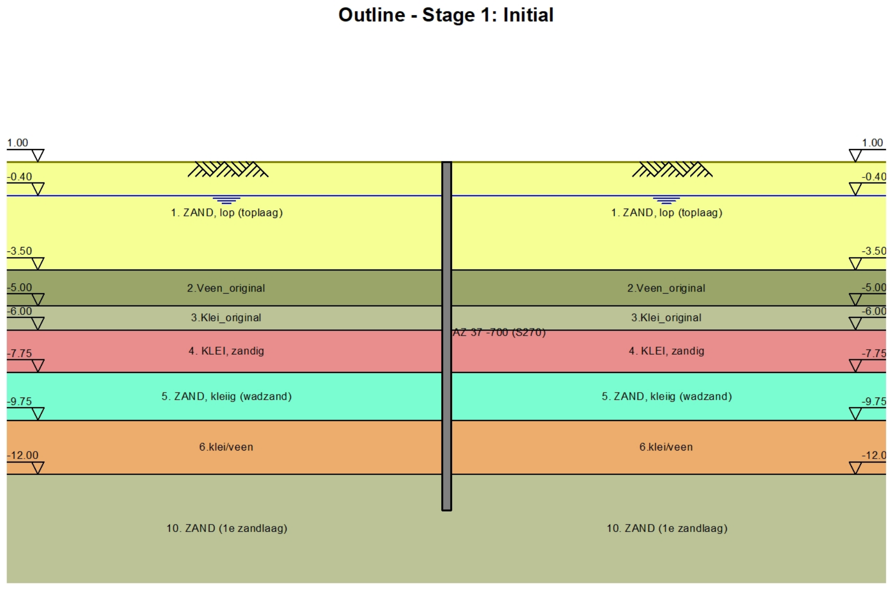
Stage name	Stability factor [-]
Initial	10000.00
BF1- ontgr tbv stempel	7.42
BF 2 - in den natte ontgraven 0.5	5.80
BF 3 - in den natte ontgraven ok grondverb	3.94
BF 4 - aanbrengen grondverbetering	2.80
BF 5 - storten owb vloer_1	4.66
BF 5 - storten owb vloer	4.66
BF 6 - half droogzetten	2.14
BF 6 - volledig droogzetten put	2.03
BF 7 - afstempelen op -1 vloer, bovenstempel verw	2.03

2 Input Data for all Stages

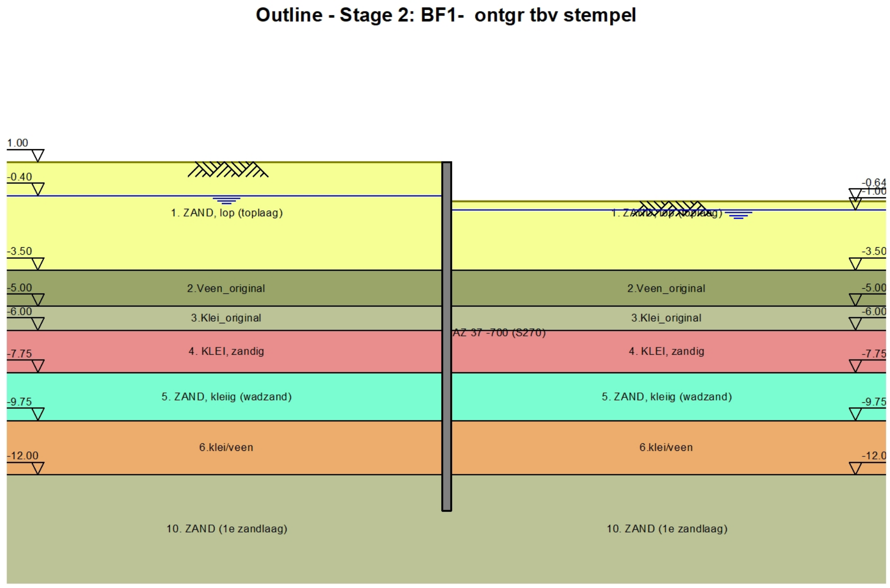
2.1 Calculation Options

First stage represents initial situation	No
Calculation refinement	Fine
Verification	EC7 NA NL - method A: Partial factors (design values) in all sta Eurocode 7 using the factors as described in the National Annex of the Netherlands. It is basically design approach III.
Multiplication factor for anchor stiffness	1.000
Used partial factor set	RC 1
Factors on loads	
- Permanent load, unfavourable	1.00
- Permanent load, favourable	1.00
- Variable load, unfavourable	1.25 User defined
- Variable load, favourable	0.00
Factors on representative values	
- Partial factor on M, D and Pmax	1.20
Material factors	
- Cohesion	1.15
- Tangent phi	1.15
- Delta (wall friction angle)	1.15
- Modulus of low representative subgrade reaction	1.30
Geometry modification	
- Increase retaining height	10.00 %
- Maximum increase retaining height	0.50 m
- Reduction in phreatic line on passive side	0.20 m
- Raise in phreatic line on passive side	0.20 m
- Raise in phreatic line on active side	0.05 m
Overall stability factors	
- Cohesion	1.30
- Tangent phi	1.20
- Factor on unit weight soil	1.00

