Increasing the Reliability of Settlement Predictions

A Bayesian Approach

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Iv-Infra

Challenge the future

Increasing the Reliability of Settlement Predictions

A Bayesian Approach

by

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in partial fulfillment of the requirements for the degree of

Master of Science
in Civil Engineering

at the Delft University of Technology,
to be defended publicly on Wednesday September 28, 2015 at 15:30 PM.

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PREFACE

This thesis forms the final product in completing the Master track Hydraulic Engineering at the faculty Civil Engineering of the Technical University Delft. The research about increasing settlement predictions with a Bayesian Approach was carried out on behalf of IV-Infra in the period of February till September of the year 2015. The thesis is written for people interested in optimization methods for dike reinforcements projects or other large projects where settlement requirements play an important role.

I would like to thank my graduation committee for guiding me through the thesis project due to their expertise, knowledge and enthusiasm. After each meeting with the committee members also my enthusiasm about the potential of the subject grew. First of all I would like to thanks my daily supervisor from IV-Infra, Ir. Wouter van der Wiel who, with his great enthusiasm, inspired me to reach my goals and with his expertise helped me where needed. His weekly feedback on my work was very useful and supported me by making my decisions. Also Dr. Ir. Timo Schweckendiek and Dr. Ir. Cor Zwanenburg partially working for TU Delft and Deltares were of great importance for the thesis project. Making my own Matlab model was not succeeded without them showing me direction and my report is greatly improved by their feedback. Prof. Dr. Ir. Matthijs Kok, thanks for being the chairman of my committee and thanks for your advice, knowledge and expertise about the subject which helped me to perform my research.

In general I would like to thank my colleagues of IV-Infra for the nice conversations during the daily walk and giving me the opportunity to work in a good working environment. A few persons I would like to thank in person, first Ir. Arco van Sabben who helped me very much with discussions about the geotechnical aspect of my thesis project and who was always ready to answer my questions. Also Ir. Harm-Jan van der Giessen and Ir. Ruud Nooij I would like to thank to show me the way in the dike reinforcement project Kinderdijk—Schoonhovenseveer. Furthermore I am grateful to Pieterjan van Boven of Wiertsema & Partners, Peter Damen of Water Board Rivierenland and Emile Crusifix of Mourik Groot-Ammers B.V. for their help in obtaining specific data about the project.

Finally I would like to thank my study friends for giving me a great time at the University, this great time would not have been possible without the unconditional support of my parents and family. Last but certainly not least I would like to thank my girlfriend who is always supporting me by giving good advice and confidence.

Rick Cornelis Johannes van der Meijs
Delft, September 2015

Executive Summary

Introduction

As a result of the negative results of the second safety assessment of flood defences in the Netherlands, Water Board Rivierenland needed to improve the safety of the dike section Kinderdijk—Schoonhovenseveer (KIS). The KIS project is included in the flood protection program of the Netherlands. About 10 km of the dike stretch between Kinderdijk and Schoonhovenseveer does not fulfill the safety requirements. An insufficient safety level of the dike along the river Lek is in a large extent caused by the two failure mechanisms overflow and overtopping and macro instability of the inner slope. In order to meet the requirements of the two failure mechanisms four solutions are obtained, one of the four solutions is a new dike on the river side of the old dike. Building the new dike will cause deformations of the subsoil, however the new dike needs to meet the height and settlement requirements stated by the Water Board. Residual settlement will take place during the lifetime of the dike structure and is not easy to predict due to the heterogeneous character of the subsoil, uncertainties in soil model parameters leading to conservatives assumptions, and the relative short period of doing the settlement prediction with respect to the long term period of the lifetime of the dike. These three difficulties lead to uncertainties in the settlement prediction which can be reduced by using settlement plate information to update the prediction, see Figure 1.

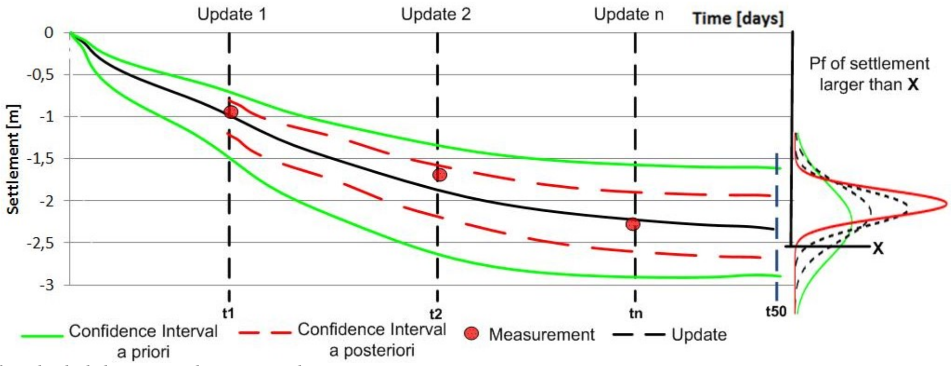


Figure 1: Updated reliability on settlement predictions

From the point of view of the client, the Water Board, it must be ensured that the requirements of the (residual) settlement are met, if not it can cause large financial consequences when it is needed to strengthen the dike again on short notice. With monitoring information the actual settlement behavior is obtained and this is used in order to reduce the uncertainties and increase the reliability of the settlement predictions. The elongated profile of a dike structure gives large opportunities in reducing construction costs by optimizing the dike design. This thesis shows how to use the monitoring data of the subsoil to increase the reliability of the settlement predictions and what the obtained information could mean for the construction process.

Probabilistic Settlement Predictions

The methods and models used to update the settlement predictions are combined in order to obtain the desired results. Modeling the settlement behavior is done by the a,b,c–isotache model together with the Terzaghi consolidation model in 1D space. Den Haan [1994] showed how to model soft soil behavior under loading conditions by making use of isotache. Strains in the soft subsoil are the result from increasing load and can be divided into direct strain, that occurs instantly and secular strain with a stress dependent and independent part (creep). With the isotache parameters a,b and c the strains in the subsoil are described, a describes direct strain while b is linked to the intrinsic time of the subsoil. Parameter c describes the creep phase of the subsoil and is therefore coupled to the stress independent part of the secular strain. By using the incremental form of the a,b,c–isotache model the settlement behavior is modeled in the software program Matlab. To execute the model in a probabilistic approach the input parameters of the settlement model are taken as stochastic parameters with a normal distribution, and with a Monte Carlo Simulation the a-priori expected settlement is computed.

Knowing the a-priori settlement prediction the uncertainties can be reduced by the monitoring data that is available. Following Bayes' Theorem the monitoring information is used in a way that the posterior settlement prediction has a conditional probability given that the obtained monitoring data is true. The failure probabilities are calculated with the use of limit state functions $Z = R - S$, where R is the resistance and S is the solicitation. Failure of the limit state function occurs when $Z < 0$. The evidence ε is resembled by the monitoring data, and to update the failure probability $P(F)$ an observation limit state function h is introduced according to the method of Straub [2011]. Monitoring data as obtained by settlement plates are of the equality type of information, the difficulty is that when using this information in a probabilistic way the probability of the observation approximates zero due to not having a surface under the probability density function. By using Straub [2011] the equality information is transformed into equivalent inequality information and the a-posteriori predictions can be obtained. This is done via the direct update method that implies that the a-priori settlement realizations are updated with each measurement by looking at the prior realizations compared to the measurement at the same point in time. When the prior realizations fall within the range of the measurement its satisfies the statement and is considered as information that resembles the actual settlement behavior. By using multiple measurements at different points in time the reliability of the settlement prediction is increased by obtaining shifted and narrowed probability density functions. Another type of updating is the indirect update method where the individual variables are updated for each measurement, however this method is not used in this thesis due to the mathematically same results (Schweckendiek [2014]) as the direct update and because this method is time consuming and must take into account correlation changes. To verify the updating method the accepted software program D-Settlement is used to compare the results. Correlations between the isotache parameters are obtained from actual data sets of the KIS project, these correlations are used in the model and the consequence of these correlations is a larger uncertainty of the settlement prediction.

CASE STUDY KINDERDIJK—SCHOONHOVENSEVEER

The KIS project is divided into several sections along a 17 km stretch. Section W is located in the municipality Streefkerk, it is more or less situated in the middle of the KIS project between the municipalities Kinderdijk and Schoonhovenseveer. The case study is done with information obtained by measurements of settlement plate W27 from section W. Water Board Rivierenland stated three requirements in the contract document regarding the settlements. These requirements focus on the minimum design height NAP +5,40 m of the dike during lifetime and two short and long term residual settlement requirements (a maximum of 0,1 m 1 year after completion and 0,3 m 50 years after completion). The project is executed in a semi-probabilistic way (partial safety factors) and consist of the following parts: the check on the reference design, adjustment of schematization, settlement calculation, check of stability during construction and determination of the stability after construction. However the computation of the expected settlement is based on expected or deterministic values for the subsoil variables. The construction method is as follows, first a soil improvement is applied after which the work floor is constructed made out of sand. Vertical drainage is installed as well as horizontal drainage to induce the outflow of water and speed up the consolidation process. After the application of the drainage system the clay layers are constructed with a thickness of 1 meter each, the period in between the raises is set on a minimum of 30 days in order to guarantee the safety regarding the stability of the dike structure. In total 6 clay layers are constructed and a 7^{th} clay layer with a variable thickness which is determined by using the monitoring information. The total height without the last clay layer is NAP +8,0 meter. In order to keep track of the soil behavior a monitoring network is installed that consist of the following instruments: piezometers, GPS devices and settlement plates. The information of the settlement plate is used in this thesis to update the settlement predictions. Measurements are in theory done according to a specific scheme at specific points during construction in order to cover the total settlement behavior of the subsoil, for instance measuring one day before and after a raise to gain insight in the direct settlement that took place due to the raise.

CASE STUDY - POSTERIOR ANALYSIS

The methods and models to execute the case study are combined in the proposed model constructed with Matlab. The update is carried out on the basis of three limit state functions that resemble the three settlement requirements. To find an optimized new dike design several scenarios are introduced that differ from each other in the way the 6^{th} and 7^{th} clay layer are constructed. Thicknesses of these layers are varied and some scenarios have instead of the 7^{th} clay layer a temporary overburden that was needed to meet the residual settlement requirements. The scenarios with the temporary overburden are giving better results with respect to the residual settlement requirements. To verify the results the most promising scenario is also computed with D-Settlement, the results of this model gave a larger reduction of the reliability and showed that the total and residual settlement that were expected after updating are smaller. A possible reason that the results from D-Settlement are more favorable is because D-Settlement uses the least square method in order to fit the predictions on the measured settlement, however by using the least square method the mean of the possible outcomes become highly important with respect to the values found in the tails of the parametric distribution. The method used in this thesis also takes the extreme values of the tails into account during the update of the settlement predictions. This could be an indication that D-Settlement actually underestimates the expected settlement due to giving less importance to the values found in the tails of the parametric distributions. It is seen from both updating models that the residual settlement requirement on short term (1 year after completion) is easily complied, and that the long term requirement is also met for several scenarios after the update. Because of modeling the settlement behavior in 1D space in this thesis it is investigated whether the results in 2D space are different due to spatial variability of the stress distributions in the subsoil. It is noticed that in 2D space the total and residual settlement are somewhat larger than in 1D space. For the proposed model measurements had to be selected in order to obtain reliable results. Because of a lack of computation strength and the filter effect of the direct updating method, measurements are chosen for specific phases on the settlement curve to cover the total settlement behavior. Due to the short period in which the measurements are done it is difficult to increase the reliability of residual settlement predictions. The settlement behavior of the subsoil finds itself alternately in the consolidation phase and creep phase during construction and therefore it is difficult to give reliable predictions about the residual settlement without the use of a temporary overburden. This lack of information about the creep part of the settlement curve can be obtained by continuing the monitoring of settlement plates in the period of the temporary overburden.

OPTIMIZATION NEW DIKE DESIGN

Due to the probabilistic approach in this thesis it was necessary to obtain information about what the accepted probability of failure is of not meeting the settlement requirements during lifetime of the Water Board Rivierenland. After contacting the Water Board, the accepted probability of failure was said to be between 5% and 15%. In this research it is chosen to be not too conservative or progressive and thus the accepted probability of failure is determined on a value of 10%. This means that not meeting the settlement requirements has an exceedance probability of 10%. With this knowledge the best scenario is determined, it seemed that scenario 16 with the posterior settlement predictions complies to all settlement requirements, although the prior settlement predictions do not meet the failure probability. An optimized design for the contractor is obtained by meeting the settlement requirements close to the accepted failure probability. Scenario 16 has a 1 meter thick 6^{th} clay layer and a temporary overburden of 0,7 meter. The financial and societal benefits for the contractor and the client are investigated by knowing the optimized clay layer thicknesses. With respect to the prior situation which complies to the settlement requirements (a 1 meter thick 7^{th} clay layer without a temporary overburden) the financial benefits are computed. It turned out that by using only the settlement plate information the reduction in costs are as high as €11,47 per m^2. This number is extrapolated to whole section W and eventually also to the total surface of riverward dike reinforcements of the KIS project. The reduction in costs are approximately €50.000 for section W and €426.000 for the whole KIS project. These economical benefits are for the contractor, however the Water Board is ensured that the settlement requirements are met with an exceedance probability of 10% during lifetime what leads to the fact that the dike does not need to be reinforced on short notice again (10 to 15 years) what could again lead to high costs and large societal impact for the surroundings. It must be noted that with the approach of the updating method in this thesis the influence of the selected measurements on the posterior settlement predictions is large, due to the choice that is made which measurements are used. It turned out that during the construction process the direct updating method is a convenient tool to update the predictions in a fast and reliable way.

CONCLUSION

After doing research conclusions are drawn to answer the main questions stated in the introduction. The actual optimization of the dike reinforcement design is made possible by the proposed model in this thesis. By using the combination of the a,b,c—isotache model and the Bayesian Updating method based on Straub in a probabilistic approach, the (residual) settlement predictions are made more reliable with the help of settlement plate information due to a reduction of epistemic uncertainty. As a result the dike reinforcement is corrected for these updated predictions into an optimized solution which is more reliable and saves costs. It is concluded that by introducing correlation coefficients for the compressibility parameters, the bandwidth of the settlement becomes larger with respect to the uncorrelated parameters. This is against the assumption often made that the use of uncorrelated variables is conservative. By means of using the three limit state functions an optimized design is found for the layer thickness. Scenario 16 which uses a temporary overburden instead of the initial designed last clay layer of 1 meter is chosen as the best scenario. The optimized scenario complies for all three settlement requirements regarding the accepted probability of failure by Water Board Rivierenland of 10%. The combination of methods and models presented in this thesis are verified by the accepted software program D-Settlement which shows similar results, but does not visualize the influences of each individual measurement on the posterior settlement prediction. By increasing the reliability of the settlement predictions, financial benefits are obtained for the contractor as well as the Water Board. It is showed in this thesis that the contractor could roughly save 0,5% of the total project budget of KIS. Water Board Rivierenland has the financial and societal benefit of not having to reinforce the dike again in short notice because of the accepted failure probability of 10%. The consequences of not meeting the settlement requirements are entirely for the Water Board. Due to a change in dike safety assessment procedures in nearby future due to the implementation of WTI 2017, possibly many new dike reinforcement projects have to be carried out. Therefore are large opportunities for contractors and clients to optimize their designs with the method presented in this thesis.

List of Symbols

a	direct strain parameter of the a,b,c–isotache model [-]
b	secular strain parameter of the a,b,c–isotache model [-]
c	creep strain parameter of the a,b,c–isotache model [-]
$cov(X,Y)$	covariance that describes the dependencies between two (or more) stochastic parameters X and Y [-]
c_v	consolidation coefficient of a soil type [m^2/s]
C	correlation matrix, shows the correlations between multiple variables [-]
D	layer thickness of layer in subsoil [m]
ε^H	natural vertical strain parameter (Hencky strain) [-]
ε_d^H	direct natural vertical strain parameter [-]
ε_s^H	secular natural vertical strain parameter [-]
ε_e	equivalent inequality domain when using equality information in Bayesian updating, it is the domain where $h_e < 0$ [-]
ε	evidence in Bayesian updating, evidence could be monitoring data, observations etc. [-]
F	failure domain for the limit state function Z where $Z < 0$ [-]
γ	volume weight of soil type [kN/m^3]
γ_w	volume weight water [kN/m^3]
h_0	initial layer thickness [m]
Δh	change in layer thickness [m]
h_e	equivalent observation limit state function obtained by using equality information in the Bayesian updating process [-]
L	likelihood, evaluates the effect of information on an uncertain parameter [-]
μ	mean of a stochastic parameter [-]
p_0	applied load on subsoil [kPa]
$P(A)$	probability of a certain event A [-]
$P(A\|B)$	conditional probability of A given that B is true [-]
$P(F\|\varepsilon)$	conditional failure probability F given that information ε is true [-]
$P(F \cap \varepsilon)$	the probability that F and ε are true [-]
R	resistance in limit state function [-]
ρ	correlation coefficient, measures the statistical dependence between two variables between -1 and +1 [-]
s_m	measurement of the settlement [m]
S	load in limit state function [-]
S	settlement of the subsoil [m]
σ	standard deviation of a stochastic parameter [-]
σ'_v	vertical effective stress in the subsoil [kPa]
σ'_i	vertical effective stress in the subsoil on i^{th} time step [kPa]
σ'_{vp}	vertical limit strain in the subsoil [kPa]
$\Delta\sigma'$	change in vertical effective stress in the subsoil due to an applied load [kPa]
t	time [day]

T equivalent time in Terzaghi's consolidation theory [day]

Δt time step taken to compute variables in time [day]

τ_i intrinsic time on i^{th} time step [day]

U consolidation degree according to Terzaghi's consolidation theory, values between 0 and 1 [-]

U standard normal distribution [-]

Z limit state function, when $Z < 0$ the system fails [-]

Glossary

1D model	Physical process modeled in one direction, no spatial varieties in model.
Bayes' Theorem	A theory for determining conditional probability named after 18^{th} century British mathematician Thomas Bayes. The theorem provides a way to update existing predictions conditional of using new evidence.
Compressibility parameters	For the a,b,c–isotache model the compaction of a particular soil type is described by the compressibility parameters a,b and c.
Cone Penetration Test	Very test common in the Netherlands to examine the type of soil in a soft subsoil environment, a cone is penetrated vertically in the subsoil and measures friction and friction number. With this information the soil type for each sub layer could be determined.
Consolidation	The process of converting the increased load on water pressure to the effective soil stress in the subsoil that result in the outflow of water. The consolidation process is faster when applying a vertical drainage system that provides an easier runoff.
Correlation	When two variables are dependent of each other they are correlated. Positive correlation: when the one increases, the other variable also increases. Negative correlation: when the one increases, the other variable decreases.
Creep	A continuously during deformation of the subsoil when in the past a change in loading took place The subsoil keeps on deforming despite that change in loading is stopped, the subsoil is in a viscous state.
Direct reliability updating	Bayesian updating method which immediately updates the uncertain parameter conditional to the evidence.
Direct strain	Strain that occurs instantly when applying a load on the subsoil.
Epistemic uncertainty	Uncertainty due to lack of knowledge about the physical processes or by a lack of data.
Equality information	Monitoring data that gives information that equals a value, not possible to use straightforward in probability theory because the probability of a measurement exactly equal to a number is approximately zero.
High water safety program	Program in the Netherlands where Water Boards and Rijkswaterstaat undertake actions to let the primary flood defenses comply to the legal standard now and in the future.
Hinterland	Land behind a primary flood defense which could be flooded when the flood defense fails.
Indirect reliability updating	Bayesian updating method which first updates the individual uncertain parameters conditional to the evidence and then updates the uncertain parameter bases on the updated individual uncertain parameters.
Inequality information	Monitoring data that gives information that is smaller or larger than a certain value, this type of information can be used in probability theory.
Inherent uncertainty	Uncertainty that represent randomness or variations in physical phenomena. An example of this inherent uncertainty is the water level of the river Lek.
Intrinsic time	The intrinsic time can be seen as an equivalent age of the subsoil which takes the pre loading characteristics of the subsoil into account respective to the subsoil characteristics in case of applying a certain load on the subsoil.
Isotache	Lines with equal rates of strain.
K_0 C.R.S. Test	Compression test where the soil characteristics of a soil type are determined, for instance to determine amongst others the compressibility parameters a,b and c.

KIS	Dike reinforcement project along the river Lek between the villages Kinderdijk and Schoonhovenseveer as a part of the High Water Safety Program of the Netherlands.
Lifetime	The period for which a structure is designed, in most cases for Hydraulic Engineering this is a period of 50 years in which the designed structure must withstand the possible occurring loads during this period.
Monitoring network	Combination of devices which measure the behavior of a physical phenomenon, with the help of the produced information the engineers can steer the project in the desired direction.
Piezometer	A device which measures the water pressure (more precisely, the piezometric head) of groundwater at a specific depth in the subsoil.
Probabilistic Design	A probabilistic approach in designing is build upon input variables using probability density functions and then propagates these density functions through the physical model to obtain uncertain outputs of the variable(s) of interest. Probabilistic Design is used to obtain an optimized design for the structure and to quantify the reliability of a system, or parameters that are part of that system.
Raise	In this thesis a raise is defined as adding a clay layer on top of the surface (in Dutch: ophoogslag) to increase the height of the dike.
Residual settlement	After construction of a structure the subsoil will continue to deform in time, the amount of settlement after completion till the end of the lifetime of that structure is called residual settlement.
Riverward dike reinforcement	Strengthen the dike structure at the river side of the dike (in Dutch: rivierwaartse dijkversterking).
Secular strain	Strain with a viscous behavior which has a stress dependent (parameter b in the isotache model) and a stress independent (parameter c, creep) part.
Sensitivity	Influence of an individual parameter on the physical model that it is part of that specific physical model.
Settlement	Compaction of the subsoil where the pore volume (filled with water and/or air) of the soil is decreased due to a load and thus the layer thickness of the subsoil also decreases.
Settlement plate	Measurement tool to monitor the settlement of the subsoil (in Dutch: zakbaak). It is made out of a bottom plate and a pole perpendicular on the plate. The pole remains visible during the raises because it can be extended if needed.
Soft subsoil	Soil types like clay or peat which are known to be soft and weak relative to the hard and strong bedrock found in much foreign countries.
Statistical uncertainty	Uncertainty related to the parametric distributions of stochastic parameters, normally the parametric distribution is determined from data with statistical tools. Due to the use of these tools there will always be some uncertainty left in the choice of parametric distribution or the parameter values itself.
Water Board	Management organizations in the Netherlands in the field of water safety and water resource management, Water Boards were the first organizations in the Netherlands that formed a local government. Water Boards are often the clients for the dike reinforcement projects.

Contents

1

INTRODUCTION

The subject of this MSc thesis is introduced in this chapter. The problem is analyzed and formulated and after that the research outline and approach are given.

1.1. BACKGROUND

1.1.1. FLOOD DEFENCES IN THE NETHERLANDS

The Water Boards in the Netherlands have the task to prevent inundation of the hinterland by floods from the sea and rivers. Flood defences are designed to withstand extreme conditions (i.e. extreme water levels) to protect the people and objects in the hinterland. The Water Board Rivierenland is responsible for an area of 200.000 ha. that is confined by the river Nether Rhine and river Lek on the north side and the rivers Meuse and Merwede on the south side from the German border till the village Kinderdijk. Rivierenland is responsible for 550 km of primary flood defenses. Since the year 1996 a safety assessment is carried out in the Netherlands for primary flood defenses, this safety assessment is repeated every six years to check whether a flood defense can withstand an extreme load. Because of the (negative) results of the second safety assessment (2001-2006) of flood defences, the Water Board Rivierenland needs to improve the safety of the dike section Kinderdijk—Schoonhovenseveer (KIS). The reinforcement of the dike is included in the flood protection program (In Dutch: Hoogwaterbeschermingsprogramma (HWBP)). See Figure 1.1 for an overview of the project area.

Figure 1.1: Project Area Kinderdijk—Schoonhovenseveer

1.1.2. PROJECT KINDERDIJK - SCHOONHOVENSEVEER

The considered dike is situated between Kinderdijk and Schoonhovenseveer at Gelkenes (municipality Molenwaard). It is part of dike ring 16 (Alblasserwaard and Vijfheerenlanden) and lies at the left bank of the river Lek. The length of the dike is approximately 17,5 km of which 10 km does not fulfill the safety requirements with respect to the stability of the dike during high water. Solutions to strengthen the dike are especially conceived for stability requirements and the height of the dike. The project is located in an area with particularly soft cohesive soil (i.e. clay and peat) on a non-cohesive Pleistocene sand layer at a depth of ± NAP - 13 meter. The soil in the transversal direction of the dike is heterogeneous, it consists of several peat, clay and sand layers. In the longitudinal direction of the dike the subsoil is relatively homogeneous, the clay, peat and sand layers can be found along the 10 km stretch of the dike.

In the project plan of Kinderdijk - Schoonhovenseveer (KIS) there are four solutions conceived to strengthen the dike sections which do not fulfill the height and/or stability requirements. From the safety assessment it is considered that the main failure mechanisms are sliding of inner slope and overflow and overtopping. To illustrate these failure mechanisms, see Figure 1.2.

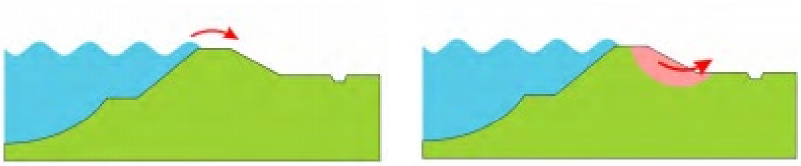


Figure 1.2: Failure mechanisms for the KIS project, left: overflow and overtopping right: macro stability inner slope Vergouwe et al. [2014]

For these two failure mechanisms four solutions are obtained, as listed below.

- A concrete structure on the land side of the dike.
- A diaphragm wall in the crest.
- A stability berm on the inner side of the dike.
- A new dike on the river side of the 'old' dike.

All four solutions listed above are carried out in the KIS project, each specific dike section will has its own solution. This could be one of the above mentioned solutions or a combination of the four solutions. This research is only related to the fourth solution of constructing a new dike. The new dike will be constructed on the river side of the old dike and the old dike becomes the inner berm that will provide a better stability of the dike system. Besides improving the stability of the dike system, also the new dike will be higher than the old dike (a raise between 0,8 meter and 1,5 meter). The new dike will get eventually a crest height of ± NAP + 6,0 meter. Figure 1.3 shows a cross-section of the new dike design in transversal direction.

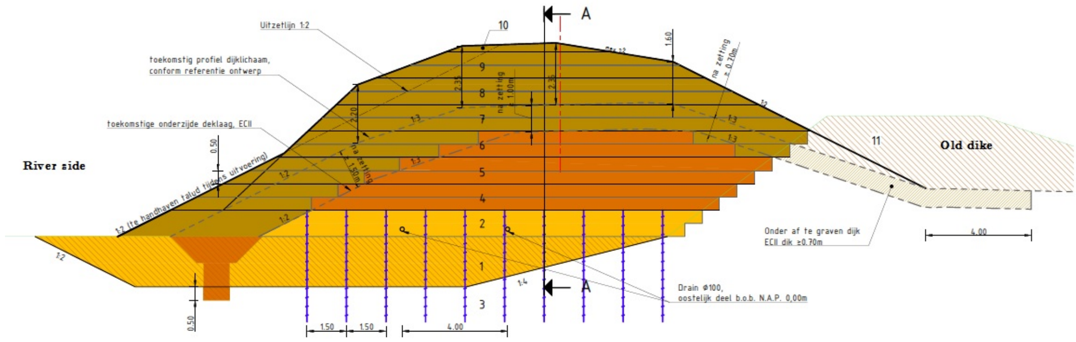


Figure 1.3: Cross-section new dike

As a consequence of building the new dike the subsoil will deform in vertical direction, called settlement. In time the new dike will settle and therefore the crest of the dike will be lowered. For the safety requirements of the dike it is highly important that the dike has a minimum height during its lifetime to prevent overtopping and next to that, strict stability requirements to avoid macro stability problems. See Figure 1.4 for the residual settlement and extra raise, needed to meet the requirements in time. The extra raise is in this thesis defined as the compensation for expected settlement during lifetime.

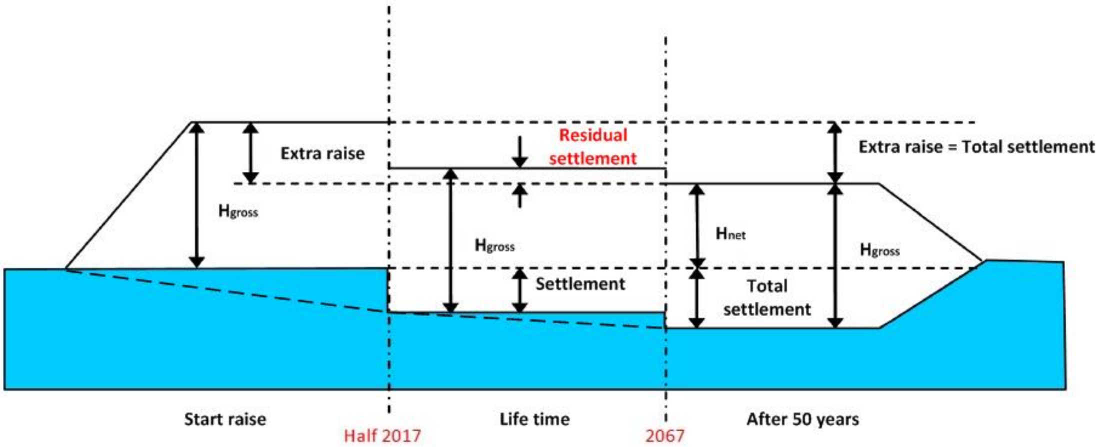


Figure 1.4: Settlement in time Wolters [2014d]

Figure 1.4 comprises the problem stated in this thesis in one figure. In horizontal direction the figure represents the time during construction period and lifetime, on the vertical axis the height of the dike. Before completion the extra raise is constructed to allow for the expected settlements. The new dike structure of the KIS project has a completion date around half 2017, by then already the largest part of settlement occurred, however during the lifetime or user period of 50 years the settlement continues (in a slower rate) and this can be called residual settlement. In the most optimized (i.e. costs) design the total settlement is exactly equal to the extra raise after the 50 years of lifetime.

In theory the prediction of settlement is straightforward, but in practice it is hard to make reliable predictions about settlements because of several reasons. An obvious reason is that the subsoil has a highly heterogeneous character, this is in contrast of the assumptions made in the design that one or two Cone Penetration Tests (CPT) are resembling a whole dike section. Another assumption often made is in the determination of the compressibility parameters, in this thesis a, b and c from the a,b,c–isotache model. These parameters are obtained by K_0 C.R.S. compression tests which tests the samples obtained from the project site. Often there are large differences between the compressibility parameters of the soil layers in the real situation. To translate the lab results of a few bottom samples to a whole soil layer conservative parameters are chosen to make the calculations, which ultimately results in overdimensioned designs. Therefore also the prediction on the residual settlements are quite unreliable on the long term, because there is a lack of knowledge (i.e. epistemic uncertainty) about the soil characteristics in the project area.

There is a need for other ways of obtaining more reliable predictions on residual settlements. For instance one way to make predictions more reliable is to update the reliability with monitoring data obtained from a geotechnical monitoring network. The goal of the thesis project is to make predictions about the residual settlements more reliable and with this new knowledge optimizing the design of a dike section. Especially when looking at the last constructed clay layer (i.e. extra raise) that probably could be less thick.

In order to know the position of this research in the already available literature, a literature study is carried out which goes into possible methods and ways to make settlement predictions more reliable. See Appendix I for the literature study.

1.2. Problem definition

1.2.1. Problem formulation

In Paragraph 1.1.2 it is stated that it is difficult to predict residual settlements of a dike with a certain reliability. Because of several reasons these predictions are hard to make, however, reliable predictions are needed not only to design a new dike but also during the construction of a dike. Taking the dike reinforcement project Kinderdijk—Schoonhovenseveer as example, it is important to know the settlements before constructing the last layer. The predicted settlements are determinative for the eventual thickness of the last clay layer. In the design of the new dike in the KIS project the thickness of the last clay layer is calculated using conservative approximations. This is done because of high uncertainties in the amount of residual settlement of the dike and to prevent that the requirements on residual settlement are not met. The consequence is that the last clay layer is probably overdimensioned and thus will have higher costs. From the point of view of the client, the Water Board, it must be ensured that the requirements of the residual settlement are met in case of a probabilistic approach with a failure probability of 10% (which is explained in Chapter 5), if they do not it can cause large financial consequences for the client when it needs to strengthen the dike again on short notice.

1. The dike reinforcement on the river side of the old dike must have a minimum design height of NAP +5,40 meter after 50 years, according to the binding document BIND-A. Waterschap Rivierenland [2014a]
2. The residual settlement of the dike reinforcement on the river side of the old dike has a maximum of 0,10 meter one year after the last raise. Waterschap Rivierenland [2013]
3. The residual settlement of the dike reinforcement on the river side of the old dike has a maximum of 0,30 meter fifty years after the last raise. Waterschap Rivierenland [2013]

These requirements are introduced by the Water Board due to the fact that the dike needs to satisfy the legal standards that are applicable for flood defences. From a certain extreme water level the needed minimum height of the dike is obtained, which resembles the first of the three requirements. The residual settlement requirements are for a different purpose, namely the usability of the dike structure during lifetime. In case a road is constructed on top of the dike it is not desired to have large settlements which cause the road to become in poor condition.

Apparently there is a need for a method to make predictions on residual settlement more reliable to optimize the design and save costs. See Figure 1.5 for a visual explanation of the advantages of updating the reliability on the settlement prediction. It can be seen that by using information on time t_1, t_2 and t_n (red dots and black dashed lines) the updated parametric distribution (black dashed lines) for the total amount of settlement after lifetime at t_{50} will become narrower and eventually the posterior parametric distribution is obtained (red lines) which is more reliable with respect to the a-priori distribution (green lines).

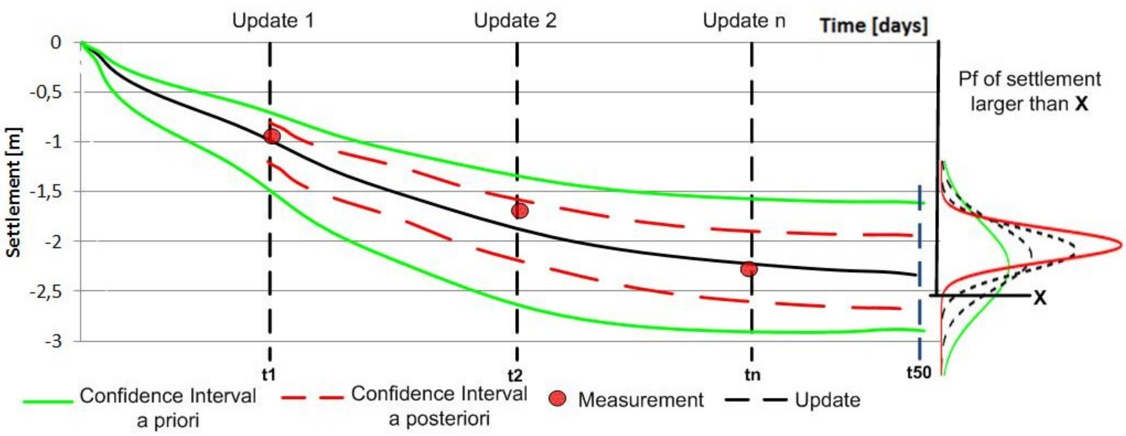


Figure 1.5: Updated reliability on settlement predictions

Figure 1.5 shows that the settlement prediction can be updated by using settlement information during the construction period, however it does not illustrate the relative short period in which this information must be obtained, processed and applied to update the settlement prediction. To illustrate the above, see

Figure 1.6 which shows that the long term settlement prediction (blue shaded area) is based on the very short period of gathering information (red shaded area). Only a small part of the settlement has taken place when doing the prediction, this makes long term prediction of (residual) settlement difficult. Again the red dots are measurements where the updated settlement prediction is based on. Completion of the project is illustrated by the vertical line t_0 and important dates by t_1 (one year after completion) and t_{50} (fifty years after completion).

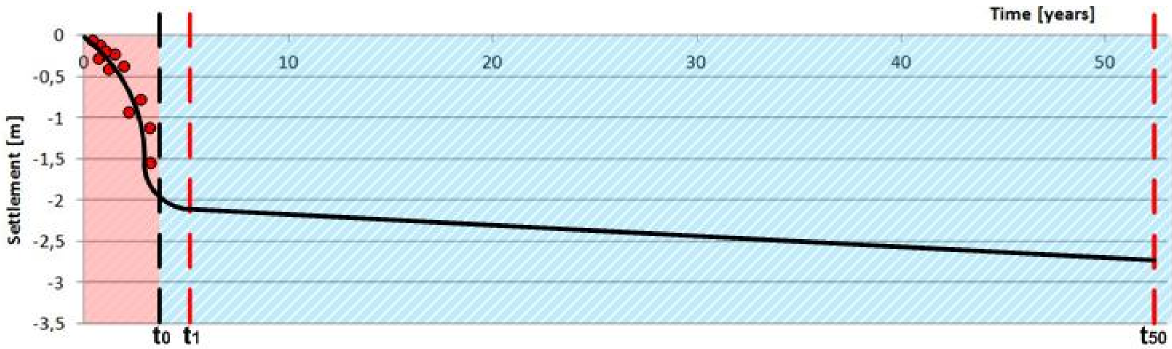


Figure 1.6: Update period vs lifetime of dike reinforcement

Taking into account the long profile such as a dike structure there are large opportunities to optimize the design and reducing the costs. The data from the geotechnical monitoring network can contribute to the reliability of the predictions, because measured[1] settlement values describe the actual settlement behavior of the dike system, instead of probably less reliable computations with a computer model due to uncertainties of the input variables. Using the information about actual settlement behavior aims only on the improvement of the uncertainties of the input variables, the model performance itself is not improved by this data.

1.2.2. MAIN QUESTIONS

The thesis project will aim to answer the following research questions:

1. ***How to make a more reliable prediction for total and residual settlements of a dike reinforcement using settlement plate monitoring data in a one dimensional model?***
2. ***And how to use this information in the construction process?***

An answer on these questions can be found by investigating the problems formulated in Paragraph 1.2.1. The main questions are formulated in a way that it covers the problem formulation.

1.2.3. KEY QUESTIONS

The key questions below are structured in such a way that the process of research starts with the most simple model and with each following question more complex modeling is involved. Eventually the model will approximate the real situation of settlement behavior for a dike structure and from this knowledge an optimization in layer thickness and financial benefits can be made. Also a feeling of which parameters are important in the construction process is obtained.

1. How to predict the residual settlement using a probabilistic a,b,c–isotache model considering a one dimensional homogeneous subsoil and one layer?
2. How to improve the predicted residual settlement of the probabilistic model using monitoring data of measured settlements considering a one dimensional homogeneous subsoil?
3. How to predict the residual settlement of the probabilistic a,b,c–isotache model considering a one dimensional stratified subsoil?
4. How to improve the predicted residual settlement of the probabilistic model using monitoring data of measured settlements and considering a one dimensional stratified subsoil?

[1]Note: there are always measurement errors involved in monitoring data.

5. What is an acceptable thickness of the last clay layer of the dike reinforcement in the KIS project, regarding the updated reliability, and what is the financial benefit of this optimization?

6. What aspects of the updating method are important during the construction process and how to use this knowledge in the construction process?

1.3. Outline & Approach

In this paragraph the report structure is described. The first chapter is used to introduce the problems regarding the prediction of settlement in time for a dike structure, the main questions and key questions are formulated on the basis of this problem formulation.

The second chapter presents the research methods that are used in this thesis, the settlement model is explained as well as the the updating method to obtain more reliable settlement predictions. Also the specific use of these methods are shown for the case study of the project Kinderdijk—Schoonhovenseveer.

To give insight in the characteristics of the case study project Chapter 3 is used to elaborate on the location, requirements regarding the settlement and how the monitoring network is set up. Also the prior settlement predictions are displayed.

Chapter 4 shows the results obtained from both the models used. These results are discussed and conclusions are drawn.

The optimization of the new dike design is described in the next chapter. Chapter 5 goes into the layer thickness of the raise and what the financial consequences are using the results of this thesis. Also the influence of variables during the construction process are described in this chapter. Finally the discussion and conclusions of the optimization are shown.

The last chapter of this thesis goes into the conclusions and recommendations about predicting settlement for dike reinforcements are given.

The research approach that is followed in this thesis is summarized in Figure 1.7. Chapter 2 gives the answer to key questions 1,2 and 3. Key question 4 is answered in Chapter 4 and the last two answers of the last key questions are described in Chapter 5. Answers on the two main questions are given in the concluding part of the thesis, Chapter 6.

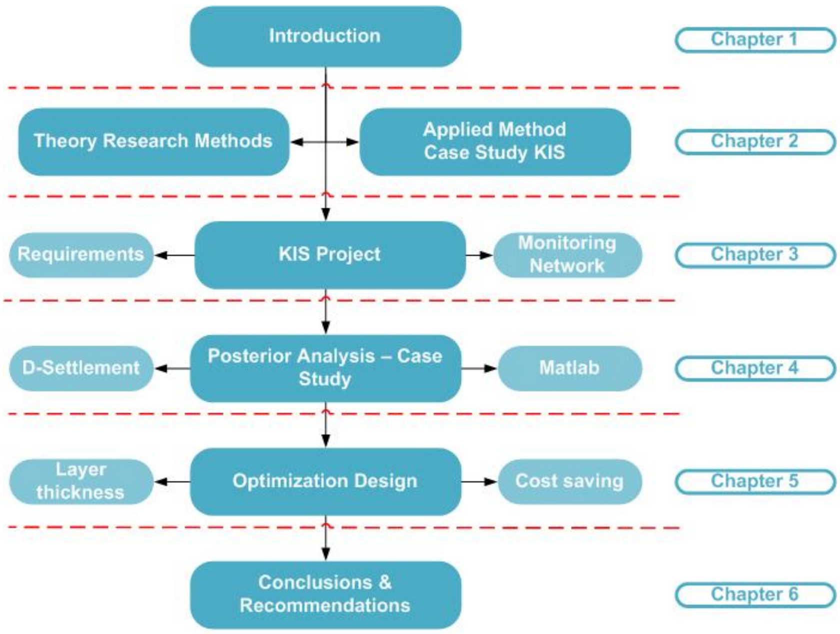


Figure 1.7: Outline & Approach of Thesis project

2

PROBABILISTIC SETTLEMENT PREDICTIONS

Chapter 2 elaborates on the models and methods used in this thesis. The settlement model is explained, as well which formulation is used. Next the statistical method that is used is explained to give insight in the approach of the reliability updating. With reliability updating uncertainties are involved, these uncertainties can be classified which is also explained in this chapter. After this the software programs for the research are described and explained for which purposes they are used. Lastly the usage of the methods in the case study is given.

2.1. OVERVIEW

In order to create more insight in the research method followed in this thesis this paragraph gives an overview on how the different parts of the investigation come together. Firstly the physical aspect of the method is modeled, the physical model used is the a,b,c–isotache model founded by Den Haan [1994] which represents the soil behavior. From this model the prior settlement estimates are obtained. Next to the physical model the statistical method of Bayesian Updating is used in combination with Monte Carlo Simulation to update the prior beliefs. This leads to probably more reliable posterior beliefs on the soil behavior. See Figure 2.1 for a visual representation of the overview.

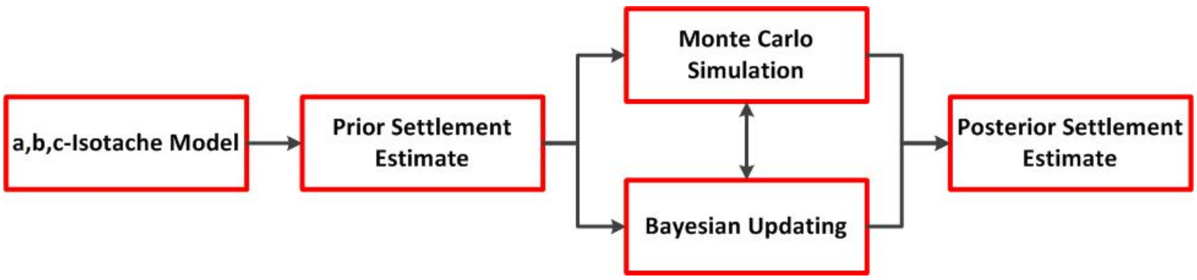


Figure 2.1: Overview of research method

2.2. A,B,C–ISOTACHE MODEL

This section will give a description of the a,b,c–isotache model. The following text is largely based on the PhD thesis of Den Haan [1994] on 'Vertical compression of soils'. The thesis gives a clear overview of the principles of the model.

2.2.1. FUNDAMENTALS OF THE MODEL

Due to increasing the load on the soft subsoil, by adding clay layers, the subsoil will deform. This results in strains, direct strain and secular strain. The direct strain occurs instantly with a change of the soil stress, while the secular strain (i.e. creep strain) behaves viscous. The model is described using a diagram with the horizontal axis as the natural logarithm of the effective soil stress and the vertical axis as the natural strain. The diagram shows a line of direct strain but also lines of equal rates of creep (i.e. creep isotaches). The gradient of the line of the direct strain is described by the parameter *a*. The secular strain is more complicated, this phenomenon is described by a stress dependent and a stress independent part. Parameter *b* gives the slope of the isotache lines and these lines are directly coupled to the intrinsic time. According to Den Haan [1994], founder of the a,b,c–isotache model, intrinsic time is:

"During creep at constant effective stress, intrinsic time is proportional to the reciprocal of creep strain rate, and equals true time in any reference frame (e.g. time since load increase) shifted over a value called the time shift.".

In other words, the intrinsic time can be seen as an equivalent age of the subsoil which takes the pre loading characteristics of the subsoil into account respective to the subsoil characteristics in case of applying a certain load on the subsoil.

Due to the coupling between parameter *b* and the intrinsic time, the intrinsic time can also be displayed in the diagram, see Figure 2.2.

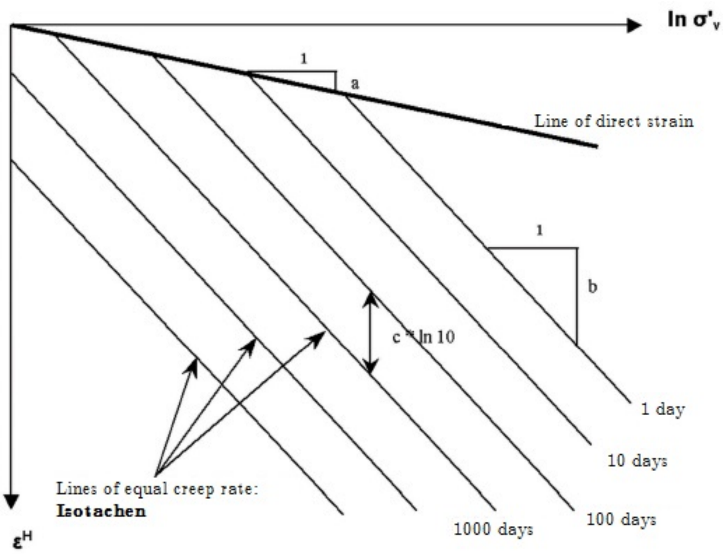


Figure 2.2: The isotache diagram

Parameter *c* is a measure for progressing strain during constant stress (pure creep), therefore it is coupled to the stress independent part of the secular strain. As can be seen in Figure 2.2, the lines of equal creep rate are diagonally displayed in the field of the natural logarithm of the effective soil stress and the natural strain. With a shift along an isotache with increasing stress, also the natural strain increases. The parameter *b* is therefore coupled with the stress dependent part of the viscous strain.

When combining the above description of the a,b,c–isotache model with Figure 1.4 in Paragraph 1.1.2 the settlement at start raise depends largely on the direct strain parameter *a* and after the consolidation period also on the secular strain parameters *b* and *c*. The extra raise showed in Figure 1.4 is especially to cover the settlements as a results of the direct strain parameter *a* but also to cover the residual settlement during lifetime. At completion, mid 2017 in Figure 2.3, the direct strain is zero (assuming that the degree of consolidation is 100%[1]) and no extra loading is being applied anymore (and thus no change in effective stress), therefore only the creep parameter *c* is of influence on the settlements during the lifetime of the structure. Therefore the residual settlements only depends on the creep parameter *c*. At the point in time where the lifetime of the structure is reached the extra raise equals the direct settlement plus the residual settlement. If the predictions about the settlement already during the construction stage can be made more reliable the extra raise before completion probably could have a smaller (or larger) thickness and thus could

[1]In practice the consolidation process never reaches 100% of consolidation, a value of 99,4% is considered to be full consolidation Sipkema [2006]

be optimized to save costs. However it is important to look at the residual settlement during lifetime because the Water Board Rivierenland stated in their Programme of Requirements (in Dutch: Vraagspecificatie en Eisen) Waterschap Rivierenland [2014b] as a requirement that the residual settlement must not exceed 0.1 meter after one year and 0.3 meter in the lifetime of the dike structure. To illustrate this see Figure 2.3.

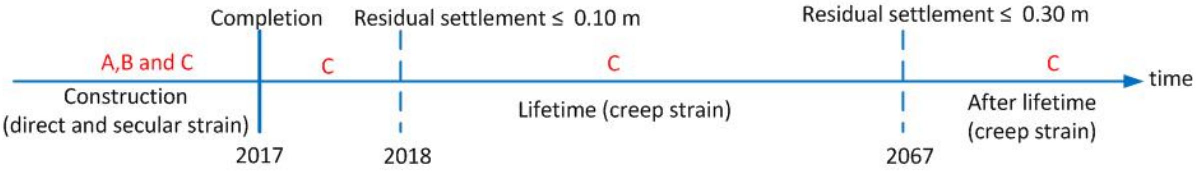


Figure 2.3: Requirement and relevant isotache parameters during lifetime

2.2.2. MODEL PARAMETERS

For the KIS project the three parameters a, b, c are obtained by executing K_0 C.R.S. oedometer tests on soil samples from borings. The value of parameter a can be obtained by the measured natural strain that occurs after one day of loading in the reloading part of the K_0 C.R.S. test. Den Haan and Kamao give a good description how the K_0 C.R.S. tests are executed in Den Haan and Kamao [2003].