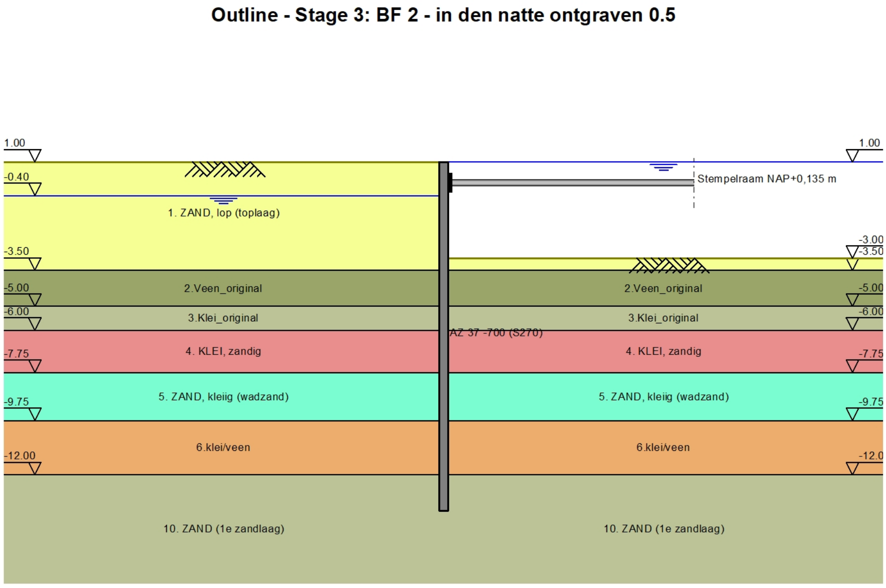
3 Outline Stage 1: Initial



4 Outline Stage 2: BF1- ontgr tbv stempel

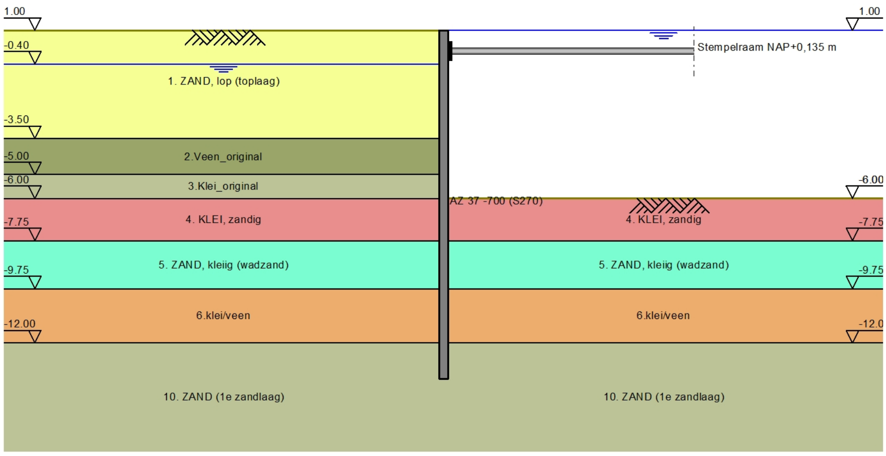


5 Outline Stage 3: BF 2 - in den natte ontgraven 0.5