See Figure 2.4 for a graphical representation of the compressibility parameters a, b and c.

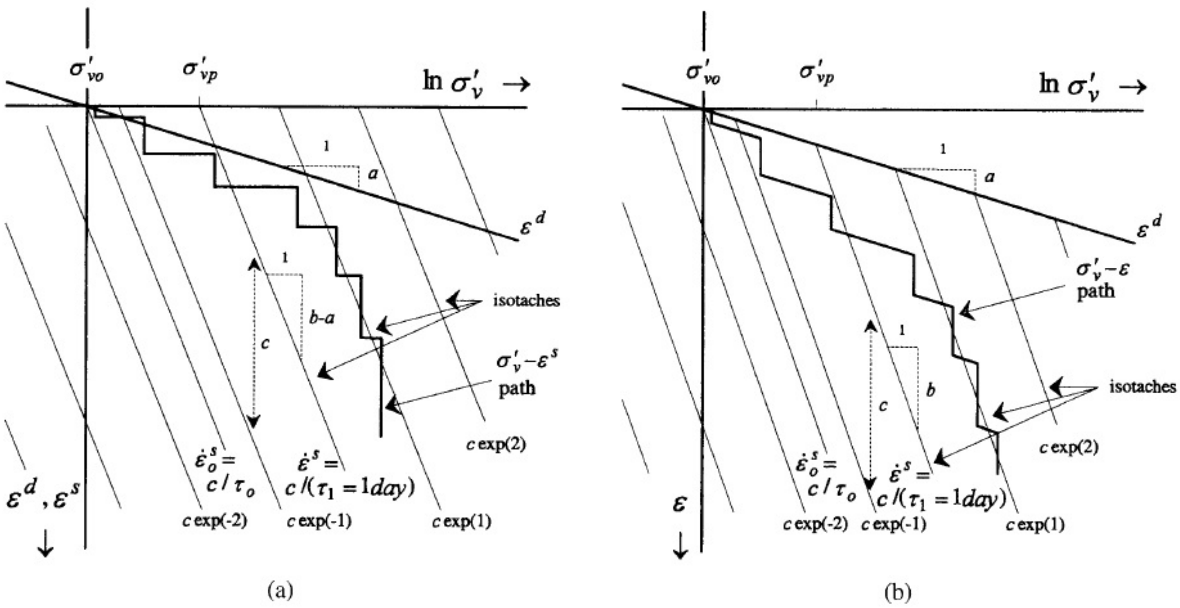


Figure 2.4: Isotache parameters Den Haan and Kamao [2003], direct and secular (creep) strain separated (left) and combined (right)

Equation 2.1 gives the definition of parameter a from Figure 2.4 (a) which is found on the reloading part of the test:

$$a = \frac{\Delta\varepsilon_v^H}{ln\left(\frac{\sigma'_{vi}}{\sigma'_{vi+1}}\right)} \tag{2.1}$$

Figure 2.4 (b) gives the definition of parameter b, namely b defines the slope of the normal compression of the diagram for an occurring strain as a result of an increase in load. See equation 2.2.

$$b = \frac{\Delta\varepsilon_v^H}{ln\left(\frac{\sigma'_{vi}}{\sigma'_{vi+1}}\right)} \tag{2.2}$$

Parameter c is defined in Figure 2.4 (b). It relates to the creep of the subsoil is shown in Figure 2.4 (b), here it can be seen that c is related to the slow deformation processes in time. The following equation 2.3 is determined for parameter c:

$$c = \frac{\Delta\varepsilon_v^H(t)}{ln\left(\frac{t_i+\Delta t}{t_i}\right)} \tag{2.3}$$

This is how the compressibility parameters are derived from oedometer tests. For the KIS project several oedometer tests are carried out on soil samples and data sets about the a,b and c parameters are constructed for each soil type Wiertsema & Partners [2014]. To determine the values of the compressibility parameters often there is chosen to obtain parameter b from the laboratory tests and define a and c relative to b. Den Haan [1996] shows ratios for peat, namely a is between 1/5 b and 1/10 b and for c a ratio between 1/20 b and 1/25 b is given. However in present day the ratios are slightly changed with more research done in time. The KIS project also uses this type of ratios Muntinga [2013], they are showed in Table 2.1.

Soil type	**a**	**b/a**	**b**	**b/c**	**c**
Peat	0,0348	9	0,3130	15	0,0209
Clay	0,0118	12	0,1410	20	0,0071

Table 2.1: Average isotache parameters a,b,c KIS project

From Table 2.1 it can be noticed that the ratios of the soil parameters are relatively conservative because of the choice of relatively high compressibility parameters when comparing the ratios for peat stated by Den Haan [1996].

2.2.3. FORM OF MODEL

Multiple forms of the a,b,c–isotache model can be used to compute the settlements. In this thesis there is chosen to use the incremental form of the model, in this way the model can be modeled in the software program Matlab, see Appendix C for the Matlab scripts. The settlements are computed with the following formula.

The total strain ε^H is the sum of the direct strain ε_d^H and the secular strain ε_s^H:

$$\varepsilon^H = \varepsilon_d^H + \varepsilon_s^H \tag{2.4}$$

for the direct strain ε_d^H holds:

$$\varepsilon_d^H = a \cdot ln\left(\frac{\sigma'_i}{\sigma'_{i-1}}\right) \tag{2.5}$$

where a is a compressibility parameter and σ' the effective stress. For the secular strain ε_s^H holds:

$$\varepsilon_s^H = c \cdot ln\left(\frac{\tau_{i,end}}{\tau_{i,begin}}\right) \tag{2.6}$$

where c is the creep strain compressibility parameter and $\tau_{i,end}$ is the equivalent age in days at the end of a time step Δt. The intrinsic time $\tau_{i,begin}$ is the equivalent age in days at the beginning of a time step. The compressibility parameter b of the a,b,c–isotache model is coming back in the parameter $\tau_{i,begin}$, as can be seen in the following equations:

$$\tau_{i,begin} = \tau_{i-1}\left(\frac{\sigma'_{i-1}}{\sigma'_i}\right)^{\frac{b-a}{c}} \tag{2.7}$$

and:

$$\tau_{i,end} = \tau_i + \Delta t \tag{2.8}$$

As a value for Δt 1 day is taken, this means 18250 steps in the design life of 50 years. To improve computation time larger time steps can be taken which represent specific points in time, such as start raise and end raise. To illustrate the approach described above, see Figure 2.5.

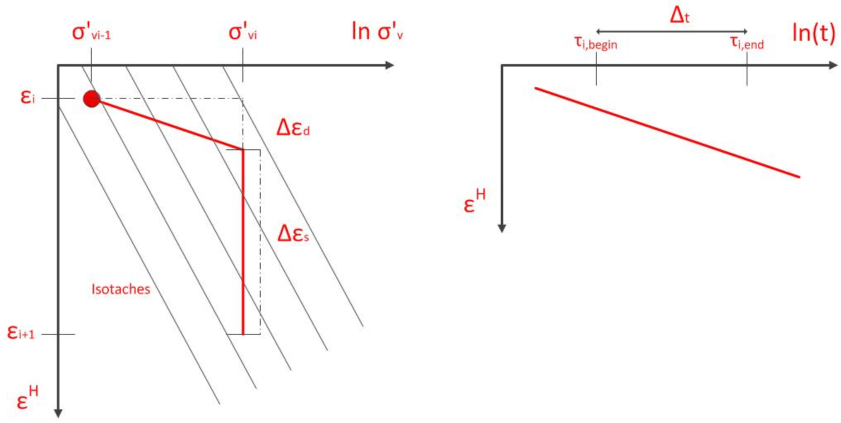


Figure 2.5: Illustration of incremental form of the a,b,c–isotache model

When the direct strain and secular strain are known the total strain ε^H can be computed:

$$\varepsilon^H = a \cdot ln\left(\frac{\sigma'_i}{\sigma'_{i-1}}\right) + c \cdot ln\left(\frac{\tau_{i,end}}{\tau_{i,begin}}\right) \tag{2.9}$$

To compute the settlement Δh the following formula is used:

$$\Delta h = h_0 \cdot \left(1 - exp(-\varepsilon^H)\right) \tag{2.10}$$

where h_0 is the initial thickness of the particular layer.

Note: Equation 2.10 is used because the total strain is given here in so called Hencky strain (natural strain) and not in the traditional linear strain. The advantage of using natural strain in the a,b,c–isotache model is that it performs better in situations where large deformations take place Den Haan [1994].

2.3. MONTE CARLO SIMULATION

To model the uncertainties of the variables, these variables are given a mean and standard deviation. From the parametric distribution a large number of realizations are obtained and with help of the Monte Carlo Simulation the calculation is made.

Probabilistic design is nowadays widespread in Hydraulic Engineering projects. These projects have typically a small target failure probability with very high consequences when a system fails (like nuclear facilities). The discussion about how to deal with these kind of systems is assessed by the societal acceptability of a certain risk and the interpretation of the consequences and the failure probability of these systems. Because of the type of system dealt with here there are large uncertainties involved in how to assess these systems. The systems are modeled and the uncertainties of the system are incorporated within the model parameters. The benefit of approaching the problem this way is that the risk of failure is quantified and that engineers have a tool to design their dike or flood defense and to make choices on the basis of these failure probabilities.

Because of the uncertainties involved in the system the model can give a wrong representation of reality when using only deterministic values for parameters. By giving the parameters a certain probability distribution, with for instance for the normal distribution a mean and a standard deviation, the distributions can be propagated through a model using the Monte Carlo method and something can be said about the reliability of a model or a prediction. In other words, Monte Carlo Simulation is a method for exploring the sensitivity of a system by varying parameters within the statistical constraints. With a Monte Carlo Simulation a physical process is simulated randomly a lot of times, the result is given by a set of possible outcomes which can give a certain reliability. For the thesis project the Monte Carlo Simulation will be used to get a certain reliability of the expected residual settlements of the new dike using the a,b,c–isotache model. Each parameter in the equation for total strain, equation 2.9, will be modeled by a parametric distribution. In this research it is assumed that the individual parameters have either a normal distribution or a lognormal distribution. This corresponds with the software program D-Settlement from Deltares Deltares Systems [2011], where also only these two parametric distributions can be chosen.

2.4. BAYESIAN UPDATING

Paragraph 2.4 explains how the monitoring data is used to update the a-priori prediction. This is done with the Bayesian approach based on Bayes' rule.

2.4.1. DESCRIPTION

One way to connect the monitoring data to the predicted settlements with the a,b,c–isotache model is by using Bayesian Updating. Bayesian Updating enables more reliable predictions of the settlement by using the monitoring data obtained during construction. The monitoring data is the update, or evidence, of a certain scenario that will happen in the future. Occurrence of this future scenario is uncertain and by calculating a certain reliability for this scenario the risks and the consequences are weighed. Knowing this, decisions about the design and construction of the dike structure can be made. Bayesian Updating is based on the principles of Thomas Bayes, an English statistician, who first showed how to use evidence to update beliefs. Bayes himself did not publish his theory, this was done by Pierre-Simon Laplace in 1812. Laplace named the developed theory after Bayes: "*Bayes' Theorem*". Bayes' Theorem or Bayes' Rule is stated as follows, see equation 2.11:

$$P(A|B) = \frac{P(B|A) \cdot P(A)}{P(B)} \tag{2.11}$$

where:

- P(A) and P(B) are the unconditional probabilities of A and B.
- P(A | B), a conditional probability, is the probability of A given that B is true.
- P(B | A), is the conditional probability of B given that A is true.

Bayes' Rule is used in this thesis in a slightly different form. Failure probabilities $P(F)$ are calculated with a limit state function $Z = R - S$, where R is the resistance (requirement about the residual settlements of the Water board Rivierenland) and S is the solicitation (the predicted settlement). Failure of the limit state function occurs when $Z < 0$. The evidence ε is resembled by the monitoring data of the settlement plates. To update the failure probability $P(F)$ an observation limit state function h is introduced. This observation limit state function is more elaborately explained in paragraph 2.4.3. Again failure of the observation limit state function is when $h < 0$. Eventually the following equation 2.12 is used in this thesis, based on Bayes' Theorem:

$$P(F|\varepsilon) = \frac{P(Z(\mathbf{X}) < 0 \cap h(\mathbf{X}) < 0)}{P(h(\mathbf{X}) < 0)} \tag{2.12}$$

where:

- P($F|\varepsilon$) is the conditional probability of failure F given the evidence ε.
- P($Z(\mathbf{X}) < 0 \cap h(\mathbf{X}) < 0$) is the probability that the limit state function Z is smaller than zero AND the probability that the observation limit state function h is smaller than zero.

The updated failure probability given the monitoring data is obtained when both the probability of the limit state function Z and observation limit state function h are smaller than zero divided by the probability of the observation limit state function smaller than zero. A more elaborate derivation of equation 2.12 can be found in Appendix B.

As a graphical example in what the Bayesian Updating means for the parametric distribution of for instance the settlement S with a measurement close to the predicted settlement and a measurement of the actual settlement smaller than predicted, see Figure 2.6 below. Left graph of Figure 2.6 represents a measurement close to the prediction, right graph is the measurement smaller than the prediction.

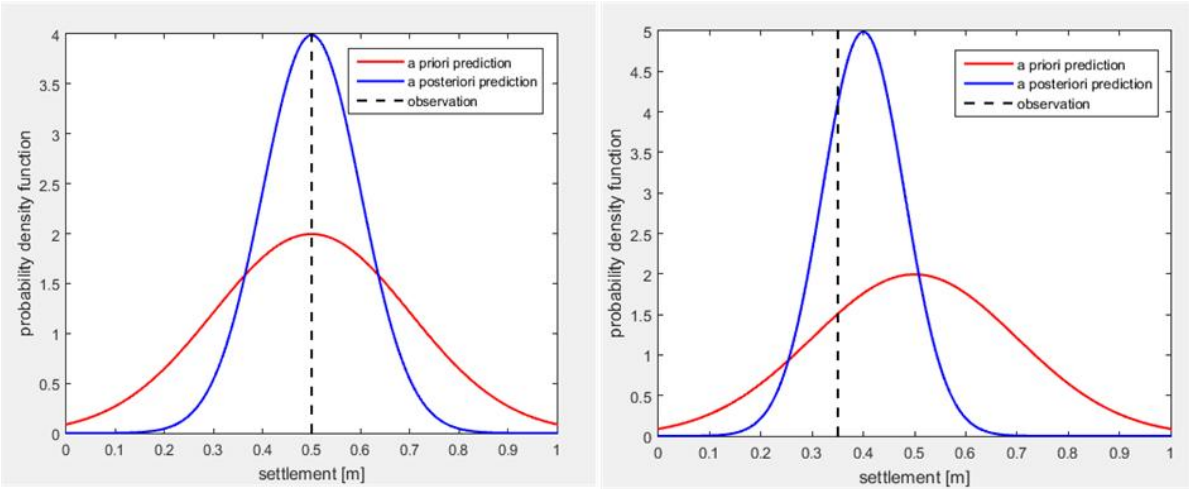


Figure 2.6: Graphical examples of Bayesian Updating

As can be seen for both a-posteriori probabilistic density functions the mass of the density function is more concentrated around the mean value of the settlement, this is because the standard deviation becomes smaller with the implementation of the measurement or evidence. The right graph of Figure 2.6 is from a measurement of the settlement which is smaller than the a-priori settlement, this is due the filtering effect of the prior realizations that fall within the range of the measurement error and prior realizations that do not fall within that range. The probability density function shifts to the left and the mean value of the expected settlement becomes also smaller. It also works the other way around when there are larger values for the settlement found with measurements. The reliability of the amount of expected settlement becomes larger because of a smaller mass in the tails of the probability density function (the values for the 5% and 95% quantiles are closer to the 50% quantile).

2.4.2. UNCERTAINTIES

In probabilistic design there are three different types of uncertainties, namely inherent, model and statistical uncertainty according to van Gelder [2000]. Figure 2.7 shows the classification of the three types of uncertainties.

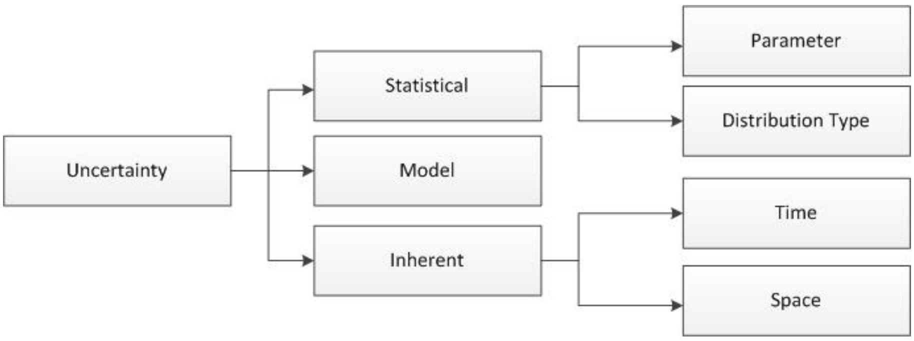


Figure 2.7: Classification of uncertainties van Gelder [2000]

Van Gelder divides uncertainty in two classes, namely inherent uncertainty and epistemic uncertainty. Inherent uncertainties are defined as uncertainties that represent randomness or variations in physical phenomena. An example of this inherent uncertainty is the phreatic level in the KIS project. The level of the phreatic line in the KIS project greatly depends of the water level of the river Lek. This water level has a long history of data, however one cannot predict what the maximum water level in the Lek next year would be and therefore the maximum phreatic level of next year cannot be predicted. The other class of uncertainty, epistemic uncertainty, has its source in the lack of knowledge about the physical processes or by a lack of data. An example of epistemic uncertainty is the lack of information about the a-priori a,b and c parameters of the isotache model. With boring samples these subsoil parameters are obtained from the K_0 C.R.S. tests executed in the lab. However it is often the case that there are too few samples or borings available to resemble reality perfectly, consequences in a lack of data and thus epistemic uncertainty.

Inherent uncertainty in time, see Figure 2.7, cannot be reduced because of little (or no) information about uncertainties in future. These uncertainties remain unknown however theoretically this uncertainty can be reduced by keeping record of (for instance) the physical process for the coming centuries, hence for now this uncertainty cannot be reduced. Example of a stochastic process running in time is for instance the earlier mentioned phreatic level in the subsoil, the future prediction about the phreatic level remains uncertain despite all the gathered records of phreatic levels in the past. An example of inherent uncertainty in space is lack of information about the subsoil (actually epistemic uncertainty). With Cone Penetration Tests (CPT's) and borings the subsoil is investigated in the laboratory and classified. These samples are taken very local, however they represent a whole soil layer. All knowledge about the subsoil could be collected, however that would be a very expensive way of minimizing this uncertainty and is not feasible. This also holds for the KIS project, here there are for section W only two CPT's executed which represent the whole section, while section W has an area of approximately 220 x 20 m². The density of the soil investigation[2] can be considered low.

Parameter uncertainty is introduced by a lack of data. The parametric distribution is fitted on the data set which is available with for instance the Maximum Likelihood Estimate method Dekking et al. [2005] or Kolmogorov–Smirnov tests Massey [1951]. In the case of too few data the fit parameters become uncertain because of the uncertainty in the chosen parametric distribution. Above there is already referred to the uncertainty of the chosen parametric distribution, for example it is not clear whether the compression parameters a,b, and c of the isotache model are normal distributed or that they have another parametric distribution. Distribution uncertainty and parameter uncertainty are highly dependent on each other, Van Gelder says that it is not always possible to draw the line between them. Parameter and distribution uncertainty combined represent the statistical uncertainty. In the KIS project the input parameters that are made available by the client have deterministic (expected) values and are not given a parametric distribution. In this thesis assumptions are made for the parametric distributions of the variables which automatically introduces parameter uncertainty.

Model uncertainty is introduced in how the the physical processes are described in the model. In this thesis the a,b,c–isotache model is used to describe the settlements of the subsoil. Describing the physical process of settlement in a model is very difficult and therefore the a,b,c–isotache model could be imperfect (however it is the best settlement model for soft soil there is at the moment). Model uncertainty can be reduced by improving the model itself, this means for this thesis to improve the physical a,b,c–isotache model or the mathematical Bayesian Updating part of the model. Reducing the model uncertainty can also be done by reducing the local bias of the measurements used in the Bayesian Updating, this means reducing the consistent error made when taking the measurements.

Note: There is no relation between parameter uncertainty and model uncertainty. Parameter uncertainty depends on the quality and quantity of the data, model uncertainty depends on how well the physics are resembled in the model.

[2]Note: Due to the very weak top soil in section W it was difficult (on many places not possible) to execute a CPT.

2.4.3. INEQUALITY INFORMATION VS. EQUALITY INFORMATION

Two types of information can be distinguished in the implementation of reliability updating: inequality information and equality information. Information is in this thesis defined as monitoring data or measurements. For the inequality information it applies that if the evidence ε implies that the observed quantity is greater than or less than a function of random variables, the evidence ε can be formulated as:

$$\varepsilon \equiv \{h(\mathbf{x} < 0)\} \qquad \text{OR} \qquad \varepsilon \equiv \{h(\mathbf{x} > 0)\} \tag{2.13}$$

where h(x) is the observation limit state function. Inequality information is for instance exceedance of a limit state or survival of the limit state (for instance: the observation of the settlement is smaller than 0.5 meter).

Equality information is defined as follows:

$$\varepsilon \equiv \{h(\mathbf{x} = 0\} \tag{2.14}$$

To implement equality information in the computations some difficulties arise, for instance it is difficult to define the a-priori probability of a dike failure when the observation exactly equals some particular value **x** (the probability of the observation appearance would have a value of approximately zero![3]). Therefore there is a need to transform the equality information into inequality information, in a way that this valuable information can be used in reliability analysis methods to update predictions. A solution to this problem is introduced by Straub Straub [2011], with his method the equality information is reformulated into inequality information.

The monitoring data obtained from the monitoring network of KIS is of the equality information type. In order to use this data in the project it has to be reformulated into inequality information. This can be illustrated by Figure 2.8 which shows for a problem with two random variables x_1 and x_2 and one measurement ε that the inequality (updated) failure domain ($Z(x) < 0$ and $h(x) < 0$) is a surface and the equality (updated) failure domain ($Z(x) = 0$ and $h(x) = 0$) is a line. As the definition of domain already holds, a domain is a space where outcomes of random variables are present, thus it is impossible for a line holding the equality failure domain (i.e. that the failure probability becomes approximately zero) and thus the equality information must be transformed in a way that the information can be presented in a failure domain. Figure 2.8 also shows that the surface of the a-priori limit state function $Z(x) < 0$ is larger and thus more uncertain than the updated limit state function (i.e. observation limit state function) $h(x) < 0$. The same holds for their equality counterparts, here the line of $Z(x) = 0$ is less convex shaped than the updated line of $h(x) = 0$.

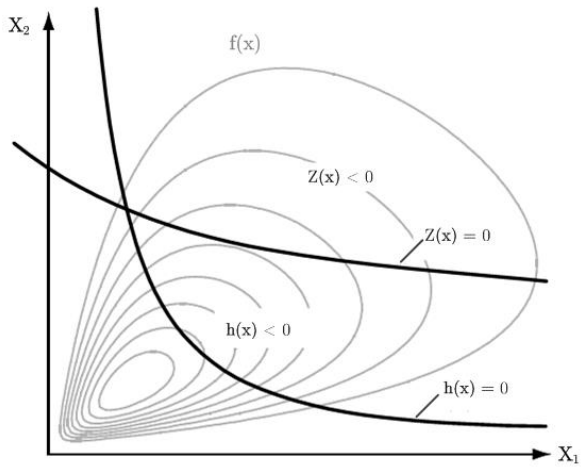


Figure 2.8: Illustration of the reliability updating problem Straub [2011]

[3]Note that the observation also has a measurement error and therefore the probability of appearance of that observation does not exactly equals zero

In order to give more insight in Figure 2.8 below the two probability density functions with observations and failure probability is showed for the case of inequality information and equality information. The left probability density function of Figure 2.9 equals the grey surface $Z(x) < 0$ showed in Figure 2.8 and the right probability density function of Figure 2.9 equals the black line $Z(x) < 0$ showed in Figure 2.8. The failure probability when using the equality information becomes approximately zero as mentioned earlier.

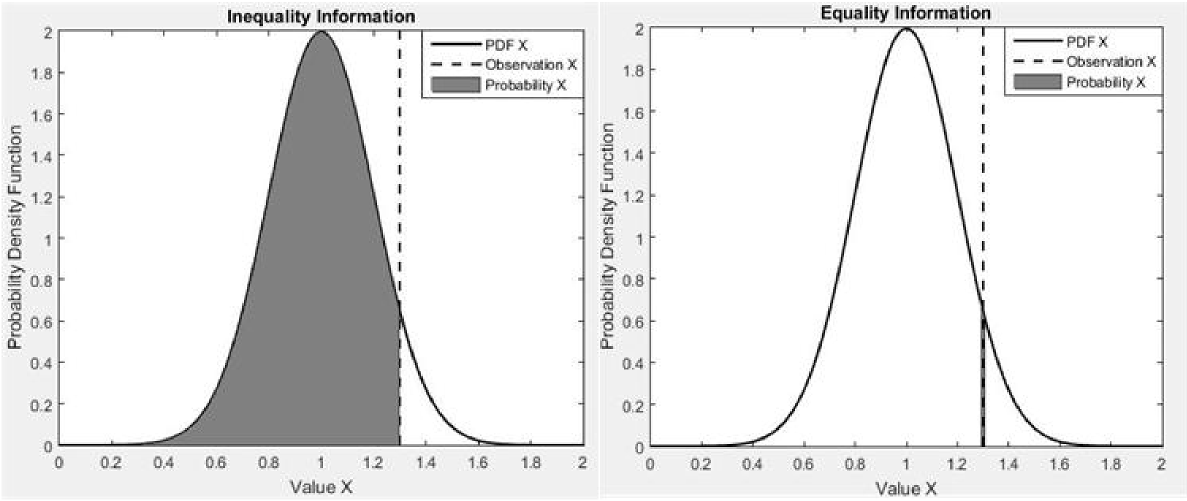


Figure 2.9: Probability problem inequality information (left) vs. equality information (right)

In Appendix A the method of transforming equality information into inequality information is elaborately described.

2.4.4. DIRECT VS. INDIRECT RELIABILITY UPDATING

Updating of information can be done in two ways, direct or indirect. The indirect method takes all the updated individual parameters of the physical model and run them through the same physical model again to update the prediction of the physical process. An advantage of this method that a lot of information about the sensitivity of an individual random variable can be obtained. All updated probability density functions can be obtained and from these updated PDF's it is immediately clear what kind of influence the measurement has on the particular random variable. On the other hand there are some properties of the indirect method that are disadvantageous, for instance the computation time of the model becomes much larger because of all the updates of the stochastic parameters and running through the model again. Another remark is that with the indirect method correlations can be introduced or changed when updating the random variables and thus another calculation step is needed to gain reliable results.

To avoid the two discussed disadvantages of the indirect method, use is made of the direct method in this thesis. The direct method only uses the conditional probability of failure as can be seen in equation 2.12. The updated probability of failure given the measurement ε is directly obtained. The results of the two updating methods are equal Schweckendiek [2014], however the direct reliability updating method is easier to compute, introduces or changes no correlations visually and has less computation time needed. The indirect and direct method are compared in Figure 2.10, it can be seen that the indirect has more steps to take, however it gives more insight in the model parameters. Also the connection to the failure probability is made, the failure probability is computed from the realizations of S a-priori and S a-posteriori.

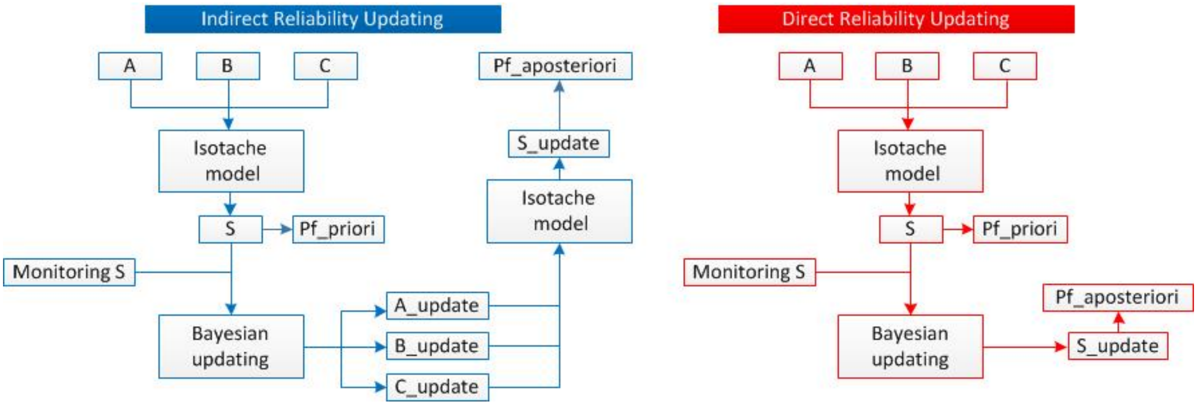


Figure 2.10: Differences between indirect (left) and direct (right) method

It could be interesting to use the indirect reliability update method during construction when the model parameters still can be adjusted with advanced knowledge about the subsoil of the project. In case of the settlement model used in this thesis it could be valuable to know more about the real compressibility parameters of the subsoil. By adjusting these parameters to less conservative values [4] the thickness of the extra raise could probably less because the residual settlement would be smaller. In this way the designer has proof that the subsoil is more resistant than thought of at beforehand and the layer thickness of the last raise could be less thick than thought of at beforehand. Also with the results from the indirect reliability update method it can be seen which variable has more importance regarding other variables. This information could be interesting to use in other parts of the KIS project or for other dike reinforcement projects in the Netherlands. It could improve the geotechnical monitoring network as well as the design of the future dike reinforcement projects.

2.5. SOFTWARE

2.5.1. D-SETTLEMENT

The software program D-Settlement can be used to calculate the expected settlements for a new dike structure or other types of embankments. The designers of the new dike structure of KIS use this program to calculate the a-priori settlement with the a,b,c–isotache model. Therefore it is interesting to compare the results obtained from D-Settlement and the proposed model build for this thesis. D-Settlement is also used to verify the a-priori predictions of the settlement resulting from the proposed model, see Appendix D.

D-Settlement contains an interesting option, namely the 'Settlement Plate Fit option. The way this fit is carried out is described in the manual of D-Settlement Deltares Systems [2011]:

"The automatic fit by means of an iterative weighted least squares procedure, which minimizes both the difference between measurement and prediction, and the difference between the original and the adapted value of the parameters. During each iteration, D-Settlement linearizes the influence of parameter modifications, by first determining the settlement variations caused by very small parameter changes."

This fit is a deterministic approach, though another interesting option is the module 'Reliability Analysis'. With this module the soil parameters in the D-Settlement can be modeled in a probabilistic way. Two parametric distributions are available, the standard normal distribution and the lognormal distribution. There is a choice to calculate with level II (FORM And SORM) or level III (Monte Carlo) methods. When using the Reliability Analysis function together with the Settlement Plate Fit function of D-Settlement the a-priori settlements are updated. D-Settlement applies Bayesian Updating of the parameter covariance matrix. The update introduces correlations between the different uncertain (stochastic) parameters (i.e. indirect reliability updating), which finally yield in a reduced bandwidth for the updated mean values of the settlement prediction. The designer could choose on which confidence bound (50% till 0.99%) he/she wants to design by adjusting this in D-Settlement.

[4] see Chapter 3 for more information about the determination of the soil parameters at the KIS project.

For a full explanation with examples the reader is redirected to the paper of Calle et al. [2005]. It explains in which fashion D-Settlement uses the monitoring data to update the predictions of the settlement. In short, Calle et al. [2005] explains that D-Settlement uses the Jacobian matrix in combination with the covariance matrix in order to compute the updated settlement prediction. With the Jacobian matrix the a-priori covariance matrix is updated and with the updated covariance matrix the a-posteriori settlement prediction can be made. The reason the Jacobian matrix is used is that the deterministic approach in D-Settlement works, as already said above, with the weighted least squares method. It is however not clear from this description how D-Settlement uses the equality information in the update.

This approach of Bayesian Updating is different than is proposed in this thesis, therefore it is interesting to compare the results from the D-Settlement model and the results from the thesis project. The differences between the Bayesian Updating method in this thesis and D-Settlement are especially found in the way the equality information is used (no measurement error in D-Settlement or equivalent inequality information) and the updating method itself (in this thesis the Jacobian matrix is not used to update the a-priori covariance matrix). Also the influence of each individual update cannot be showed by D-Settlement. Next to that the update method differs in the fact that the approach in this thesis also takes care of the tailes in the distributions, whilst the approach in D-Settlement mainly focuses on the mean values due to the use of the least square method.

2.5.2. D-GEO STABILITY

With this program the stability of the dike structure is checked. Implementing a certain cross-section with soil properties for each different layer and implementing different kind of loads the stability under extreme conditions can be checked. The relevance of D-Geo Stability for the thesis project is to check whether the optimized last clay layer does not influence the stability of the new dike design and thus if it is physically feasible to change the new dike.

2.6. APPROACH CASE STUDY KIS PROJECT

This section elaborates on how the methods described in this chapter were used to build the model and what assumptions were made in the model. It also goes into the correlations between the different compressibility parameters of the settlement model for the clay and peat layers in the KIS project.

In Appendix D the approach can be found how the proposed model was build and how it is verified. The direct updating method can easily return failure probabilities for multiple measurements (i.e. evidence obtained in time), also for knowing the values of the updated settlement predictions the direct reliability updating method is used. Straub's method, which transforms inequality information to equality information, basically filters the amount of a-priori realizations to a smaller number of a-posteriori realizations with in most cases a smaller confidence bound. Because of using multiple measurements in time the number of a-posteriori realizations becomes smaller with each update (i.e. measurement), in this thesis this characteristic of the updating method is called the filter effect.

2.6.1. DIRECT RELIABILITY UPDATING METHOD

A flowchart of the direct reliability updating model is given in Figure 2.11 to illustrate which steps are taken to obtain the updated failure probability. Below this flowchart is more elaborately described.

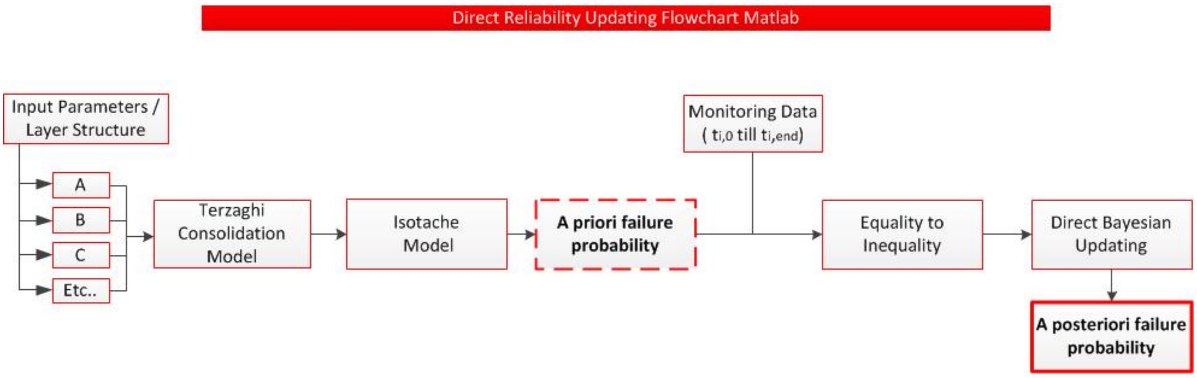


Figure 2.11: Flowchart direct reliability updating in Matlab

LAYER STRUCTURE

The input variables are structured in a separate Matlab script which is called by a function containing the isotache and consolidation model, see Appendix C for the Matlab script. By giving the input variables a normal distribution all variables are stochastic. There is chosen for a normal distribution for each variable because of a lack of data and following the assumption in D-Settlement which also only uses the normal or lognormal distribution. Correlations are obtained for the a,b and c variables from a data set obtained from the engineering company Wiertsema & Partners. This company executed the K_0 C.R.S. tests on samples taken from the subsoil at the KIS project. Main purpose of these samples was to define the limit strain of the subsoil, however also the compressibility parameters where obtained and these data was exported to Matlab to obtain the correlations. Paragraph 2.6.3 will elaborate on how the correlations are obtained and the effects of these correlations on the model results.

TERZAGHI CONSOLIDATION MODEL

The model uses the incremental form of the a,b,c–isotache model which is explained in Paragraph 2.2.3, this incremental form is connected to the consolidation equations of Terzaghi Verruijt [1999], see Equations 2.15, 2.16 and 2.17.

$$T = \frac{c_v * t}{D^2} \tag{2.15}$$

$$U = \sqrt[6]{\frac{T^3}{T^3 + 0.5}} \tag{2.16}$$

$$\Delta\sigma' = U * p_0 \tag{2.17}$$

Where T is the equivalent time, c_v the consolidation coefficient of a soil type, t is the time after loading in seconds. The loading p_0 is multiplied with the consolidation ratio U to obtain the increase (or decrease when unloading) in effective stress $\Delta\sigma'$.

With Terzaghi's equation a consolidation ratio U is obtained which has a range between 0 and 1. A value of $U = 1$ means that the consolidation is completed and that the excess of water is pressed out of the pore volumes. The consolidation has a large impact on the model, it introduces a delay in the settlements that will occur in time. On large timescales the settlement will be the same if not using the consolidation equations of Terzaghi, however the monitoring data obtained from the KIS project is taken during construction and right after construction. To model the consolidation process during the lifetime of the new dike it is important to use small time steps in the construction phase, and also to get reliable results when using the monitoring data to update the settlement prediction. The time steps in the model will become larger when the construction phase is finished (approximately after 1065 days) because the consolidation process is assumed to be completed.

VERTICAL DRAINAGE

When applying vertical drainage in a dike reinforcement project the consolidation process of the subsoil is likely to go faster than without the vertical drainage. This is because the water is drained more easy to the surface and the voids between the grains need less time to become smaller what implies an increased settlement in a shorter period. To model vertical drainage in Matlab it is chosen to give the soil types a high consolidation coefficient, in D-Settlement the soil types are given a drained status. The results with the chosen consolidation coefficient in Matlab are compared with the results of the drained status in D-Settlement and these results correspond as can be seen in Appendix D.

ISOTACHE MODEL

When combining the a,b,c–isotache model, the consolidation model and the correlations the settlement can be predicted a-priori. The limit state function $Z = R - S$ is calculated, with R is the resistance and S is the solicitation. Here the resistance is the requirement of the residual settlement stated by the Water Board Rivierenland and the solicitation is the predicted settlement and thus the results from the a,b,c–isotache model.

A Monte Carlo Simulation is executed to obtain the a-priori settlement predictions. The model is simulated a large number n times and by using a lot of simulations the real settlement is approached by taking the mean value of all these possible realizations of the settlement. For the whole lifetime of the new dike structure the settlements are calculated, this is done by computing the settlement for the first 466 days for specific points in time (i.e. points in time when raises are applied and measurements taken) and then compute the settlement on 831, 1065 days, 1430 days 10000 days and 19315 days (i.e. 50 year after completion) after construction. In total there are a number of 33 points in time where the settlement is computed. Each point in time is thus simulated n times by the Monte Carlo Simulation.

MONITORING DATA

Monitoring data is obtained from the records of the settlement plates installed at the KIS project. In particular the output from settlement plates in section W is used in the calculations. IV-Infra engineers have developed a release document for the KIS project to determine if the next raise could be executed. According to water pressure measurements which are gathered each day, it can be seen when the consolidation process has made sufficient progress after the loading of the previous raise. The time between each raise is obtained from the data set of the release document to simulate the raises in the proposed model at the appropriate time. When applying another raise on top of the previous raise while the consolidation progress is not sufficient the bearing capacity of the the subsoil can lead to instability because of the high water pressures. This could lead to dangerous situations for the dike stability and is therefore carefully monitored, the release of a next raise is therefore an important part of the construction of the new dike.

The settlement in time is obtained from a data set of the settlement plates, see Appendix E. For a couple of points in time the settlement is recorded with a GPS device which measures the height of a settlement plate respective to the initial situation of the settlement plate. All these values of the real settlement in time are also computed in the a-priori predictions of the settlement. Now these two values on a specific point in time can be compared and the settlement prediction can be adjusted by applying Bayesian Updating.

EQUALITY TO INEQUALITY AND DIRECT BAYESIAN UPDATING

With the methods discussed in Paragraph 2.4 and Appendix A the update is performed. First the measurement is implemented in the model and is given a measurement error because the measurement could have uncertainties like performance of the GPS device or a human error during the measurement. According to the GPS device used (see Paragraph 3.3), the device makes an error between -0.02 and + 0.02 meter Trimble [2003]. However not only measurement errors are made by the GPS device itself, also human errors are involved. One can think of for instance the placement of the device and the soil around the settlement plate not being equalized. These practical circumstances are responsible for a higher measurement error. The measurement error is modeled as a normal distributed random variable with $\mu_{em} = 0$ meter and $\sigma_{em} = 10$ cm. Now the settlement measured at time t_i and the prediction made a-priori about the settlement at the same time t_i can be compared and used for the Bayesian Update. Straub [2011] introduced a method to transform the equality information (monitoring data) into inequality information as discussed in Paragraph 2.4 and Appendix A, below a short explanation is given.

Straub uses two properties of Bayesian analysis, namely the Likelihood (i.e effect of information on an uncertain parameter Θ) and that the posterior probability is proportional to the Likelihood times the a-priori probability. In his method the Likelihood of the a-priori settlement $\mathbf{x}_g$ is given as follows, see Equation 2.18:

$$L(\mathbf{x}_g) = f_{e_m}[s_m - s(\mathbf{x}_g)] \tag{2.18}$$

where s_m is the measured value of the a-priori settlement $s(\mathbf{x}_g)$ and f_{e_m} is the probability density function of the measurement error.

Now Straub takes another limit state function called the observation limit state function h_e. The observation limit state function uses a standard normal variable U and the inverse of the standard normal cumulative distribution of the Likelihood function to obtain the outcomes which are located in the failure domain of the observation, see Equation 2.19.

$$h_e(\mathbf{x}_g, u) = u - \Phi^{-1}\left[cL(\mathbf{x}_g)\right] \tag{2.19}$$

The scaling factor c is to ensure that the Likelihood is always between zero and one in order to calculate the inverse cumulative distribution (always between zero and one). The scaling factor is typically $c = \frac{\sigma_{em}}{0.39}$ after Schweckendiek [2014] with σ_{em} as the standard deviation of the measurement error.

To calculate the updated conditional probability of failure the corresponding inequality domain of the observation limit state function $h_e(\mathbf{x}_g, u)$ is needed, this is where the observation limit state function is smaller than zero, see Equation 2.20:

$$\varepsilon_e \equiv \left\{h_e(\mathbf{x}_g, u) \leq 0\right\} \tag{2.20}$$

Posterior probability of failure is obtained by applying Bayes' rule, see Equation 2.21:

$$P(F|\varepsilon) = \frac{P(F \cap \varepsilon)}{P(\varepsilon)} = \frac{P(Z(\mathbf{X}) < 0 \cap h_e(\mathbf{x}_g, u) < 0)}{P(h_e(\mathbf{x}_g, u) < 0)} \tag{2.21}$$

This means that the updated conditional probability of failure given the failure domain ε equals the probability that $Z \leq 0$ and $h_e \leq 0$ divided by the probability that $h_e \leq 0$.

Now the updated failure probability is obtained, it is also possible to obtain the values belonging to the updated settlement prediction itself. Calculating the updated settlement as a function of the observation limit state function which are smaller than zero will give the a-posteriori settlement predictions (direct reliability updating). This also introduces the filter effect, if a prior realization does not satisfy the observation limit state function this realization is deleted from the prior set of realizations.

2.6.2. INDIRECT RELIABILITY UPDATING METHOD

In order to give a comparison between the direct and indirect reliability updating method this section elaborates on the indirect reliability method. It is important to mention that the indirect method is not used in this thesis, however it could give interesting results.

This method does not differ much from his direct counterpart, therefore much of the description above about the direct method holds also for the indirect method. The method differs in the fact that the individual variables are updated and with the updated parametric distributions the a,b,c–isotache model is again computed to obtain a number of realizations of the settlement but now with a smaller standard deviation. With each measurement the model needs to be run through to obtain each time the a-posteriori distributions of the individual variables. The expectation is that with each update the standard deviation of the a-posteriori settlement prediction becomes smaller and thus more reliable. Figure 2.12 shows the flowchart of the indirect reliability updating method in Matlab (**note:** for one measurement).

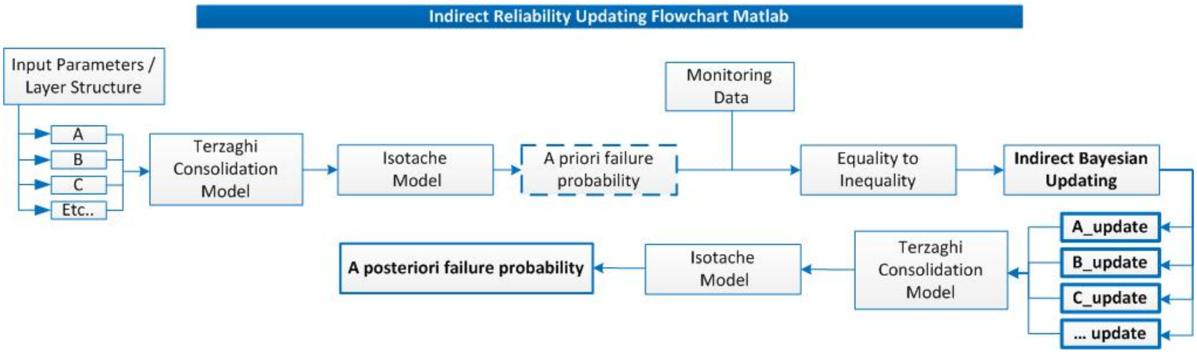


Figure 2.12: Flowchart indirect reliability updating method

It can be seen that the indirect method is much more labor-intensive than the direct method, while it gives more information about the individual variables than the direct method.

UPDATE OF PARAMETRIC DISTRIBUTION

Each individual parameter in the model has a parametric distribution, with the monitoring data the parametric distributions can be updated. When taking the observation limit state function h_e as a function of the individual variable the realizations of this individual variable that hold when $h_e < 0$ are obtained. From this new set of realizations the mean and standard deviation are calculated and these form the updated parametric distributions of the individual variable. Below the mathematical expression for the update of an individual variable a.

First the observation limit state function is obtained with the monitoring data as evidence just like equations 2.18, 2.19 and 2.20.

After the equivalent inequality domain ε_e is obtained, it can be taken as a function of the individual variable a in order to obtain the posterior realizations of a.

Now a new vector of realizations is obtained for $a_{posterior}$ with a number of realizations smaller than the a-priori number of realizations. From this new vector the mean μ and standard deviation σ are calculated.

With this approach the updated parametric distribution of a is calculated. In this thesis it is assumed that all individual variables are normal distributed. However it is quite possible that some variables are not normal distributed in reality, this holds especially when these variables are updated. Due to a lack of data (epistemic uncertainty) this could not be evaluated in this thesis, and it would be a large effort to search for good parametric distributions for each variable. An assumption is made to use only the normal distribution in this thesis. On the other hand it is acceptable to choose the normal distribution because conventional software like D-Settlement also gives only two choices regarding the parametric distributions of the stochastic parameters. The two options as mentioned earlier in this report are the normal and lognormal distribution (only in demo-mode) from which the lognormal distribution is especially used to model the stochastic parameters close to zero and which could not become negative (for instance the compressibility parameters a, b and c).

2.6.3. CORRELATIONS

This section illustrates the differences between the calculations with or without implementation of prior correlations. A more elaborate analysis can be found in Appendix F, the results are shown here. Assuming that all input variables are independent the modeling of the isotache model becomes relatively easy, however there are correlations found between the compressibility parameters a,b and c.

To model correlation coefficients there are several formulas that can be used, the two best known are the Pearson and Spearman Statistics Solutions [2015]. The main difference between these two methods is that Pearson is appropriate in case of linear data and Spearman for non-linear data. Looking at the type of data, the isotache parameters could be said to be linear. If a soil type is resisted to compress under a load the compressibility parameters will be higher, this works also the other way around. For instance peat has high compressibility parameters with respect to sand. Also the relations between the isotache parameters can be assumed linear, often only compressibility parameter b is known and with ratios the other two parameters are obtained, see 2.2.2. The correlation coefficient is as followed defined:

$$\rho = \frac{cov(X,Y)}{\sigma_X \cdot \sigma_Y} \tag{2.22}$$

A couple of approximations are made in Pearson's method, bot variables (X and Y) should be normally distributed, the variables should have a linear relation and it is assumed that the data is normal distributed about the regression line (i.e. homoscedasticity). In Matlab the correlations are modeled by computing the covariance matrix from the source data of the compressibility parameters, with this covariance matrix as input a random number generator is used to obtain correlated values for a,b and c.

As mentioned earlier the results from the K_0 C.R.S. tests of the KIS project were obtained and an analysis on this data set is carried out to find the correlations between a,b and c. From each test the values for the isotache variables were added to a data set and loaded into Matlab. Correlations were found for the initial data set for both clay and peat, in this thesis there is no distinction made for the correlations of each soil type. The correlations found for clay are representative for each type of clay. The same holds for peat, see Figures 2.13 and 2.14 for the dependence of a,b and c for clay and peat respectively. Looking at the values used in the KIS project for the compressibility parameters this assumption is valid here because for each type of clay (or peat) the same values for a,b and c are used.