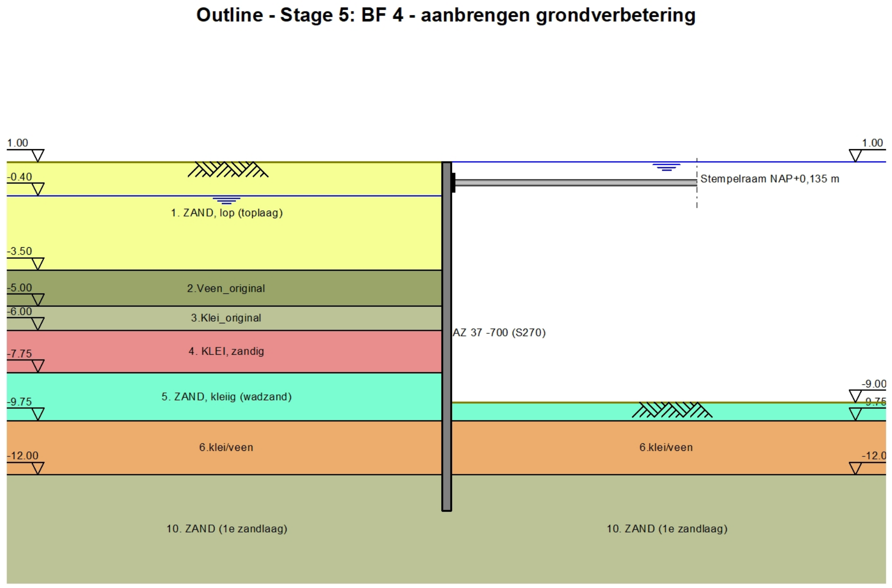


6 Outline Stage 4: BF 3 - in den natte ontgraven ok grondverb

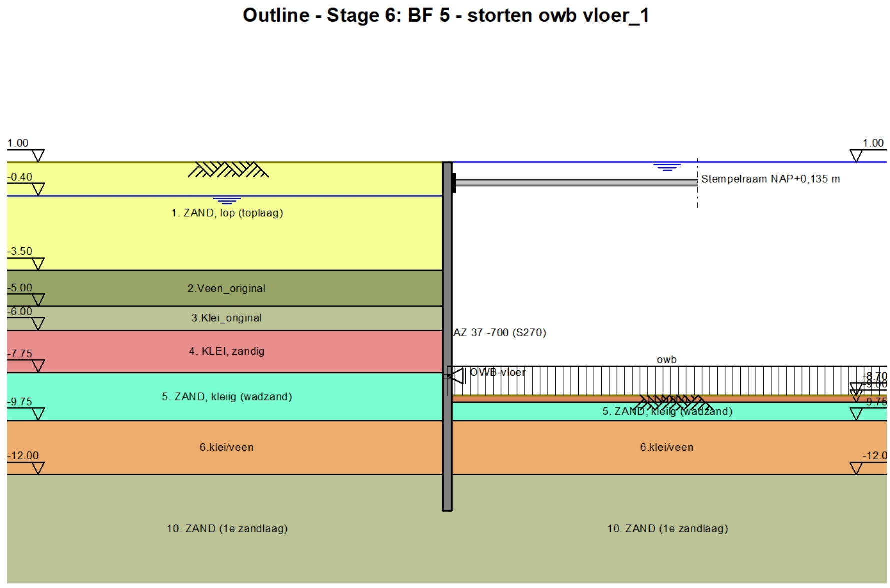
Outline - Stage 4: BF 3 - in den natte ontgraven ok grondverb