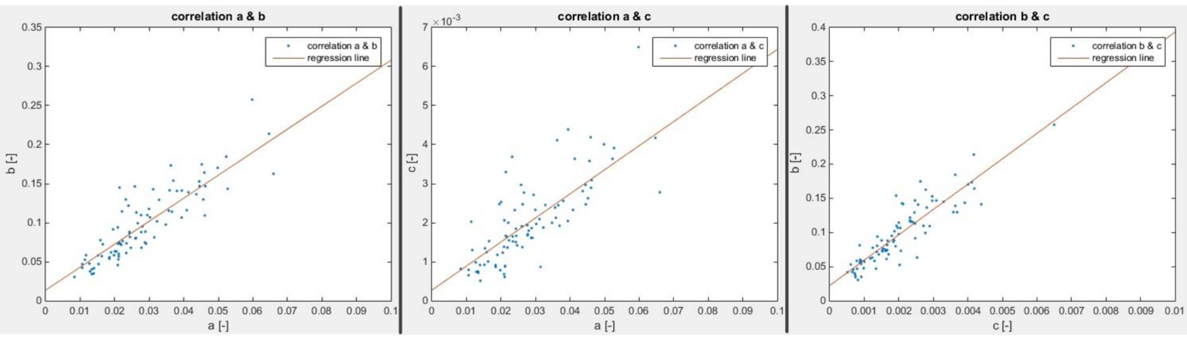


Figure 2.13: Correlations found for clay isotache variables

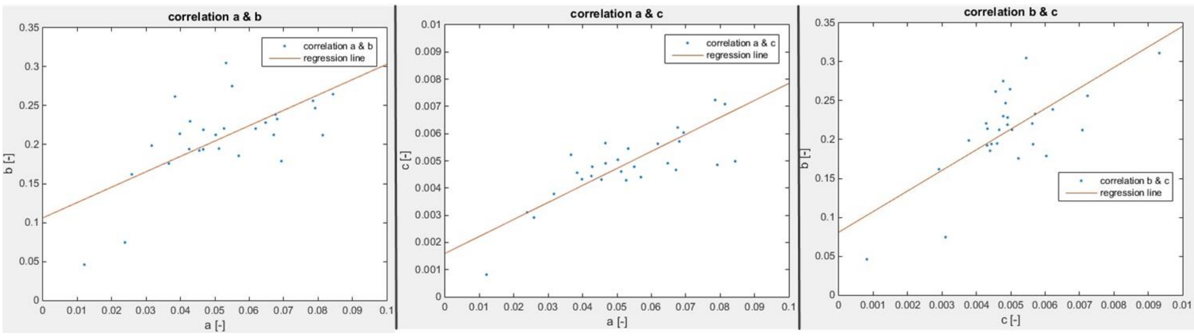


Figure 2.14: Correlations found for peat isotache variables

As can be seen in the figures there was more data available for clay than peat, also the compressibility parameters of clay are more dependent than the ones for peat because of a steeper regression line. This can also be concluded from the correlation matrices that are showed below, see Equations 2.23 and 2.24:

$$C_{clay} = \begin{bmatrix} 1 & 0.8626 & 0.7521 \\ 0.8626 & 1 & 0.8899 \\ 0.7521 & 0.8899 & 1 \end{bmatrix} \tag{2.23}$$

$$C_{peat} = \begin{bmatrix} 1 & 0.6810 & 0.8068 \\ 0.6810 & 1 & 0.7078 \\ 0.8068 & 0.7078 & 1 \end{bmatrix} \tag{2.24}$$

In the model these correlation matrices are used to describe the dependence between the random variables a,b and c for both clay and peat. In this way a large number of realizations is obtained for these variables. In order to show the dependency between all three compressibility parameters, see Figure 2.15.

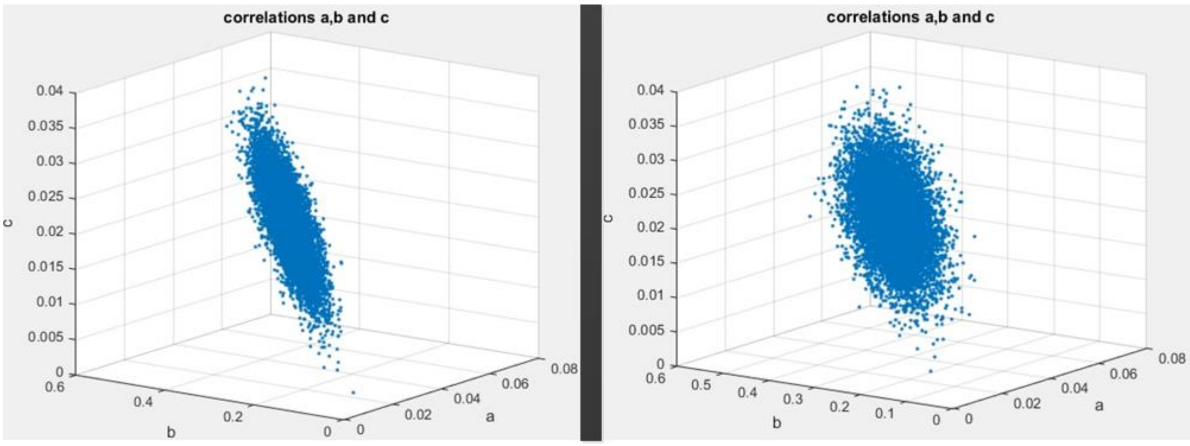


Figure 2.15: Dependency between a,b and c for clay (left) and for peat (right)

After a thorough analysis about the influence of the correlations on the a,b,c–isotache model it is noticed that the standard deviation of the settlement prediction becomes larger with respect to the uncorrelated settlement model. It is therefore not conservative to assume that the soil parameters are uncorrelated. See Appendix F for the description of this analysis.

3

CASE STUDY KINDERDIJK — SCHOONHOVENSEVEER

Chapter 3 will explain how the design and construction is carried out in the KIS project. It only elaborates on the fourth solution, the new dike on the river side of the 'old' dike. Therefore only section W, situated in the municipality Streefkerk is considered in the remainder of the thesis report. The instruments used to monitor the settlements are discussed as well as the monitoring plan. Practical issues regarding the monitoring plan and data are described which could occur during a construction project and lastly the prior settlement predictions are given.

3.1. LOCATION

The KIS project is divided into several sections along a 17 km stretch. Section W is located in the municipality Streefkerk, it is more or less situated in the middle of the KIS project between the municipalities Kinderdijk and Schoonhovenseveer. In the KIS project section W is divided into two parts, one part in the west and a part in the east. When naming section W in this thesis, the eastern part of section W is referred to, this is the normative part according to the design report of IV-Infra Wolters [2014d] regarding the settlement. Figure 3.1 shows the exact location of section W.

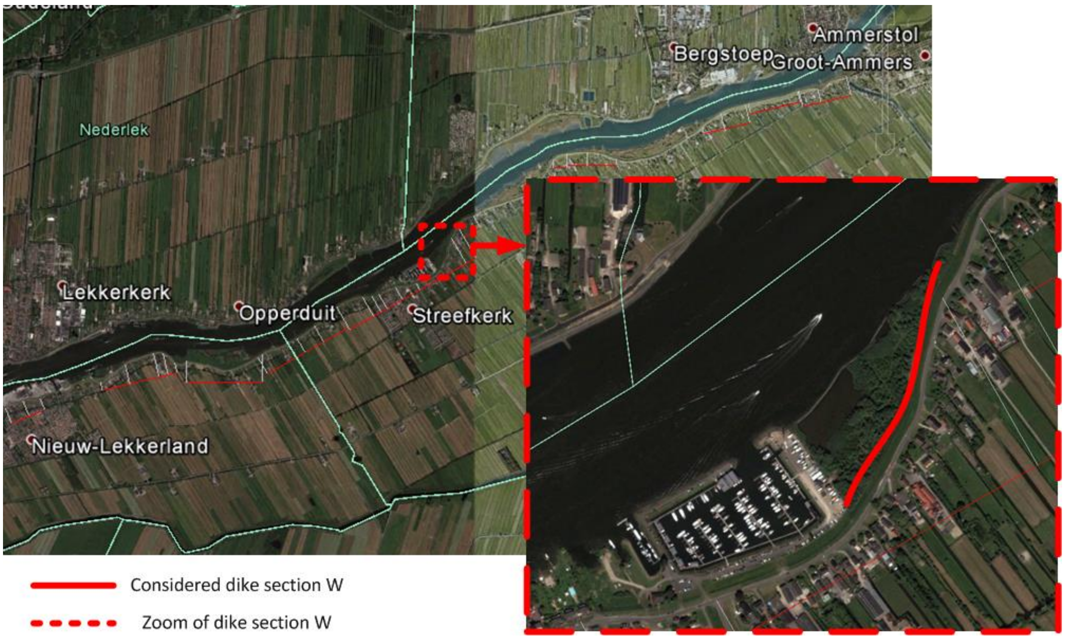


Figure 3.1: Location of section W

3.2. REQUIREMENTS

There are three important requirements given by the client Water Board Rivierenland Waterschap Rivierenland [2014b] with respect to the settlement that are applicable to this thesis as already stated in Chapter 1:

1. The dike reinforcement on the river side of the old dike must have a minimum design height of NAP +5,40 meter after 50 years, according to the binding document BIND-A. Waterschap Rivierenland [2014a]

2. The residual settlement of the dike reinforcement on the river side of the old dike has a maximum of 0,10 meter one year after the last raise. Waterschap Rivierenland [2013]

3. The residual settlement of the dike reinforcement on the river side of the old dike has a maximum of 0,30 meter fifty years after the last raise. Waterschap Rivierenland [2013]

3.2.1. DESIGN

IV-Infra was selected by Water Board Rivierenland to design the new dike on the riverside of the old dike. Water Board Rivierenland already made a reference design in collaboration with the engineering firm Witteveen + Bos. This is the basis of the design made by IV-Infra, which is far more detailed than the reference design. The engineers of IV-Infra took five steps to come up with the design of the new dike Wolters [2014d]. These steps hold several assumptions which are not discussed in this report if they are not related to the thesis subject. The five steps will be discussed briefly below:

1. **Check on reference design:** The schematization of the reference design made in D-Settlement and D-Geo Stability was checked on the following points: the normative profile, the soil parameters, layer structure, traffic load, design water level, schematization phreatic line, schematization water pressure and some computation settings in both software programs.

2. **Adjustment of schematization**: From the findings of the check, the settlement calculation was adjusted because of differences in method between the reference design and the implementation design. The settlement calculations were changed on the following points: adjust the layer structure outside the dike to the normative layer structure closest to where a CPT is done on the subsoil and possibly change the phreatic line. The CPT's for section W are taken on the inside of the dike because of the very low strength of the soil outside the dike (it was not possible to use heavy machinery on this weak soil). Furthermore it is mentioned that errors/inaccuracies were taken care of in the implementation design and that the parameters of the soil raise could still change during construction.

3. **Settlement calculation**: First the residual settlement was calculated. Using D-Settlement the vertical drainage was modeled. Also here the a,b,c–isotache model used, but in a two dimensional situation. The obtained geometry for the year 2067 (50 years after completion in 2017) with the settlement calculation must comply with the prescribed geometry in 2067 by the client. If the design requirements were not met than adjustments were made: change the phasing of the raises, add vertical drainage and apply a temporary overburden.

4. **Check stability during construction:** The waiting times between the raises were calculated with D-Settlement and D-Geo Stability by looking at the consolidation percentages for several scenarios. With the obtained waiting times between the raises the outer macro stability of the dike was checked during construction. Three extreme scenarios for the water level as load on the dike were calculated, namely: the fall of high water level to average high water level, extreme low water level, average low water level in combination with extreme precipitation (higher phreatic level).

5. **Determination of stability after construction:** Because of the large similarities between the prescribed phasing by the client and the computed phasing by IV-Infra there were no more calculations made for the macro stability after construction.

By following these steps the initial cross-section for section W is obtained, see Figure 3.2. The numbers in this figure represent the phases of the construction. The phases are more elaborately described in the next Paragraph 3.2.2.

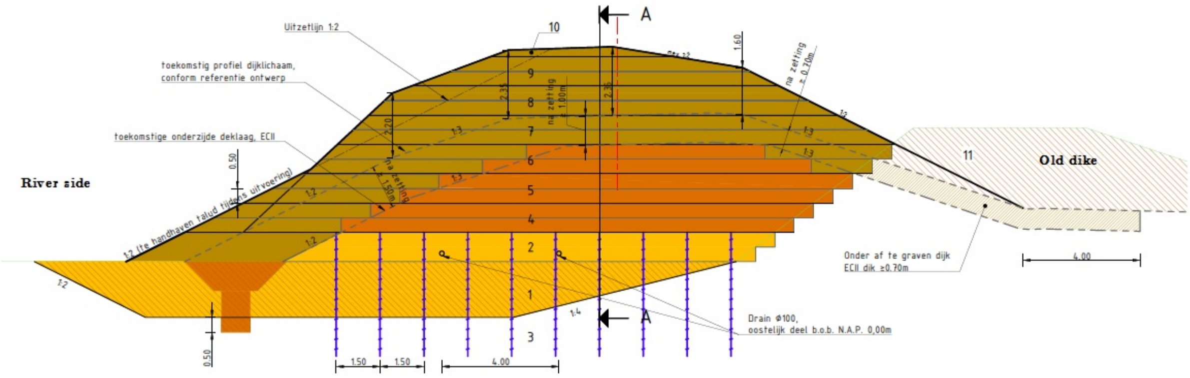


Figure 3.2: Design of new dike section W Wolters [2014d]

3.2.2. CONSTRUCTION METHOD

This section will explain how the new dike is constructed, the construction steps will be described by means of the different phases in the construction. The numbers of Figure 3.2 correspond with the numbers in the list below, only phase 11 is not depicted in the figure.

- **Phase 1:** Constructing the new dike will start by replacing the top layer (sludge, debris) with sand. See Figure 3.3 for the pile of sludge and debris depicted top left in the picture.
- **Phase 2:** The first actual raise is the work floor of the project with a layer thickness of 1 meter. Application of a work floor enables the contractor to work in dry circumstances. It also gives room for the horizontal and vertical drainage systems.
- **Phase 3:** In this phase the vertical and horizontal drainage are being installed. The vertical drainage is designed as follows, synthetic wick drains of 10 x 0,3 cm are applied and installed in a triangular grid with center to center distance 1,0 meter, see Figure 3.3. Minimum distance till the aquifer (Pleistocene sand) is 1,5 meter to prevent connection between the aquifer and the drains[1]. Also the horizontal drainage is installed in the work floor, this drainage system is coupled on to the vertical drainage as well as the pumps to make it possible for water to flow out of the weak soil layers. Applying these drainage systems increases the consolidation rate significantly and thus benefits the speed of construction.

Figure 3.3: Vertical drains section W and replacement of top layer

[1] When the aquifer comes into contact with vertical drains pipes are formed which transport sand and water to the surface because of the hydraulic head and can cause serious instability problems!

- **Phase 4:** Construction of the first clay layer. Category III[2] is applied here with a layer thickness of 1 meter. The choice is made to use category III clay because this layer is located in the core of the new dike. Therefore it is not needed to have a high erosion standard, a more important requirement is that the permeability is low. **Phase 5** and **phase 6** are respectively the second and third clay layer and these layers are also constructed with category III clay and are 1 meter thick.
- **Phase 7:** A clay layer of category II, this category has a higher erosion standard than category III. Therefore it is the first outer layer to protect the dike core of eroding during extreme conditions. This clay layer has a thickness of 1 meter. **Phase 8** and **phase 9** are also clay layers of category II and 1 meter thick.
- **Phase 10:** Construction of the last and outermost clay layer, the first protective layer of the dike against overtopping and overflow. This layer is of category II clay and has a variable thickness. The thickness depends on the amount of settlement during construction, if this is low the thickness could be less and vice versa. This is the layer which is of importance in this thesis. If predictions of the settlement can be made more reliable the thickness of this layer could be less and therefore probably resulting in lower costs.
- **Phase 11:** Lastly there is phase 11, this phase is the completion of the new dike. The geometry is profiled in a way that it corresponds to the required geometry set by the client. No extra clay is added, it will only be shifted to other locations.

As can be noticed from Figure 3.2 a clay coffin (in Dutch: kleikist) is depicted, however this clay coffin is not mentioned in the above list. It is of importance to construct this clay coffin to prevent water seeping through the dike. In some circumstances (i.e. large difference in water pressure inside the dike and outside the dike) the process of piping could be initiated. This could lead to unfavorable conditions regarding the stability of the dike, therefore it is important to construct this clay coffin.

Another remark about Figure 3.3 is the large pile of debris that can be seen in the top left of the picture. This is the weak top layer that is removed in phase 1 and replaced by sand. Here the vertical drains are connected to the horizontal drainage system and pumps as mentioned earlier. Also in this picture the settlement plates can be observed, these are the steel pipes with the orange marks on top placed in a row in cross-sectional direction.

From the design plan Wolters [2014d] it is noticed that the construction of the dike is as fast as possible to save as much time before completing the work. By doing this the time of settlement of the new dike is longest before completion and therefore it becomes easier to meet the requirements regarding the residual settlements. A disadvantage of a quick raising method is that a higher probability of instability of the dike is involved during construction.

[2] Clay used for construction of dikes can have different erosion standards, in the KIS project category II and III are used where category II clay better protects the dike structure against erosion. See TAW [1996] for the different clay categories regarding their erosion resistance.

3.3. MONITORING NETWORK

This section elaborates on the monitoring network and the used instruments to execute the monitoring in the KIS project. Also the monitoring plan is discussed. The network is important to monitor the settlements when a raise is applied, adjustments in the design or planning can be made if the settlements are not as expected or when it takes more/less time to achieve the desired level of consolidation before starting with constructing the next raise.

3.3.1. INSTRUMENTS

Monitoring in the KIS project, only taking the solution of the new dike on the river side into account, is done with settlement plates, GPS surveying system and piezometers Everts [2015].

SETTLEMENT PLATE

As mentioned earlier in this thesis, IV-Infra has installed a geotechnical monitoring network at the KIS project. Settlement data of the subsoil is obtained by the settlement plates. With a GPS device the exact position and depth of the measuring pole is determined during the construction. By adding a clay layer the settlement plate will settle more and this difference in height is measured. The pole of the settlement plate can be extended when the raises become to high, in this way the settlement plate grows along with the progress of the work. Some practical issues about these settlement plates are discussed in Appendix G. Figure 3.4 shows how the settlement plate looks like in a vertical intersection of the subsoil (left) and the right picture gives a settlement plate in section W.

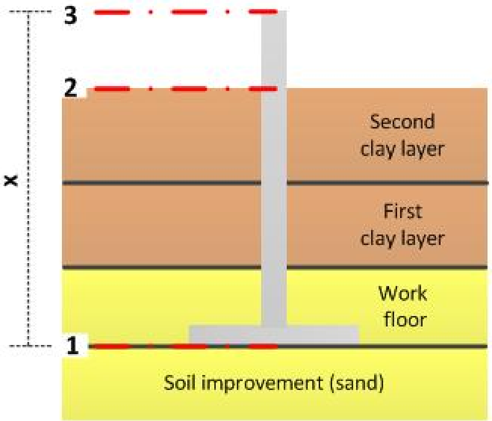


Figure 3.4: Intersection of subsoil with settlement plate (left) and actual settlement plate at section W (right)

The plate itself is placed on top of the soil improvement layer and embedded in the work floor to gain stability. Three heights as depicted in Figure 3.4 are measured to monitor the amount of settlement of the subsoil and the actual applied layer thickness. The settlement in time is obtained by measuring the settlement of position **1**, it is obtained by the distance **X** between position **1** and **3** (where the actual measurement is done) and is reliable under the assumption that the pole is not deformed. To keep track of the length of the settlement plate (i.e. distance **X**) a log is being kept. With the measurement at height **2** the actual applied layer thickness can be obtained by comparing the height of position **2** with the measurement done at **2** before the raise.

GPS surveying system

Measuring the positions **2** and **3** is done by a GPS device. The Trimble R8 GNSS System is used which is a highly accurate device working with satellites to determine the position of the settlement plate accurately on ± **0,02 meter**. See Figure 3.5 for the GPS device.

The white head of the GPS device is the actual part that determines the position, here the communication with several satellites takes place to determine the exact position. In the display next to the pole the user can change settings or store data that is measured. Also the computer corrects the position determined by the white head for the length of the pole. Another small but important feature is the level to keep the device in a good position.

For KIS the same person measures the positions of all settlement plates in one day when measurement of the settlement plates are carried out. The execution of the measurements itself is therefore reliable regarding the human error that is made. Each measurement is taken by the same person and taken under assumable same conditions, thus it is likely that the error made in the measurements is consistent throughout the project.

Figure 3.5: GPS surveying system: Trimble R8 GNSS System

Piezometers

Water pressure in the subsoil is monitored with gauges placed in the subsoil on different depths. Three or four gauges are installed at one specific spot and are installed at a depth of around 3 meter, 6 meter and 9 meter below the initial surface, thus below the top of the soil improvement layer. See Figure 3.6 on how the piezometers are installed.

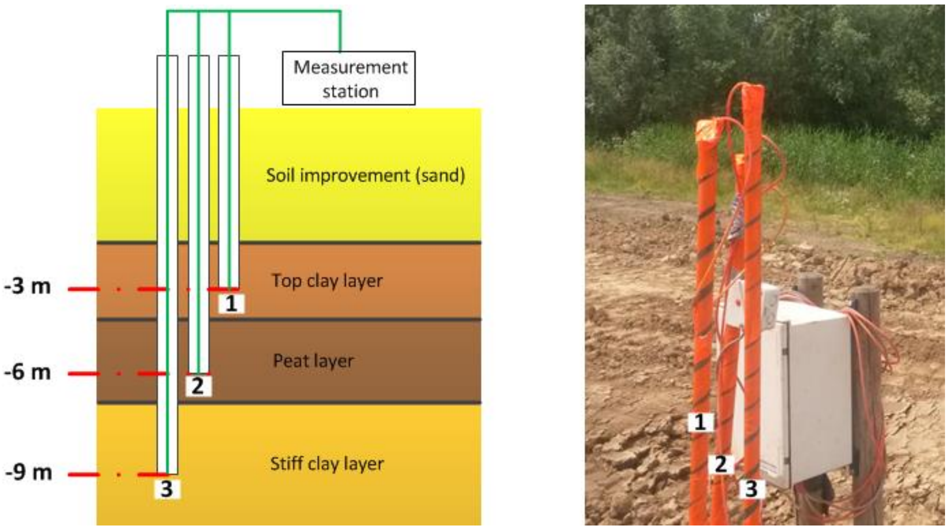


Figure 3.6: Intersection of piezometers in the subsoil (left) and measurement station at section W (right)

First hollow pipes are bored into the subsoil till the specific depth named above, then the gauges are lowered into the pipes and connected to the station. When constructing a raise the pipes can be extended in the same way as the settlement plates. The station takes every 10 minutes a measurement and uploads these measurements automatically to the Internet where they can be observed.

These measurements are used to determine the degree of consolidation of the subsoil after constructing a raise. When the gauges show that the water pressure at the different depths are at an acceptable level the next raise can be executed.

3.3.2. MONITORING PLAN

SETTLEMENT PLATES

The KIS monitoring network of settlement plates is installed as follows, the plates are placed with a centre-to-centre distance of 50 meter in longitudinal direction, in perpendicular direction a distance of about 5–7 meter, see Figure 3.7.

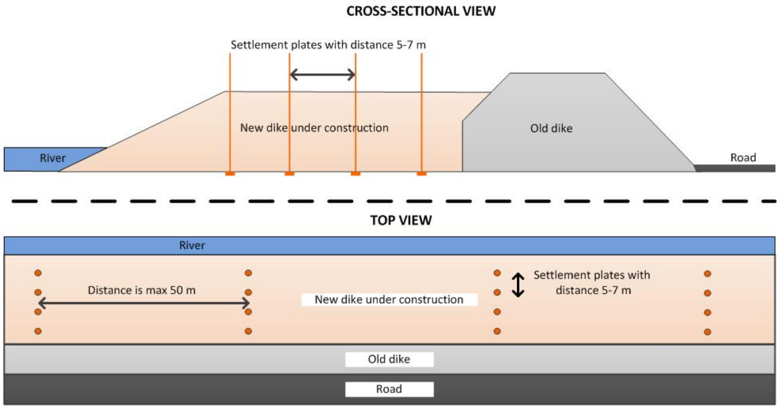


Figure 3.7: Monitoring grid for riverward dike reinforcement, top figure: perpendicular direction, bottom figure: longitudinal direction

Representative positions in the cross-section of the new dike are covered by the measurements of the settlement plates to gain enough insight in the behavior of the subsoil. These representative positions are mostly the positions where the settlement will be largest (i.e. not at the edges of the cross-section).

In order to obtain as much information as needed during construction to optimize the execution of the work and to keep a certain safety level the settlement plates are monitored on specific moments in time[3]. The frequency of measuring is as follows:

- After each new layer in the first month 5 measurements: just before the raising and 1, 7, 14 and 28 days after raising
- Then 3 months every 2 weeks: 6 measurements
- Then 8 months once per month: 8 measurements
- Then during 1 year once every 2 months: 6 measurements

Thus in total there are 5 + 6 + 8 + 6 = 25 measurements in 24 months. The measurements of the settlement plate just before and just after the raise gives the designer the information about the layer thickness of the raise that is applied. Measurements done at day 1 and 7 after a raise give insight in the amount of direct settlement (note: creep also plays a part in these measurements) and the other measurements give information about the amount of creep. Due to the application of vertical drainage the direct settlement will only be present during the first few days after the raise.

In case the actual amount of settlement does not correspond with the settlement predictions the following actions could be taken:

- Change the compressibility parameters a,b and c of the isotache model.
- Change the pore water pressure in the model.
- Apply less/more overburden.
- Improve the drainage in order to improve the consolidation process.
- Partly remove the executed raise.

[3]This is the theoretical basis of the monitoring plan, in practice the measurements are taken more or less every two weeks.

PIEZOMETERS

As mentioned earlier in Paragraph 3.3.1 the piezometers are installed on the same location but at different depths. Monitoring of the water pressure is important to check the stability of the dike structure during construction. Measurements are taken 6 times per hour and uploaded to Internet. Figure 3.8 shows the data obtained and the water pressures on different depths (different colors in figure) at one specific location at section W.

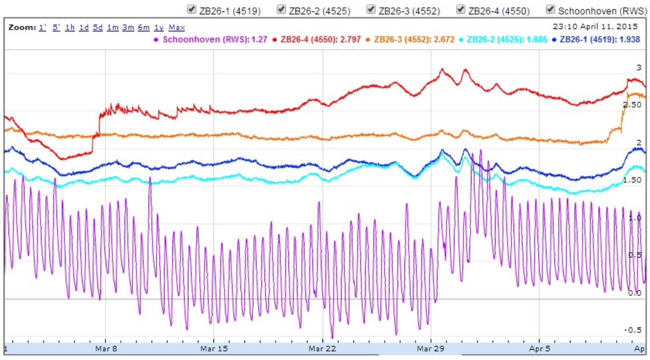


Figure 3.8: Graph of water pressure at section W for three different depths and the water level of the river Lek

3.4. PRIOR SETTLEMENT PREDICTION

This paragraph elaborates on the prior settlement prediction for the representative vertical which is also used in the 1D model for the posterior analysis. The prior settlement prediction is obtained by selecting the representative vertical in the D-Settlement model and take the results from the computation. Figure 3.9 shows the cross-section of the old and new dike together with the subsoil and vertical drainage (blue arrows). The old dike lies on the right of the new dike in the figure.

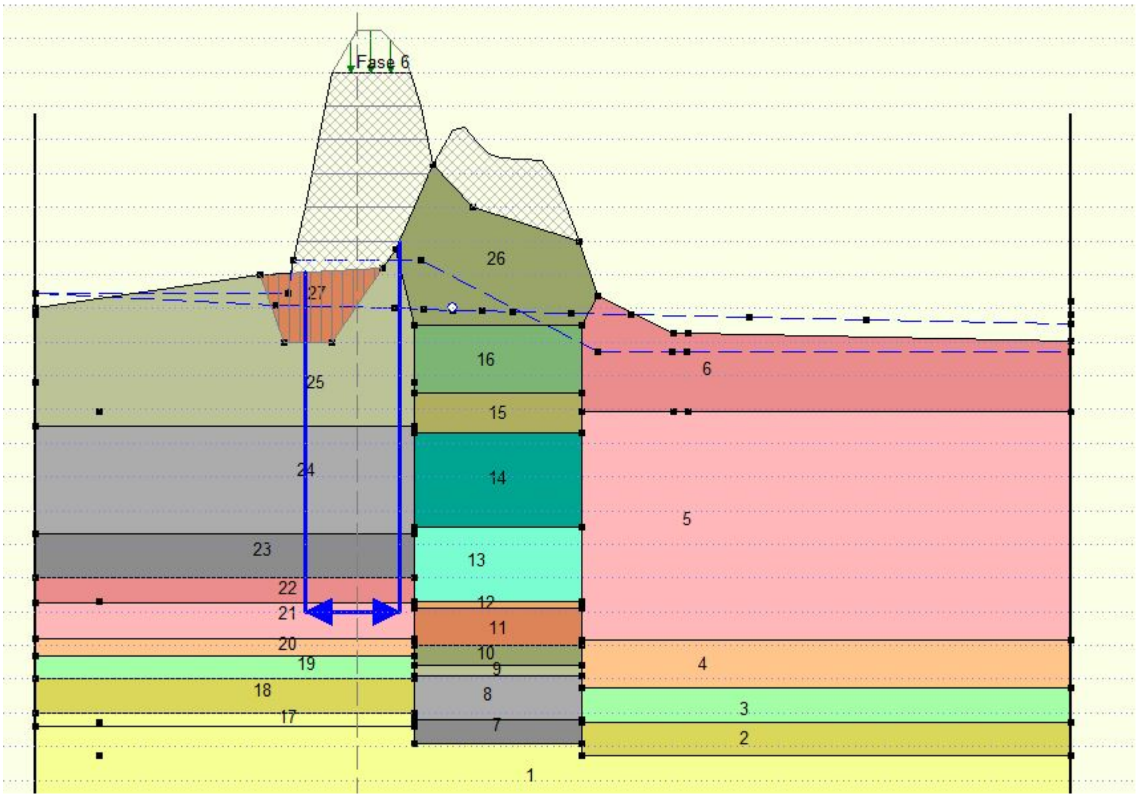


Figure 3.9: Representative cross-section

Below the settlement curve is showed for the representative vertical for the prior settlement prediction. In the calculation the soil parameters, raises and waiting times are implemented as described in the design report. See Figure 3.10 for the theoretical settlement curve as was obtained for the initial design of the riverward dike reinforcement.

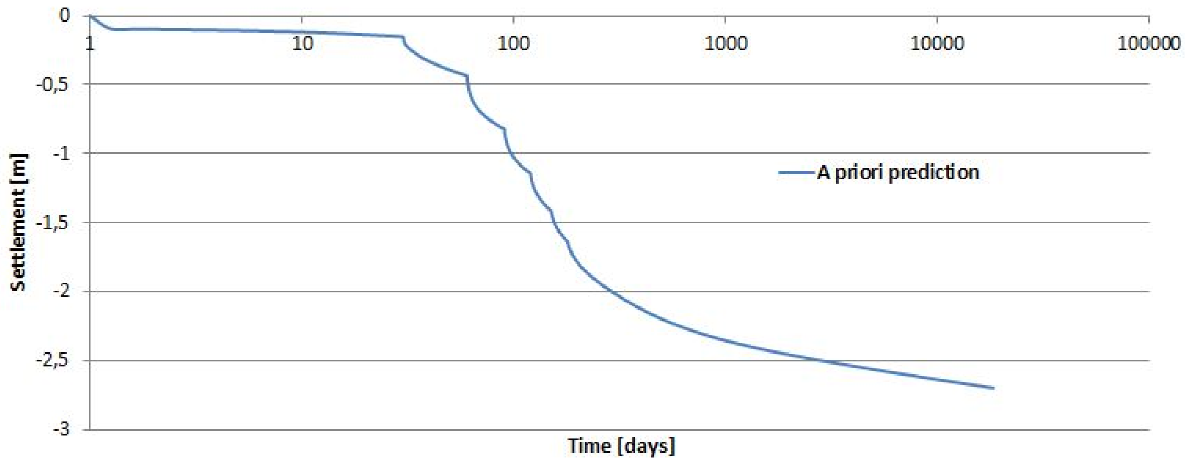


Figure 3.10: Theoretical settlement curve

From Figure 3.10 it can be noticed that no confidence bounds can be given due to the deterministic approach. For the subsoil variables in the KIS project expected or deterministic values are used to compute the settlement Blinde [2013] and Muntinga [2013]. In case there is no monitoring network of settlement plates it is important to use partial safety factors in the design (level I probabilistic method Vrijling et al. [2002]), however in the KIS project there is settlement plate information obtained and with this information the expected settlement can be adjusted based on actual settlement. No failure probability of meeting the total and/or the residual settlement requirements can be calculated and there is no feeling obtained about how good this prediction is without the use of a probabilistic approach and settlement plate information.

4

CASE STUDY - POSTERIOR ANALYSIS

The fourth chapter of this thesis explains the case study of the dike reinforcement project at the river Lek between Kinderdijk and Schoonhovenseveer. A probabilistic method is followed which implies the need for limit state functions and parametric distributions of the random variables. The methods and models used are described in Chapter 2 and the results from the predictions of D-Settlement and the proposed model are presented and compared in this chapter.

4.1. LIMIT STATE FUNCTIONS

As described in Paragraph 3.2 the Water Board Rivierenland has three requirements that need to be met during the lifetime of the dike structure. These three requirements are translated to limit state functions to use in the Monte Carlo Simulation and obtain failure probabilities and a-posteriori settlement values. Below these three limit state functions are listed. S is defined as the settlement, and the accepted failure probability of these limit state functions in case of a probabilistic approach is 10% as stated by the Water Board Rivierenland (see Chapter 5 for an explanation).

- Limit state function 1: $Z1 = S_{allowed_{(t_0 \dots t_{end})}} - S_{predicted_{(t_0 \dots t_{end})}}$

  Z1 = Total allowed settlement during lifetime - Total predicted settlement during lifetime

- Limit state function 2: $Z2 = S_{allowed_{(t_{1year})}} - S_{predicted_{(t_{1year})}}$

  Z2 = Maximum allowed residual settlement one year after completion **(0,10 m)** - Model prediction residual settlement one year after completion

- Limit state function 3: $Z3 = S_{allowed_{(t_{50year})}} - S_{predicted_{(t_{50year})}}$

  Z3 = Maximum residual settlement during lifetime **(0,30 m)** - Model prediction residual settlement during lifetime

From the limit state functions Z1 and Z3 it is made clear that these are long term requirements, here the difficulties arise as already stated in Chapter 1 with respect to the long term settlement prediction based on only a short period of obtaining information about the behavior of the subsoil. It can be imagined that there are relatively large uncertainties involved by doing these predictions, it is therefore important that the designer could use tools to design within a certain confidence bound. The probabilistic reliability updating method in this thesis gives the designer these tools and insight. Limit state function Z1 will differ for each scenario (later introduced in this chapter). The total allowed settlement depends on the total applied amount of clay, a larger amount of clay means a larger height and therefore the allowed settlement is also larger, however with a larger amount of clay the settlements will also be larger due to the increased weight on the subsoil. The total allowed settlement for Z1 is based on the requirement that the design dike height after lifetime needs to have a minimum of NAP +5,40 m Waterschap Rivierenland [2014a].

4.2. PARAMETRIC DISTRIBUTIONS OF VARIABLES

An assumption is made in this thesis to model all input parameters with the normal distribution. Because of a lack of data of all variables there could be no tests carried out to find a parametric distribution that fits the data well. This assumption is supported by the fact that D-Settlement could also use only the normal or lognormal distribution. More research should be carried out to find the appropriate parametric distributions for each input parameter.

In order to compare the results of the proposed model and D-Settlement model the input parameters are kept the same for both models when possible. For instance the layer thickness of the individual layers cannot be given a standard deviation per layer, only one and the same standard deviation for all soil layers for the D-Settlement model. Also the values of the variables are taken the same as how they are used in the KIS project, however the subsoil variables in the KIS project are taken deterministic (expected values). To obtain the normal distribution for each variable the mean is taken as the deterministic value and a standard deviation is chosen in the order of 10 to 30%. For instance the POP (Pre Overburden Pressure) values of the soil are given a standard deviation of 30% because of relative large uncertainty in their values, which is also confirmed by geotechnical advisers in the KIS project. Lower standard deviations are given for instance to the unit weight of the soil types. These soil characteristics can well be represented by borings and test in the laboratory. The compressibility parameters a,b and c are obtained from compression tests and in the KIS project the mean values of the results of these tests are used Muntinga [2013]. As input for the Matlab and D-Settlement models these values are taken as the mean with a coefficient of variation of 20% in the D-Settlement model. The proposed model uses different standard deviations due to correlations between a,b and c, the parameter sets of these parameters are computed using the correlation coefficients found with the data set of the parameters. These correlation coefficients are also used in D-Settlement.

4.3. SCENARIOS

To find the optimized thickness of the last layer regarding to reliability aspects as well as financial aspects, there are 16 scenarios conceived that are computed. These scenarios have different layer thicknesses and the planning of raising the new dike deviates from each other in the last phases of the project. To obtain the optimized last layer it is chosen to use scenarios that cover a range of possible solutions instead of iterating to an optimized solution what could give a smaller range of possible solutions and needs more computation time. Though the use of scenarios is implicitly an iterative way to find the optimized solution. Below in Figure 4.1 the different scenarios are showed. Appendix H shows the precise values of all scenarios. The blue bars represent the total height with respect to reference level NAP with the 6^{th} clay layer, red bars show the thickness of the 7^{th} layer and the green bars show the thickness of the temporary overburden that replaces the 7^{th} layer. The different heights per scenario cause the limit state function Z1 to change regarding the required design height of the dike after lifetime. For instance scenario 1 has a total height (without settlement) of NAP + 9 m and thus the total allowed settlement during construction and lifetime is 9-5,40 = 3,6 m.

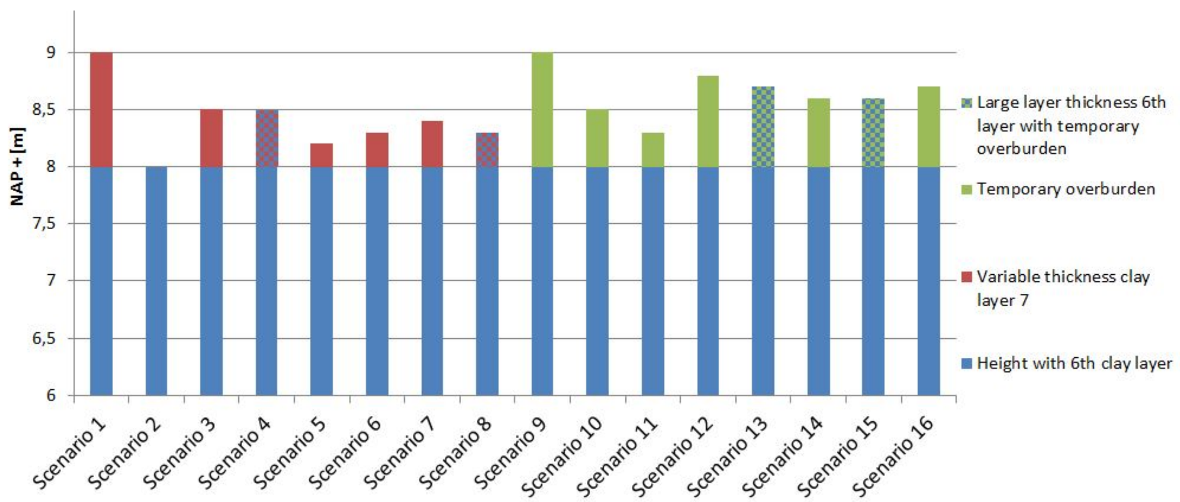


Figure 4.1: Scenarios

As can be seen from Figure 4.1 scenarios 9-16 are having a temporary overburden with the 6^{th} layer as final top layer instead of using the 7^{th} clay layer. With the temporary overburden the rate of settlement is increased before completion, which benefits the residual settlement requirements because of creating more settlement before completion and thereby reducing the amount of settlement during lifetime. However the temporary overburden probably causes an increase in total settlement and therefore affects the first requirement stated in Paragraph 4.1 (Note that for each scenario limit state function Z1 is different). The temporary overburden is not explicitly stated in the design report of the KIS project Wolters [2014d], although the designers are taking this option also into account. In this thesis the temporary overburden is taken into account as a possible solution to meet the residual settlement requirements. A period of one year is taken to apply the temporary overburden, looking at the planning of the KIS project the temporary overburden is removed about 200 days before completion which should be enough to finish the last work on the dike reinforcement.

4.4. SETTLEMENT PREDICTION PROPOSED MODEL

Below the results are showed for the 16 scenarios obtained with the proposed model. Figures are displayed to visualize the (updated) failure probabilities and the (updated) settlement predictions. Only the first scenario is fully displayed with figures, figures from the other scenarios can be found in Appendix H. The measurements that are used to update the settlement prediction can be found in Appendix E with an explanation how the measurements are used to obtain the results below.

SCENARIO 1

Scenario 1 is the most conservative scenario (largest amount of clay applied), the layer thickness of the last phase is 1 meter thick. There is no temporary overburden added to speed up the settlement process. Figure 4.2 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

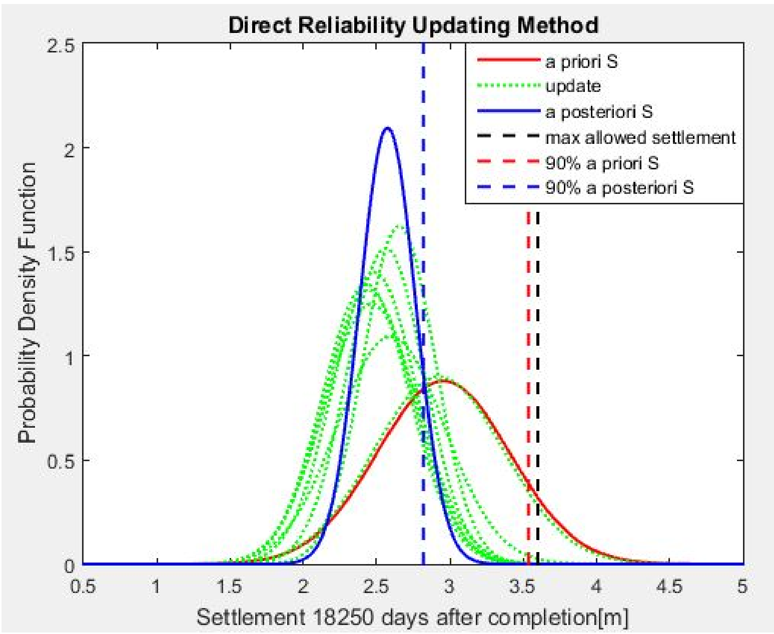


Figure 4.2: Total settlement prediction, a-priori and a-posteriori

As can be seen from Figure 4.2 the a-priori prediction (red) is updated (green) to the a-posteriori prediction (blue). The green lines show all the updates (measurements) in time, the more updates are computed the smaller the parametric distribution becomes. The green lines show the updated settlement prediction from each individual measurement that is used, in this way the influence of each measurement on the a-posteriori settlement prediction and the behaviour of the direct update method can be visualized. The a-posteriori distribution has a shifted mean and smaller standard deviation due to the filter effect of the direct update method. The mean is shifted to a lower value and also the 90% confidence interval is shifted to a lower value which shows that the total expected settlement will be less than a-priori predicted. It can also be noticed that the dashed black line lies right to the 90% failure probability which indicates that this scenario complies with the first limit state function Z1. The dashed black line is the requirement that follows from the applied scenario regarding the total allowed settlement during lifetime, for scenario 1 this means that the total applied height of the dike raises is NAP + 9 m (without settlement) and the required design height of the dike after lifetime is set on NAP + 5,40 m, this means that the total allowed settlement during construction and lifetime can be 9-5,40 = 3,6 m. This is represented by the black dashed line in Figure 4.2. The following Figure 4.3 shows the updated residual settlement one year after completion.

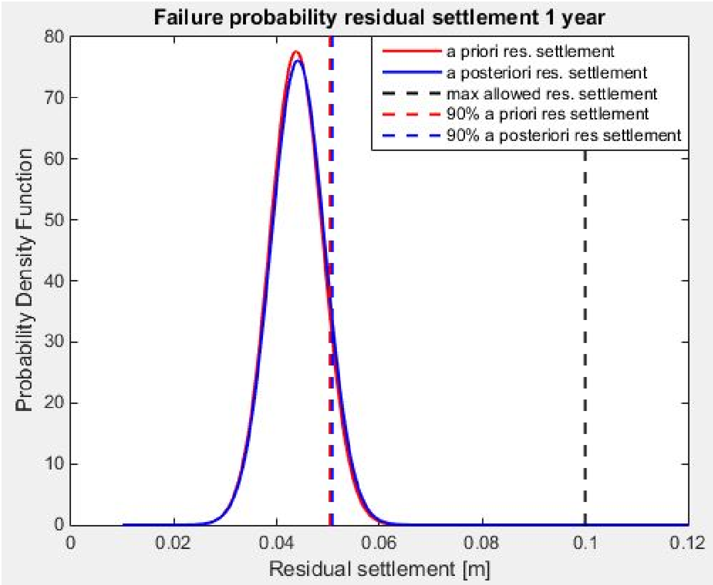


Figure 4.3: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure 4.3 shows that the probability of failure is approximately zero, because of the dashed black line, the requirement of 0,10 meter residual settlement one year after completion, is not crossing the probability density functions of the a-priori and a-posteriori prediction. The probability of failure is zero for the limit state function Z2. It can also be noticed that the a-priori and a-posteriori residual settlement do not differ much and are thus situated on top of each other.

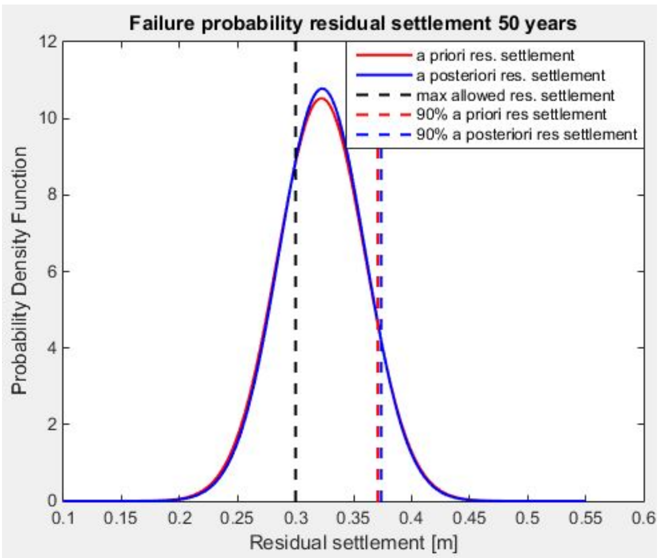


Figure 4.4: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

Figure 4.4 shows that the failure probability does not decrease with the a-posteriori prediction. Again as with 4.3 the update does not give a more reliable a-posteriori prediction for the residual settlement fifty years after completion. Because of the larger mass of the probability density functions to the right of the requirement (0,3 meter) this scenario has a high probability of failure for the limit state function Z3.

The above three figures showed the accompanying results of the three limit state functions mentioned in 4.1, however it also interesting how the a-posteriori predictions behave in time. To illustrate this see 4.5, note that the time is represented on the horizontal axis with a log scale.

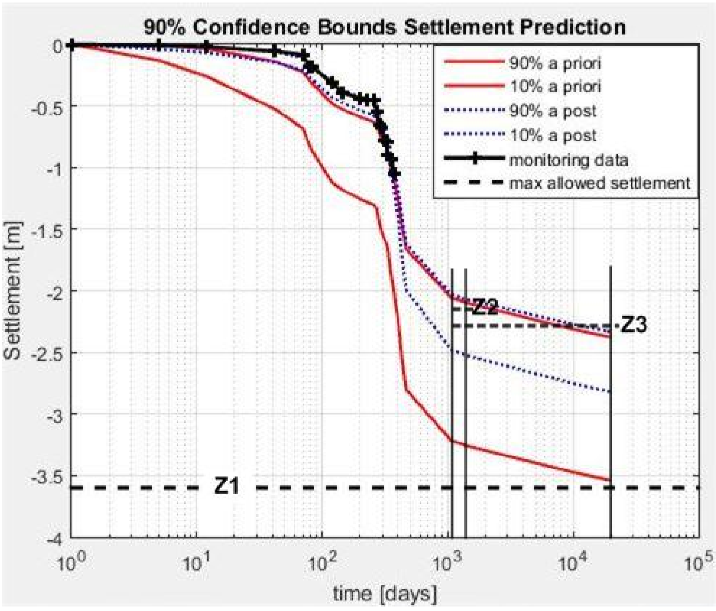


Figure 4.5: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

Above Figure 4.5 shows the development of the settlement in time with the 90% confidence intervals for the a-priori realizations (red) and a-posteriori realizations (blue). Also the monitoring data is added to give insight in how well the a-posteriori realizations perform. In order to show all three requirements these are displayed again with the black dashed lines. Requirement Z1 for the total settlement after lifetime can be read from the vertical axis and is displayed with the lowest horizontal black dashed line, while the residual settlement requirements Z2 and Z3 are the amount of settlement in certain periods illustrated with the horizontal black dashed lines with respect to the solid black lines that represents the date of completion (about 1065 days after start construction, most left line), one year after completion and fifty years after completion (most right line). The amount of expected residual settlement can be determined by looking at the slope of the settlement curve between the time span of each residual settlement requirement. The total settlement requirement shows that the a-priori and the a-posteriori predictions fulfill the requirement of the total settlement in time (limit state function Z1). Figure 4.5 and Figure 4.2 are actually visualizing the same results for the total settlement that is expected when the lifetime of fifty years is reached.

Below Figure 4.6 summarizes the results obtained with the proposed model regarding to the failure probabilities of the three limit state functions explained in Paragraph 4.3. A failure probability of at most 10% is accepted by the Water Board Rivierenland to meet the settlement requirements, which is further explained in Chapter 5. However the 10% failure probability is already showed in the figures of this chapter.

In the following text about the different scenarios only figures like 4.6 are showed to summarize the results. Appendix H shows all results and figures of all scenarios.

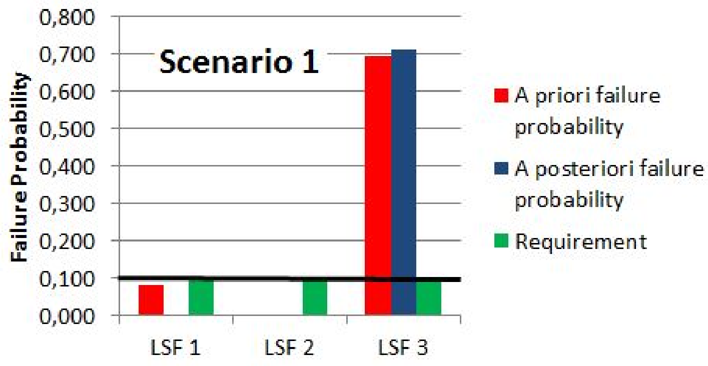


Figure 4.6: Results scenario 1

SCENARIO 2

This scenario leaves layer 7 out of the construction and thus uses 1 meter clay less than scenario 1. See Figure 4.7 for the results.

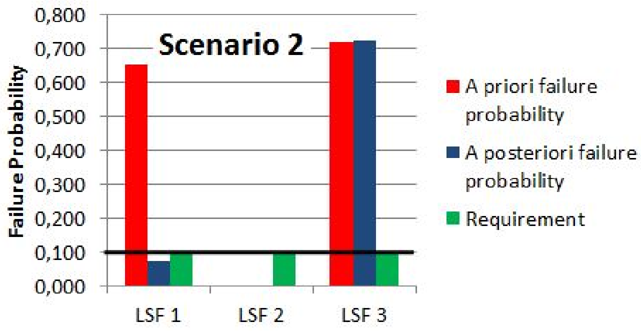


Figure 4.7: Results scenario 2

SCENARIO 3

Figure 4.8 shows the results of scenario 3 where the layer thickness of the 7^{th} clay layer is 0,5 meter.

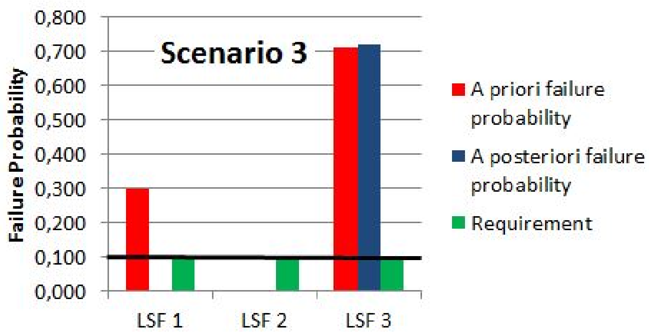


Figure 4.8: Results scenario 3

SCENARIO 4

Scenario 4 combines the last two raises to one raise with a layer thickness of 1,5 meter. It should be investigated if the stability requirements are met. Below Figure 4.9 shows the failure probabilities of the three limit state functions

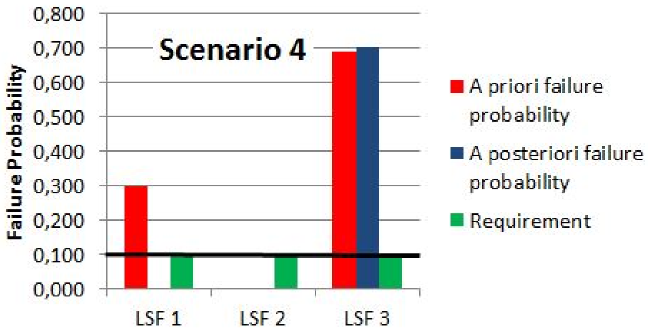


Figure 4.9: Results scenario 4

SCENARIO 5

For scenario 5 a less thick 7^{th} clay layer is used, see Figure 4.10 for the results.

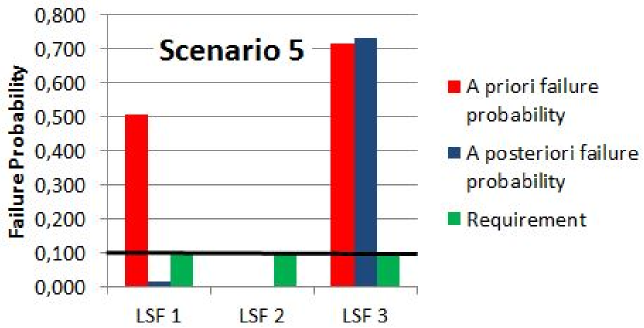


Figure 4.10: Results scenario 5

SCENARIO 6

The sixth scenario has a 7^{th} layer thickness of 0,3 meter. Figure 4.11 shows the results.

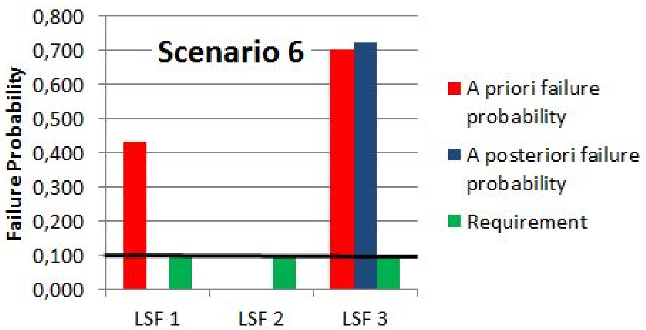


Figure 4.11: Results scenario 6

SCENARIO 7

Figure 4.12 shows the results of scenario 7.

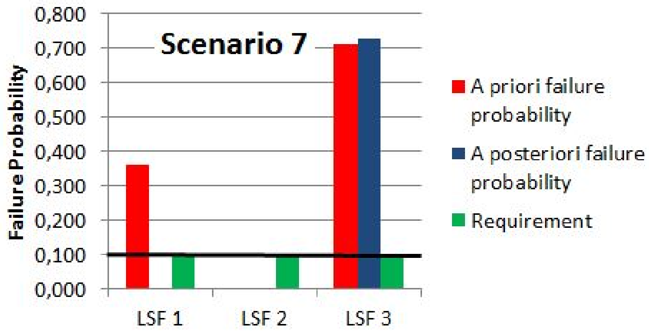


Figure 4.12: Results scenario 7

SCENARIO 8

Figure 4.13 shows the results of scenario 8. Here again the last two raises are combined to one raise but with a smaller layer thickness, namely 1,3 meter.

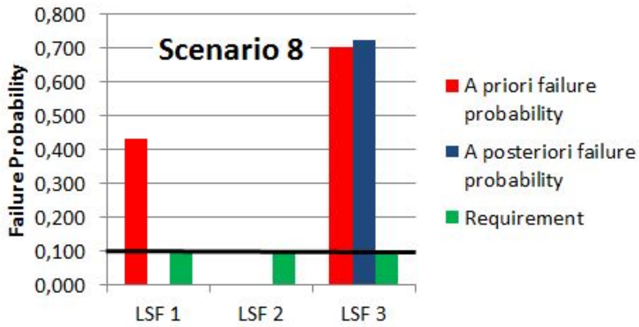


Figure 4.13: Results scenario 8

SCENARIO 9

Scenario 9 is different from the other previous scenarios because a temporary overburden is applied in the construction method. Thirty days after applying clay layer 6 an overburden is constructed with a layer thickness of 1 meter. One year later the overburden of 1 meter is totally removed. This method increases the amount of settlement in an earlier stage and is advantageous for the residual settlement requirements. Figure 4.14 shows the results of the failure probabilities.

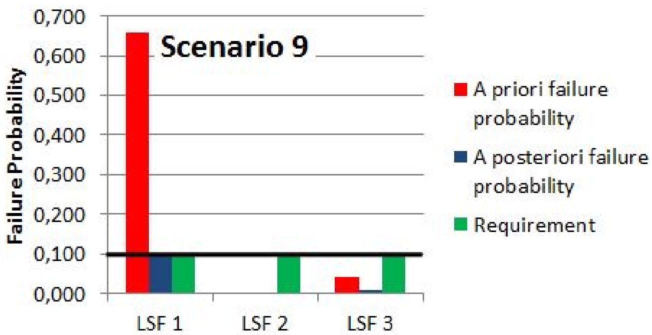


Figure 4.14: Results scenario 9

SCENARIO 10

Scenario 10 is the same as scenario 9 but now with a temporary overburden of 0,5 meter, see Figure 4.15.

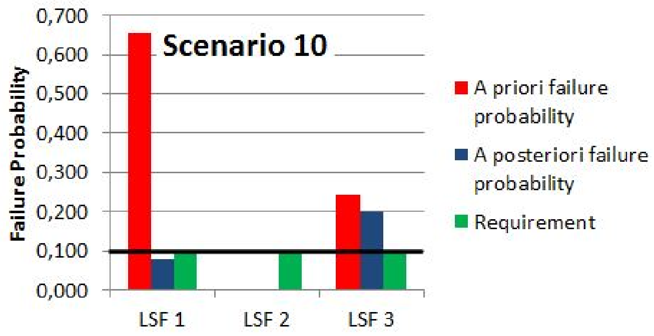


Figure 4.15: Results scenario 10

SCENARIO 11

Same as the two previous scenarios but now with an overburden of 0,3 meter. See Figure 4.16 for the results of scenario 11.

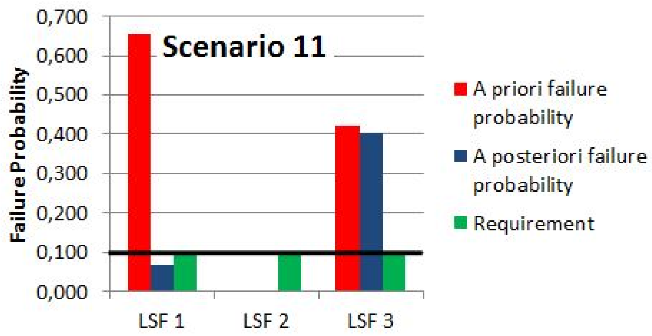


Figure 4.16: Results scenario 11

SCENARIO 12

This scenario has a temporary overburden of 0,8 meter. See Figure 4.17 for the results of scenario 12.

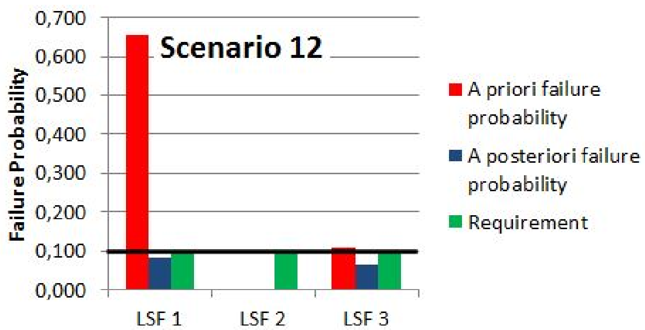


Figure 4.17: Results scenario 12

SCENARIO 13

This scenario has a temporary overburden of 0,7 meter. This temporary overburden is together with the 6^{th} clay layer constructed, thus the last applied raise has a layer thickness of 1,7 meter. Due to this thickness, stability requirements of the dike must be investigated. See Figure 4.18 for the results of scenario 13.

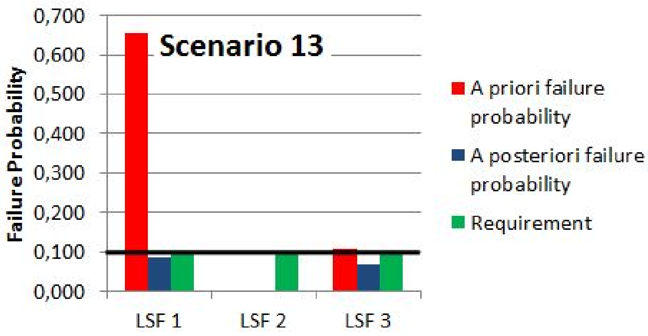


Figure 4.18: Results scenario 13

SCENARIO 14

Scenario 14 also has a temporary overburden but with a thickness of 0,6 meter. This overburden is constructed separately of clay layer 6. See Figure 4.19 for the results of scenario 14.

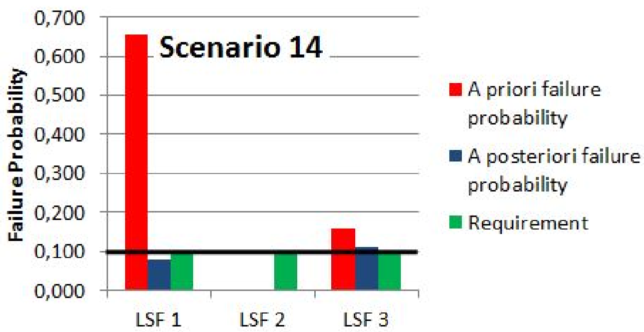


Figure 4.19: Results scenario 14

SCENARIO 15

Scenario 15 is the same as scenario 13 but now with a temporary overburden of 0,6 meter. See Figure 4.20 for the results of scenario 15.

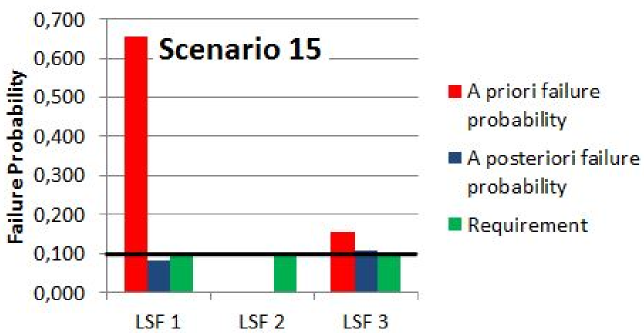


Figure 4.20: Results scenario 15

SCENARIO 16

Scenario 16 has a temporary overburden with a thickness of 0,7 meter. This overburden is constructed separately of clay layer 6. See Figure 4.21 for the results of scenario 16.

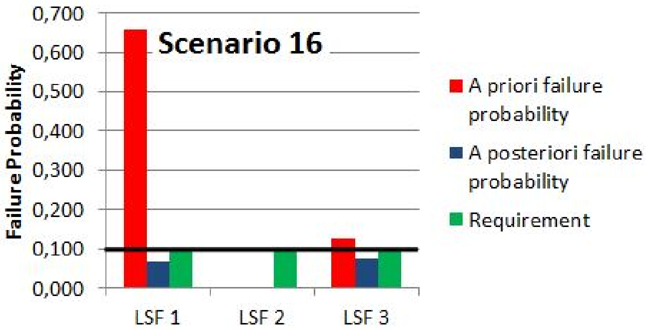


Figure 4.21: Results scenario 16

Conclusions of the results can be found in Paragraph 4.7 and are further elaborated in Chapter 5. However the preliminary conclusion that can be drawn here is that scenario 16 looks the most promising of all scenarios.

4.5. SETTLEMENT PREDICTION D-SETTLEMENT

To compare the results obtained from the proposed model the most favorable scenario is also used in D-Settlement to compute settlements. From Paragraph 4.4 scenario 16 is the most promising regarding the results of the failure probabilities for the three limit state functions. From stability calculations with the software program D-Geo Stability, the Factor of Safety (F.O.S.) is close to 1 when applying scenario 16. When computing the stability for scenario 13 (which combines the 6^{th} layer and overburden at once) the F.O.S. is about 0,85 what indicates that this scenario is more likely to cause macro instability of the dike during construction. To be safe scenario 16 is chosen as the most favorable one regarding the amount of settlement as well as the macro stability during construction.

As already explained in Paragraph 2.5.1 D-Settlement also uses monitoring data to update the prior beliefs about the expected settlement. However there can be no limit state functions implemented in D-Settlement and thus the way of obtaining failure probabilities is different. With a Monte Carlo Simulation in combination with the Settlement Plate fit option, a form of Bayesian Updating is obtained and the results of the settlement predictions are displayed within a certain confidence bound. Now the total settlement can be chosen on what reliability the designer wants to have for its predictions about total and residual settlement. This is the other way around with respect to the proposed model where at the end of the calculation the failure probabilities are computed and than checked with the accepted probability of failure (see Paragraph 5.1.1 for the accepted probability of failure).

A major difference between the Matlab and D-Settlement model is the use of consolidation model, D-Settlement models the consolidation with the Darcy model. In Deltares Systems [2011] some limitations of the Terzaghi model with respect to the Darcy model are listed. In general the Darcy model is more accurate than Terzaghi, however the most important limitation of Terzaghi with respect to Darcy is that the consolidation process is more accurate for Darcy during reloading or unloading of the subsoil. This is because Darcy uses the permeability k which depends on the void ratio e. When loading the subsoil compression takes place and the pore volumes decrease which leads to a smaller permeability. Consolidation of the subsoil depends on the permeability as well as the stiffness of the subsoil, using Darcy this is accounted for, when using Terzaghi the consolidation process not takes into account the changing stiffness in time and space. Unloading the subsoil means a more stiff response of the subsoil than in the case of loading, in the case of unloading actually a different consolidation coefficient should be used. For instance, Terzaghi would show the same period of consolidation when reloading the soil, where Darcy shows a faster consolidation process in the case of reloading[1]. This is also noticed in the proposed model where the degree of consolidation is the same for each raise.

[1]The use of the Darcy consolidation model only works when the permeability and the stiffness of the subsoil are stress-dependent. When the stiffness is not stress-dependent and the permeability is, the consolidation period is overestimated. When the permeability is constant the consolidation period is underestimated with the use of the isotache model in combination with Darcy.

Another difference between Matlab and D-Settlement settings is the number of Monte Carlo Simulations, Matlab uses one million realizations of the settlement on each point in time incorporated in the model and D-Settlement uses 10.000 possible settlement curves of the isotache model. This difference is mainly because the updating method presented in this thesis needs a large number of realizations to handle the number of measurements used to update the prediction, as mentioned earlier. Both models are stable for the number of Monte Carlo Simulations used.

When using the 1D model in D-Settlement it is not possible to simulate vertical drainage, to solve this the same approach as for the proposed model is followed. Each layer in the subsoil is marked as a drained layer which implicates a high consolidation coefficient.

SCENARIO 16, TOTAL SETTLEMENT

The total settlement is computed with a Monte Carlo Simulation with and without the Settlement Plate Fit option. In this way the a-priori and a-posteriori settlement prediction of D-Settlement is obtained. Just as for the proposed model the confidence bounds are set to 90%. With this calculation the failure probabilities of the limit state functions Z1 and Z3 can be computed. Below in Figure 4.22 the results of the a-priori and a-posteriori settlement predictions are shown. The same features are showed as for the results obtained with the proposed model.

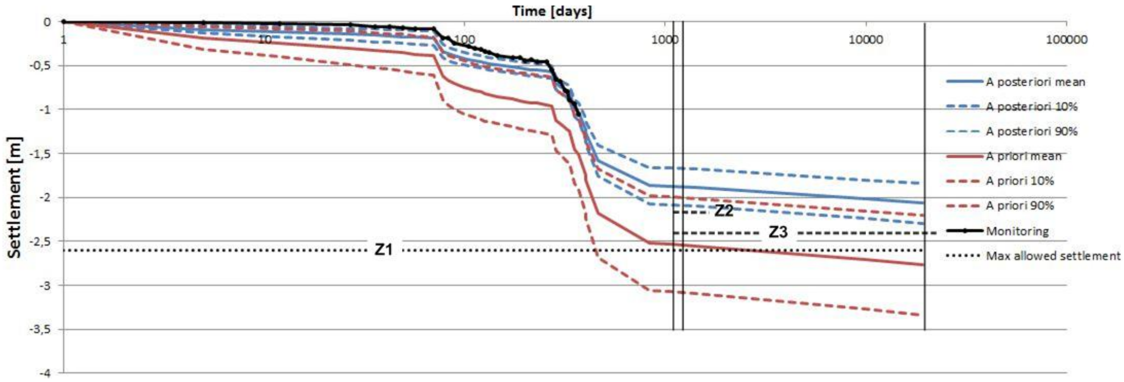


Figure 4.22: a-priori and a-posteriori settlement prediction D-Settlement for scenario 16

Figure 4.22 looks similar to the figures obtained by the proposed model (see Chapter 5 and Appendix H), the a-posteriori prediction shows that the amount of settlement will be lower as initially thought. Comparing the monitoring data with the a-posteriori settlement prediction it seems that there is a good fit. As can be noticed D-Settlement shows an amount of settlement that is lower than the predictions with the proposed model, the 90% confidence bound of D-Settlement is located about 45 cm lower than the 90% confidence bound of the proposed model (see Appendix H). The same results of the total settlement are also shown in Figure 4.23 but now the probability density functions of the prior and posterior distribution are given. Again the red and blue lines are the 90% probability of exceedance of respectively the prior and posterior distribution. It can be seen that the a-posteriori probability of exceedance lies much lower than the black dashed line representing the requirement of the total settlement. Another remark that must be made is that the consequence of the updates in time cannot be visually showed when using D-Settlement as was the case for the proposed model, see Figure 4.2. Only the posterior distribution is obtained, not the distributions in between for each update and therefore less information of the updating process is obtained. Each measurement has a different effect on the posterior distribution as can be seen from the green lines in Figure 4.2 of the proposed model, that shows that the posterior distribution changes in time.

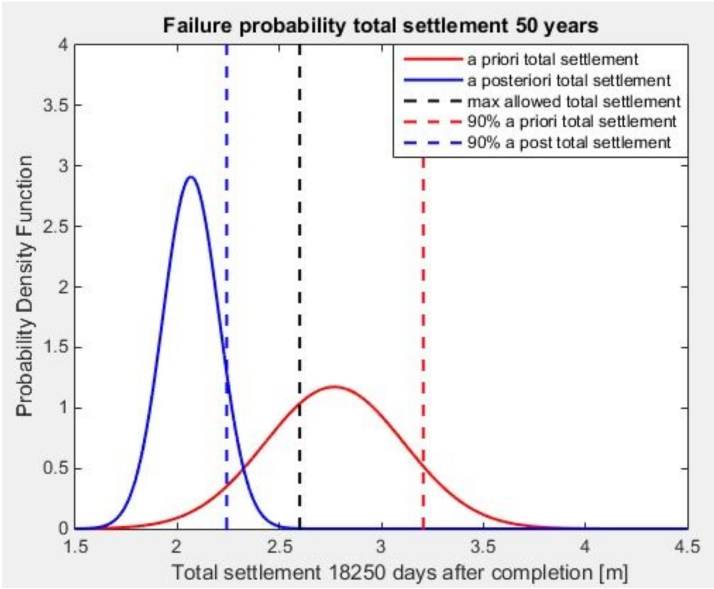


Figure 4.23: Probability density function for total settlement with D-Settlement for scenario 16

SCENARIO 16, RESIDUAL SETTLEMENT ONE YEAR

Figure 4.24 shows the updated settlement prediction of the one year residual settlement requirement. Also in this case the residual settlement requirement for the first year after completion is easily met according to the probability density functions of the resulting residual settlement after one year. When using the monitoring data the prediction of the residual settlement becomes even more reliable and gets a value close to 1 cm.

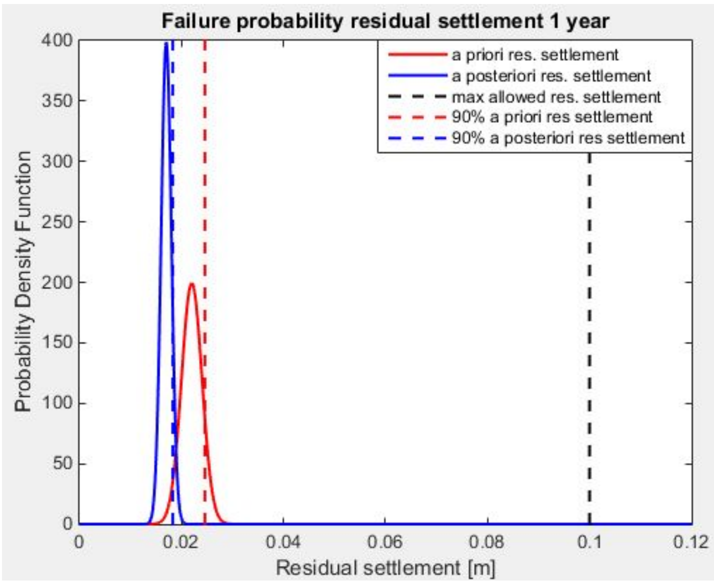


Figure 4.24: a-priori and a-posteriori residual settlement prediction 1 year D-Settlement for scenario 16

Comparing the results of the proposed model and D-Settlement model with respect to the requirement for the one year residual settlement, D-Settlement shows more reliable predictions than the proposed model. However this requirement is easily met by both models because they both show a failure probability of 0 for limit state function Z2.

SCENARIO 16, RESIDUAL SETTLEMENT FIFTY YEARS

Figure 4.25 shows the updated settlement prediction of the fifty years residual settlement requirement.

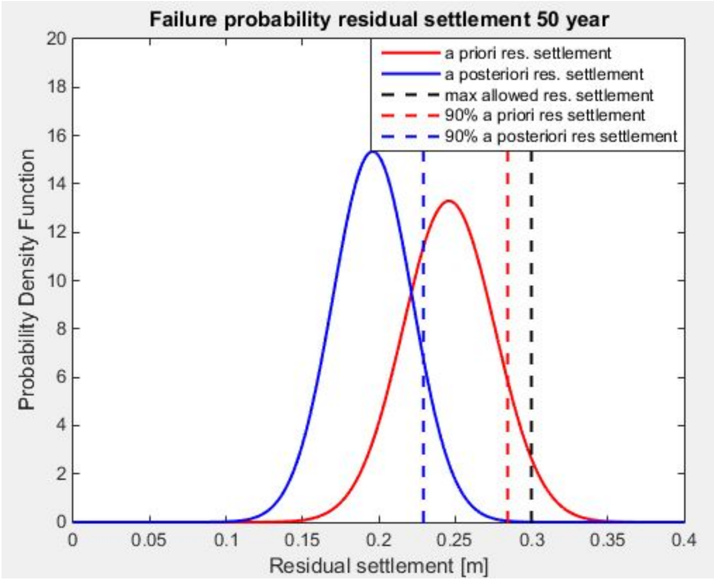


Figure 4.25: a-priori and a-posteriori residual settlement prediction 50 years D-Settlement for scenario 16

Again D-Settlement shows similar results as the proposed model, what indicates that the reliability updating method used in this thesis works for the settlement predictions. However the Bayesian Updating method in D-Settlement has a larger influence on the a-priori prediction comparing it to the reliability updating method by Straub.

SCENARIO 16, FAILURE PROBABILITIES D-SETTLEMENT

Below in Figure 4.26 and Table 4.1 the failure probabilities obtained with D-Settlement are showed for the limit state functions.

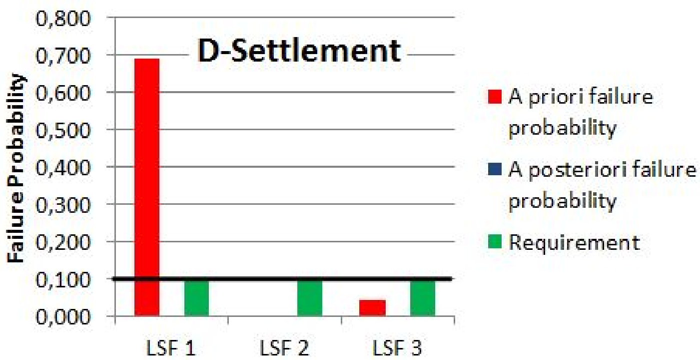


Figure 4.26: Results scenario 16 D-Settlement

It can be seen that the results in D-Settlement with respect to the posterior failure probabilities (approximately zero) are more favorable than the results obtained by the proposed model.

	Failure probability total amount of settlement
a-priori	0,6912
a-posteriori	0,00054
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,044
a-posteriori	0,00044

Table 4.1: Results scenario 16 D-Settlement

A possible reason that the results from D-Settlement are more favorable is because D-Settlement uses the least square method in order to fit the predictions on the measured settlement, however by using the least square method the mean of the possible outcomes become highly important with respect to the values found in the tails of the parametric distribution. The method used in this thesis also takes the extreme values of the tails into account during the update of the settlement predictions. This could be an indication that D-Settlement actually underestimates the expected settlement due to giving less importance to the values found in the tails of the parametric distributions.

4.5.1. SCENARIO 16, 1D TO 2D

The results showed in this chapter are resembling a 1D model, this model can only deform in vertical direction. However the 1D model does not cover the whole soil behavior in reality because of spatial variability of the subsoil. The soil particles can also move in horizontal direction and therefore introducing a horizontal deformation of the subsoil. Modeling the horizontal deformation acquires a finite element model which is not carried out in this thesis. However to compare the 1D model used in this thesis to the 2D situation where the stress distribution in 2D space can take place Figure 4.27 is showed below. This figure shows the differences found between the 1D and 2D models computed with D-Settlement. The 2D model takes into account the boundaries of the load (raises of clay) and the different layer thickness of subsoil layers in the cross-section.

The differences in the results of both models can most likely be found in the two-dimensional effects and stress distributions of the effective stresses and water pressures which are better represented in a 2D model. As can be seen from Figure 4.27, between the raises the 1D model shows a steeper curve than the 2D model what indicates that the consolidation process is faster in the 1D situation. A reason for this difference is that the vertical drainage in the 1D model is simulated by using a high consolidation coefficient in contrast to the 2D model where actual vertical drainage can be modeled, which is more representative for the actual soil behavior.

Analyzing the illustration below it becomes clear that the total and residual settlements are slightly underestimated by the 1D model. The creep part of the settlement curve of the 2D model is steeper than its 1D counterpart and also the total settlement is a bit more than in the 1D situation. In the unloading part of the settlement curve, at about 830 days, the 2D curve starts to deviate from the 1D curve. Probably the unloading of the subsoil is more advantageous for the 1D model and causes a steeper creep curve for the 2D model due to the differences in stress distribution. However when comparing these results with the 1D model it shows that the approach explained in this thesis closely approximates the 2D model although the differences found will affect the third limit state function.

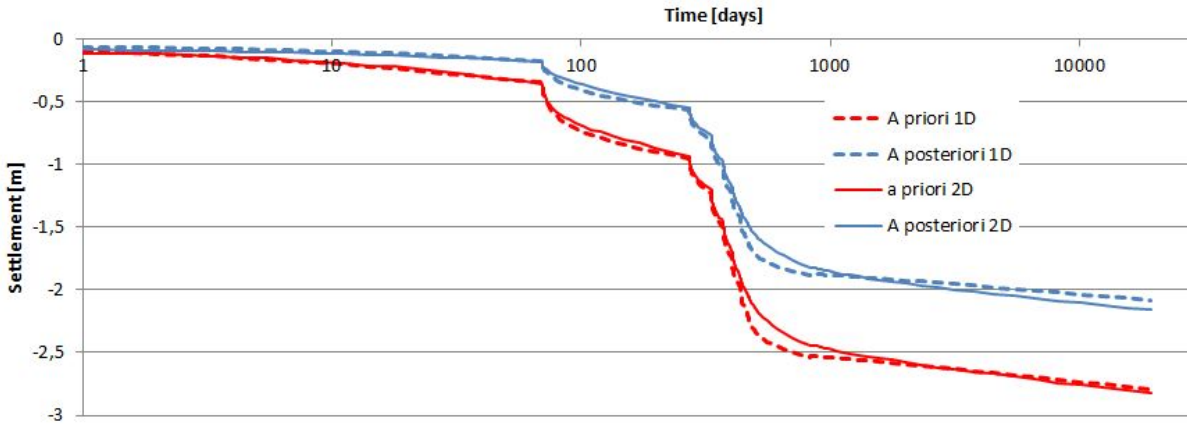


Figure 4.27: Comparison of 1D model to 2D model

4.6. REMARKS OF RESULTS PROPOSED MODEL

Chapter 4 presents the results of the reliability updating method used in this thesis, some remarks can be made about how this method is used and what the influence is on the obtained results.

The first remark that is made is from the fact that in combination with a lack of computation strength and the fact that the filter effect of the direct update method is strong, induce a too small amount of a-posteriori realizations that are left after using all measurements available to update the settlement prediction. Due to this limitation of the model measurements are chosen to represent the settlement behavior. The choice of the selected measurements play an important role in the final results of the settlement prediction. During the process of obtaining the results it was tested how the proposed model responded on the use of specific measurements in time. For instance, only five measurements were used which were obtained during the winter period where practically no work was executed for the new dike. These measurements are located in the creep part of the settlement curve, and induces low probabilities of failure with respect to the three limit state functions of Paragraph 4.1. The same is done but now for the last five measurements that are done during this research, these measurements were taken when the raises where applied in a relatively short period after each other (i.e. period of thirty days). Now the measurements are located in the steep part of the settlement curve, the consequence of using these measurements was that the failure probabilities of the three limit state functions became higher. It can be concluded that the reliability updating method is sensitive of which information is used to update the settlement predictions. This follows directly from the fit of the realizations on the measurements, when only measurements used where the settlement rate is high it is likely that the a-posteriori realizations show higher total and residual settlement. The model chooses only realizations that comply to this relatively high settlement rate which indicates that a larger settlement is expected a-posteriori. The same can be said vice versa, when only using measurements which indicate a relatively slow settlement rate it is likely that the model results are realizations where similar settlement rates can be found. From this observation the choice is made to represent the settlement behavior as good as possible by choosing the measurements of specific points in the settlement curve that together resembles the settlement behavior as good as possible. The consequence of using only specific points in the settlement curve is a loss of information and accuracy between the specific point on the curve.

For each scenario the same assumptions are made, one assumption that need to be mentioned here is that the construction of the new dike at section W was not yet finished when working on this thesis. The proposed model is build till clay layer 5 with actual waiting times between each raise and layer thicknesses. Layer 5 and 6 were not yet constructed when doing this research, therefore the assumption is made that the waiting time was 31 days between raise 4-5, 5-6, and the layer thickness for both raises equals the 1 meter that was designed. This assumption can be made with confidence because it it noticed from the monitoring data that each raise was approximately according to the design, and the waiting times in this phase of the KIS project were also as designed. The assumptions made for the construction of Layer 5 and 6 are used in the settlement prediction part of the model.

Another assumption made for each scenario is the independence in time between the measurements of the settlement plate. When obtaining settlement plate measurements close to each other in time it could be the case that due to ongoing deformation processes the measurements are correlated. A consequence of using Straub's reliability updating method is that with each measurement used the number of a-posteriori realizations becomes smaller. This happens because the a-posteriori realization after updating must comply to each measurement. If for one update the realization does not comply to the measurement (the realization is not located between the ranges of the measurement error) this realization is deleted from the a-priori set. In this way the number of a-posteriori realizations becomes smaller when using more measurements. It can be imagined that when using too much measurements there is no solution possible because no a-posteriori realization is located within all update ranges (if the data is in agreement of the predictions this should not be a problem due to a much smaller filter effect). Due to this model characteristic the number of measurements to obtain the a-posteriori solution is limited. However this feature introduced an advantage for the assumption that the measurements are independent from each other. The used measurements in the updating method have a period of at least 16 days between them. A period of 16 days between two measurements should be enough to justify the assumption of independence in time between the measurements, although this assumption is not necessary for using the method described in this thesis. See Appendix E for the measurements of settlement plate W27 used. From this appendix it is made clear that the monitoring planning is not carried out as initially proposed. Chapter 3 shows the theoretical planning to cover the settlement behavior as goods as possible, however in practice the measurements were done every 2 weeks and not at the specific points in time as initially proposed. This introduces a lack of information about the settlement behavior that can be prevented.

The reliability updating method filters the realizations to a smaller number with each update. As already mentioned earlier by using a large number of updates the number of a-posteriori realizations can be become zero in theory and no solution is possible. To ensure reliable results obtained with the updating method it is necessary that at least a number of 100 realizations comply to all updates. The number of a-posteriori realizations for each scenario presented in this chapter is at least 100 but often the number of a-posteriori realizations is between 200 and 300. The only exception is for the limit state function Z2 where for all scenarios no solutions are found (see Paragraph 4.4, this is because the requirement has a far larger value compared to the predicted settlements. This approach of Bayesian Updating is different than is used in D-Settlement, with the Monte Carlo Simulation the actual update is carried out by obtaining a large number of possible realizations of the settlement. As already stated before, each measurement enhances the reliability of the settlement prediction. In D-Settlement the approach is first to perform a Monte Carlo Simulation and obtain the a-priori results, than do the (deterministic) Settlement Plate Fit to fit some individual variables onto the measurements and than again execute a Monte Carlo Simulation with the updated variables. Also only five variables are updated, namely the three isotache parameters, the preconsolidation stress (POP) and the vertical permeability where the proposed model uses all individual parameters to update the prediction, see Appendix C for the modeling of all individual parameters as written in the Matlab code.

The standard deviation of the measurement error determines how strong the updating method filters out the realizations, with a large standard deviation more realizations are kept and vice versa. This is verified by the fact that more realizations fall within the range of the measurement error and are therefore not filtered out of the posterior solution. It is important to choose a good representation of the standard deviation of the measurement error, too large and the settlement prediction is not made more reliable, to small and too few (or no) realizations are left to determine the updated settlement prediction. The most important fact is that the measurement error needs to resemble reality as good as possible, in this research a standard deviation of the measurement error is set on 10 cm. The GPS device used for KIS, see Chapter 3, has a measurement error of 2 cm however the settlement plates are likely to deform under the large weight on top and due to heavy equipment. Also around the poles small heaps of clay are formed which can influence the measurement. There is epistemic uncertainty involved in the way the measurements are obtained.

As mentioned earlier the proposed model uses the Terzaghi consolidation model to calculate the development of water pressures after applying a load. The Darcy consolidation model is actually more accurate in case of reloading and unloading behavior.

The last remark is that if purely looking at the update itself, the update does not depend on time. The last measurement does not necessarily give more information than the first measurement used in the update. Both updates only take into account the prior realizations that fall within the range of the measurement error. Individually the updates contribute to the increase in reliability however this is small compared with the increase in reliability that is obtained by the use of multiple measurements working together and the Bayesian approach that the a-posteriori prediction must comply to all these measurements. However when looking at the settlement behavior in time the last measurement does give the most information about the settlement behavior because this is the latest information about the actual settlement behavior.

4.7. CONCLUSION

Chapter 4 describes the answers on the key question 4 stated in Chapter 1, the first three key questions are also implicitly answers while building the proposed model.

Key question 4:

How to improve the predicted residual settlement of the probabilistic model using monitoring data of measured settlements and considering a one dimensional stratified subsoil?

Below the answer on this question is given.

The requirements stated by the Water Board Rivierenland regarding the total settlement and residual settlement are represented by the limit state functions introduced in this chapter. To satisfy these requirements there are 16 scenarios developed which differ in having a temporary overburden or not and also in the thickness of the last clay layer that is applied during construction. All scenarios are processed with the proposed model which updates by using settlement plate information the a-priori settlement prediction to the a-posteriori prediction. Looking at the results of the scenarios it is concluded that to reduce the a-posteriori failure probability of the residual settlement requirement during lifetime of the dike structure, a temporary overburden is needed. This overburden introduces more settlement before completion, which favors the residual settlement expected during lifetime.

By comparing all failure probabilities of all scenarios, executing scenario 16 is the best way to meet the requirements stated by the client. This scenario shows that the a-priori failure probability of the total settlement prediction is strongly reduced by the reliability updating method. The measurements obtained from the settlement plates show that the actual settlement is lower that initially thought. Also the probability of failure of the residual settlement requirement during lifetime is reduced due to the use of the temporary overburden of 0,7 meter in scenario 16. The required amount of residual settlement after one year is satisfied by all scenarios with a failure probability of approximately zero. All posterior failure probabilities for scenario 16 are below 10%.

When only looking at a-priori failure probabilities, only scenario 1 will satisfy the total settlement requirements (not requirement of the residual settlement during lifetime). This scenario consumes a complete clay layer more when compared to scenario 16, here the advantage of updating the settlement prediction with monitoring data is made clear. On beforehand scenario 16 was not able to meet the requirements but with the use of monitoring data it became clear that this scenario is the most appropriate for both the contractor and client. The client has proof that the total and residual settlement will not exceed the requirement with 90% confidence and the contractor optimizes its design to probably save costs with the construction of the dike structure.

The reliability updating method of Straub combined with the a,b,c–isotache model showed that the settlement prediction indeed can be made more reliable with the use of monitoring data. The settlement plate data is in a proper way described in the model by translating the equality information to the inequality information so that it can be used in updating the prior beliefs. With a million realizations in the Monte Carlo Simulation a more reliable settlement prediction is obtained which showed that the posterior parametric distribution becomes less wide and the 90% exceedance probability shifted to lower values. When looking at the posterior confidence bounds it is seen that the monitoring data is nicely located within these bounds. This indicates that the update works good.

To compare the results of the reliability updating method modeled with Matlab, the software program D-Settlement is used. With the Reliability Analysis module in combination with the Settlement Plate Fit option also a form of Bayesian updating of the a-priori settlement prediction is carried out. The results of this analysis are shown in this chapter and it can be concluded that the proposed model is more conservative than D-Settlement. The smaller amount of settlement obtained with D-Settlement could amongst others lie in the fact that D-Settlement uses the Darcy consolidation model in D-Settlement, which is more accurate than Terzaghi as used in the proposed model. Another possible explanation of better results with the D-Settlement model is that the monitoring data has a larger influence on the a-posteriori settlement prediction due to the least square method to fit the predictions which focuses less on the tails of the parametric distributions. However the results obtained by D-Settlement lack the development of the a-posteriori settlement prediction, it does not show the influence of a measurement on the settlement predictions and could underestimate the actual settlements because of the use of the least square method. With the method presented in this thesis this is accounted for and during the construction process the designer could see which measurement affects the prediction more than others. Also how the equality information is dealt with in D-Settlement is not clear. In order to better compare the two methods the modeling of the consolidation process in the proposed model should be improved by using Darcy.

Using the direct update method the results of the a-posteriori settlement predictions are obtained in a relatively quick way and there is no need to take into account correlation changes because the method implicitly takes care of this feature. However more information about the changing parametric distributions of individual variables can be obtained when using the indirect update method. It shows which variable changes more than others, however now the correlation changes must be accounted for and for using a lot of measurements this method will take considerable time with respect to the direct update. Therefore during the construction process it is more convenient to use the direct updating method.

The reliability updating method of Straub is carried out for a 1D a,b,c–isotache model, this is a simplified representation of reality. In order to see what the implications are also a 2D calculation with D-Settlement is carried out and compared to the 1D model. It turns out that the 1D model slightly underestimates the amount of total settlement as well as the amount of residual settlement. Influence of spatial variability of pore water pressures and effective stresses play a significant role in the differences between the 1D and 2D model.

It is concluded that the reliability updating method is sensitive to measurements located on specific places on the settlement curve. When using only measurements located in the steep part of the curve (during fast raising) it turned out that the failure probabilities are larger a-posteriori than a-priori. When the same is done for measurements located in a gentle part of the settlement curve the a-posteriori failure probabilities were much lower. In order to resemble the settlement behavior as good as possible it is needed to take the measurements that together cover the total settlement behavior till the moment in time the measurements were taken. Doing this, reliable settlement predictions are obtained on basis of the data that is available. It is acknowledged that the choice of these measurements is arbitrary and depends on the experts opinion which measurements to use. When more data about the creep phase is available the predictions for the residual settlement can be made more reliable.

Also the choice of the standard deviation of the measurement error strongly influences the reliability updating method presented in this thesis. Choosing a too large value will not give more reliable results and choosing a too low value and the settlement prediction becomes unreliable due to too few realizations left for the a-posteriori predictions.

5

OPTIMIZATION NEW DIKE DESIGN

With the obtained results from Chapter 4 the variable thickness of the clay layer in phase 10 can be determined. Due to the probabilistic approach the probability of failure can be obtained for the applied thickness of the last clay layer. In this chapter the accepted failure probability (i.e. stated by the Water Board Rivierenland) is obtained and the optimal layer thickness is found. How much influence has this thickness on saving costs in the KIS project? This question is answered in this chapter.

5.1. LAST CLAY LAYER

This section elaborates on the results of Chapter 4 and evaluates what the thickness of the last clay layer should be with respect to an accepted probability of failure stated by Water Board Rivierenland. In this way the optimization of the last clay layer can be carried out.

5.1.1. ACCEPTED PROBABILITY OF FAILURE

The settlement models for the KIS project are calculated in a deterministic way, therefore there are no failure probabilities stated in the binding documents of the KIS project. In the current state of affairs the engineers of IV-Infra need to prove in some way that the residual settlement requirements are met and also that the height of the new dike at the end of the lifetime is as required. To gain this information geotechnical engineer Peter Damen of the Waterboard Rivierenland was contacted to give insight in what the accepted probability of failure could be regarding the settlement requirements. Some previous dike reinforcement projects carried out by the Water Board Rivierenland proved to have a lifetime of only 10 to 15 years instead of the designed lifetime of 50 years.

Figure 5.1: Impression of impact dike reinforcement project KIS

The lifetime of the dike reinforcement was shorter due to larger settlement of the dike than expected, insufficient geotechnical research or larger horizontal deformations than expected. The fact that the societal costs plus the large impact on the surroundings (think of villages, people but also environmental impact see Figure 5.1) the dike reinforcement needs to have a minimum lifetime of 40 years by having enough strategic reserves. In the KIS project the minimum lifetime of the dike reinforcement is, as already mentioned earlier in this thesis, set to 50 years. With the information above the Water Board Rivierenland chose an accepted probability of failure between 5% and 15%. For this research it is chosen to be not over conservative or too progressive and thus the accepted probability of failure in this thesis is set to 10%. A failure probability of 10% is common in Hydraulic Engineering.

5.1.2. LAYER THICKNESS

From Chapter 4 the results are analyzed here and the most favorable scenario for the probability of failure is chosen and explained below.

Scenario 16 can be considered the most favorable scenario regarding the accepted probability of failure. As well as for the total settlement and the residual settlement in 50 years after completion the posterior failure probability is below 10% which satisfies the accepted failure probability of the Water Board. See Table 5.1 for the results.

	Failure probability total amount of settlement
a-priori	0,6589
a-posteriori	0,068511
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,12467
a-posteriori	0,0751

Table 5.1: Results scenario 16

As can be noticed the a-priori failure probabilities are higher than the a-posteriori failure probabilities, which represents the strength of using monitoring data to update the prior belief about the expected settlement. Another remarkable feature of the scenario is that both failure probabilities of the residual settlement after 1 year are zero what means that this requirement is certainly met, according to the proposed model.

Figure 5.2 shows the expected settlement behavior in time for the a-priori and a-posteriori settlement. Also the requirement of total settlement and the monitoring data are added. The posterior settlement prediction nicely follows the monitoring data and has a smaller 90% confidence bound than the a-priori settlement prediction.

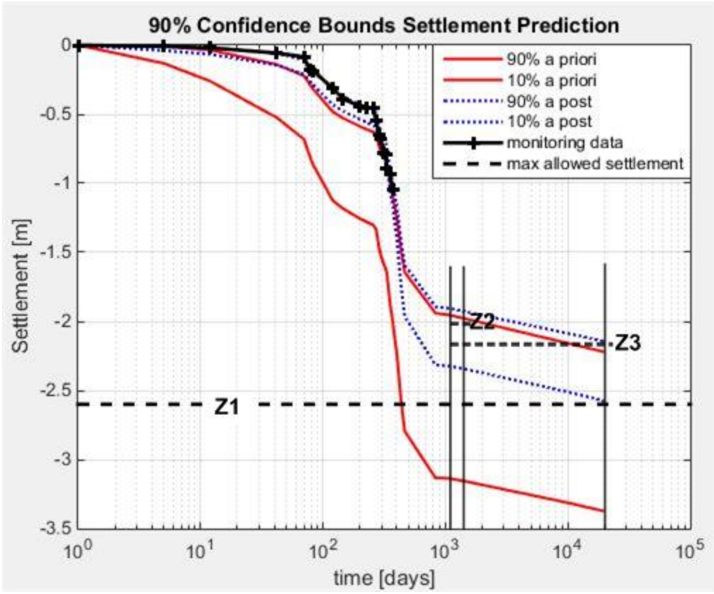


Figure 5.2: Most favorable scenario settlement prediction in time

It can be seen from Figure 5.2 that the maximum allowed settlement is just below the 90% confidence bound what indicates that a failure probability smaller than 10% is found for the a-posteriori settlement prediction. All three requirements, Z1, Z2 and Z3, are met as can be noticed from the Figure 5.2 and the optimization can be found in the fact that the maximum allowed settlement lines cross the blue 90% confidence lines. This can also be showed with the updated parametric distribution of the total settlement 50 years after completion, see the blue and black dashed lines in Figure 5.3.

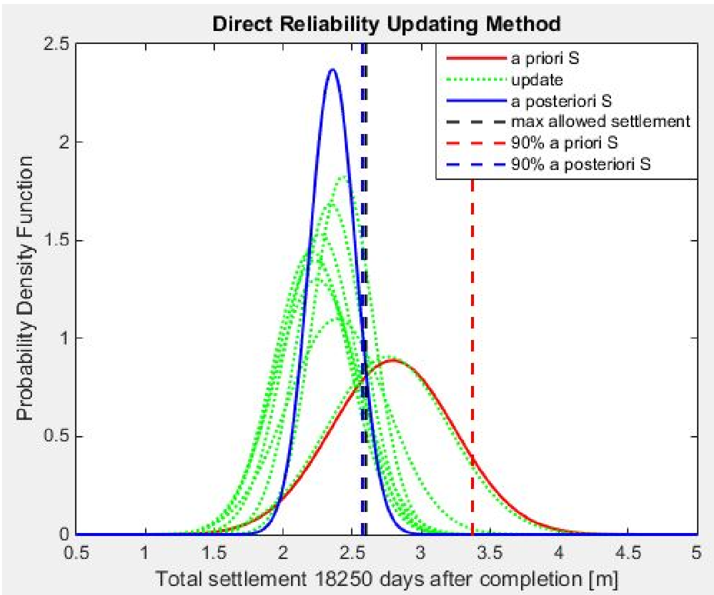


Figure 5.3: Most favorable scenario for settlement prediction 50 years after completion

The a-priori settlement was expected to be too large with respect to the requirement due to the red dashed line placed at the right of the black dashed line in Figure 5.3, but with the updates showed in green the a-posteriori distribution becomes less uncertain with respect to the a-priori distribution and also has a lower 10% probability of exceedance. With regard to the residual settlement requirements stated by the Water Board, Figures 5.4 and 5.5 are showed for the 1 year and 50 years requirement respectively.