7 Outline Stage 5: BF 4 - aanbrengen grondverbetering

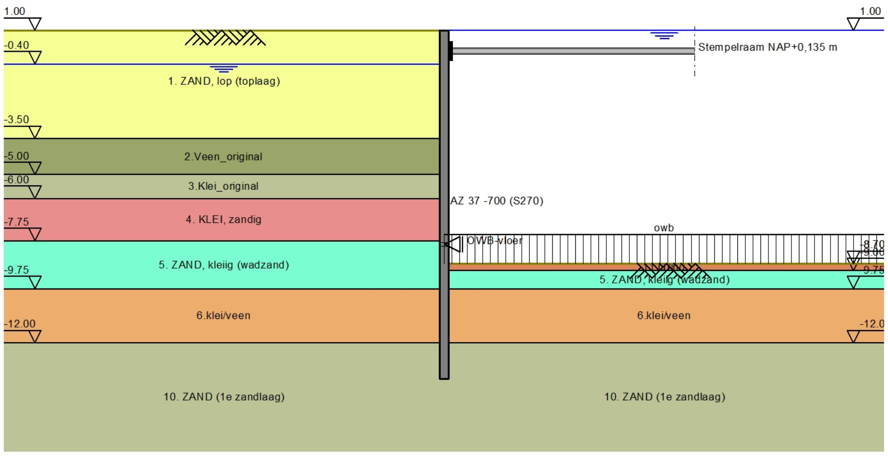


8 Outline Stage 6: BF 5 - storten owb vloer_1



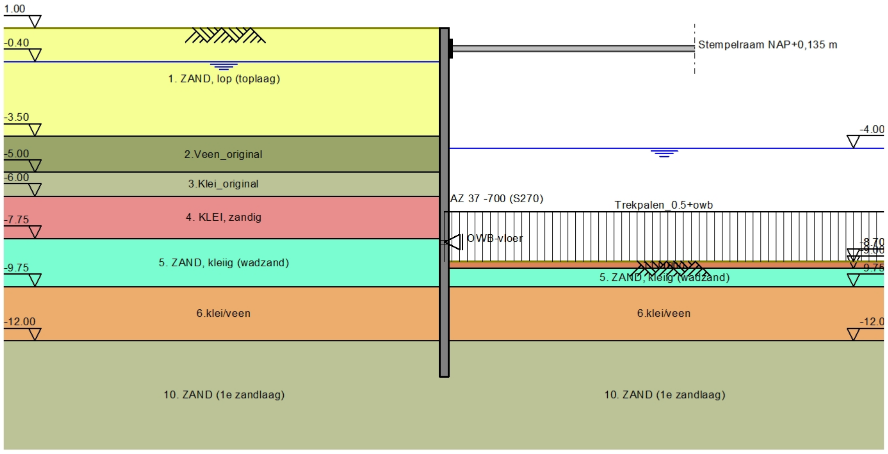
9 Outline Stage 7: BF 5 - storten owb vloer

Outline - Stage 7: BF 5 - storten owb vloer



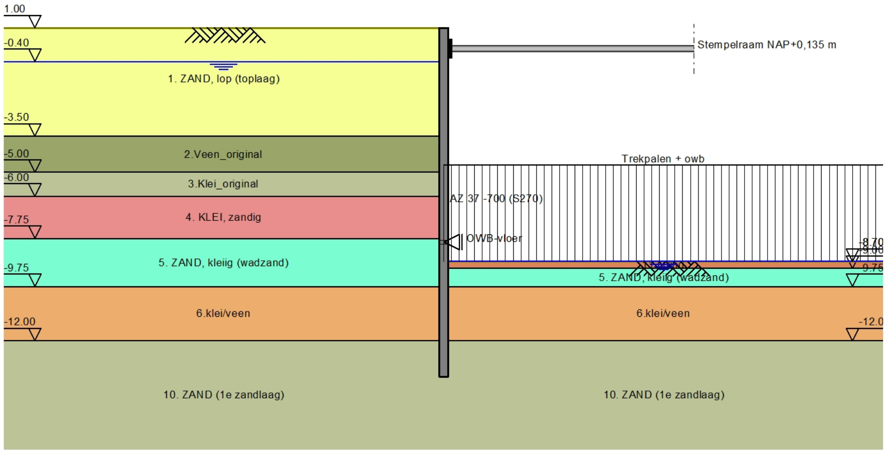
10 Outline Stage 8: BF 6 - half droogzetten

Outline - Stage 8: BF 6 - half droogzetten



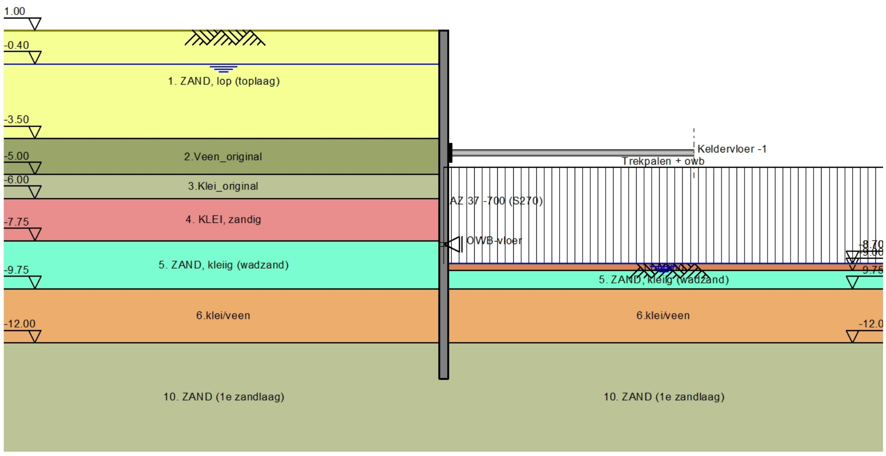
11 Outline Stage 9: BF 6 - volledig droogzetten put

Outline - Stage 9: BF 6 - volledig droogzetten put



12 Outline Stage 10: BF 7 - afstempelen op -1 vloer, bovenstempel verw

Outline - Stage 10: BF 7 - afstempelen op -1 vloer, bovenstempel verw



End of Report