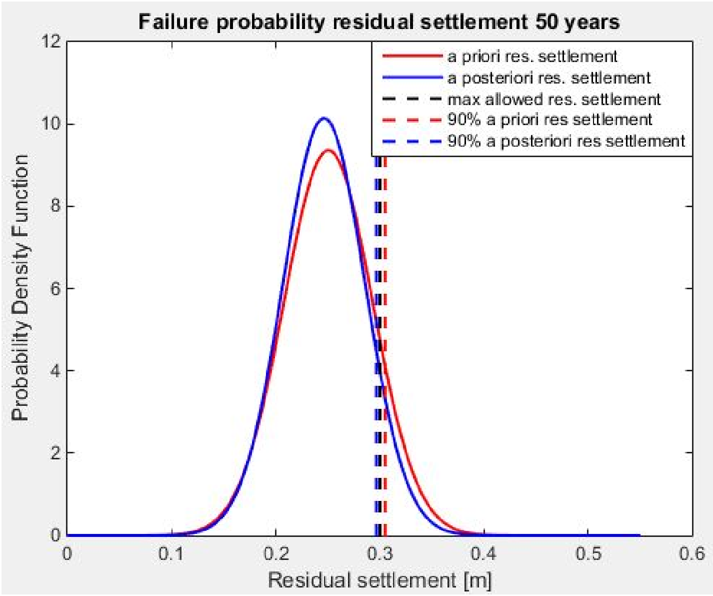


Figure 5.4: Most favorable scenario for residual settlement prediction 50 years after completion

The posterior parametric distribution of the residual settlement after fifty years has a smaller standard deviation and a 90% exceedance probability that lies lower than the prior distribution. This means that the failure probability is lower for the posterior distribution. As mentioned earlier the requirement of the 1 year residual settlement is met because of that the requirement does not cross the prior and posterior parametric distributions. What can be concluded from Figures 5.4 and 5.5 is that the residual settlement predictions become more reliable than initially thought.

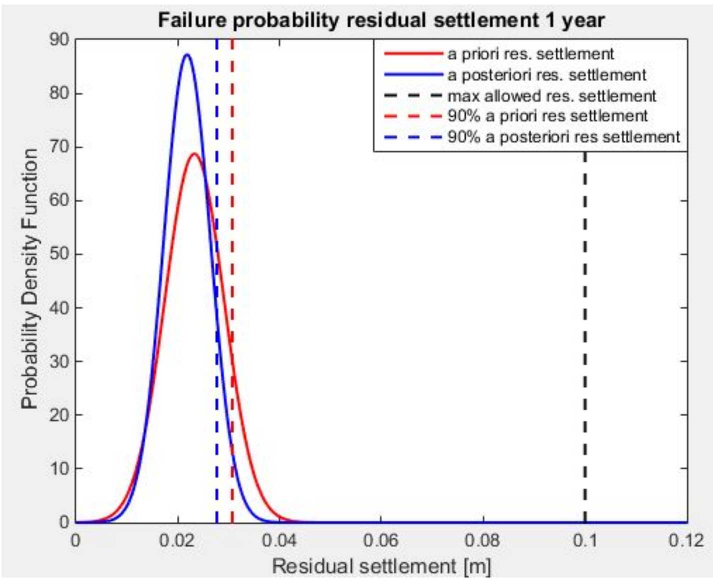
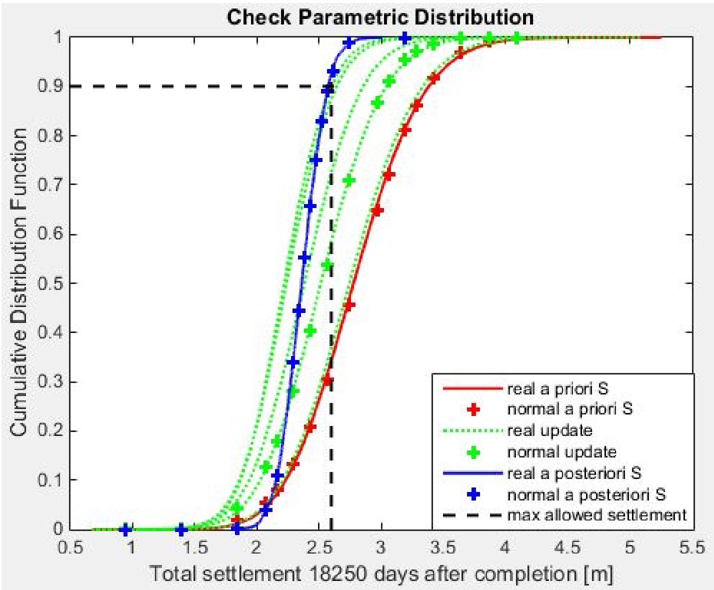


Figure 5.5: Most favorable scenario for residual settlement prediction 1 year after completion

The optimized layer thickness of the last layer therefore can be set on 0 meter (clay layer 7 is not applied in the final design) with a temporary overburden of 0,7 meter. The initial needed 7^{th} layer to meet the settlement requirements and accompanying failure probabilities is now replaced with the temporary overburden. This temporary overburden will be applied for one year which is enough to meet the settlement requirements. In this period of a year the monitoring of settlement behavior should be continued and with this new information even more reliable predictions of the residual settlement can be made because there is more information available. Scenarios are possible where a thicker layer of clay than the temporary overburden can be removed when settlement predictions are favorable, on the other hand it could also be the other way around where the layer that is removed is less thick than the temporary overburden and thus more clay is needed as initially thought.

INTERMEZZO: Check Parametric Distribution
It is assumed that the results shown in the previous Figures 5.2, 5.3, 5.4 and 5.5 are normal distributed. However due to the updates and changes in correlation (not visualized in the direct updating method), it is possible that this assumption is not true. In order to verify this assumption, the real data from the posterior analysis is plotted as a cumulative distribution function, which shows the exceedance probability on the y axis and the value of the total settlement on the x axis. This real data is compared with the normal cumulative distribution function of the data set. In the figure below the comparison between the normal cumulative distribution function (+) and the real cumulative distribution function is shown for the a-priori settlement prediction, the update and the a-posteriori settlement prediction.



It is seen that the real cumulative distribution functions comply to the normal cumulative distribution functions for the posterior as well as the prior prediction, and therefore the assumption made to show the results as normal distributions is verified. The figure above shows again that the posterior settlement prediction is more reliable than the prior settlement prediction, the blue line is steeper and thus more reliable than the red line because of a smaller standard deviation. Also the settlement requirement nicely crosses the 90% exceedance probability of the posterior prediction which indicates that scenario 16 is an optimized solution.

5.1.3. STABILITY

Scenario 13 and scenario 16 are almost the same, they differ in the fact that scenario 13 raises the last layer at once with a layer thickness of 1,7 meter of which 0,7 meter is the temporary overburden. Scenario 16 is, as already explained, executed with the temporary overburden separately. A safe choice would be scenario 16 because it does not exceed the 1 meter raise of the dike as used in the conventional design. It could be interesting choosing scenario 13 in case of the cost savings that come with it, because looking at the production costs scenario 13 should cost less than executing scenario 16. However applying a 1,7 meter raise at once could give stability problems of the new dike. This is checked and confirmed with D-Geo Stability calculations, a Factor of Safety of close to 1 is found for scenario 16 in contrast to scenario 13 where the Factor of Safety is about 0,85 (a Factor of Safety of 0,90 during construction is accepted in the KIS project). A possible situation could be that by applying 1,7 meter clay at once the loading of the subsoil is too high, which leads to exceeding the pore pressure limits. When the pore pressures in the subsoil become too high, the effective soil stresses (which resembles the strength of the dike) are low which can cause macro stability problems for the new dike. The dike could slide off and fail, therefore when applying a load on the subsoil caution is needed.

5.1.4. LIFETIME DIKE STRUCTURE

It is interesting to see what the expected lifetime of the dike structure based on the updated settlement prediction is. For scenario 16 the settlement prediction is extended to moments in time that come after the design lifetime of 50 years. Several moments are taken: for 51, 52, 55, 57, 60, 65, 70 and 75 years after completion the settlement prediction is made. A computation is made to investigate what lifetime of the dike structure can be expected for the 10% failure probability in case of scenario 16 based on the posterior prediction. The failure probability is plotted in a graph together with the years after lifetime, see Figure 5.6.

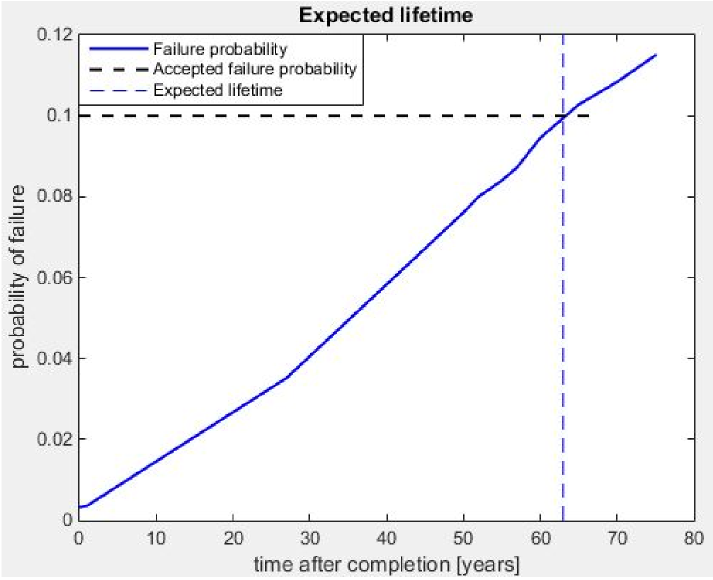


Figure 5.6: Expected lifetime after update

The expected settlement prediction crosses the requirement of 10% failure probability at about 63 years after completion. Thus the failure probability that the total settlement requirement is met at 63 years after completion is 10%. This computation is made under the assumption of unchanged conditions of the knowledge available now.

5.2. COST SAVINGS IN KIS PROJECT

Calculations for the expected save in cost are made for the 1D situation where only the results of the 1D model in Matlab are used to determine the optimized layer thickness. These cost savings are extrapolated to whole section W, however this cannot be linearly extrapolated because of the fact that soil behavior is different in a 3D situation. With the knowledge of cost savings in section W the benefits of the whole KIS project are calculated with respect to the riverward dike reinforcements.

Scenario 16 seems the most promising of all scenarios to actually carry out. When looking at the a-priori predictions it was needed to have a last clay layer of 1 meter thick (see scenario 1 in Paragraph 4.3), however with the use of the monitoring data this perception is changed and now the whole last clay layer is not needed anymore with respect to the 10% probability of exceedance of not fulfilling the settlement requirements stated by the Water Board. For both parties, the contractor as well as the client, this is beneficial. For the contractor it means a cost saving regarding the costs of the amount of clay to use, for the client it means that the a-posteriori expected settlement will only have a 10% exceeding probability of the requirements and therefore the new dike should be high enough during its lifetime. With the results from this thesis the new dike can be considered future proof (during lifetime) regarding the failure mechanisms of overflow and overtopping.

To say something about the practical use of the updating method in this thesis the financial benefits for the contractor are shown. With the help of indicators of the costs coming with the construction of one clay layer the financial benefits could be calculated, see Table 5.2. This table is based on producing 2000 m^3 clay[1].

[1] These numbers are obtained from the contractor of the KIS project, Mourik Groot-Ammers B.V., the numbers are representative for the market but are not the actual numbers of the KIS project.

Equipment	Amount of units	Price per unit	Total price
Bulldozer	8,5	€120,00	€1.020,00
Shovel	8,5	€75,00	€637,50
Hydraulic excavator	10	€85,00	€850,00
Dump trucks	36	€80,00	€2.880,00
Sheepsfoot roller	8	€85,00	€680,00
Profiling	5	€60,00	€300,00
Traffic regulator	3	€35,00	€105,00
Total			**€6.472,50**

Table 5.2: Indicators of productions cost 2000 m^3 of clay

Cost savings for the contractor can be obtained in two ways, namely applying less amount of clay or to keep the production rates as high as possible. The costs of 1 m^3 clay[2] in the KIS project is said to be around 10,50 €/m^2. Because of the magnitude of the dike reinforcement project the production rates of processing the clay are quite high, and therefore applying less clay does not have a large influence in saving costs. If the clay is not needed for section W it is used in another section. However when applying less clay there are also less production costs involved. When looking at scenario 16 it uses a temporary overburden, this means that there is approximately no financial benefit because this overburden needs to be constructed and also removed again one year later.

5.2.1. SETTLEMENT PLATE W27

This thesis showed that for a 1D model the expected settlements in time can be reduced significantly by using Bayesian Updating. From the results of Chapter 4 it can be concluded that there is one meter less clay needed as initially designed in this thesis. Below a simple calculation makes clear what this means for the financial benefits in case of 1 m^2 representing the settlement plate W27.

The reduction in production costs when applying scenario 16 can be obtained by taking 70% (0,70 meter instead of 1 meter clay) of the price per m^3 of clay.

$$\text{Estimated production costs} = \frac{€6472,50}{2000m^2} = 3,24€/m^2 \tag{5.1}$$

$$\text{Actual production costs} = 70\% \times 3,24€/m^2 = 2,27€/m^2 \tag{5.2}$$

The other way of reducing costs is the reduction of the amount of clay. Per m^2 there is one meter less clay needed what means a reduction in costs of 10,50 €/m^2. The total cost reduction is therefore:

$$\text{Total cost reduction} = (3,24€/m^2 - 2,27€/m^2) + 10,50€/m^2 = 11,47€/m^2 \tag{5.3}$$

In theory there is a total cost reduction for section W of 11,47 €/m^2 with respect to the processing of clay and the cost of clay. In the following paragraphs the consequences are calculated for whole section W and for the entire KIS project.

5.2.2. SECTION W

To translate the total cost reduction per m^2 calculated in the previous paragraph first the total surface of section W must be known:

$$\text{Surface section W} = 220m \times 20m = 4400m^2 \tag{5.4}$$

In case of scenario 16 the last last clay layer of 1 m thick is not needed to meet the settlement requirements. This has the following financial benefit:

$$\text{Estimated amount of clay} = 4400m^2 \times 1m = 4400m^3 \tag{5.5}$$

$$\text{Estimated amount of costs} = 4400m^3 \times 10,50€/m^3 = €46.200 \tag{5.6}$$

[2]This number is also obtained from the contractor Mourik Groot-Ammers B.V.

Equation 5.6 shows that initially the cost of clay was €46.200 for one clay layer in section W, however with one layer less this means an economical benefit of €46.200 for section W only on the cost for a less amount of clay needed.

Also the production costs of the clay in scenario 16 give an economical benefit, although much smaller compared to the cost of clay itself. The production costs of the clay for 4400 m^3 are showed in Table 5.3. From this the cost savings regarding the production can be calculated.

Equipment	Amount of units	Price per unit	Total price
Bulldozer	18	€120,00	€2.160,00
Shovel	18	€75,00	€1.350,00
Hydraulic excavator	22	€85,00	€1.870,00
Dump trucks	72	€80,00	€5.760,00
Sheepsfoot roller	16	€85,00	€1.360,00
Profiling	10	€60,00	€600,00
Traffic regulator	6	€35,00	€210,00
Total			**€13.310,00**

Table 5.3: Estimated production costs one clay layer section W

Table 5.3 is in case of scenario 16 not representative because the last layer is a temporary overburden of 0,7 meter. Therefore about 30% of the production costs of a full layer can be taken to calculate the production cost savings:

$$\text{Production cost savings} = 30\% \times €13.310 = €3.993,00 \tag{5.7}$$

It can be seen that the production cost savings are small compared to the cost reduction of the costs of clay itself. The total cost reduction is shown in Equation 5.8.

$$\text{Total cost reduction section W} = €46.200 + €3.993 = €50.193,00 \tag{5.8}$$

Due to the use of settlement plate information in the reliability updating method presented in this thesis, the financial benefit for the contractor is about €50.000,00 for section W. In the next paragraph an approximation is made in the case of using the settlement plate information for the entire KIS project.

5.2.3. RIVERWARD DIKE REINFORCEMENTS KIS

What does these cost reductions described in the previous paragraphs mean for the entire KIS project regarding the riverward dike reinforcements? This thesis only elaborated on section W and used this information to calculate the financial benefits. It can be imagined that not in every section of the project such favorable circumstances can be found, although it could also be the case that there are sections where even less settlement takes place than thought on beforehand. The assumption is made here that section W resembles the average cost reduction when using the reliability updating method. To know the total cost reduction for the KIS project with respect to the settlement of the riverward dike reinforcements the total surface is calculated and multiplied with the cost reduction per m^2 obtained with the calculation for section W. The total length of the riverward dike reinforcement in the KIS project is 2,5 km with approximately an averaged width of 15 meter Combinatie Dijk Verbetering Molenwaard [2013].

$$\text{Average cost reduction per meter}^2 = \frac{€50.000}{4400m^2} = 11,36€/m^2 \tag{5.9}$$

$$\text{Total surface of riverward dike reinforcements KIS} = 2500m \times 15m = 37.500m^2 \tag{5.10}$$

$$\text{Average total cost reduction KIS} = 37.500m^2 \times 11,36€/m^2 = €426.000,00 \tag{5.11}$$

The cost reduction number presented in Equation 5.11 is just an indication due to large assumptions made when extrapolating results from a 1D model to a whole project. However this number is an indication for the potential financial benefits for a dike reinforcement project like KIS. When putting the cost reductions of the settlement in perspective to the total costs of the KIS project (about €65 million), the financial benefit is more than 0,5% of the entire project budget.

However the real profit, obtained by using a probabilistic approach combined with the reliability updating method in this thesis, is that with 90% confidence can be said that the new dike is high enough to endure its design lifetime of fifty years.

5.3. INFLUENCE OF MEASUREMENTS

During the construction process the engineer can still exert some influence, the influences of the measurements during the construction process as observed during the research are discussed here.

As showed in Chapter 4 the reliability of the settlement prediction can be significantly increased relative to the initial design. With each measurement the reliability of the settlement prediction is increased. It is noticed that the choice of the measurements is important for the construction process, the measurements need to resemble the initial settlement behavior but also resemble the settlement behavior during fast raising of the dike structure. When only using measurements obtained in the steep part of the curve the predicted residual settlement is higher as the case where only measurements are used located in the gentle part of the curve. The proposed model is not able to use all measurements done for settlement plate W27 because of no posterior realizations left of the initial set of 1 million realizations. Therefore it was necessary to choose a set of measurements that resembles the total settlement behavior as good as possible. The selection of measurements was done in a way that representative behavior of the settlement is covered, for instance taking the measurement where the behavior changed from creep phase to the consolidation phase again when loading the subsoil.

Without the temporary overburden the failure probabilities of the residual settlement requirements where not met although the measurements show that the predicted settlements are not as large as thought on beforehand. Apparently results from the reliability updating method of Straub in this case are realizations where the settlements during construction are relatively low and residual settlements (after completion) that are high. This means that the uncertainties in predicting residual settlement are too high. A reason is that there are not enough measurements done yet for which the behavior of the soil is in the creep phase. This is again exactly the problem that is stated in Chapter 1 that within a relative small period of time settlement predictions must be made for relative large timescales. The monitoring data that is used in this thesis is located alternately in the direct strain (consolidation), secular strain and creep strain part of the settlement curve. The uncertainties in residual settlement therefore remain in the a-posteriori predictions. To avoid this situation it is proposed to use the common solution of a temporary overburden, which introduces larger settlement before completion and less residual settlement as can be seen from the results in Chapter 4. It is recommended that during the period of the temporary overburden the monitoring of settlement plates continues, the data will then show the settlement behavior of the creep part of the curve. By using this information the final a-posteriori settlement prediction can be made.

5.4. DISCUSSION

In the KIS project the approach followed to prove that the requirements stated by the Water Board Rivierenland are met is based on a deterministic approach. In this thesis the settlement model is full probabilistic and with the Monte Carlo Simulation failure probabilities can be obtained. It can be used as a tool for the designer to prove to the Water Board that the settlement requirements are met with a certain amount of exceedance probability.

The results about the amount of total and residual settlement have a probability of exceedance of 10%. Water Board Rivierenland was asked to give a desired failure probability in case the KIS project was calculated in a probabilistic way (as in this research). The amount of settlement that is used to determine the optimized layer thickness and optimized costs is therefore taken with a failure probability of maximal 10%. This is a different approach than common in the design of dikes where in general the mean of the total amount of settlement is chosen. However it gives the Water Board the benefit of a certain reliability that the dike reinforcement project actually endures the design lifetime with the assumption that changing conditions in time are incorporated in the requirements. A dike reinforcement project with the size as the KIS project has large consequences on the society, such as houses need to be bought from people, a long time of disturbance for people living close to the project, and environmental issues. The Water Board Rivierenland must be ensured therefore that the requirements that are stated by this institution are met, and with the approach in this thesis it can be proven to the Water Board that the requirements are met with a probability of exceedance of 10%.

Scenario 16 is chosen as the best scenario to execute for section W. With a temporary overburden and completely leaving out the 7^{th} clay layer in the construction, the a-posteriori failure probabilities are smaller than 10%. The reliability updating method does not give satisfying results when the temporary overburden is left out of the design, this probably has to do with the lack of settlement plate information relative to the phase where the settlement of the dike is in. The influence of the updates on the residual settlement prediction is therefore not well represented when using the available monitoring data. In order to meet the requirements the solution in this stage of the project is to apply a temporary overburden into the design, however maybe with monitoring data obtained in a later phase of the project (in any case before completion) the requirements are met without application of the temporary overburden.

By using the reliability updating method the a-posteriori settlement predictions show a more reliable and less amount of settlement prediction as compared with the a-priori predictions. This has its consequences of the expected costs of the construction of the new dike. When looking at the scenarios it was initially thought that the last layer had to be 1 meter thick, by using the settlement plate information this thickness is converted into a temporary overburden with a thickness of 0,7 meter and per m^2 1 meter less clay needed. A reduction in costs is now obtained by the posterior belief about the total expected settlement.

In the previous paragraph the total cost savings for the KIS project are calculated, however this amount of reduction in cost is obtained by the results obtained from a 1D model. It is a large assumption that the reduction in the amount of clay can represent the total clay reduction of all riverward dike reinforcements of the KIS project. This number is therefore indicative for the potential economic benefits of optimizing construction stages with monitoring data. Though it is clear that the potential in economic benefit is large in case of settlement predictions, the method presented in this thesis could also be used in other areas of engineering what have to do with settlement predictions (e.g. road construction).

As noticed in the part where the influence of the measurements are discussed, the chosen measurements to use in the updating method is of great importance. It gives different results when only using measurements located in one part of the settlement curve or using measurements that cover the total settlement behavior. The method and thus the results are therefore quite sensitive to the measurements. Eventually in this thesis 10 measurements are used to update the settlement predictions, these 10 measurements are located on specific points in time that resemble the total settlement behavior as good as possible and also the last measurement is used. The last measurement is the most valuable measurement because it is located in the last known part of the settlement curve and therefore gives the most information about the expected settlements that still need to take place.

5.5. CONCLUSION

Chapter 5 describes the answers on the key questions 5 and 6 stated in the introduction:

Key question 5:

What is an acceptable thickness of the last clay layer of the dike reinforcement in the KIS project, regarding the accepted probability of failure, and what is the financial benefit of this optimization?

Key question 6:

What aspects of the updating method are important during the construction process and how to use this knowledge in the construction process?

Below the answers on these questions are given.

Water Board Rivierenland did not demanded a probabilistic approach or a certain failure probability on which the dike reinforcement could be designed for the KIS project, instead of a full probabilistic approach a deterministic approach is followed in which the consequences of not meeting the settlement requirements are for the Water Board. The approach in this thesis is probabilistic and therefore to come up with a design of the last clay layer of the dike reinforcement a certain failure probability was needed. After contact with the Water Board it was stated that a failure probability of the settlement requirements of 5% to 15% was accepted. To be not too conservative or too progressive an accepted failure probability of 10% was chosen for which the most favorable scenario must comply. Designing on this failure probability satisfies the Water Board Rivierenland and is common in Hydraulic Engineering.

The best scenario had to comply to all three settlement requirements, the 1 and 50 years residual settlement requirement and the total settlement requirement. The optimized last clay layer thickness was found in scenario 16 which uses a temporary overburden of 0,7 meter separately constructed from the 6^{th} clay layer. The 7^{th} clay layer that was initially needed to satisfy the requirements is completely left out of the design due to the use of the reliability updating method. A reduction in the amount of needed clay is found to be as high as 1 meter in the 1D situation with respect to the initial design.

This chapter has shown the potential cost savings of the KIS project. These are obtained by the a-posteriori settlement prediction of scenario 16 which saves 1 meter of clay per m^2. The profit that the contractor could obtain is set on €11,47 per m^2, this means a total profit of section W as high as €50.000. However it gets interesting when looking at the whole KIS project, by extrapolating the benefit of section W to the total surface of riverward dike reinforcements the potential financial benefit for the contractor is found to be around €426.000 which is more than 0,5% of the total project budget. The reliability updating method used in this research showed that potentially large economical benefits can be obtained.

Scenario 13 and 16 are very similar, however in scenario 13 the 6^{th} clay layer of 1 meter and temporary overburden of 0,7 meter are constructed at once where in scenario 16 they are separately constructed. Looking at the stability of the dike structure during construction the Factor of Safety for scenario 13 is considered to be too low and in order to be safe and having less risks of macro instability of the dike structure scenario 16 is chosen. Looking at the costs savings between the two scenarios, scenario 13 is giving slightly better results however its not worth the risk of macro instability.

Advantages for the contractor to use the reliability updating method are made clear, however for the Water Board Rivierenland there are also large benefits with the approach of the settlement requirements in this thesis. The designer now has a tool to increase the reliability of meeting the settlement requirements and therefore ensures the Water Board to an exceedance probability of at most 10% of not meeting the requirements during the lifetime of the dike structure. With respect to the two failure mechanisms of the dike structure, overflow and overtopping and macro stability, the dike reinforcement should endure its lifetime of 50 years with a failure probability of 10%. Using the deterministic approach optimization of the dike reinforcement is probably lower and no exceedance probabilities can be given as evidence of meeting the requirements of the

Water Board. In a worst case scenario this can lead to needing a new dike reinforcement already after 10 or 15 years after completion of the KIS project which has large societal and financial impacts.

The a-posteriori settlement prediction obtained by the use of Straub's method in combination with the a,b,c–isotache model greatly depends on the used measurements to make the update. Because of the different phases (consolidation phase, creep phase) in which the subsoil can be in under loading conditions, the selection of the measurements determine the final results. In order to get a reliable posterior settlement prediction from the total settlement behavior till the moment of the last measurement taken, it is necessary to use the measurements that cover the different phases of the subsoil behavior. A drawback of the direct updating method in combination with the Monte Carlo Simulation is that the amount of measurements that can be used to update the prediction is limited due to the filtering of prior realizations to a diminishing number of posterior realizations with each update.

6

Conclusion & Recommendation

Chapter 6 goes into the conclusions that can be drawn from the research that is carried out of updating settlement predictions. During this research it is noticed that settlement plate information is successfully used to improve the settlement predictions and thereby optimize the design of a dike reinforcement. However some improvements can be made in the proposed model to get even better results of increasing the reliability of settlement predictions. These findings are stated as recommendations about long term (residual) settlement prediction.

6.1. Conclusion

Chapters 2 to 5 showed the answers on the key questions as defined in Chapter 1. With the knowledge obtained from doing research into increasing the reliability of settlement predictions, the following research questions stated in Chapter 1 can now be answered:

1. ***How to make a more reliable prediction for total and residual settlements of a dike reinforcement using settlement plate monitoring data in a one dimensional model?***
2. ***And how to use this information in the construction process?***

The main reason to define these questions is that predictions on (residual) settlement are quite unreliable because of the relative short period of time, these predictions need to be made with respect to the lifetime of the dike structure. The bandwidth of the predictions is large due to several types of uncertainties, for instance due to a lack of information about the input parameters of the models, or a lack of knowledge of the physical processes in the subsoil (epistemic uncertainties). By using settlement plate information the epistemic uncertainties found in the lack of data can be reduced. The consequence of this reduction in uncertainty is that the settlement prediction can become more reliable as initially thought. Another reason of increasing the reliability of settlement predictions lies in the fact that potentially economical benefits can be obtained by the client and contractor of a dike reinforcement project. Due to a conservative approach in the design phase of the project the reinforcements are likely to be overdimensioned and with the use of information about the actual settlement behavior of the subsoil the dike structure can be optimized.

The actual optimization of the dike reinforcement design is made possible by the proposed model in this thesis. By using the combination of the a,b,c—isotache model and the Bayesian Updating method based on Straub in a probabilistic approach, the (residual) settlement predictions are made more reliable with the help of settlement plate information. As a result the dike reinforcement is corrected for these updated predictions into an optimized solution which is more reliable and saves costs.

Water Board Rivierenland stated three requirements for the total and residual settlement during lifetime of the dike structure. Using the software program Matlab it was possible to combine the methods and models in order to get the a-posteriori settlement predictions. By using the direct updating method introduced by Straub the obtained settlement plate information was used to update the a-priori settlement predictions of the one dimensional a,b,c–isotache model. The monitoring data was used to describe the actual behavior of the subsoil under loading conditions and with the measurements the reliability of the residual and total settlement predictions was increased. The three requirements are translated into limit state functions and with the Monte Carlo Simulation the failure probabilities are obtained with respect to the requirements. When looking at the results of the settlement predictions on the requirement that the residual settlement 1 year after completion must not exceed 0,1 meter, it is concluded that this requirement is easily met by all scenarios. However this requirement stays important in order to prevent too large residual settlements for the road structure on top of the dike (there is no road on top of the dike in case for section W).

For searching the optimized solution on basis of the gathered settlement plate information, several scenarios are conceived to obtain the optimal solution in an iterative way. Because of the reason that a less thick last clay layer induces less settlement, it was necessary to think of these scenarios. Choosing the best scenario was done on the basis of the accepted probability of failure as was set by the Water Board Rivierenland. Eventually the exceedance probability of not meeting the settlement requirements during lifetime was set on 10%. With this number the best scenario was chosen that satisfied the failure probability, this turned out to be scenario 16. This scenario does not need the last clay layer but can meet the requirements with a temporary overburden of 0,7 meter which is applied for one year and removed before completion. Regarding the macro stability of the dike structure during construction it was chosen to be safe and not construct the 6^{th} clay layer and temporary overburden at once. This would cost slightly more however when looking at the cost in case of sliding of the new dike, these extra costs are negligible. Scenario 16 fulfills settlement requirements with a probability of failure of 10% and gives the riverward dike reinforcement (in fact a whole new dike) enough strength to withstand the two failure mechanisms that are responsible for the KIS project, namely overtopping and overflow of the dike and macro instability.

Some remarks are made with respect to the results obtained by the model to update the settlement predictions. By implementing the correlation coefficients between a, b and c of the isotache model it became clear that these coefficients increase the bandwidth of the settlement predictions, thus it is necessary to know the correlation coefficients between the soil parameters for reliable predictions. Also it can be concluded that the model is very sensitive for the selection of the measurements that are used to update the a-priori prediction. Besides that not all measurements that are available could be used because of the filter effect of the direct update method, it is also important to resemble the total settlement behavior obtained with the settlement plate information. The selection of measurements must represent specific points on the settlement curve. In order to predict better the residual settlement it is concluded that monitoring of the settlement must continue during the period before removing the temporary overburden, to obtain as much as possible information to predict the final posterior settlement behavior. Due to the lack of information in this phase of the project and the alternately settlement behavior (consolidation phase alternated with creep phase) it is not yet possible to give more reliable results on residual settlements without the use of a temporary overburden. However these two features of the model are related, the temporary overburden is already needed before a reliable prediction can be made about the expected residual settlement. Again it is showed that the process of determining a reliable long term prediction of the expected settlement is difficult to achieve in the relative short period that is available to do those predictions.

Another important sensitivity in the proposed model is the measurement error that is used in the updating method, choosing it too low and the update method does not work and choosing it too high the range of the measurement error becomes to small and after a couple of updates no realizations are left. The most important aspect is that the standard deviation of the measurement error approximates the actual made error. The standard deviation used in this research was set on 10 cm because of uncertainties in the GPS device which does the actual measurements, the human error made while placing this device and also the circumstances in which the measurements are done (heaps of clay around the settlement plate poles and deformation of the pole due to the weight of the construction).

Verifying the method and models that are combined in Matlab was needed to trust the results from the model. First, a relatively easy one dimensional loading of the subsoil was modeled in Matlab and the same was done in D-Settlement, an accepted software program in the field of Hydraulic Engineering, in order to verify the initial results of the proposed model. After this was done the actual execution of the research could take place and it was seen that the proposed model gives similar results as D-Settlement, although the results from D-Settlement are somewhat better. This is mainly because the least square method used in D-Settlement which concentrates around the mean and considers less the extreme values in the tail of the parametric distribution in contrast to the proposed model. Also the type of consolidation model used in this research is an important difference, instead of using the better Darcy consolidation model the Terzaghi model was used. The updating method in D-Settlement shows more influence of the measurements in the posterior settlement prediction, due to the different approach of Bayesian Updating. The proposed model is fully probabilistic as well as obtaining the posterior realizations. The prior settlement predictions are obtained with the Monte Carlo Simulation in D-Settlement, and for the posterior prediction D-Settlement uses the deterministic least square method to fit the settlement prediction on to the settlement plate information and with the changed variables a Monte Carlo Simulation is carried out.

An advantage of the approach to update the settlement predictions in this thesis with respect to the update method in D-Settlement is that it does give information about how much each measurement influences the posterior probability density function. It can be made clear how the probability density function changes in time until the update with the last measurement. The designer could use this information to make choices about the last phase of the construction of the new dike. In case that the posterior probability density function moves in beneficial direction from the prior probability density function the designer could even more optimize its design, and vice versa it means that the designer must be cautious and build in enough safety to meet the settlement requirements.

It is concluded that updating the settlement prediction can potentially save a lot of costs. The model used in this thesis is one dimensional and represents only one settlement plate, however the reduction in the amount of clay is significant with only the use of information of one settlement plate. Especially savings in amount of clay contribute to the financial benefits of the contractor. It is showed that with the assumption that the investigated section W resembles the average section of the whole KIS project that the potential economical benefits could be as high as 0,5% of the total project budget of KIS. As already stated in the introduction, due to the elongated profiles of dike reinforcement project the potential economical benefits for the contractor can be large when the reliability of settlement predictions is increased. However the above explanation holds only if the actual settlement is less than initially thought during the construction period. Another reason for the saving in costs found in this thesis for the KIS project, is that the contractor only has to prove that the settlement requirements are satisfied. Because when the requirements are not met on long term conditions the financial consequences are for Water Board Rivierenland, and thus the risk costs in the case of the KIS project are for the client and not for the contractor.

On the other side, approaching the settlement requirements in a probabilistic way the Water Board Rivierenland as client obtains a dike structure that meets the settlement requirements with a probability of failure of 10% during the lifetime of the dike structure. In a couple dike reinforcement projects in the past it was shown that after 10 or 15 years the dike did not meet the requirements anymore with respect to the amount of settlement and thus the height of the dike. A dike reinforcement project greatly impacts the surroundings, and with the probabilistic approach combined with the direct update method in this thesis there are also societal and financial benefits for the Water Board Rivierenland.

Extrapolating the results of the case study done for the KIS project to other dike reinforcement projects in the Netherlands, this thesis showed that a lot of profit can be obtained for both contractors as clients by using the monitoring information about the subsoil and the probabilistic approach combined with the updating method presented in this thesis. In nearby future the procedures of the dike safety assessment are changed, and possible many dike reinforcement projects need to be undertaken to satisfy this new approach which is described in WTI 2017 (in Dutch: Wettelijk Toets Instrumentarium 2017). Therefore there are large opportunities for contractors and clients to optimize their designs with the method presented in this research.

6.2. RECOMMENDATIONS

After doing research on how to increase the reliability of settlement predictions there are several recommendations to make that resulted from this research. Below these recommendations are described to give a clear picture on which parts of the reliability updating method presented in this thesis could be improved in order to obtain even more reliable settlement predictions and which topics are interesting to investigate.

When looking at the basis of the updating method, the input data of the variables, it is noticed that there is no research done to the parametric distributions of the soil variables. With data sets of for instance the a,b and c isotache parameters the parametric distributions can be obtained and therefore more accurate modeling can be done and results obtained. In the present day it is assumed that all input parameters have a normal or lognormal distribution although this is unlikely to be true. Another recommendation is to investigate the correlation coefficient between the soil parameters further because of the influence on the settlement predictions. In practice often the assumption is made that computations with uncorrelated variables are conservative, however from the analysis made in this thesis it became clear that this is not always true and thus needs more investigation.

In order to increase the reliability of the settlement predictions it is recommended to do as much soil investigation as possible within the project budget before starting the construction of the dike reinforcement. The designer can use this information already to adjust its preliminary design in order to save time and money during the construction process.

To improve the proposed model it is recommended to replace the Terzaghi consolidation model with the Darcy consolidation model which gives more accurate results in case of unloading and reloading of the subsoil. This is important for the updating model because of the short time span where the measurements are done, these measurements are also done during the unloading and reloading of the subsoil and therefore it is recommended to use Darcy as consolidation model.

Another limitation of the proposed model used in this thesis is that the settlement model is one dimensional. Settlement behavior can better be represented when modeling in two or even three dimensions due to spatial variability of the soil characteristics. It is recommended to model the settlement in 2D space in order to better represent the stress distribution in the subsoil, and in order to model the horizontal deformations in the subsoil a finite element model is recommended. This means for the updating method that is also should be changed because in this thesis it is in line with the 1D settlement model by using the information of one settlement plate.

Using more measurements is another recommendation that can be made, this recommendation can be seen in several ways. First the updating model needs to be improved in speed and strength to be able to use all measurements that are obtained by the settlement plate and secondly it is recommended to continue with the monitoring during the period of the temporary overburden. With this information even more reliable predictions can be made because the subsoil will be in the creep phase. On the basis of this information the final settlement prediction could be given and the optimization of the design can be even improved further.

More types of monitoring data were obtained in the KIS project, for instance the water pressures in the subsoil are monitored for several layers. This information is valuable in order to see how the actual consolidation of the subsoil behaves. It can be imagined that this information could also be used to support the increase of the reliability of settlement predictions. Another interesting field of research is the impact of the horizontal deformation of the subsoil on adjacent buildings in a dike reinforcement project like KIS.

In this research there is no sensitivity analysis done on the individual input variables for the a,b,c–isotache model on the updating method. To update the settlement predictions the direct updating method of Straub is used, however there is a similar method that indirectly updates the prediction. This indirect updating method gives more insight in the change of parametric distributions of the individual parameters, and from this information it can be seen which stochastic parameter changes more than other parameters. When using the indirect updating method some caution must be taken with respect to the introduction and change of the correlations with each update. The method could give interesting information about which stochastic parameters play an important role during the updating of the settlement prediction and from the results the steering of the monitoring network or the whole project could be improved.

Other sampling techniques like for instance Importance Sampling can be used to improve the update method and make the model more efficient by sampling from a distribution that overweights the important region, like the tail of the parametric distributions (small probability, high consequence events). The plain Monte Carlo Simulation used in this thesis does not give extra weight to these important regions.

Furthermore it is recommended to Water Boards in the Netherlands to execute the settlement prediction for dike reinforcement projects by using a probabilistic approach. The settlement prediction in the KIS project is carried out deterministic which does give certain benefits but also drawbacks. The main advantage of the probabilistic approach used in this thesis is that the designer could say something about the reliability of the settlement predictions, by using deterministic methods it is difficult to prove that the predictions made are satisfying the requirements of the Water Boards and the predictions made are therefore less reliable. Also the high consequence, low probability events are better represented by a using the probabilistic approach.

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A

EQUALITY INFORMATION VS BAYESIAN UPDATING

This appendix explains how to deal with equality information in Bayesian Updating. Information about a system can be obtained by measurements, monitoring or observations of the system behaviour. This information can be used to update the system reliability, for instance by using the Bayesian Updating method. Obtained information can be expressed in an inequality type (e.g. the observation that the settlement is not higher than a certain value) and equality type (e.g. monitoring data about settlements). In this research the information is of the equality type and by definition the a-priori probability of equality information is zero. For example, what is the a-priori probability that the settlement of the new dike is **exactly** 0,2324 m? That would be zero in the field of probabilistic design. Therefore there is a need to transform the equality information into inequality information, in a way that this valuable information can be used in reliability analysis methods to update predictions. In the paper of Straub [2011] this transformation of information is explained. Below the derivation of this transformation method is explained.

In an engineering project information ζ is obtained by monitoring and ζ is thus of the equality type. In Bayesian analysis the effect of information ζ on the uncertain parameter Θ is expressed through the likelihood function, see equation A.1.

$$L(\theta) = P(\zeta|\Theta = \theta) \tag{A.1}$$

Another rule in Bayesian analysis is that the posterior probability is proportional to the likelihood times the a-priori probability, see equation A.2.

$$P(\theta|\mathbf{x}) \propto L(\theta|\mathbf{x}) \times P(\mathbf{x}|\theta) \tag{A.2}$$

Straub combines these two properties in his method and starts by partitioning the random variables $\mathbf{X}$ in two sets: $\mathbf{X}_g$ and $\mathbf{X}_h$. $\mathbf{X}_h$ contains the random variables in $\mathbf{X}$ that only appear in $h(\mathbf{x})$ (the observation function), a typical example for an element of $\mathbf{X}_h$ is a random variable describing measurement uncertainty. $\mathbf{X}_g$ contain the remaining variables in $\mathbf{X}$, these are for instance the monitoring data. If $\mathbf{X}_h$ is a scalar variable, the likelihood function (in general: $L(\mathbf{x}|\varepsilon) = P(\varepsilon|\mathbf{X} = \mathbf{x})$) can be written as A.3:

$$L(\mathbf{x}_g) \propto \sum_{j=1}^{n_h} f_{x_h}[\hat{x}_{h,j}(\mathbf{x}_g)] \tag{A.3}$$

where $\hat{x}_{h,j}$ are the n_h roots of $h(x_h, \mathbf{x}_g)$ for given $\mathbf{x}_g$. In this thesis the likelihood simplifies because X_h represents a measurement error e_m, see A.4:

$$L(\mathbf{x}_g) = f_{e_m}[s_m - s(\mathbf{x}_g)] \tag{A.4}$$

where s_m is the measured value of the settlement $s(\mathbf{x})$ and f_{e_m} is the probability density function of the measurement error.

Now it is necessary to express the equality information ζ by the likelihood function $L(\mathbf{x}_g)$ in the form of equation A.3, in the way that the information can be described as inequality information. Straub notes that the following identity holds for any likelihood function $L(\mathbf{x}_g)$, see equation A.5:

$$L(\mathbf{x}_g) = \frac{1}{c} P\left\{U - \Phi^{-1}[cL(\mathbf{x}_g)] \leq 0\right\} \tag{A.5}$$

here U is a standard Normal variable, Φ^{-1} is the inverse standard Normal cumulative distribution function and c is a positive constant to ensure that $0 \leq cL(\mathbf{x}_g) \leq 1$ for all $\mathbf{x}_g$ (the value for the inverse cumulative distribution function must be between 0 and 1). Equation A.5 enables expressing the likelihood function by the equivalent observation limit state function, see equation A.6.

$$h_e(\mathbf{x}_g, u) = u - \Phi^{-1}\left[cL(\mathbf{x}_g)\right] \tag{A.6}$$

With the use of the standard Normal variable U the outcomes of $h_e(\mathbf{x}_g, u)$ are in the failure domain of the observation. From the new generated data set the mean and standard deviation of an model input parameter can be obtained. This is done for example by taking each value of parameter a of the a,b,c–isotache model where $a(h_e(\mathbf{x}_g, u) < 0$ and then calculating the mean and standard deviation of this set of outcomes. In this way the distribution of the model input parameter is updated with the use of equality information ζ.

To calculate the updated conditional probability of failure we need the corresponding inequality domain of the observation limit state function $h_e(\mathbf{x}_g, u)$, this is where the observation limit state function is smaller than zero, see A.7:

$$\varepsilon_e \equiv \left\{h_e(\mathbf{x}_g, u) \leq 0\right\} \tag{A.7}$$

And then applying Bayes' rule, A.8:

$$P(F|\varepsilon) = \frac{P(F \cap \varepsilon)}{P(\varepsilon)} = \frac{P(Z(\mathbf{X}) < 0 \cap h_e(\mathbf{x}_g, u) < 0)}{P(h_e(\mathbf{x}_g, u) < 0)} \tag{A.8}$$

This means that the updated conditional probability of failure given the failure domain ε equals the probability that $Z \leq 0$ and $h_e \leq 0$ divided by the probability that $h_e \leq 0$.

B

Bayesian Updating

The next part is based on the paper of Schweckendiek Schweckendiek et al. [2014] in which the Bayesian Updating method is explained by using inequality information and the direct method.

The theory of Schweckendiek is a basis on how to approach the Bayesian Updating in a straightforward way, in this thesis this basis is used and elaborated on further.

Reliability updating with inequality information Schweckendiek et al. [2014]

Prior analysis

Let the failure event F be defined as the limit state function Z assuming negative values. The probability of failure is then given by equation B.1:

$$P(F) = P(Z(\mathbf{X}) < 0) = \int f_{\mathbf{X}}(x)d\mathbf{x} \tag{B.1}$$

Where $\mathbf{X}$ is the vector of random variables and $f_{\mathbf{X}}(x)d\mathbf{x}$ its (prior) probability density function (PDF). Some ground-related uncertainties, in particular the stratification of the subsoil, are modeled as scenarios E_i. The joint PDF of the random variables can depend on the scenario. The total probability of failure over all (mutually exclusive) scenarios is given by equation B.2:

$$P(F) = \sum_i P(F|E_i)(PE_i) \tag{B.2}$$

where the set of scenarios is complete (i.e. $\sum_i P(E_i) = 1$).

Fragility curves

Fragility curves are representations of the aggregated resistance $\mathbf{R}$ with respect to a dominant load variable S (i.e. $\mathbf{R}$ contains all variables in $\mathbf{X}$, except S), providing the conditional probability of failure with respect to S:

$$P(F|\{S = s\}) = \int_{Z(\mathbf{R},s<0)} f_{\mathbf{R}}(\mathbf{r})d\mathbf{r} = F_{\mathbf{R}}(s) \tag{B.3}$$

Posterior analysis

The information used is of the inequality type where the observation ε is described in terms of the exceedance of an observational limit state expressed in terms of an observational limit state function h:

$$\varepsilon \equiv h(\mathbf{X} < 0) \tag{B.4}$$

Direct reliability updating

Direct updating exploits the definition of the conditional probability, which is expressed below:

$$P(F|\varepsilon) = \frac{P(F \cap \varepsilon)}{P(\varepsilon)} \tag{B.5}$$

$$= \frac{1}{P(\varepsilon)} \int_{Z(\mathbf{R},s<0)} P(\varepsilon \cap \mathbf{x}) f_{\mathbf{X}}(\mathbf{x})d\mathbf{x} \tag{B.6}$$

Another way of expressing the posterior probability of failure is:

$$P(F|\varepsilon) = \frac{P(Z(\mathbf{X}) < 0 \cap h(\mathbf{X}) < 0)}{P(h(\mathbf{X} < 0)} \tag{B.7}$$

which implies looking for the probability that failure (in the future) and the observation (in the past) hold at the same time, divided by the probability of the observation (based on prior probabilities).

C

Matlab Script

Appendix C shows the scripts used for the Matlab model. There are several scripts constructed to gain the results of the settlement predictions. These scripts work together, and are chronological presented in this appendix. Note that for most scripts only parts are shown.

Layer structure

```
%% This function resembles the subsoil in section W, it consists of several
% layers of clay peat and sand.
%  The isotachen parameters are the same for the different sorts of clay,
%  as well as for peat and sand. The same holds for the weight gamma, POP values}
%and abc power.

% FUNCTION LAYER STRUCTURE SECTION W
function [b_sand,b_clay,...,gammaw] = layer_structure_section_W_corr (n)

%% Correlations a,b,c peat
A = dataset('File','corrpeat.txt');
B = dataset('File','corrclay.txt');

%% make data workable
A = dataset2cell(A);
B = dataset2cell(B);
A(any(cellfun(@(x) any(isnan(x)),A),2),:) = [];
B(any(cellfun(@(x) any(isnan(x)),B),2),:) = [];

ap = cell2dataset(A(:,2)); ap = double(ap);
bp = cell2dataset(A(:,3)); bp = double(bp);
cp = cell2dataset(A(:,4)); cp = double(cp);

ac = cell2dataset(B(:,2)); ac = double(ac);
bc = cell2dataset(B(:,3)); bc = double(bc);
cc = cell2dataset(B(:,4)); cc = double(cc);


data_peat=[ap bp cp];
SIGMA_peat=diag(cov(data_peat))';
MU_peat=[0.0348 0.313 0.0209];
rabc_peat = mvnrnd(MU_peat,SIGMA_peat,n);

data_clay=[ac bc cc];
```

```
SIGMA_clay=diag(cov(data_clay))';
MU_clay=[0.0118 0.141 0.0071];
rabc_clay = mvnrnd(MU_clay,SIGMA_clay,n);

% abc values peat and clay
a_peat = rabc_peat(:,1);
b_peat = rabc_peat(:,2);
c_peat = rabc_peat(:,3);

a_clay = rabc_clay(:,1);
b_clay = rabc_clay(:,2);
c_clay = rabc_clay(:,3);

 % Layer 4 Peat
 D_peat1 = normrnd(1.05,0.2*1.05,1,n);

 % Layer 5 Soft Clay
 D_soclay = normrnd(0.52,0.3*0.52,1,n);
 gammasoc = normrnd(13.5,0.1*13.5,1,n);

 % Layer 6 Peat
 D_peat2 = normrnd(0.7,0.3*0.7,1,n);

  % Layer 8 Peat
 D_peat3 = normrnd(0.4,0.3*0.4,1,n);

 % Layer 9 Sand
 D_sand = normrnd(2.6,0.3*2.6,1,n);
 gammas = normrnd(20.3,0.1*20.3,1,n);
 a_sand = normrnd(5e-4,0.2*5e-4,1,n);
 b_sand = normrnd(5.8e-3,0.2*5.8e-3,1,n);
 c_sand = normrnd(2e-4,0.2*2e-4,1,n);
 abc_sand = (b_sand-a_sand)./c_sand;

 cv = 1e-4; % weighted consolidation coefficient for whole subsoil
 gammaw= 9.81;

end
```

A,B,C—ISOTACHE MODEL

This is the script of one randomly chosen scenario to calculate the settlements. Each scenario has its own script for the a,b,c—isotache model. It is used as a function that can be called from a run file, see the next script.

```
function [S_total] = KIS_apriori_1(..)
% function with soil improvement, sand layer of 2 m thick which is placed
% where very soft material is removed.
% function is computed two times, for each soil layer at 0.25 D and 0.75 D
% to gain better results the sum is taken of these two calculations.
% settlements are calculated at 0.5 D.

%% Input random variables
D_loading_opt = normrnd(1,0.005,1,n);


D_total = D_improve + D_clay + ... + D_peat3 + D_sand;
%% Time variables
```

```
t=[1 5 1 ... 1065 1430 10000 19315]; %number of days
tau0=1;                                % reference time at t=0


%% Effective stresses in subsoil
sigma0_0= sect1*D_improve .*(gammas-gammaw);
sigma0_1= D_improve .*(gammas-gammaw) + sect1*D_clay.*(gammac-gammaw);

%% Limit strain                           % POP peat = 24, only POP clay = 35
pg_0 = sigma0_0+POP_peat;                 % limit strain soil improvement
pg_1 = sigma0_1+POP_clay;                 % limit strain clay

for m=1:n
% Initial conditions

tau_0(m,1)= tau0*(pg_0(m)./sigma0_0(m)).^abc_sand(m);
% equivalent age at time t=0
tau_end0(m,1) = tau_0(m,1);
% equivalent age at time t=0

tau(m,1)= tau0*(pg_1(m)./sigma0_1(m)).^abc_clay(m);
% equivalent age at time t=0
tau_end(m,1) = tau(m,1);
% equivalent age at time t=0

p0(m)=D_loading(m).* gammal(m);
p0_opt(m) = D_loading_opt(m) .* gammal(m);

    for i=2:numel(t);

        if t(i)<=69 % first layer (project floor phase 0 (phase 0 soil improvement))
            T(m,i)=(cv*t(i)*86400)/((D_total(m))^2);
            U(m,i)=((T(m,i)^3)/(T(m,i)^3+0.5))^(1/6);
            delta_sigma(m,i)= U(m,i) * p0(m);
        end

        % calculation of variables in time
        sigma0(m,i)= sigma0(m,1) + delta_sigma(m,i);
        %change in effective stress in soil layer 0 (soil improvement)
        sigma(m,i) = sigma(m,1)  + delta_sigma(m,i);

        tau_0(m,i)= tau_end0(m,i-1)*(sigma0(m,i-1)./sigma0(m,i))^abc_sand(m);
        % reference time of soil layer to calculate equivalent age
        tau(m,i)  = tau_end(m,i-1) *(sigma(m,i-1) ./sigma(m,i)) ^abc_clay(m);

        tau_end0(m,i)    = tau_0(m,i)+(t(i)-t(i-1));
        % change of equivalent age in time per soil layer
        tau_end(m,i)     = tau(m,i)+(t(i)-t(i-1));

        eps_d0(m,i) = a_sand(m) * log(sigma0(m,i)/sigma0(m,i-1));
        % amount of direct strain in time per soil laye
        eps_d(m,i)  = a_clay(m) * log(sigma(m,i)/sigma(m,i-1));

        eps_s0(m,i) = c_sand(m) * log(tau_end0(m,i)/tau_0(m,i));
        % amount of secular strain in time per soil layer
        eps_s(m,i)  = c_clay(m) * log(tau_end(m,i)/tau(m,i));
```

```
                eps_total0(m,i) = eps_total0(m,i-1)+eps_d0(m,i) + eps_s0(m,i);
                % amount of total (natural) strain per soil layer
                eps_total(m,i)  = eps_total(m,i-1) +eps_d(m,i)   + eps_s(m,i);

                S0(m,i) = -1/sect*D_improve(m).* (1-exp(-eps_total0(m,i)));
                % amount of settlement per soil layer
                S(m,i)  = -1/sect*D_clay(m)    .* (1-exp(-eps_total(m,i)));

                % total settlement of subsoil
                S_total(m,i) = S0(m,i) +... + S8(m,i);
      end
end
end
```

RUN FILE A PRIORI SETTLEMENT

This file is used to compute the a priori settlement predictions with the use of the presented function of the previous Paragraph. As can be seen each individual layer is calculated twice and from this to calculations the mean is taken to get reliable results.

```
%% abc apriori

load layer_structure_section_W_corr(n)


sect=2;
sect1=[0.25 0.75];
S_total1=KIS_apriori_9(sect,sect1(1),n,...,gammaw);
S_total2=KIS_apriori_9(sect,sect1(1),n,...,gammaw);
S=S_total1(:,:)+ S_total2(:,:);

B=zeros(n,1);
y=zeros(n,1);

for i=1:n   % filter bad results
          B(i)=S(i,2);
          y(i)=length(B(1:i));
     if S(i,2)>0
          y(i)=0;
     end
end

x=y(1:n);
x(x==0)=[];
S1=S(x,:);
```

BAYESIAN UPDATING FILE

In order to use one million calculations all settlement predictions are placed in a vector belonging to one specific point in time. Without the temporary overburden there are 32 points in time and with the overburden 33. These are loaded into the updating file and than the a posteriori settlement predictions are calculated using Straub's method. Only part of the script is shown.

```
%% Limit State Functions & Monte Carlo Simulation apriori
```

```
R=2.6;              % resistance
S_req=S33;          % sollicitation
Z=R-S_req;          % lsf
omega_F=(Z<0);      % failure space

R1=0.10;            % resistance
S_req1=S31-S30;     % sollicitation
Z1=R1-S_req1;       % lsf
omega_F1=(Z1<0);    % failure space

R2 = 0.3;           % resistance [m]
S_req2=S33-S30;     % sollicitation
Z2 = R2 - S_req2;   % lsf
omega_F2 = (Z2<0);  % failure space


%% Direct update variables

mu_err_m = 0;                    % mean measurement error
sigma_err_m = 0.1;               % std measurement error
k = sigma_err_m/0.40;            % scaling factor (likelihood (here) always
%between 0 and 1)

%% Computation a posteriori settlement values

s_m = A(2,2);               % measurement
S_ap = S2;                  % all a priori settlement predictions belonging to the
%same day of the measurement

% straub: equality to inequality
Likelihood = normpdf((s_m-S_ap), mu_err_m, sigma_err_m);  % Likelihood of observation
vs a priori prediction
E=norminv(k .* Likelihood);                               % Observation Limit State
Function Sollicitation
U2 = normrnd(0,1,n,1);                                    % Standard normal variable
h = U2 - E;                                               % Observation Limit State
Function
em2= double(h < 0);                                       % failure domain (equivalent
%inequality)

%% A posteriori failure probabilities
number = sum(omega_F.*em2.*em4.*em6.*em8.*em10.*em12.*em14.*em16.*em18.*em20);
% number of realizations after update
pf_priori=mean(omega_F); % a priori failure probability
pf_post=mean(omega_F.*em2.*em4.*em6.*em8.*em10.*em12.*em14.*em16.*em18.*em20)/
mean(em2.*em4.*em6.*em8.*em10.*em12.*em14.*em16.*em18.*em20); % a posteriori failure
probability

number1=sum(omega_F1.*em2.*em4.*em6.*em8.*em10.*em12.*em14.*em16.*em18.*em20);
Pf_priori1=mean(omega_F1);
pf_post1=mean(omega_F1.*em2.*em4.*em6.*em8.*em10.*em12.*em14.*em16.*em18.*em20)/
mean(em2.*em4.*em6.*em8.*em10.*em12.*em14.*em16.*em18.*em20);
```

D

MODEL VERIFICATION

In order to rely on the results produced in Matlab the model needs to be verified. In this thesis the way of verifying is done by comparing the Matlab model with D-Settlement. It is important to know whether the used form of the a,b,c–isotache model is right and if the consolidation equations giving similar results as in D-Settlement. D-Settlement is an acknowledged and accepted piece of software which gives reliable results Deltares Systems [14]. The results are obtained and plotted over each other to compare the settlement predictions. This is done for three different cases, from an easy case till a representative case resembling reality. All the input parameters are the same for both models and for both models the Terzaghi consolidation method is used. The cases are build up according to the key questions in Chapter 1, from a homogeneous subsoil to a stratified subsoil with multiple raises in time.

ONE DIMENSIONAL HOMOGENEOUS SUBSOIL

The first case considered is a clay layer which is loaded with a sand layer for 10.000 days. Figure D.1 shows the geometry of the considered subsoil and the subsoil pressure.

Figure D.1: First case

As can be seen from Figure D.1 the water pressure in the subsoil is considered to be hydrostatic and the sand and clay layer are uniform (there are no anomalies present in the soil). The phreatic line is equal to the surface level.

This case is modeled with Matlab and the D-Settlement software and the following settlement predictions are derived from these models, see Figure D.2:

As can be concluded from Figure D.2 the obtained results of the first simple case are very similar for both models. A remark must be made about the consolidation here, it is chosen to work with a high consolidation coefficient c_v to simulate vertical drainage.

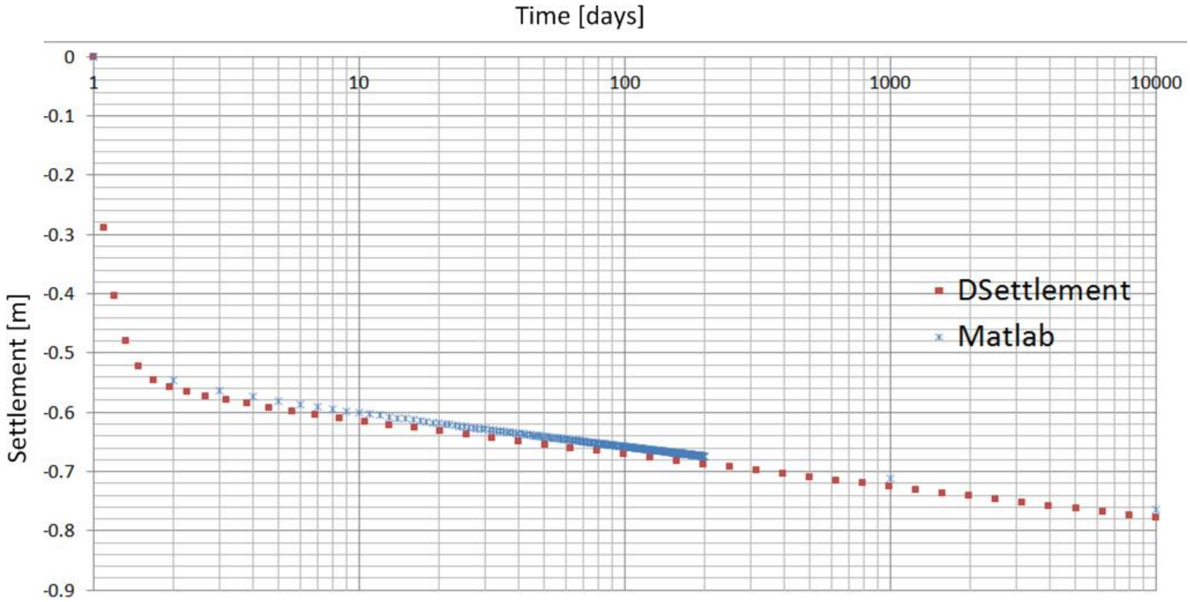

Figure D.2: Comparison between D-Settlement and Matlab model first case

ONE DIMENSIONAL HOMOGENEOUS SUBSOIL WITH MULTIPLE RAISES

The next case was to simulate not one raise for the clay layer but multiple raises. See Figure D.3 for the geometry of this case. The same assumptions are made as for the first simple case.

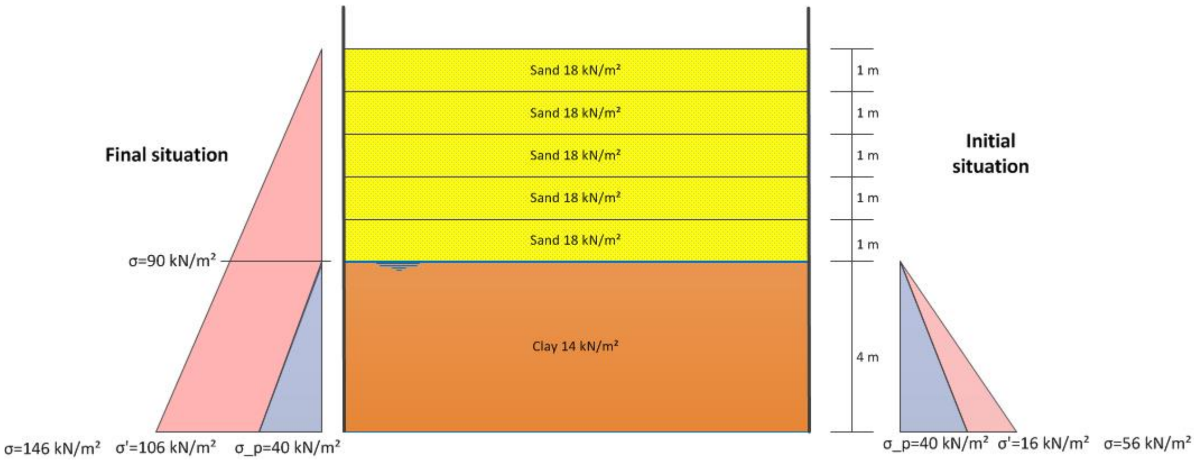

Figure D.3: Second case, multiple raises

These raises are constructed with a period of 30 days in between, so at day 1 the first layer is executed and at day 121 the last layer in executed. This is approximately the same as the construction method in KIS.

Figure D.4 compares the results obtained from Matlab and D-Settlement. The settlement predictions are very similar for both models and thus the Matlab model is reliable for this case. Notice the raises in time, there the settlement is high the first days because of the high direct strain and then passes into the consolidation period where only the secular strain is present until the next raise.

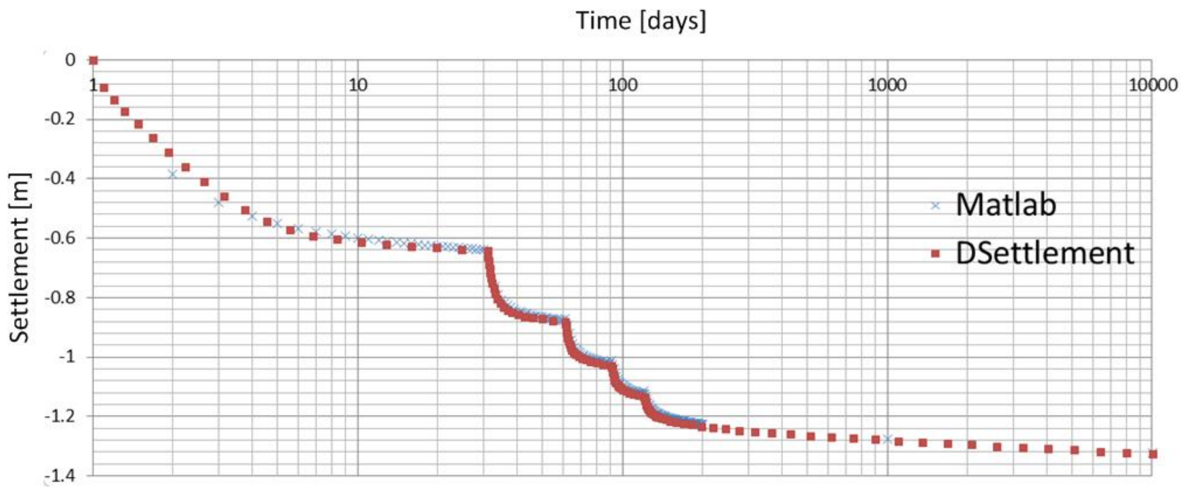

Figure D.4: Comparison between D-Settlement and Matlab model second case

ONE DIMENSIONAL STRATIFIED SUBSOIL WITH MULTIPLE RAISES

The third case to check whether the Matlab model is valid is a subsoil with different layers. Again multiple raises are resembling the loading on the subsoil. This case approaches the real construction of the new dike in the KIS project. This project also has a stratified subsoil which is loaded by constructing the new dike. Figure D.5 shows the geometry of the third case.

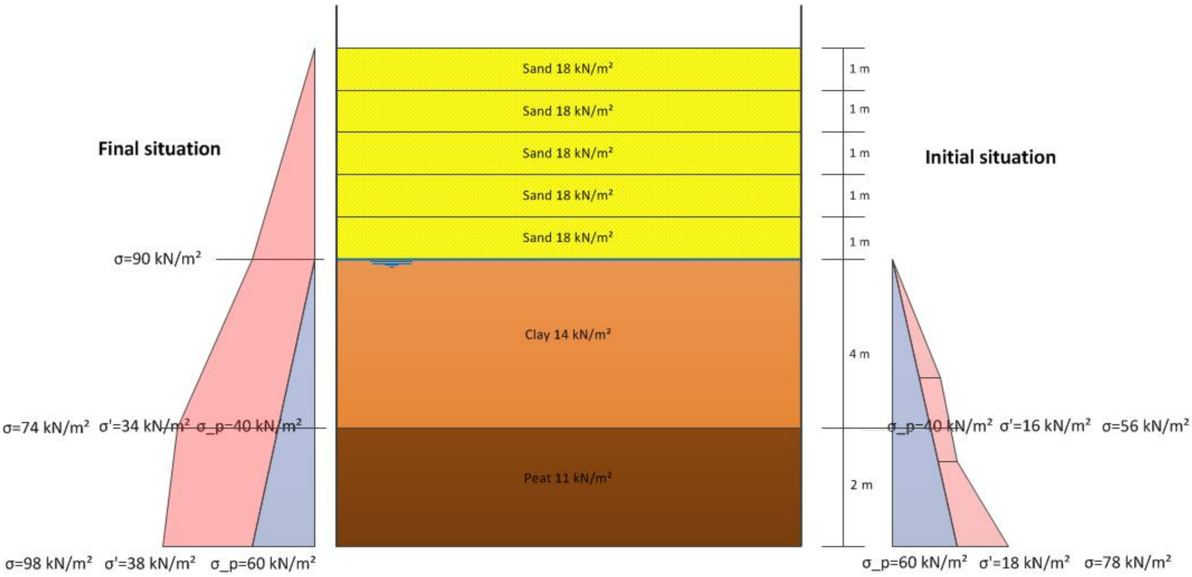

Figure D.5: Third case, stratified subsoil and multiple raises

The subsoil in the KIS project is also consists largely of peat and clay layers, hence a very soft subsoil. Figure D.6 shows the comparison between Matlab and D-Settlement models.

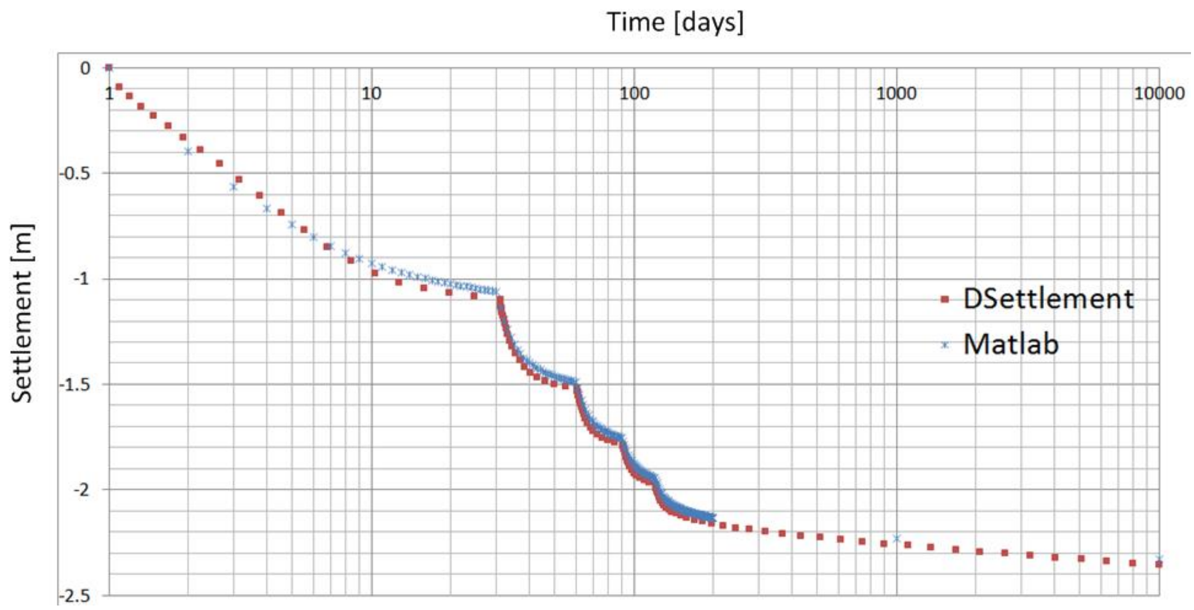

Figure D.6: Third case, close resemblance to subsoil and construction method in KIS project

With this last case, the verification of the Matlab model about the settlement predictions (i.e. a-priori knowledge) is complete. The Matlab model can be considered reliable because of the large similarities in the results based on the same input parameters.

E

Data Settlement Plates

Monitoring data is collected during the KIS project with settlement plates. The data used in this thesis is presented below in Table E.1.

Date	Head	Foot	Surface level
17-7-2014	3,189	1,189	2,152
22-7-2014	3,189	1,18	2,121
29-7-2014	3,17	1,17	2,189
13-8-2014	3,154	1,154	2,077
22-8-2014	3,137	1,137	2,102
28-8-2014	3,132	1,132	2,093
4-9-2014	3,119	1,119	2,112
12-9-2014	3,111	1,111	2,095
25-9-2014	3,105	1,105	2,071
03-10-2014	4,006	1,006	2,995
08-10-2014	4,005	1,005	3,02
14-10-2014	3,943	0,943	2,762
30-10-2014	3,91	0,91	2,793
07-11-2014	3,882	0,882	2,782
14-11-2014	3,877	0,877	2,779
21-11-2014	3,855	0,855	2,747
26-11-2014	3,834	0,834	2,775
10-12-2014	3,799	0,799	2,712
07-01-2015	3,782	0,782	2,703
21-01-2015	3,781	0,781	2,693
03-02-2015	3,749	0,749	2,655
17-02-2015	3,753	0,753	2,68
04-03-2015	3,736	0,736	2,66
31-03-2015	3,734	0,734	2,632
15-04-2015	4,645	0,645	3,137
29-04-2015	4,539	0,539	3,701
11-05-2015	4,509	0,509	3,676
29-05-2015	4,405	0,405	3,566
08-06-2015	4,396	0,396	3,58
15-06-2015	6,296	0,296	4,532
06-07-2015	6,253	0,253	4,513
24-07-2015	6,141	0,141	5,238

Table E.1: Settlement plate data W27

The highlighted rows in Table E.1 show the measurements that are actually used in this thesis to update the settlement predictions. These are measurements on representative points in time to approach the actual settlement behaviour, this means actual behaviour of the settlement during a period of no raising and a period of fast raising of the new dike. Because of a lack of computing strength it was necessary to choose specific measurements instead of using all measurements what is preferable. The consequence of filtering the a-priori realizations to a-posteriori predictions is that with each update (measurement) less realizations are left, therefore when using a large number of measurements the realizations that are left after updating could be zero and thus no solution. For a reliable a-posteriori prediction with the Monte Carlo Simulation at least a number of 100 realizations must be left after the update.

As explained in Chapter 3 the column with settlement data of the foot of the settlement plate is used as actual settlement. The applied layer thickness can be checked with the fourth column, which shows the surface level in time.

The values shown above are already filtered on 'bad' measurements, these are measurements especially in the initial phase of the project where the foot of the settlement plate was located higher than the previous measurement.

F

Correlations a,b,c–isotache model

It is important to know what the influence is of introducing correlations is the a,b,c–isotache model. It appeared that correlations had a large influence on the standard deviations of the settlement predictions. Normally one would expect that by introducing correlations between parameters the reliability of the predictions would get higher, however this doesn't hold for the a,b,c–isotache model. The use of correlations in the model is important because the in physical reality some combinations of a,b and c are not possible, with implementation of the correlations these values are filtered out of the computation. This appendix shows the analysis done on how the isotache model behaves under correlated compressibility parameters and uncorrelated parameters.

For all three compressibility parameters a,b and c the influence of the correlations between these parameters are investigated by running the isotache model in Matlab. First the parameters were given no correlations to obtain extreme values in each quadrant. The quadrants are Northeast (NE), Northwest (NW), Southwest (SW) and Southeast (SE). When the three parameters are not correlated the realizations of these parameters are approximately equal divided over each quadrant, see Figure F.1.

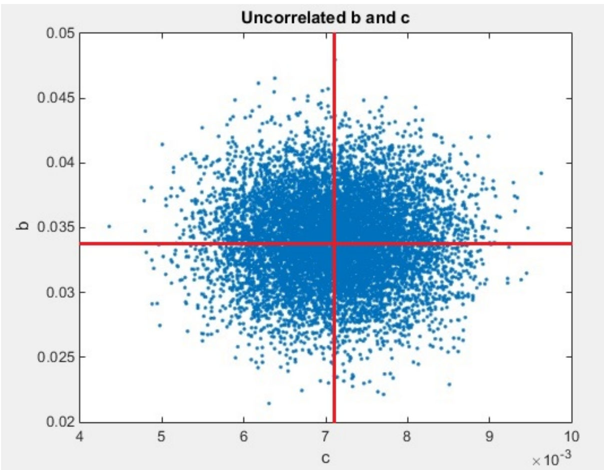

Figure F.1: Scatter plot of uncorrelated b and c

However when introducing correlations the NW and SE quadrant values will shift more to the NE and SW quadrants (when positively correlated), see Figure F.2:

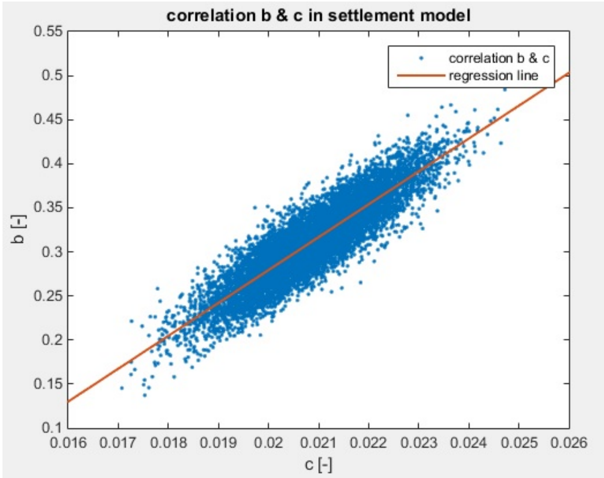

Figure F.2: Scatter plot of correlated b and c

As a result of shifting parameter values induced by correlations the values of the *a*,*b* and *c* parameters will have more extremes in the NW and SW quadrant. What this means for the settlement predictions by the a,b,c–isotache model is showed below. Here the isotache model is calculated with extreme values for each quadrant in the plot, for example for the NE quadrant there is chosen to use a low *a* value but high *b* value. The same holds for the SE quadrant but than vice versa. By doing this the influence of a combination between *a*,*b* and *c* is obtained for each quadrant and therefore the influence of the introduced correlations can be measured. Figures F.3, F.4 and F.5 show that the standard deviation found for the settlement predictions is smaller for uncorrelated values of *a*,*b* and *c* (green lines) than when they are correlated (red lines). Especially the correlations between *a* and *c*, and *b* and *c* introduces more spread in the settlement predictions.

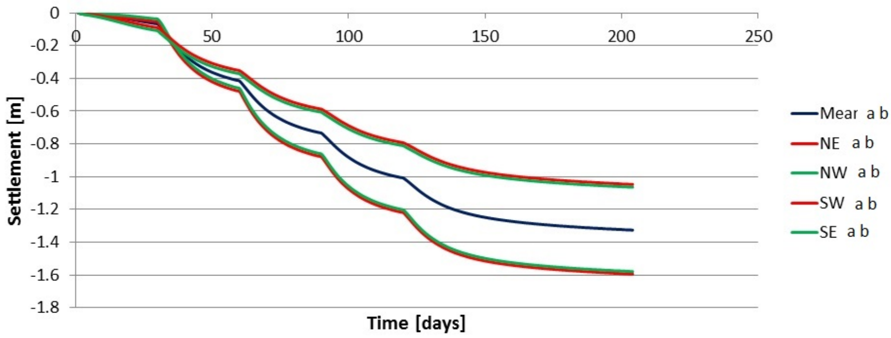

Figure F.3: Influence of correlations between a and b for the a,b,c–isotache model

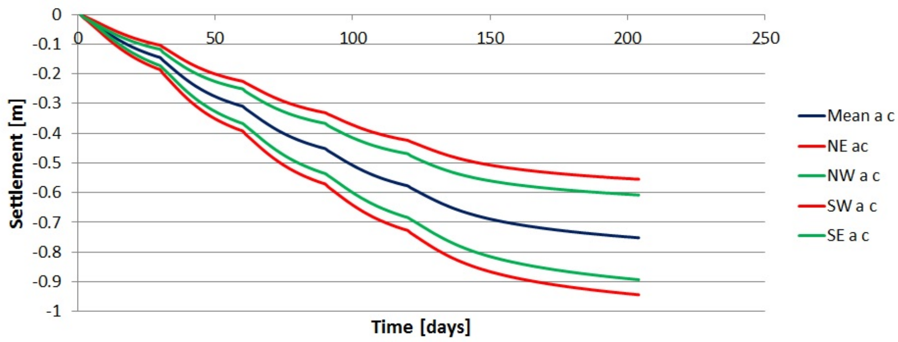

Figure F.4: Influence of correlations between a and c for the a,b,c–isotache model

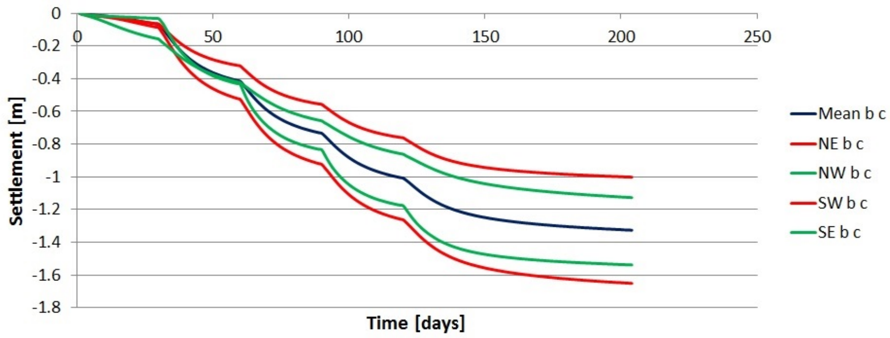

Figure F.5: Influence of correlations between b and c for the a,b,c–isotache model

A very clear graphical representation of the influence of the correlations can be found in Figure F.6, the mean does not change to much however the standard deviation gets bigger.

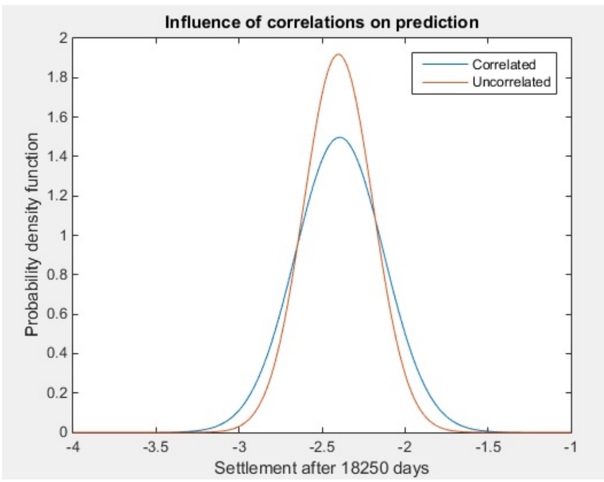

Figure F.6: Influence of correlations on probabibility density function of settlement after 18250 days

G
Practical Issues Monitoring Data

This appendix elaborates on the difficulties found in measuring the settlement plates in combination with the ongoing construction activities.

Engineers of IV-Infra have come up with a monitoring plan, which is based on how the monitoring data should be obtained. With this plan the client is informed in how the data will be obtained and how this data is used in the design and construction of the KIS project, see section 3.3.2. Theoretically this plan is good, however in practice it turns out that there are some difficulties that need to be taken into account. For example one can imagine that when the space between two settlement plates is set to 5 m, two heavy vehicles who need to pass each other during construction can hit the settlement plates. It is important to be aware of these difficulties and therefore they are named in the list below, however this thesis does not elaborate further on this topic. Some of the difficulties listed below are generally found in projects, some are specifically to the KIS project.

1. Settlement plates are run over, or they are moved by people/machinery which need to work with heavy vehicles at the locations of the monitoring equipment.
2. Settlement plates are removed because of construction works need to take place at that location, like the (re)placement of cables and pipelines. The consequence of removing the settlement plate, removing soil, place cables and pipelines, place back the soil and place back the settlement plate is that the soil is stirred. When the settlement plate is placed back on the same spot it records also the (slow) compaction of the stirred soil. An assumption made in the settlement predictions is that the soil is compacted and that the compaction is negligible to the amount of settlement.
3. A very practical and easy to solve issue is the use of different GPS devices for measuring the subsidence of the settlement plates. Each device has its own deviation when executing a measurement, therefore when only the same device is used each measurement contains the same measurement error.
4. Taking the measurements is sometimes not carried out according to the monitoring plan, a couple of examples:
   (a) The initial situation is not well measured and therefore a good reference level is missed.
   (b) In the monitoring plan is stated to measure the settlement plates at the day before constructing a raise and a day after the construction. In this way the exact layer thickness is obtained that is applied, this is of importance for the update on the settlement predictions. However this is not always done and it causes a lack of information.
   (c) An inevitable error is the placement of the GPS device, this is an error with the label human error. If placed just on a rock or just next to it, it can give other values for the measurement.
   (d) As point b, but now for all the measurements. The time between two measurements is sometimes to large and not following the monitoring plan. In KIS there are cases found where there is

sometimes no data for 70 days, while the construction process continued in this period. A consequence is that it is difficult to know the precise amount of settlement due to a raise, because of no information about the actual layer thickness and the measured settlement.

5. No log of all specifications of each construction process. For instance, what is the layer thickness applied for a certain date and location and what kind of material is used? Collecting this data gives the designers a lot of insight in how to design the remaining part of the structure. Optimization of the design would probably become even better with more information.

6. Communication between the contractor and the engineering firm is sometimes not good. For example the contractor applies 2 m of clay, while instead in the design plan it is stated that the raises are executed with a layer thickness of 1 m each. Sometimes it is not possible for the contractor to strictly follow the design plan, however when the contractor deviates from the design plan it could have consequences on the whole design.

7. Another example of difficulties between the theory and practice the design plan assumes that a whole raise is executed at once, however in the KIS project two categories of clay are used in one raise. Category II for the outer layer of the dike and category III for the inner layer and core. It is very difficult for a contractor to construct a whole raise at once, it is possible but probably very expensive. As a consequence the monitoring data found in one section can give different information about the settlement in cross-sectional direction.

H

RESULTS MATLAB MODEL

This appendix shows all figures belonging to the results obtained with the Matlab model for the 15 scenarios. Table H.1 shows the precise values of all scenarios with respect to layer thicknesses and temporary overburdens.

Scenario	Details last layer	Height of total raise
1	7^{th} layer = 1 m	NAP + 9 m
2	7^{th} layer = 0 m	NAP + 8 m
3	7^{th} layer = 0,5 m	NAP + 8,5 m
4	6^{th} layer = 1,5 m	NAP + 8,5 m
5	7^{th} layer = 0,2 m	NAP + 8,2 m
6	7^{th} layer = 0,3 m	NAP + 8,3 m
7	7^{th} layer = 0,4 m	NAP + 8,4 m
8	6^{th} layer = 1,3 m	NAP + 8,3 m
9	6^{th} layer = 1 m with temporary overburden 7^{th} layer = 1 m	NAP + 8 m
10	6^{th} layer = 1 m with overburden 7^{th} layer = 0,5 m	NAP + 8 m
11	6^{th} layer = 1 m with temporary overburden 7^{th} layer = 0,3 m	NAP + 8 m
12	6^{th} layer = 1 m with temporary overburden 7^{th} layer = 0,8 m	NAP + 8 m
13	6^{th} layer = 1,7 m with temporary overburden = 0,7 m	NAP + 8 m
14	6^{th} layer = 1 m with temporary overburden 7^{th} layer = 0,6 m	NAP + 8 m
15	6^{th} layer = 1,6 m with temporary overburden = 0,6 m	NAP + 8 m
16	6^{th} layer = 1 m with temporary overburden 7^{th} layer = 0,7 m	NAP + 8 m

Table H.1: Scenarios

SCENARIO 1

Figure H.1 shows the updated settlement prediction of the total amount of settlement at the end of the life-time of the dike structure.

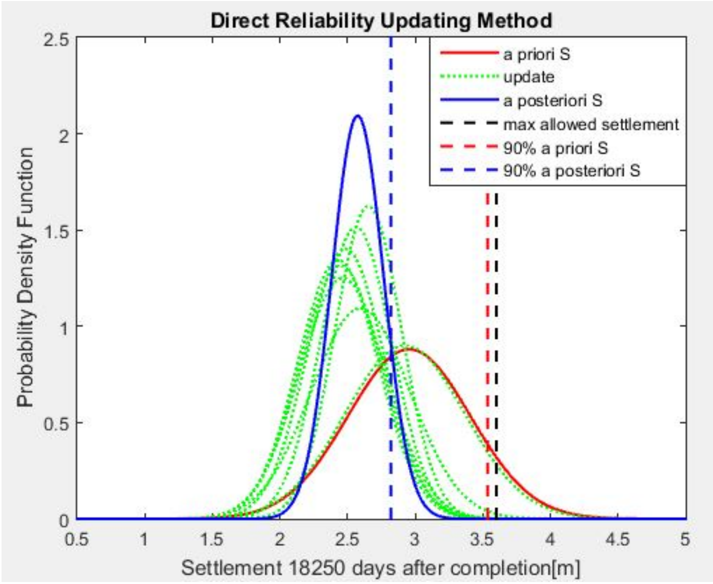

Figure H.1: Total settlement prediction, a-priori and a-posteriori

The following Figure H.2 shows the updated residual settlement one year after completion.

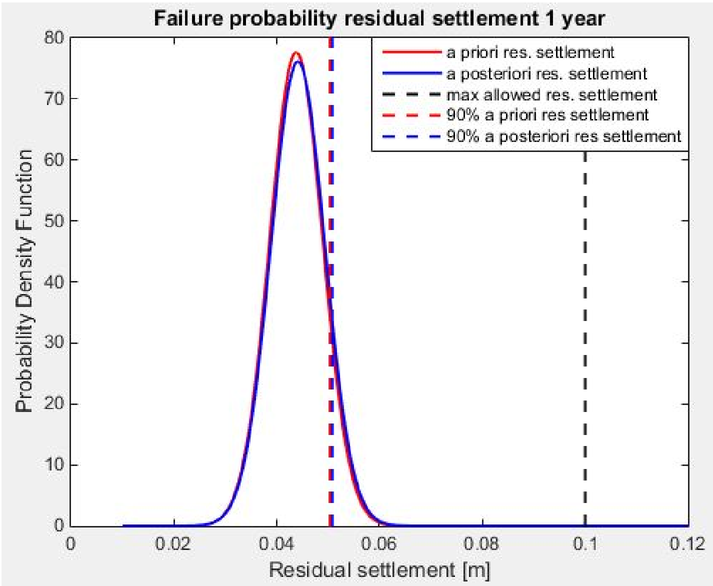

Figure H.2: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.3 shows the a-priori and a-posteriori residual settlement 50 years after completion.

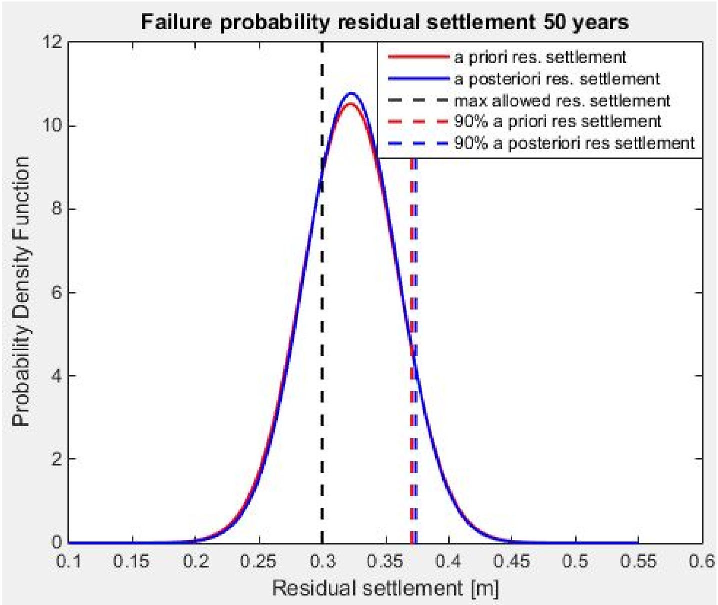

Figure H.3: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.4.

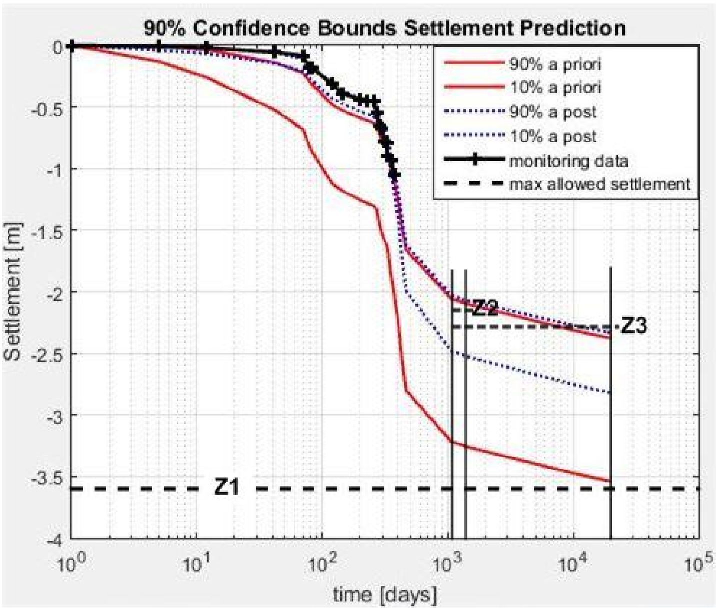

Figure H.4: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 2

Figure H.5 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

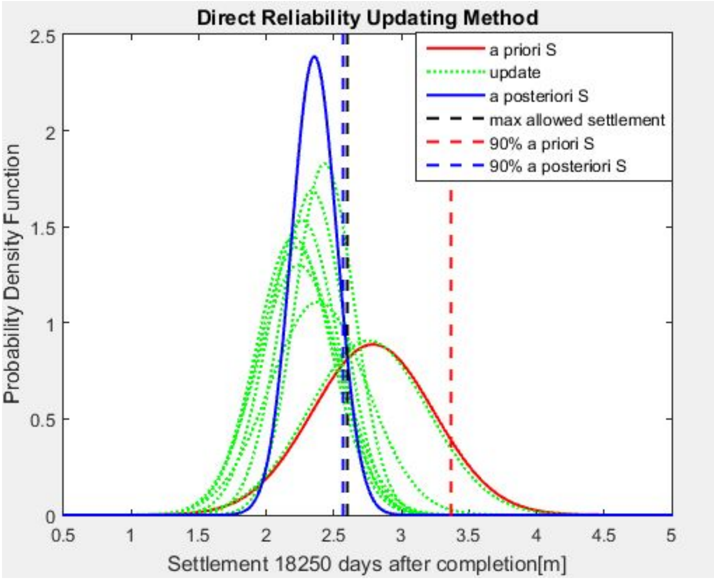

Figure H.5: Total settlement prediction, a-priori and a-posteriori

The following Figure H.6 shows the updated residual settlement one year after completion.

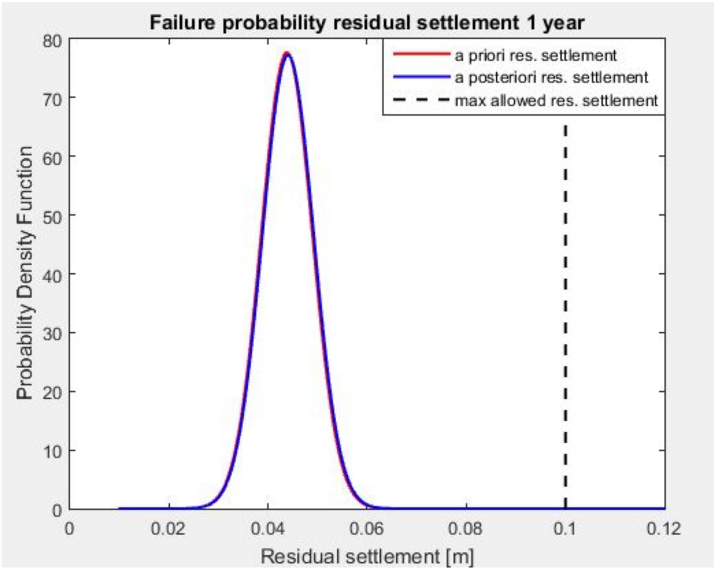

Figure H.6: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.7 shows the a-priori and a-posteriori residual settlement 50 years after completion.

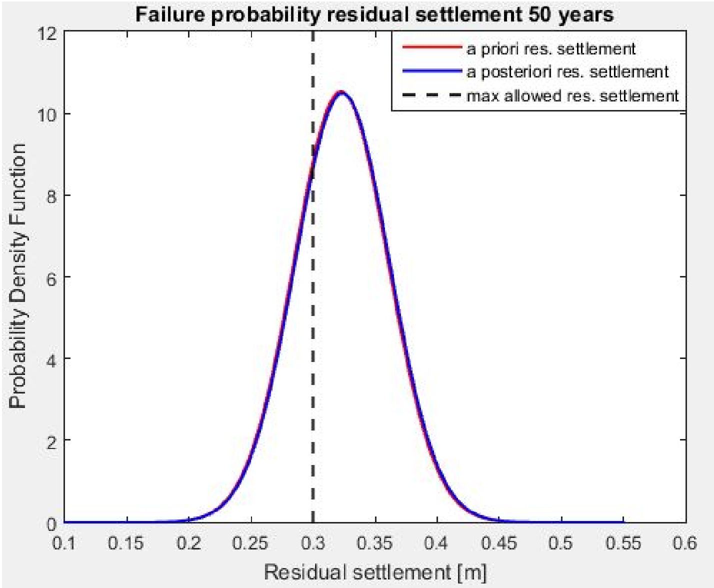

Figure H.7: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.8.

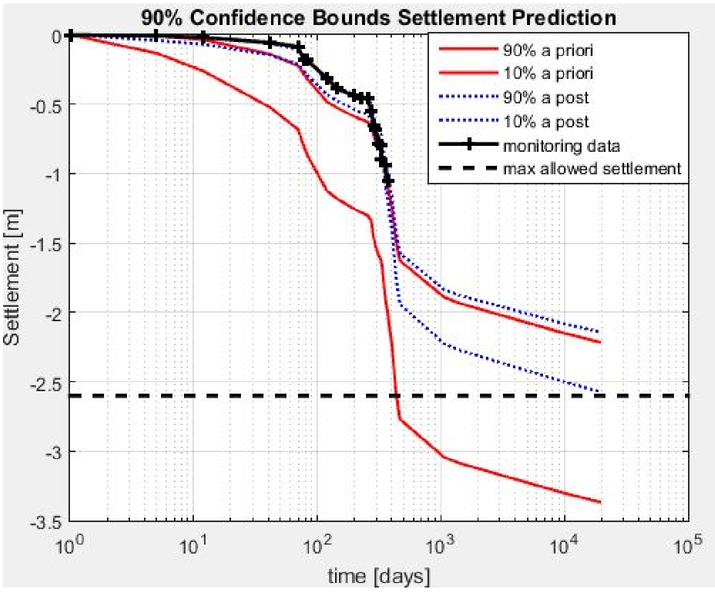

Figure H.8: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 3

Figure H.9 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

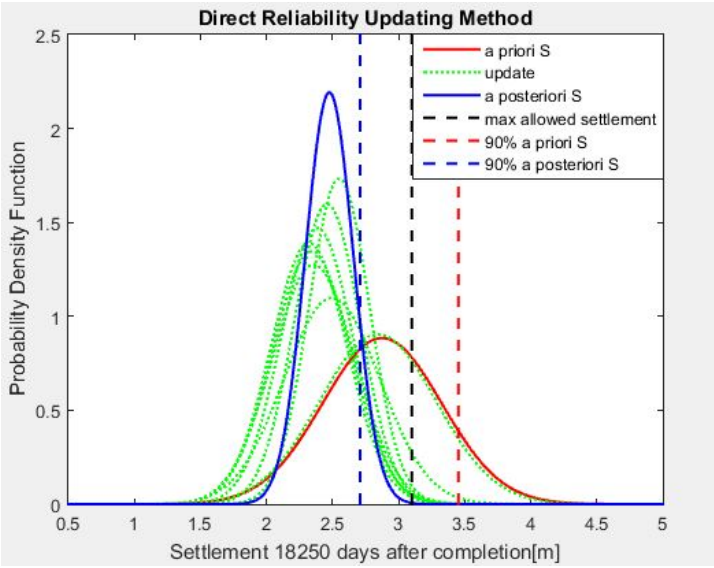

Figure H.9: Total settlement prediction, a-priori and a-posteriori

The following Figure H.10 shows the updated residual settlement one year after completion.

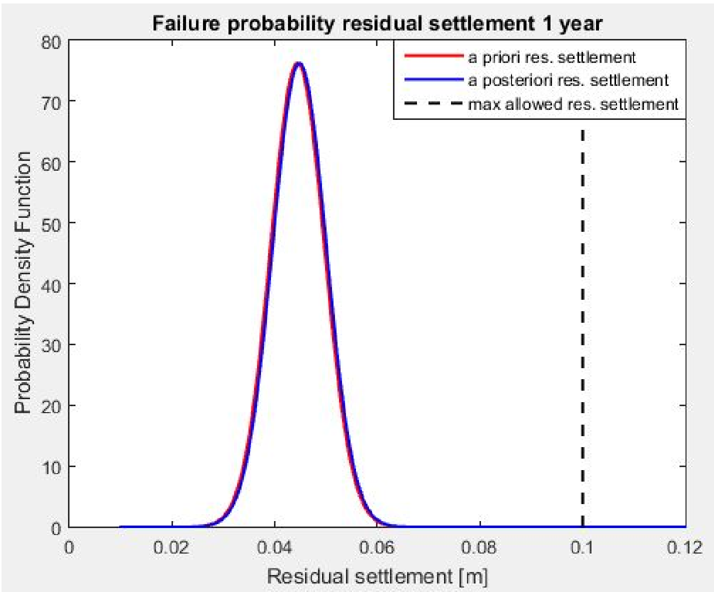

Figure H.10: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.11 shows the a-priori and a-posteriori residual settlement 50 years after completion.

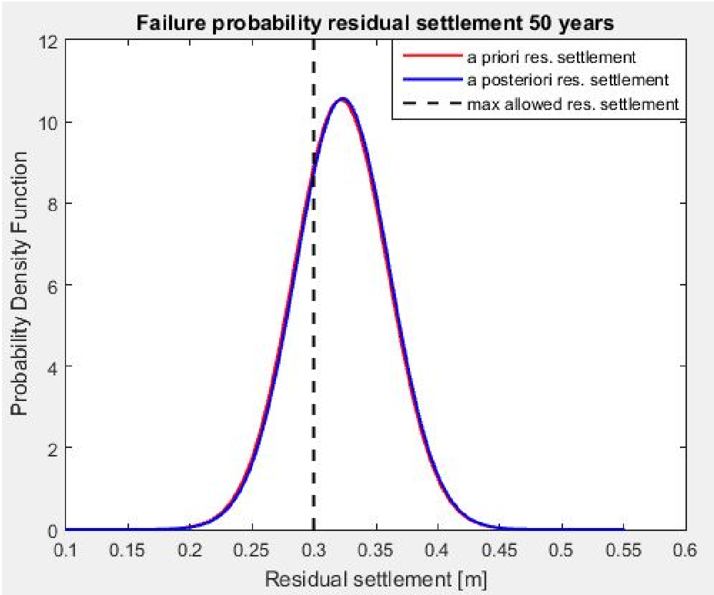

Figure H.11: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.12.

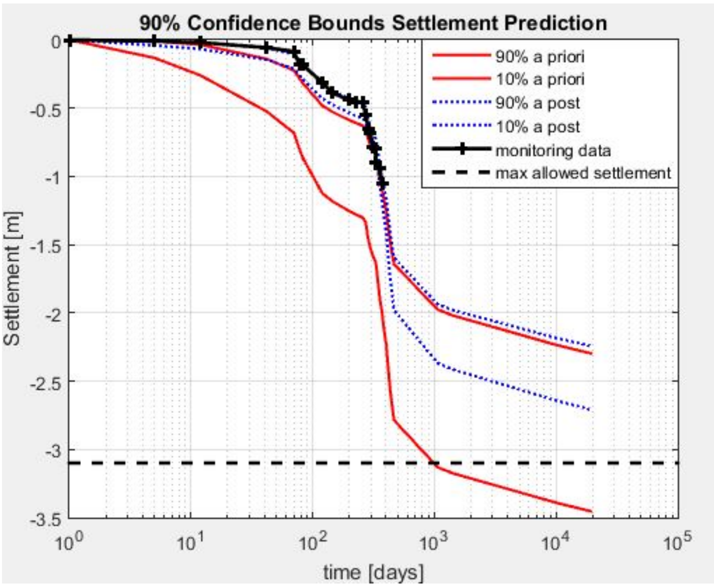

Figure H.12: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 4

Figure H.13 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

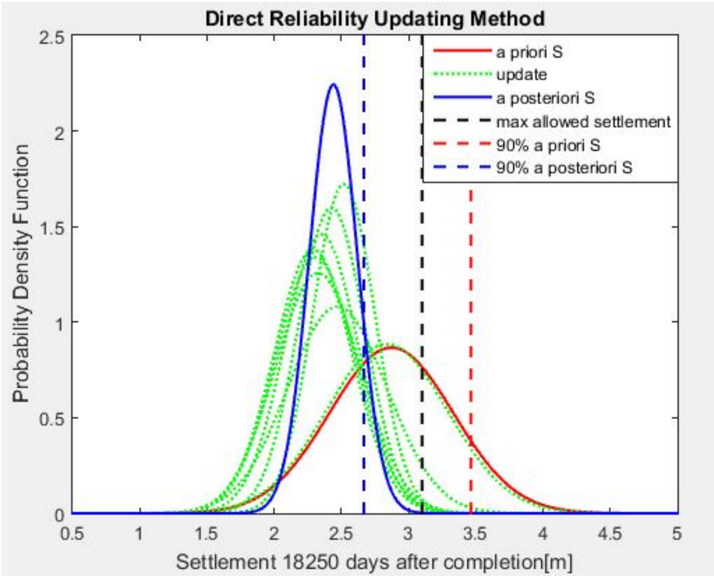

Figure H.13: Total settlement prediction, a-priori and a-posteriori

The following Figure H.14 shows the updated residual settlement one year after completion.

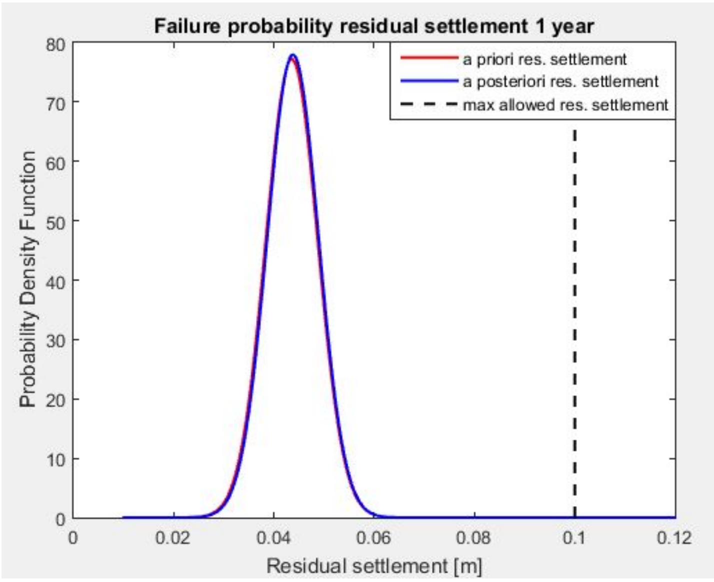

Figure H.14: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.15 shows the a-priori and a-posteriori residual settlement 50 years after completion.

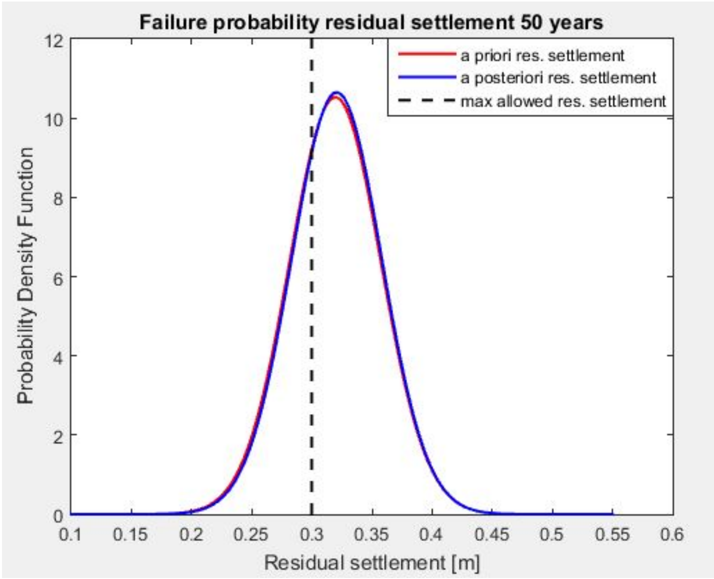

Figure H.15: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.16.

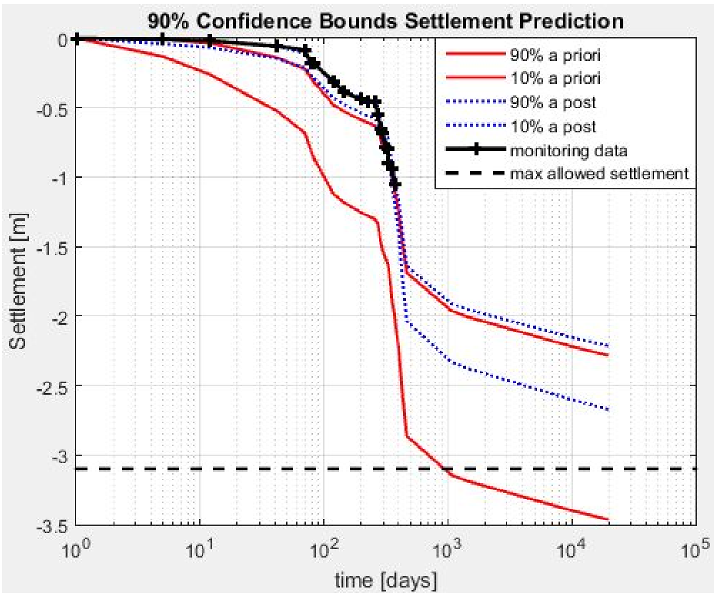

Figure H.16: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 5

Figure H.17 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

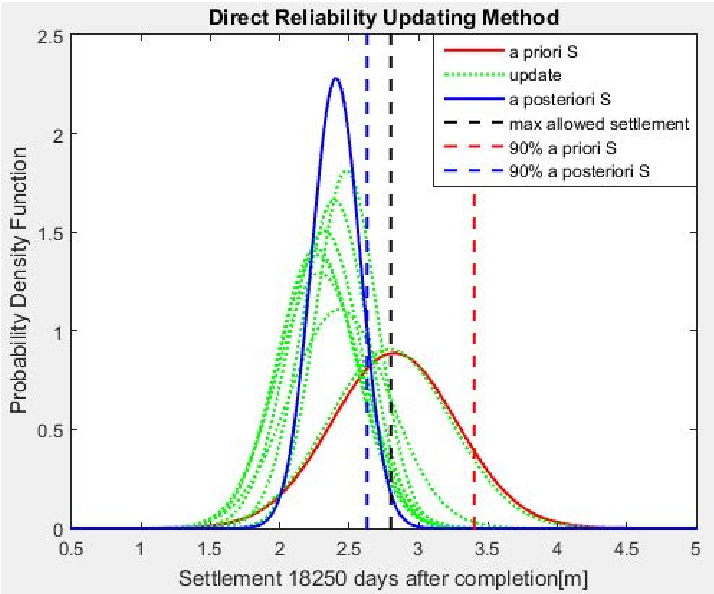

Figure H.17: Total settlement prediction, a-priori and a-posteriori

The following Figure H.18 shows the updated residual settlement one year after completion.

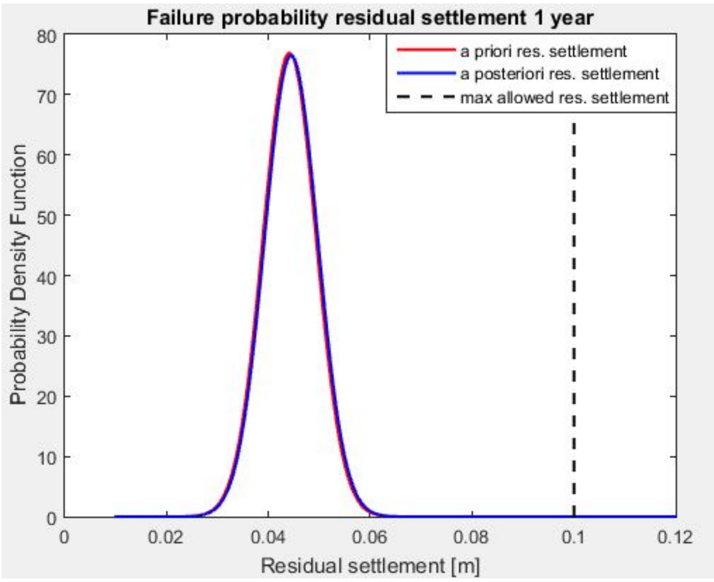

Figure H.18: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.19 shows the a-priori and a-posteriori residual settlement 50 years after completion.

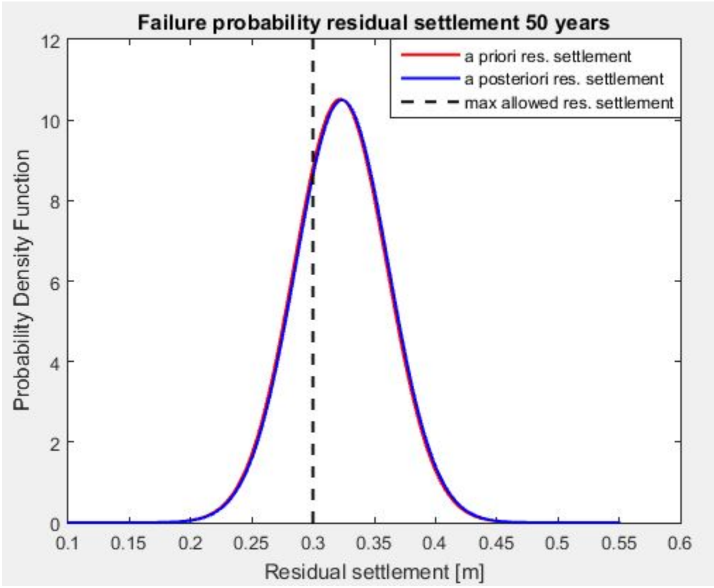

Figure H.19: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.20.

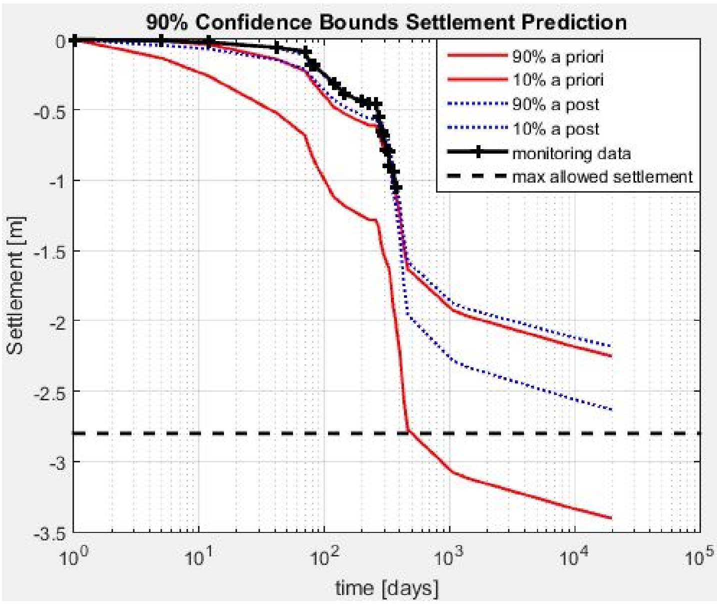

Figure H.20: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 6

Figure H.21 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

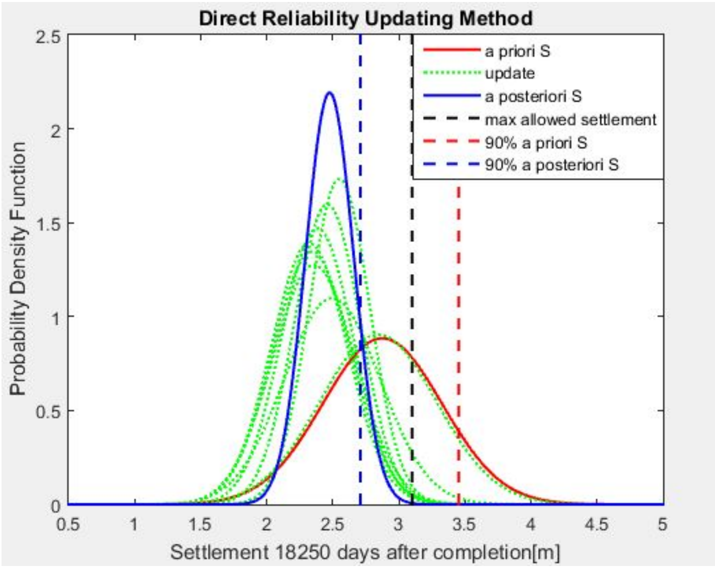

Figure H.21: Total settlement prediction, a-priori and a-posteriori

The following Figure H.22 shows the updated residual settlement one year after completion.

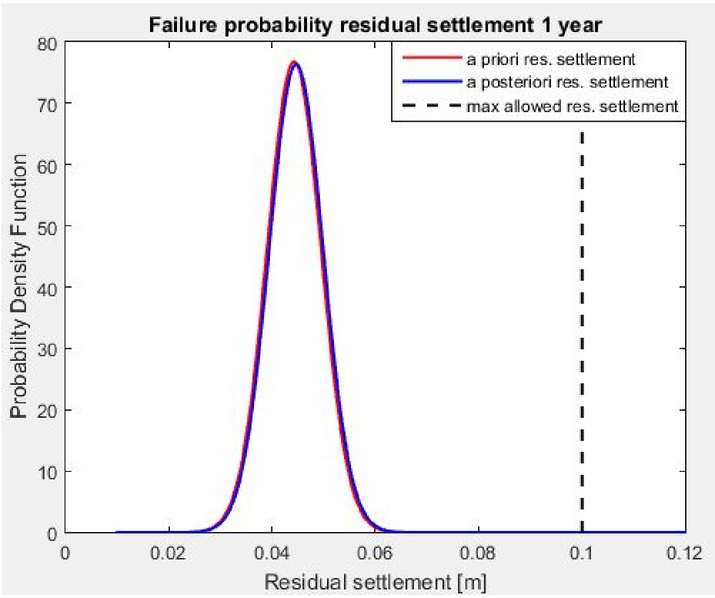

Figure H.22: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.23 shows the a-priori and a-posteriori residual settlement 50 years after completion.

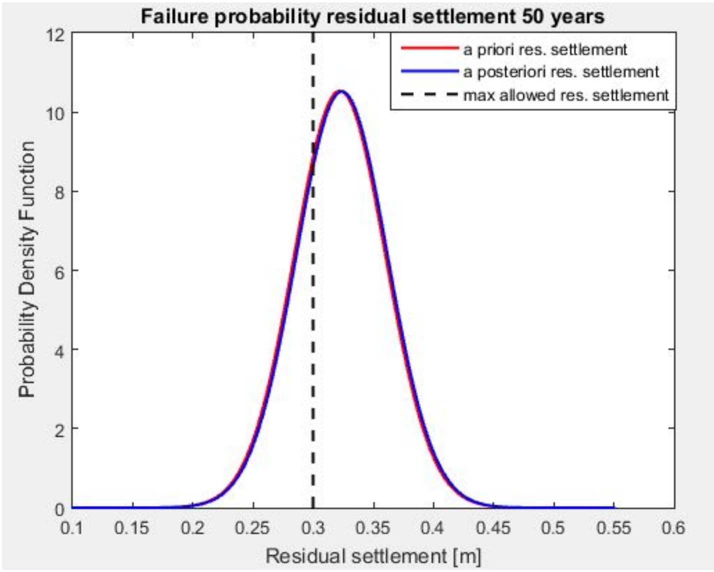

Figure H.23: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.24.

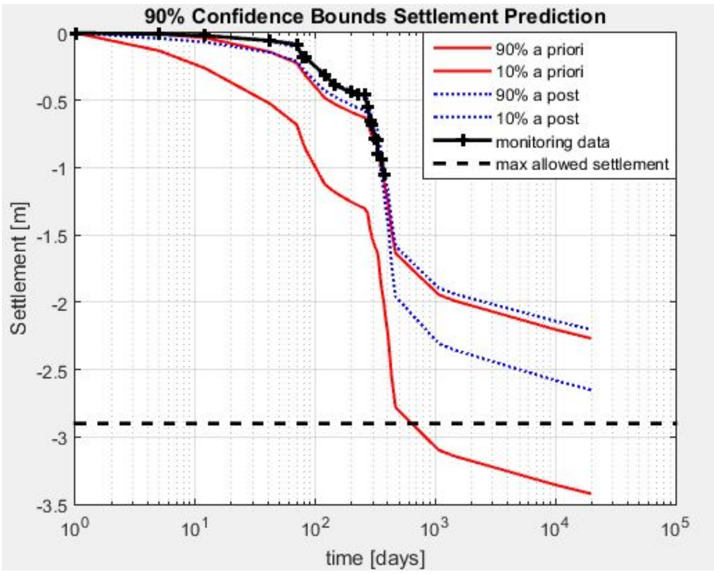

Figure H.24: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 7

Figure H.25 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

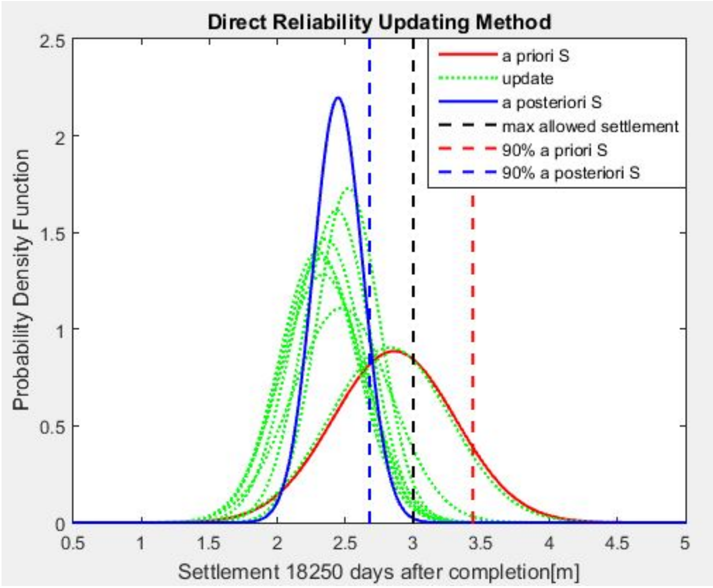

Figure H.25: Total settlement prediction, a-priori and a-posteriori

The following Figure H.26 shows the updated residual settlement one year after completion.

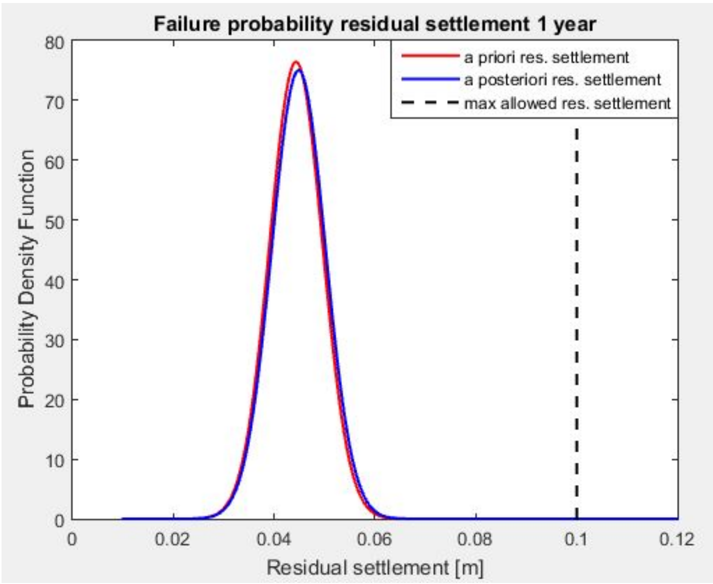

Figure H.26: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.27 shows the a-priori and a-posteriori residual settlement 50 years after completion.

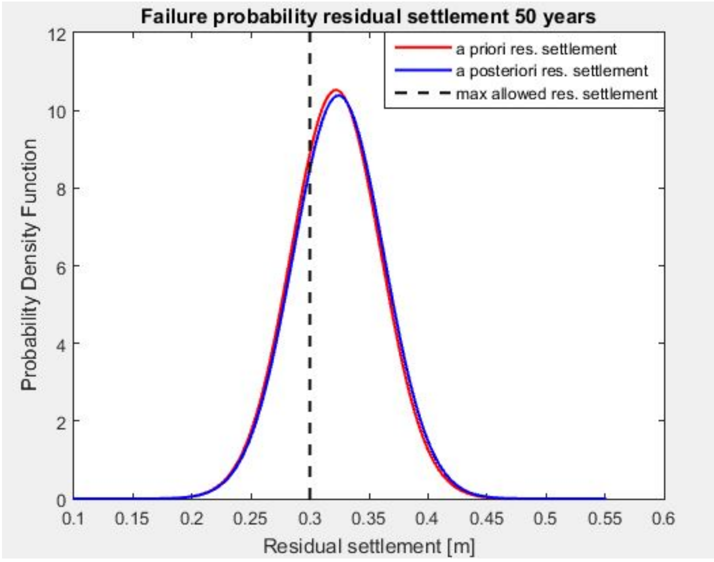

Figure H.27: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.28.

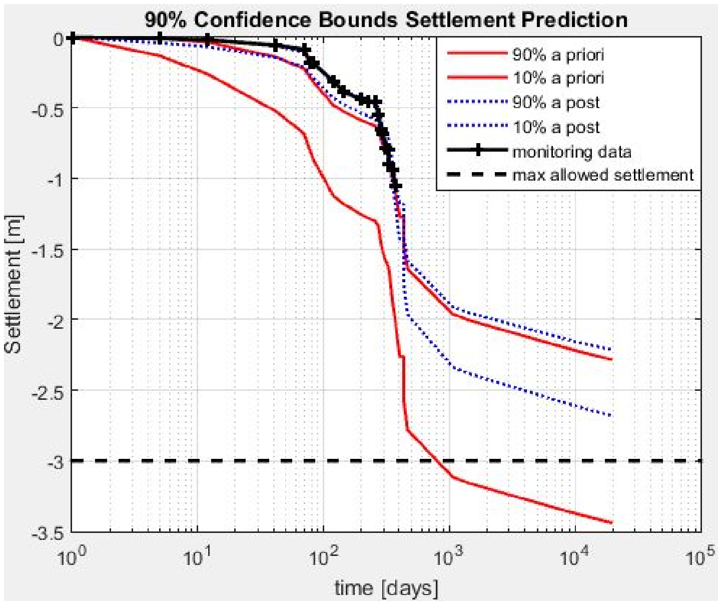

Figure H.28: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 8

Figure H.29 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

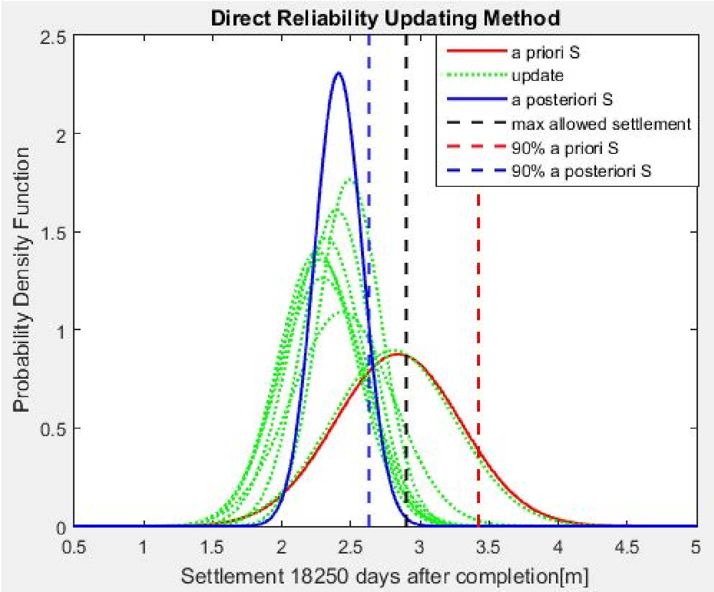

Figure H.29: Total settlement prediction, a-priori and a-posteriori

The following Figure H.30 shows the updated residual settlement one year after completion.

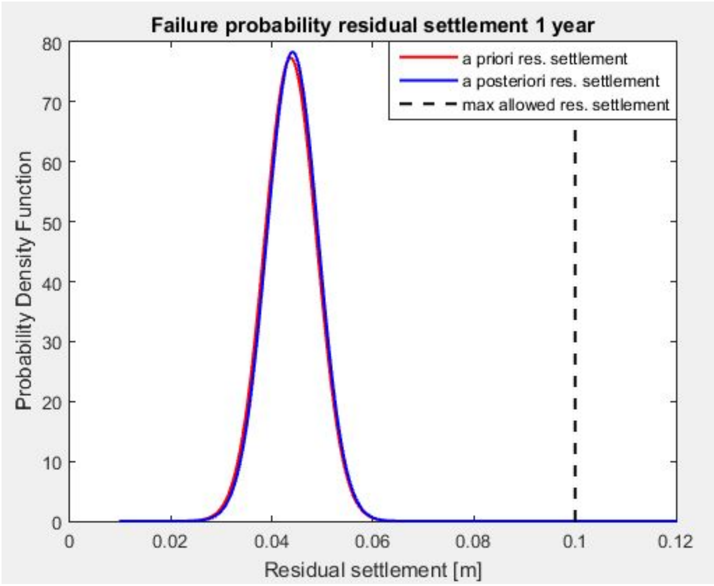

Figure H.30: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.31 shows the a-priori and a-posteriori residual settlement 50 years after completion.

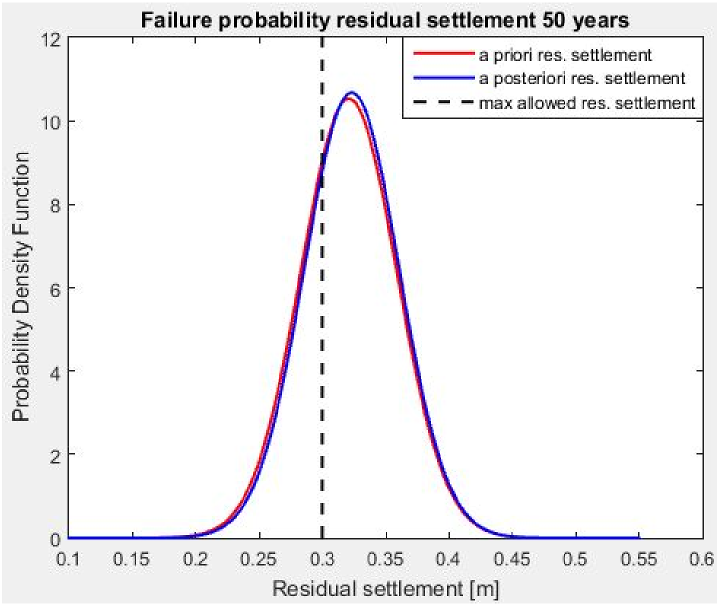

Figure H.31: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.32.

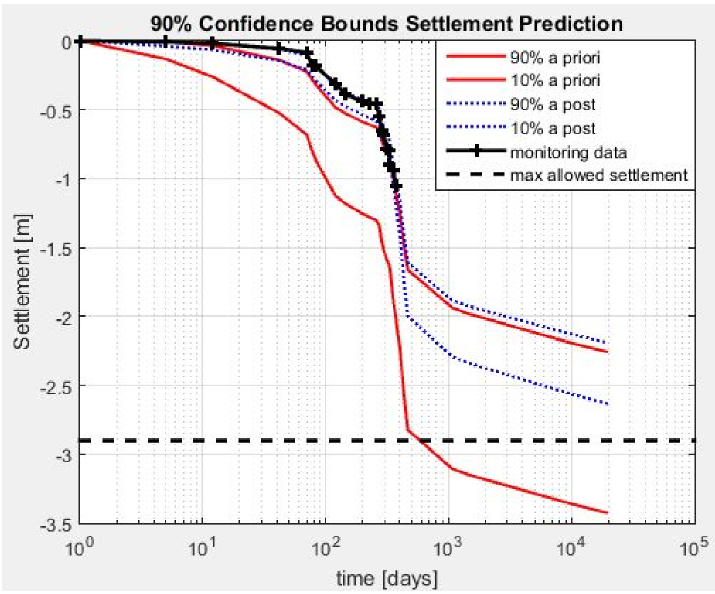

Figure H.32: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 9

Figure H.33 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

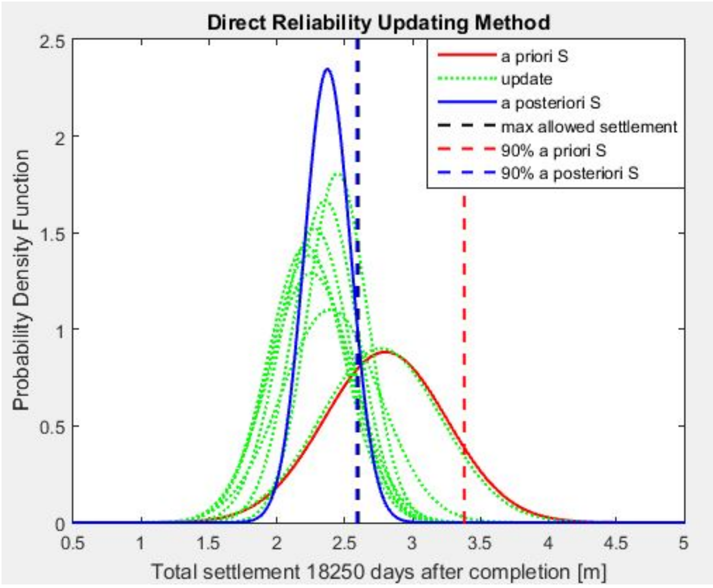

Figure H.33: Total settlement prediction, a-priori and a-posteriori

The following Figure H.34 shows the updated residual settlement one year after completion.

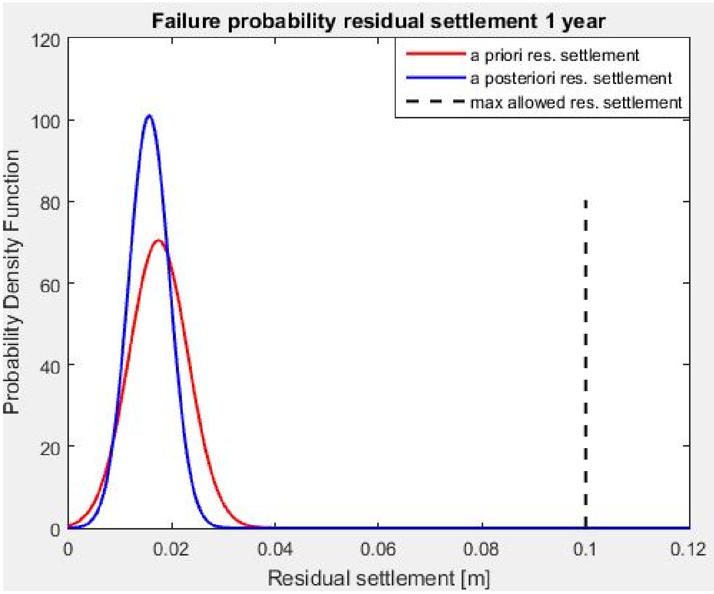

Figure H.34: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.35 shows the a-priori and a-posteriori residual settlement 50 years after completion.

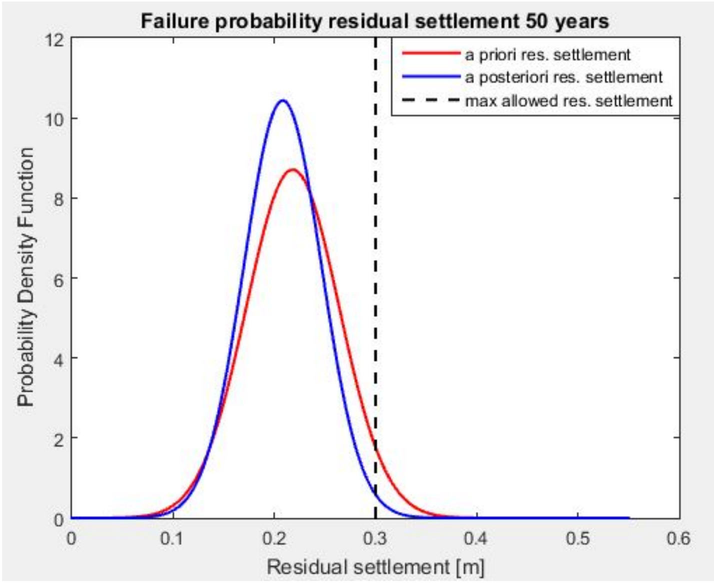

Figure H.35: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.36.

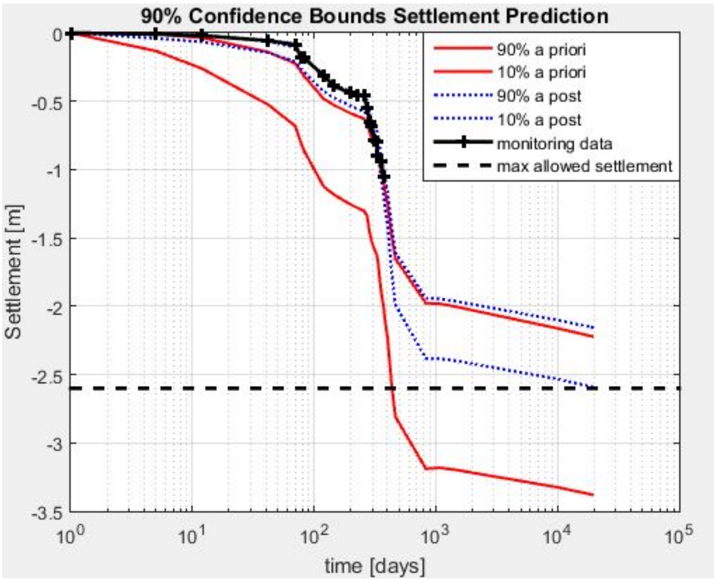

Figure H.36: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 10

Figure H.37 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

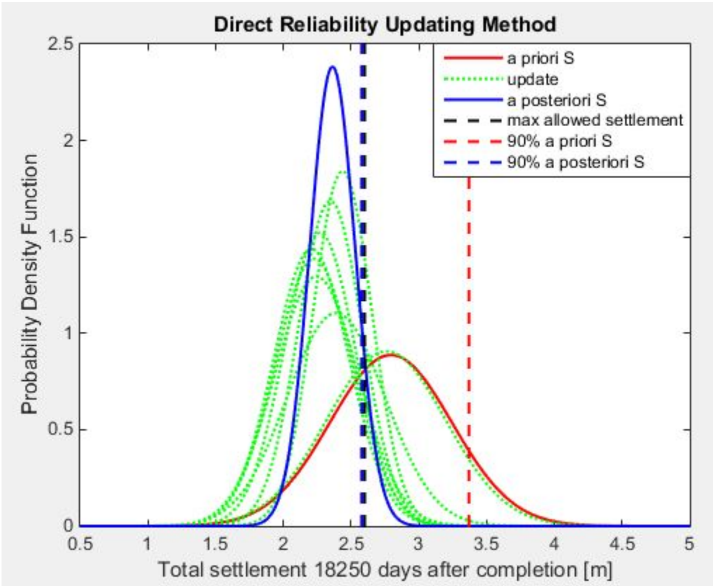

Figure H.37: Total settlement prediction, a-priori and a-posteriori

The following Figure H.38 shows the updated residual settlement one year after completion.

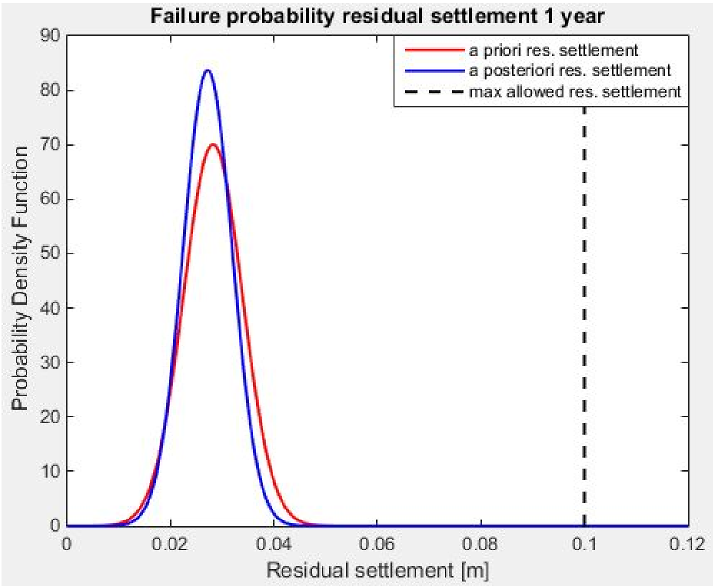

Figure H.38: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.39 shows the a-priori and a-posteriori residual settlement 50 years after completion.

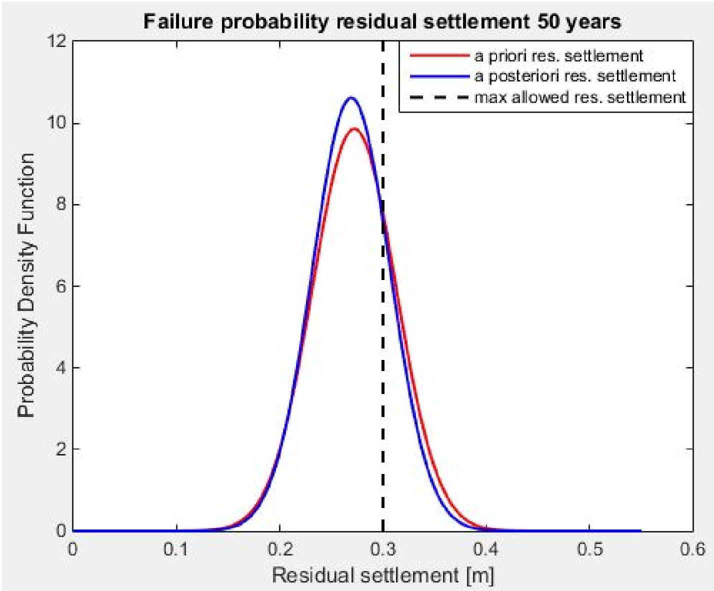

Figure H.39: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.40.

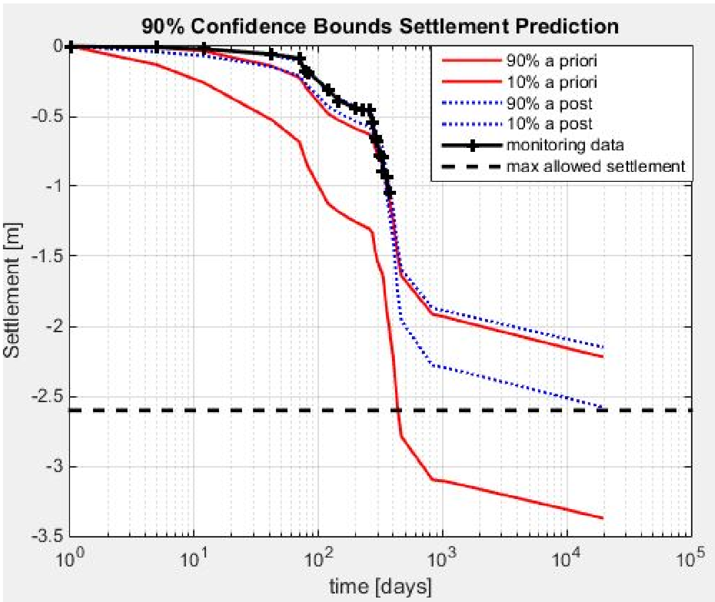

Figure H.40: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 11

Figure H.41 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

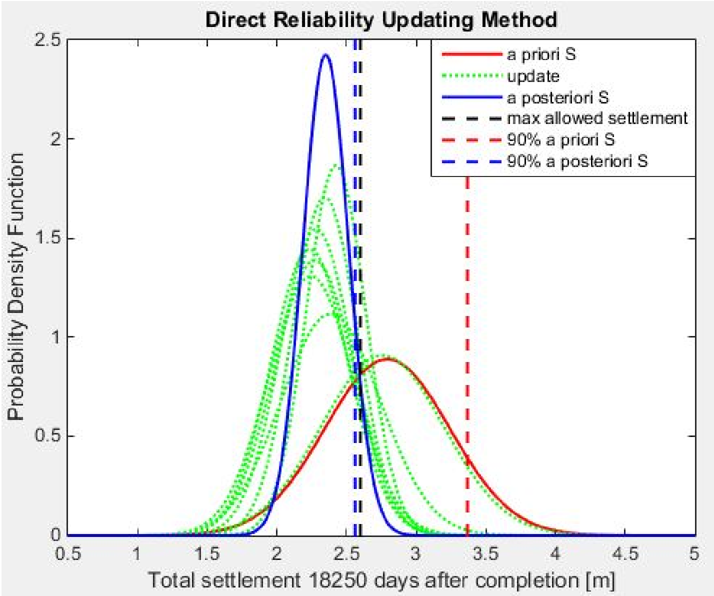

Figure H.41: Total settlement prediction, a-priori and a-posteriori

The following Figure H.42 shows the updated residual settlement one year after completion.

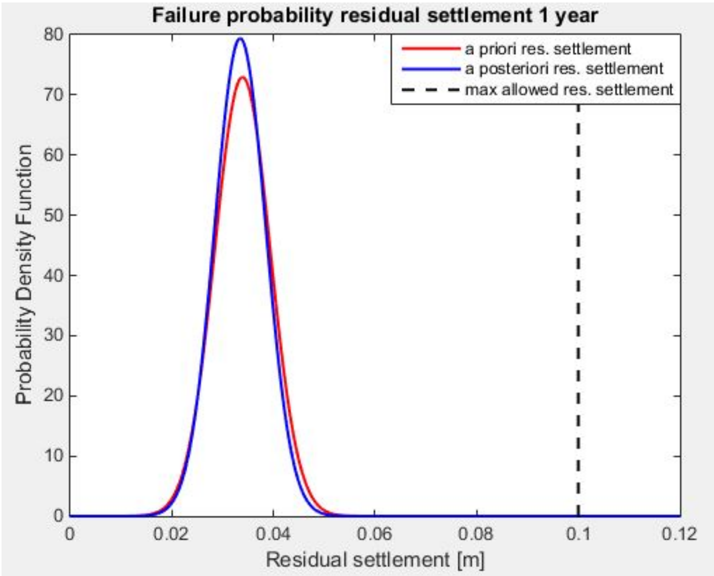

Figure H.42: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.43 shows the a-priori and a-posteriori residual settlement 50 years after completion.

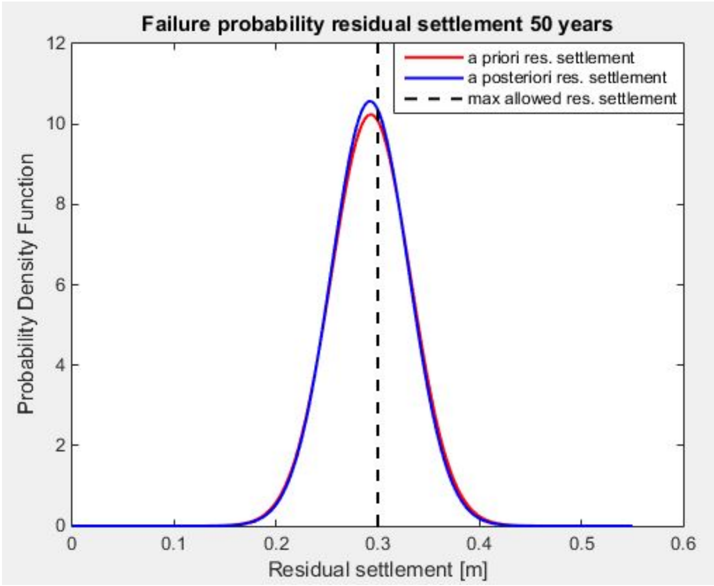

Figure H.43: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.44.

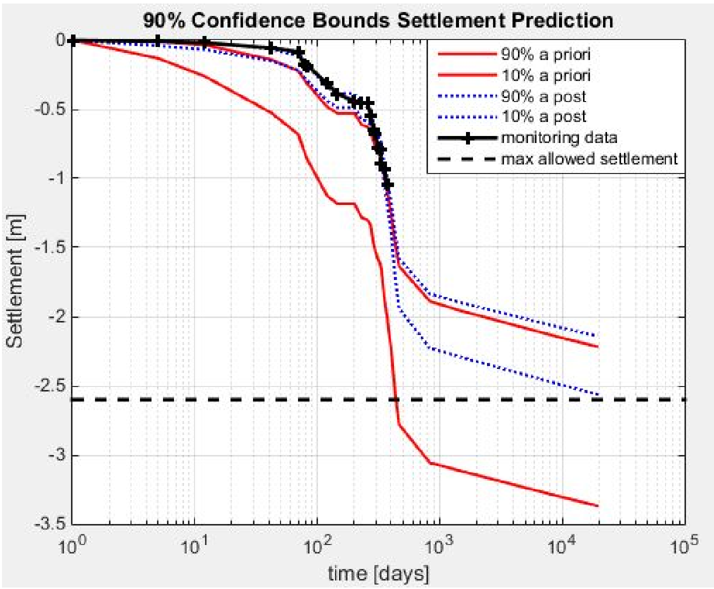

Figure H.44: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 12

Figure H.45 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

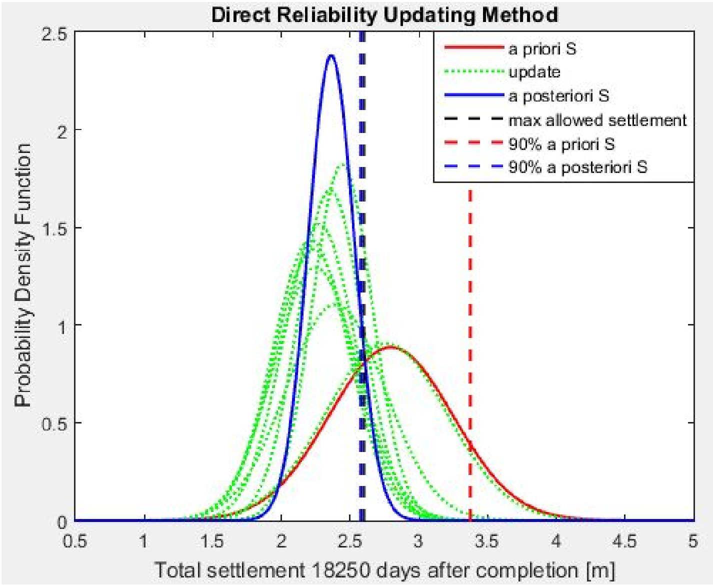

Figure H.45: Total settlement prediction, a-priori and a-posteriori

The following Figure H.46 shows the updated residual settlement one year after completion.

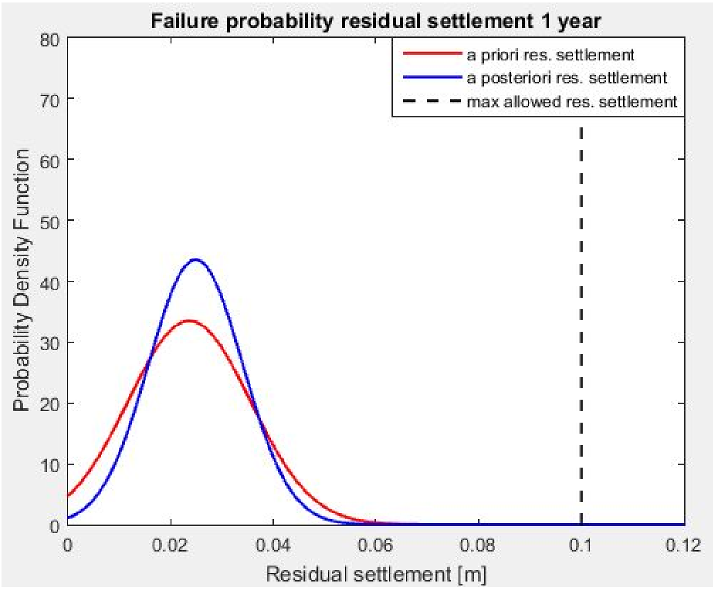

Figure H.46: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.47 shows the a-priori and a-posteriori residual settlement 50 years after completion.

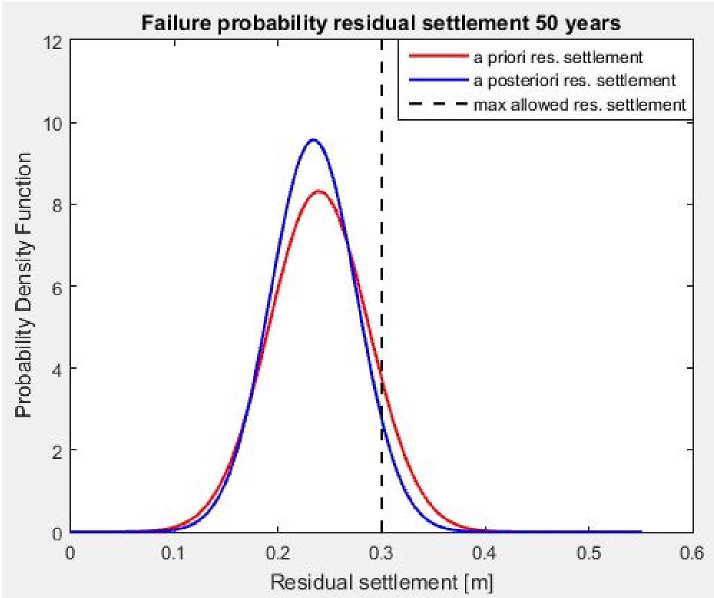

Figure H.47: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.48.

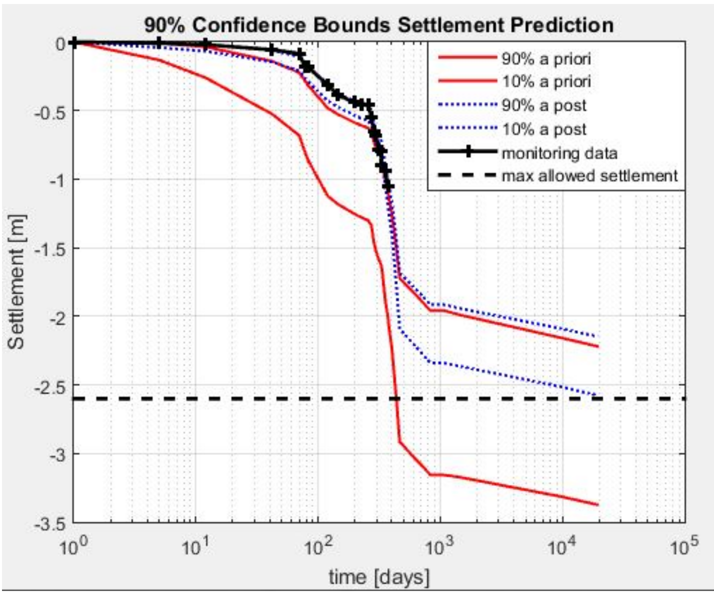

Figure H.48: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 13

Figure H.49 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

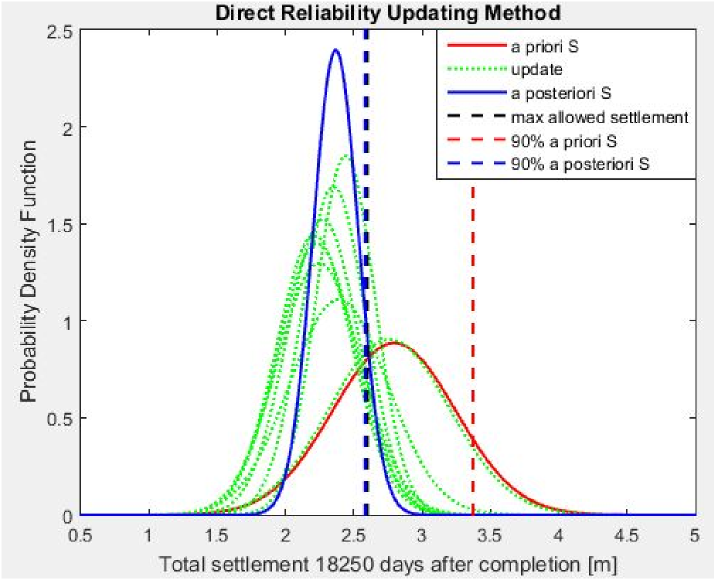

Figure H.49: Total settlement prediction, a-priori and a-posteriori

The following Figure H.50 shows the updated residual settlement one year after completion.

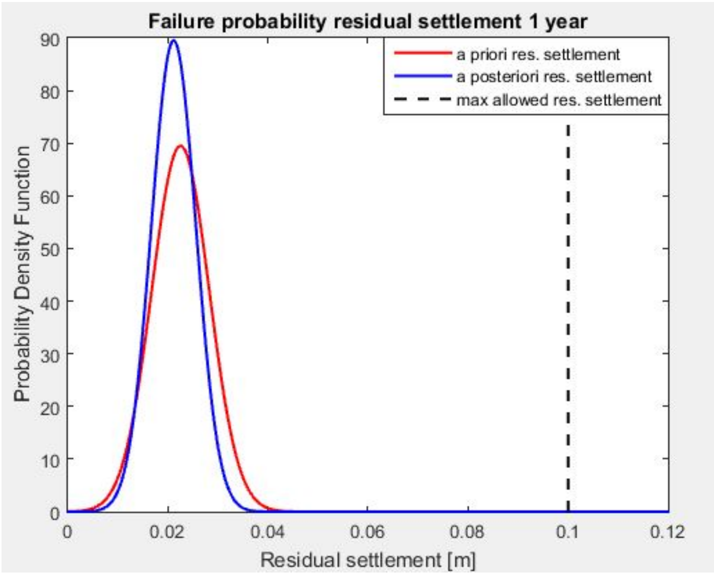

Figure H.50: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.51 shows the a-priori and a-posteriori residual settlement 50 years after completion.

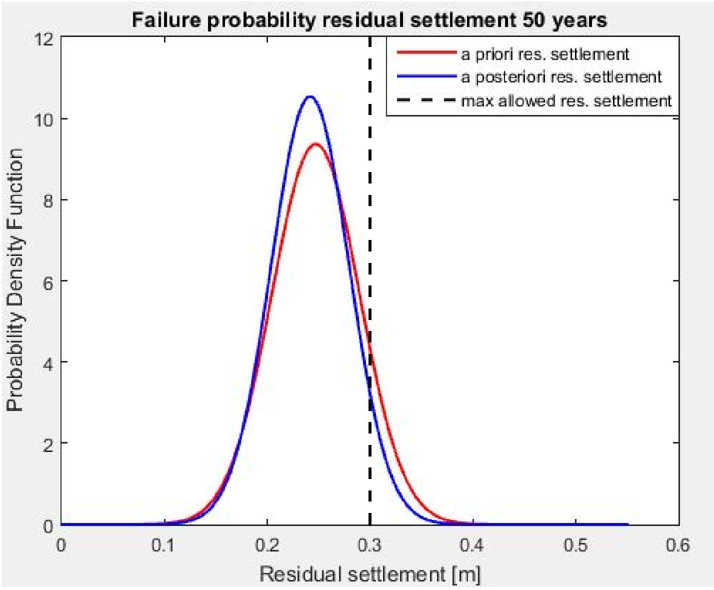

Figure H.51: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.52.

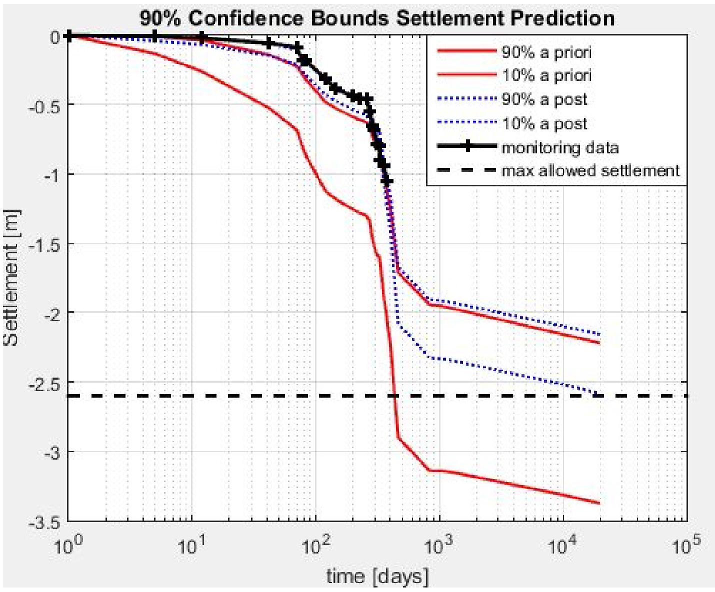

Figure H.52: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 14

Figure H.53 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

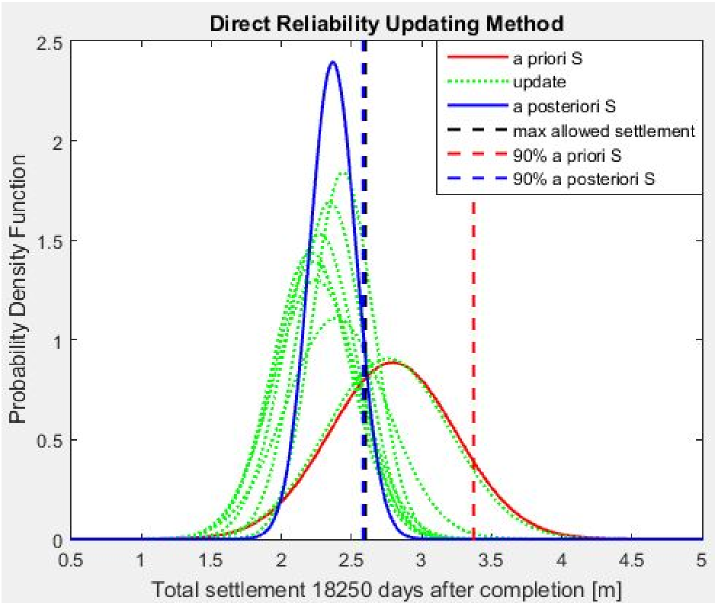

Figure H.53: Total settlement prediction, a-priori and a-posteriori

The following Figure H.54 shows the updated residual settlement one year after completion.

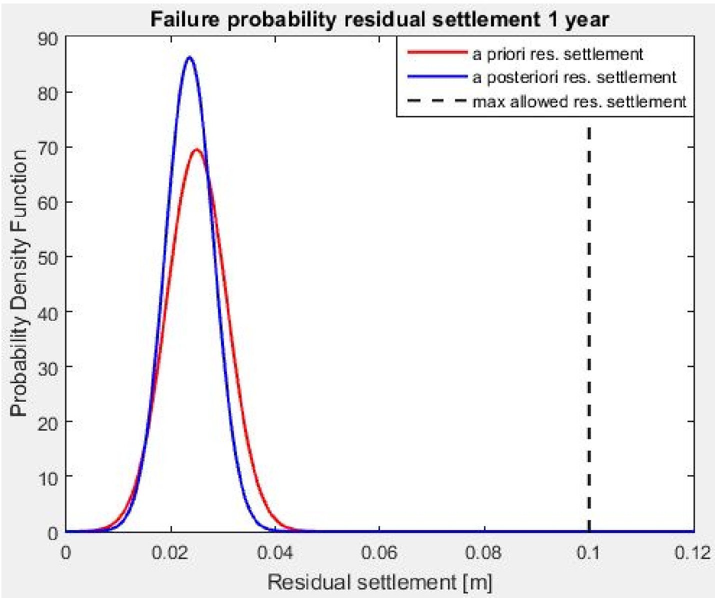

Figure H.54: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.55 shows the a-priori and a-posteriori residual settlement 50 years after completion.

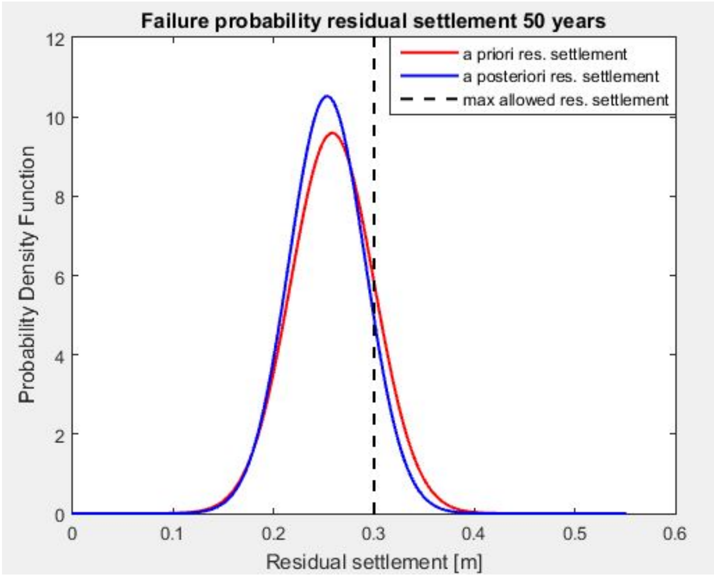

Figure H.55: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.56.

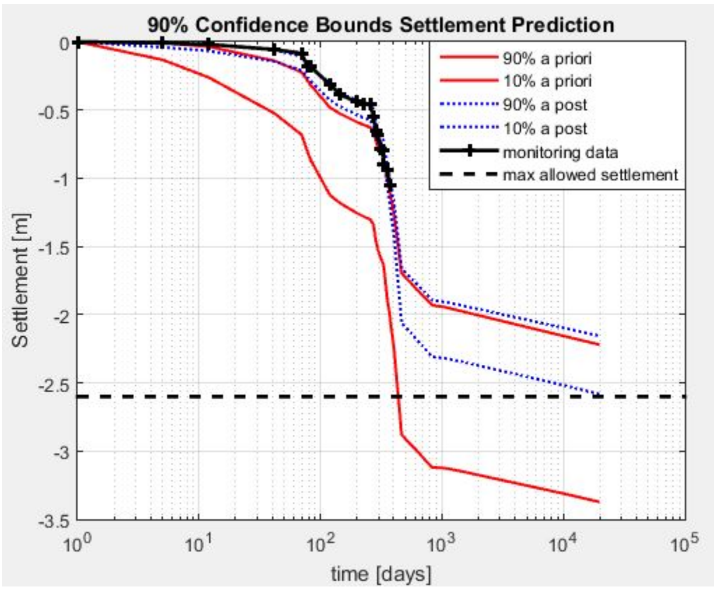

Figure H.56: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 15

Figure H.57 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

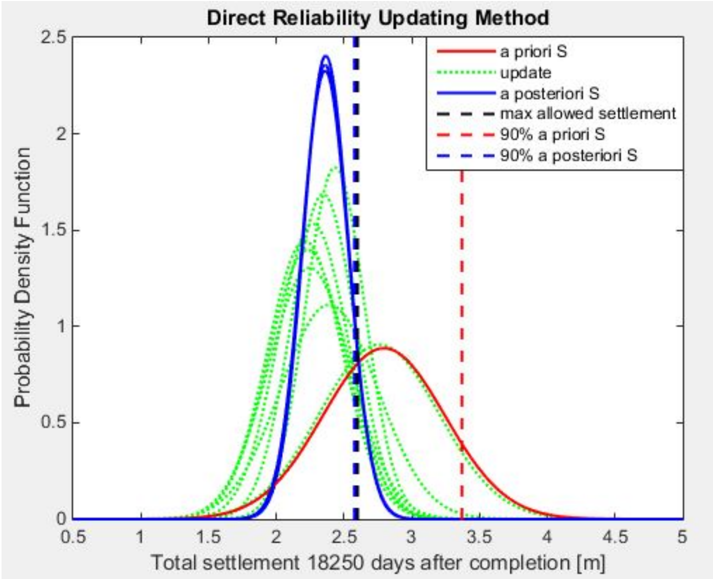

Figure H.57: Total settlement prediction, a-priori and a-posteriori

The following Figure H.58 shows the updated residual settlement one year after completion.

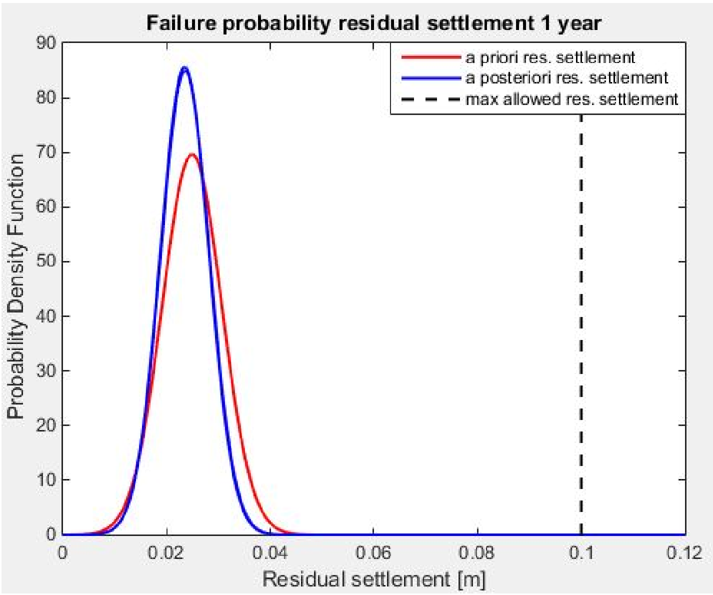

Figure H.58: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.59 shows the a-priori and a-posteriori residual settlement 50 years after completion.

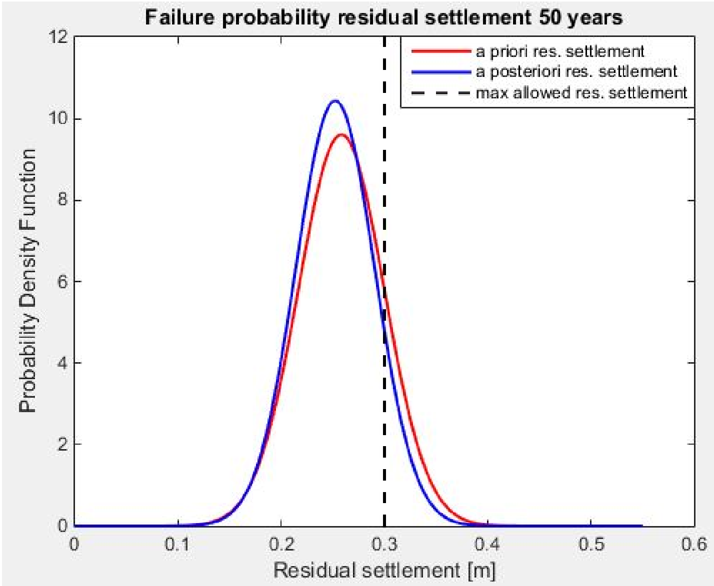

Figure H.59: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.60.

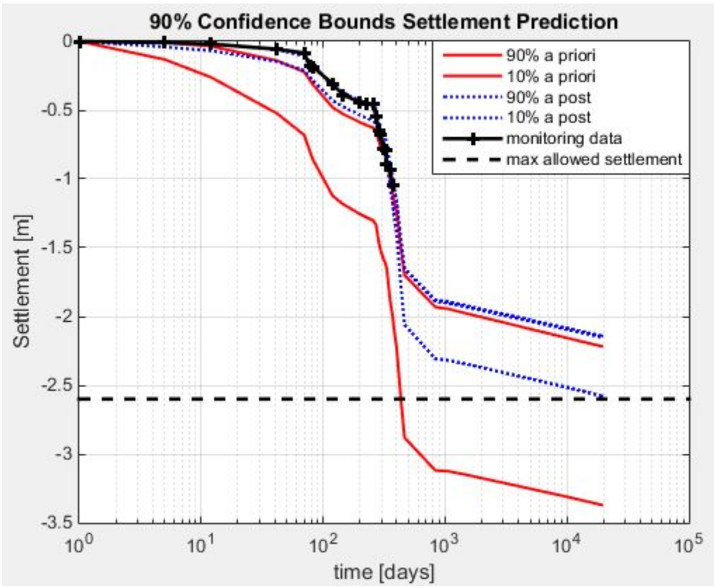

Figure H.60: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

SCENARIO 16

Figure H.61 shows the updated settlement prediction of the total amount of settlement at the end of the lifetime of the dike structure.

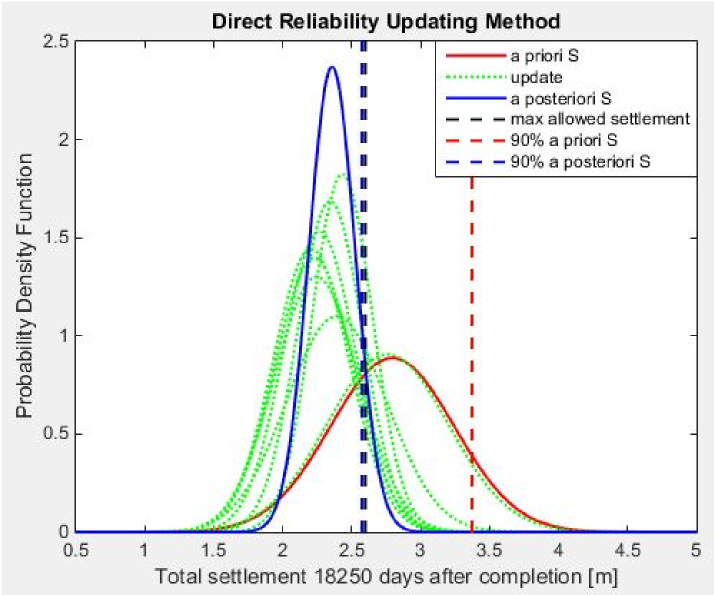

Figure H.61: Total settlement prediction, a-priori and a-posteriori

The following Figure H.62 shows the updated residual settlement one year after completion.

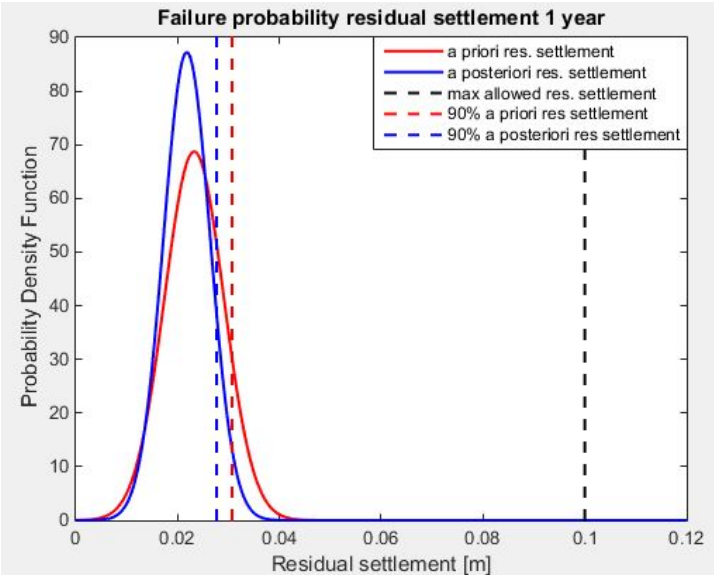

Figure H.62: Residual settlement prediction one year after completion, a-priori and a-posteriori

Figure H.63 shows the a-priori and a-posteriori residual settlement 50 years after completion.

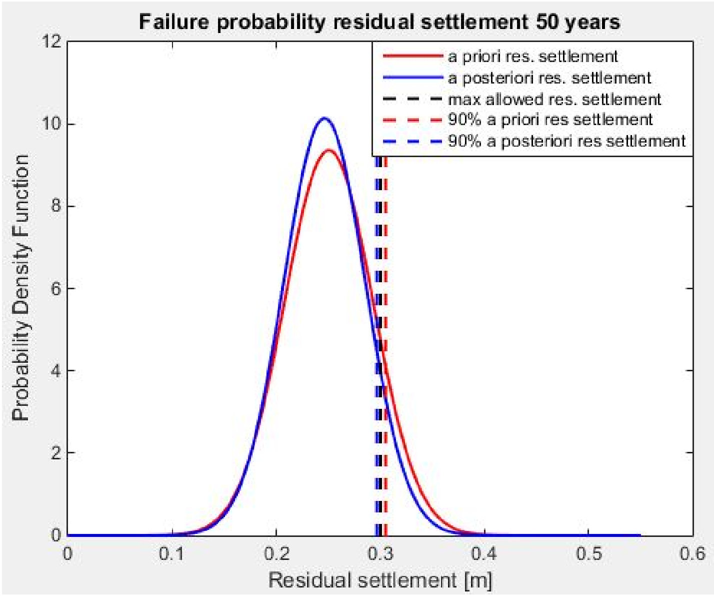

Figure H.63: Residual settlement prediction fifty year after completion, a-priori and a-posteriori

To illustrate the a-priori and a-posteriori 90% confidence intervals of the settlement in time, see H.64.

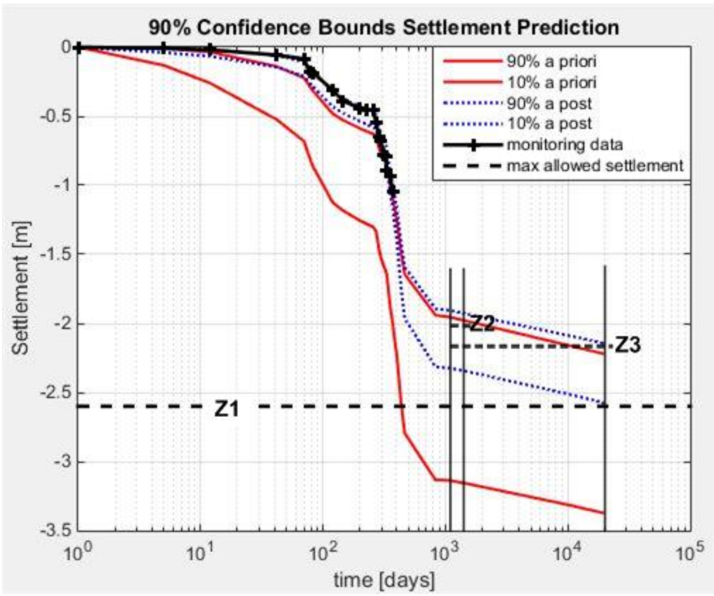

Figure H.64: Total settlement in time with a 90% confidence interval, a-priori and a-posteriori

RESULTS FAILURE PROBABILITIES PER SCENARIO

Below Table H.2 summarizes the results obtained with the Matlab model regarding to the failure probabilities of the three limit state functions.

	Failure probability total amount of settlement
a-priori	0,0827
a-posteriori	0
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,69276
a-posteriori	0,71082

Table H.2: Results scenario 1

SCENARIO 2

This scenario leaves layer 7 out of the construction and thus uses 1 m clay less than scenario 1. See Table H.3 for the results.

	Failure probability total amount of settlement
a-priori	0,65269
a-posteriori	0,072016
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,7177
a-posteriori	0,72542

Table H.3: Results scenario 2

SCENARIO 3

Table H.4 shows the results of scenario 3 where the layer thickness of the 7^{th} clay layer is 0,5 m.

	Failure probability total amount of settlement
a-priori	0,29855
a-posteriori	0,00032415
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,71139
a-posteriori	0,71831

Table H.4: Results scenario 3

SCENARIO 4

Scenario 4 combines the last two raises to one raise with a layer thickness of 1,5 m. It should be investigated if the stability requirements are met. Below Table H.5 shows the failure probabilities of the three limit state functions

	Failure probability total amount of settlement
a-priori	0,29968
a-posteriori	0
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,69068
a-posteriori	0,70411

Table H.5: Results scenario 4

SCENARIO 5

For scenario 5 a less thick 7^{th} clay layer is used, see Table H.6 for the results.

	Failure probability total amount of settlement
a-priori	0,5058
a-posteriori	0,0140505
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,71635
a-posteriori	0,73279

Table H.6: Results scenario 5

SCENARIO 6

The sixth scenario has a 7^{th} layer thickness of 0,3 m. Table H.7 shows the results.

	Failure probability total amount of settlement
a-priori	0,43363
a-posteriori	0,004648
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,71566
a-posteriori	0,72662

Table H.7: Results scenario 6

SCENARIO 7

Table H.8 shows the results of scenario 7.

	Failure probability total amount of settlement
a-priori	0,3631
a-posteriori	0,0013201
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,71342
a-posteriori	0,7294

Table H.8: Results scenario 7

SCENARIO 8

Table H.9 shows the results of scenario 8. Here again the last two raises are combined to one raise but with a smaller layer thickness, namely 1,3 m.

	Failure probability total amount of settlement
a-priori	0,43239
a-posteriori	0,003513
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,70374
a-posteriori	0,72472

Table H.9: Results scenario 8

SCENARIO 9

Scenario 9 is different from the other previous scenarios because a temporary overburden is applied in the construction method. Thirty days after applying clay layer 6 a overburden is constructed with a layer thickness of 1 m. One year later the overburden of 1 m is totally removed. This method increases the amount of settlement in an earlier stage and is advantageous for the residual settlement requirements. Table H.10 shows the results of the failure probabilities.

	Failure probability total amount of settlement
a-priori	0,65948
a-posteriori	0,093196
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,042279
a-posteriori	0,010319

Table H.10: Results scenario 9

SCENARIO 10

Scenario 10 is the same as scenario 9 but now with a temporary overburden of 0,5 m, see Table H.11.

	Failure probability total amount of settlement
a-priori	0,65505
a-posteriori	0,080354
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,24244
a-posteriori	0,19908

Table H.11: Results scenario 10

SCENARIO 11

Same as the two previous scenarios but now with an overburden of 0,3 m. See Table H.12 for the results of scenario 11.

	Failure probability total amount of settlement
a-priori	0,65404
a-posteriori	0,066263
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,4205
a-posteriori	0,4039

Table H.12: Results scenario 11

SCENARIO 12

This scenario has a temporary overburden of 0,8 m. See Table H.13 for the results of scenario 12.

	Failure probability total amount of settlement
a-priori	0,65681
a-posteriori	0,08129
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,10745
a-posteriori	0,065838

Table H.13: Results scenario 12

SCENARIO 13

This scenario has a temporary overburden of 0,7 m. This temporary overburden is together with the 6^{th} clay layer constructed, thus the last applied raise has a layer thickness of 1,7 m. Due to this thickness, stability requirements of the dike must be investigated. See Table H.14 for the results of scenario 13.

	Failure probability total amount of settlement
a-priori	0,65582
a-posteriori	0,08752
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,10957
a-posteriori	0,06793

Table H.14: Results scenario 13

SCENARIO 14

Scenario 14 also has a temporary overburden but with a thickness of 0,6 m. This overburden is constructed separately of clay layer 6. See Table H.15 for the results of scenario 14.

	Failure probability total amount of settlement
a-priori	0,6551
a-posteriori	0,07851
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,15959
a-posteriori	0,11144

Table H.15: Results scenario 14

SCENARIO 15

Scenario 15 is the same as scenario 13 but now with a temporary overburden of 0,6 m. See Table H.16 for the results of scenario 15.

	Failure probability total amount of settlement
a-priori	0,65582
a-posteriori	0,08752
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,15736
a-posteriori	0,1099

Table H.16: Results scenario 15

SCENARIO 16

Scenario 16 has a temporary overburden with a thickness of 0,7 m. This overburden is constructed separately of clay layer 6. See Table H.17 for the results of scenario 16.

	Failure probability total amount of settlement
a-priori	0,65589
a-posteriori	0,068511
	Failure probability residual settlement 1 year
a-priori	0
a-posteriori	0
	Failure probability residual settlement 50 year
a-priori	0,12467
a-posteriori	0,0751

Table H.17: Results scenario 16

I

Literature Study

Below the literature study is presented. A small summary of the most relevant articles or books is given below. The summaries describe the importance or relevance of the articles regarding the thesis project. Besides these summaries, other interesting reports, documents or articles which are less relevant but could give a small contribution to the thesis project are included in the bibliography[1] *. This chapter is just to remind the student of the available literature and the position of the thesis project within the scope of settlement predictions.*

1. Schweckendiek et al. [2014]

Author(s): T. Schweckendiek & A.C.W.M. Vrouwenvelder & E.O.F. Calle

Title: 'Updating piping reliability with field performance observations'

This article is part of the PhD research of T. Schweckendiek. It elaborates on Bayesian Updating method to increase reliability on piping uncertainties. Field performance observations (i.e. observed water levels) are used to increase the reliability. Concepts like inequality information, time (in)variance, (in)direct reliability updating and (non)-reducible uncertainty. The groundwater flow model is modeled in a probabilistic way based on Monte Carlo principles. Furthermore it shows a case study within the VNK2 project where piping uncertainties are reduced using a Bayesian Updating method. In the article it is said that the main aspect in posterior analysis based on field observations is to consider if a variable is time-invariant and, if not, how much of its variance can be attributed to the intrinsic variability. In his method, Schweckendiek assumes all basic random variables as epistemic (i.e. reducible) except the hinterland water level, the permeability of the hinterland blanket layer and model uncertainty factors are of the aleatoric (intrinsic) type of uncertainty (non-reducible). An interesting conclusion is that the more uncertainty is reducible, the larger are the effects of updating, which sounds rather logical. Another conclusion drawn in the article is that if not all resistance uncertainty is reducible, there is no guarantee that a previously survived load level will be survived again in future. One more interesting conclusion is that the effect of updating on the fragility curves is greatest close to the observation (e.g. observed water level). The last interesting note is that updating the reliability of a particular failure mechanism can lead to a conclusion that a structure is less reliable than initially estimated.

[1] Vrijling et al. [2002], Verruijt [1999], Casarin [2013], Vrijling and van Gelder [2002], Geyer [2004], Nooij and van der Giessen [2014], Den Haan [2008], Molenaar and Houben [2003], Den Haan and Kamao [2003], Ditlevsen and Madsen [2007], Rijkswaterstaat [2009], Servais [2006], Straub [2012], Box and Tiao [1992], Veraart [2014], Vrouwenvelder [2003], Sellmeijer et al. [2004], West [1986], de Wit [2001], Peng et al. [2013], Wolters [2014b], Wolters [2014c], Wolters [2014a], Everts [2015], KIS [2013], Waterschap Rivierenland [2014b], Waterschap Rivierenland [2014a], Waterschap Rivierenland [2013]

2. Calle et al. [2005]

Author(s): Ed Calle & Hans Sellmeijer & Marcel Visschedijk
Title: 'Reliability of settlement prediction based on monitoring'
The paper presents an alternative method tot predict expected mean values and standard deviations of embankment settlement, as function of time. The method is based on prior assumptions, as well as actually observed settlement behavior applying a Bayesian Updating concept. This paper focuses on the embankments needed to construct roads or railways and on the settlement on the long term in case of flood levees. The method presented in this article to update the reliability is a weighted least squares method within a Bayesian framework. This method minimizes not only the residuals between measurements and predictions, but also the residuals between the initial and the updated estimate of the mean value of the key parameters. The method presented in this paper especially focuses on making the weighted least squares method more robust and the Bayesian approach is complementary to the weighted least squares method. An interesting question raised in the concluding remarks is about for how long the settlement process should be monitored to come to a reliable solution. Another concluding remark is the assumptions made about model imperfections and measurement errors have a significant influence. From this research the function 'zakbaakfit' in the software program DSettlement is developed. The 'zakbaakfit' function works also in a Bayesian framework with the weighted least squares method.

3. Deltares Systems [2011]

Author(s): Deltares
Title: 'User Manual DSettlement, chapter 18: Special Calculations'
The software program DSettlement shows large similarities with the proposed research in the thesis project. There is an option to give input data about monitoring by settlement plates. As described in the second small summary this option in DSettlement uses the weighted least squares method. This method is a fit through the predicted and measured settlements by searching for minimum differences between the predicted and measured settlements. On top of that it also minimizes the difference between the initial value and the updated value of the fit parameters. Another option in DSettlement is the function 'Reliability Analysis' where the parameters can be given a probability distribution. Currently only the standard normal distribution can be used and the lognormal distribution is currently only available for testing purposes. Also probabilistic methods as FORM, FOSM and Monte Carlo Simulation are available in DSettlement, however IV-Infra does not obtain a license yet for this function.

4. Faber [2007a] Faber [2007b]

Author(s): Prof. dr. M.H. Faber
Title: 'Risk and Safety in Civil Engineering'
Two chapters of the lecture notes of M.H. Faber are read, namely chapter 3 and 12. Chapter 3 elaborates on the Bayesian decision approach explained step by step. It gives a clear overview of the approach of the Bayesian updating method. It also mentions three possibilities for certain situations in the updating method. The first case is the prior information strong and the likelihood weak (small sample size), in the second case the prior information and the likelihood are of comparable strength and in the last case the prior information is relatively weak in comparison to the likelihood. Chapter 12 is elaborating more in case of inspection or monitoring of structures. It gives a couple of examples for prior, pre-posterior and posterior analysis and from there the decisions to be made. The updating method is explained in case of inspections and is constructed in a Bayesian framework. Furthermore this chapter gives the difference between equality an inequality information. For updating purposes the use of inequality information gives a more straightforward approach of updating the reliability. When using equality type of information it is needed to also model the measurement errors as random variables. An important characteristic of the Bayesian approach is that always the 'engineering judgement' is included in the probabilistic model. The more measurement uncertainty the weaker is the likelihood. A couple of examples are given in the last paragraphs.

5. Schweckendiek [2010]

Author(s): T. Schweckendiek
Title: 'Reassessing reliability based on survived loads'
The paper shows how the probability of failure can be updated by applying Bayesian techniques with historical survival data (posterior analysis), it also treats how to determine the expected increase of reliability in time (pre-posterior analysis). A mathematical background is presented of reliability theory and Bayesian Updating. There are basically two different fashions for Bayesian updating: By updating the basic random variables and repeating the reliability re-determining P_f, or by directly updating P_f making use of the definition of conditional probability and the correlation between the historically observed event and the future event to be assessed. The second option is computationally less demanding, hence recommended. A consideration given in this article is: Survival information may be taken into account in design for newly built structures, using the information leads to cost-savings either in terms of lower construction cost or in terms of lower risk. The calculations in this article have been carried out mainly by means of numerical integration techniques, for even higher dimensional problems it is recommended to us for instance Markov Chain Monte Carlo sampling.

6. Straub [2011]

Author(s): Daniel Straub
Title: 'Reliability updating with equality information'
Daniel Straub presents in this article a method to update the reliability with equality information. Making probabilistic models using equality information is less straightforward then when use is made of inequality information. This is because it is hard to model probabilities with exact numbers. For instance, what is the probability of a settlement of exactly 10 cm in one year? The probability will approach zero in this case because the probability of a settlement of exactly 10 cm in one year would be very small. Modeling with inequality information gives the advantage that the data can is equal or smaller/greater than the requested probability. In this article a dummy parameter δ is introduced to cover this problem. In this way the equality information is replaced by the inequality event $\{h(x) - \delta \leq 0\}$. Furthermore it elaborates on how to use this in reliability updating methods.

7. Schweckendiek [2014]

Author(s): T. Schweckendiek
Title: 'On Reducing Piping Uncertainties'

This is the dissertation of T. Schweckendiek, the earlier mentioned articles of T. Schweckendiek in the Literature Study are supportive to the dissertation. Here some of the main findings of the research is discussed. An aspect with considerable impact is the distinction between reducible (epistemic) uncertainties and irreducible (aleatory) uncertainties. For the applications treated in this thesis, time-invariant variables (e.g., blanket thickness) essentially fall into the reducible category whereas variables modeling (intrinsic) variability in time (e.g., the water level) fall into the irreducible category. The larger the share of reducible uncertainty in a problem, the greater the effect of updating. Whether uncertainty is reducible is an input to the analysis, not an outcome. Another conclusion: it is important to point out that the proposed approach is only as good as the models and the scenarios or prior uncertainties identified by the modeler. Inherently, Bayesian posterior analysis is incapable of detecting previously unidentified conditions or mechanisms which, nevertheless may contribute to the probability of failure. In the context of monitoring data the dissertation concludes regarding field observations: The discrete uncertainties like stratification can be the most important ones causing the most significant changes. Another related common misconception is that the probability of failure up to the highest observed and survived water level is zero. This is an assumption often made in simplified survival analyses, which is only true if all resistance uncertainties were time-invariant (i.e., fully reducible). And: the assumption of time-invariant resistance can be very conservative as may lead to severe overestimation of the reliability. The following conclusion is also interesting regarding cost savings on a project: Since the difference between retrofitting and monitoring cost is usually several orders of magnitude, even a relatively low probability of meeting the target and realizing savings in retrofitting cost can make investments in monitoring attractive. An important statement that is made in the dissertation are the resistance uncertainties due to the stratification of the subsoil, these uncertainties are high sensitive for the updating method using monitoring data.

8. Den Haan [2003]

Author(s): E.J. den Haan
Title: 'Het a,b,c Isotachenmodel: Hoeksteen van een nieuwe aanpak van zettingsberekeningen'

The article presented in the journal Geotechniek is an introduction to the a,b,c Isotachenmodel and gives the comparisons and differences between the new model and the settlement models of Buisman-Koppejan. It is stated in the article that the superposition rule in the Buisman method is wrong and that the seculair effect of settlements must be explained in another fashion. Bjerrum introduced the relation between equal creep rate with the equal creep time, this relationship gives so called isotachen. Den Haan used this theory in his dissertation but instead of using linear strain it uses natural strain. The difference between the two types of strain is that the linear strain is measured from the initial state and the natural strain determines the strain incremental regarding the current state. With this approach the lack of the linear strain is overcome in the way that the strain could never be larger than the layer thickness. This is the case especially for large strains. Obtaining the parameters a, b and c can be done with the K_0-C.R.S. oedometer test. The gradient of the line of the direct strain is described by the parameter'a'. The secular strain is more complicated, this phenomenon is described by a stress dependent and a stress independent part. Parameter 'b' gives the slope of the isotachen lines and these lines are directly coupled to the intrinsic time.

9. Den Haan et al. [2004]

Author(s): E.J. Den Haan et al.
Title: 'Isotachenmodellen: Help, hoe kom ik aan de parameters'

In this article the way to obtain the isotachenparameters is explained. Both the a,b,c-isotachenparameters as the NEN-Bjerrum isotachenparameters are given with an equation. Next to the formal definitions the interrelationships between the different isotachenparameters are given. In this way the a,b,c-isotachenparameters can be written in the NEN-Bjerrum isotachenparameters and the other way around. Also there is a description about the execution of the soil compression test which can be used to obtain the parameters.

10. Heemstra [2013]

Author(s): J. Heemstra
Title: 'Met Buisman naar de Isotachen'

In his article Heemstra describes the history of the development of soil settlement prediction models. In the '30 Keverling Buisman introduced an equation based on compression tests which could predict the settlements of soft subsoil. Buisman was the inventor of the secular effect, the deformation that occurs when after an increase of loading the water pressures are disappeared (after consolidation). The secular effect is proportional to the logarithm of time. Per definition the effect is equal to zero when the time equals 1 day. There is a difference between the secular effect and the secondary effect. The secular effect describes super imposable deformation changes at changes of loading (creep changes). The secondary effect describes the deformations (creep) itself. Buisman used a linear relationship between stress and deformation. Combining this approach with the secondary effect and change the formulas in a logarithmic stress-strain relationship the isotachen method is obtained. Den Haan demonstrated that the superposition principle of Buisman and Koppejan is wrong because with each loading step there is an error is introduced. Therefore these methods cannot be divided in a couple of small loading steps because of the growing error with each loading step. However, the isotachenmodel does not have this problem.

11. Hölscher [2003]

Author(s): P. Hölscher
Title: 'Influence monitoring on reliability of predictions of settlements; Application isotache model'

The report is part of a study to search for a method to introduce the results of monitoring of settlements of an embankment into the prediction of residual settlements. The studied case in the report is the construction of an embankment for a new railway on soft soil. The settlements are predicted using the isotachen model, in this case the uncertainty of the parameters in the model are taken into account. The soil parameters as well as the loading are considered as stochastic variables and described by a mean value and the standard deviation. Use is made of a Monte Carlo simulation, in this way the predicted settlements are given a mean and a standard deviation. Implementing the monitoring data into the model the improved settlement predictions are obtained. This research is executed within the Delft Cluster project for the Rational Monitoring HerMes project. This study was carried out within the same question that arises when looking for optimizing the total amount of elevation. The case study in this report is the construction of the embankment for the Betuweroute. A challenge within this project was to construct the embankment in relative short time, while there were strict requirements regarding the residual settlements. The considered stretch of the Betuweroute finds itself in a virgin territory, which means before the construction of the embankment there has never been a load on top of the soft soil (a human induced load). The lop layer of the soft soil is removed and the embankment is constructed in 5 phases. The a,b,c-isotachenmodel is carried out with a Monte Carlo simulation, a normal or lognormal distribution is given to the soil parameters and 10.000 calculations are made. The interpretation of the results are discussed and it is noticed that there is a large difference between the mean and median settlement and the large scatter between 1000 and 10000 days. It turned out that the scatter in the consolidation coefficient makes the probability of exceeding the required residual settlement very large. With implementing the monitoring data an assumption is made that a calculated curve not more than 0.10 m may deviate. The amount of curves is therefore reduced to 83 of the 10000 curves. Until t=300 days the fit on the monitoring data is very good, after 300 days the confidence area is widening. After about 3000 days the limits of the confidence area are parallel to the mean settlement, because of the creep. A remark on this method is that the total settlement after 10000 days is hardly influenced due to the implementation of monitoring data. A conclusion is that it is necessary to begin with the monitoring of settlement right at the start of the project to get a more reliable prediction.The uncertainty in the measurements is based on engineering judgement, it is the question how much this influences the end result.

12. den Adel and Van [2002]

Author(s): Den Adel & Van
Title: 'Uitwerking K_0-C.R.S. proef, bepaling abc-parameters'
With the K_0-C.R.S. device a soil sample is charged with a constant rate of strain. From the registrations of this test the a and b parameters of the Isotachenmodel are obtained. The c parameter is obtained by the classic oedometer test. This article explains how to determine the parameters from the K_0-C.R.S. test and the oedometer test. Below a description of the device is given. The hart of the K_0-C.R.S. device is made of a oedometer ring. De test is executed with a constant rate of strain, the rate can be positive, negative or even be zero. As well as the force on top of the sample as the force at the bottom of the sample is measured. The difference between both the forces is due to the friction forces between the sample and the walls. It is assumed that the friction force is constant. When assuming the friction force is low than there is a simple approach for the mean stress in the sample. Due to deformation of the sample water will flow out of the sample at the top, here the water can flow out freely. At the bottom the sample is undrained and here the water pressure can build up. A mean water pressure is determined over the height of the sample and the difference between the total stress and the mean water pressure gives the mean effective stress of the soil sample. The horizontal force K_0 is determined by the strain gauges in the oedometer ring at half the height of the soil sample. The deformation of the sample is measured and the horizontal force is determined. The water pressure is always isotropic, and is subtracted from the total horizontal stress determined with the oedometer ring, in this way the effective horizontal stress is obtained. Temperature is an important parameter during the test because a change of temperature during the test will influence the strain gauges and therefore the measured values. The value of parameter a is determined as followed. Through the reloading part of the graph the tangent is drawn and the slope of the tangent determines a. See equation I.1.

$$a = \frac{\Delta\varepsilon^H}{\Delta ln\sigma'_v} = \frac{(\varepsilon_2^H - \varepsilon_1^H)}{ln(\frac{\sigma'_{v,2}}{\sigma'_{v,1}})} \tag{I.1}$$

The value of parameter b is obtained likewise, through the **virgin** part of the graph the tangent is taken. The slope of the tangent determines the parameter b, see equation I.2.

$$a = \frac{\Delta\varepsilon^H}{\Delta ln\sigma'_v} = \frac{(\varepsilon_2^H - \varepsilon_1^H)}{ln(\frac{\sigma'_{v,2}}{\sigma'_{v,1}})} \tag{I.2}$$

Parameter c describes the creep behaviour of the soil sample, c is measured when the constant rate of strain in the K_0 test equals zero. However in this article, from the year 2002, it is said that determining c works better with the classical oedometer test. Probably the techniques to obtain c are improved nowadays.

13. Maccabiani et al. [2003]

Author(s): J. Maccabiani & R. Spruit & J.K. van Deen
Title: 'Duurzame OnderhoudsSystematiek (DOS) voor voorzieningen op slappe bodem – onderdeel zettingsprognose'
This article presents a research to compare different methods to predict the settlements of weak sub soil in the Netherlands. Below the executive summary is given: First, a planning tool is developed with which the moment of rehabilitation of infrastructure works due to excessive settlement can be determined. The Koppejan, Isotach and Asaoka settlement model have been reviewed for use in this tool. The Koppejan and Isotach method are more accurate than the Asaoka model, but the latter is more congruous with the financial restrictions of the municipalities. Using the Asaoka model, an adequately accurate prediction can be made of the progress of settlement in time using only measurements of the settlement of the surface, thus providing an easy and cost efficient solution. In the research it is shown that the Asaoka model can be used to predict settlements during consolidation, even in very creep-sensitive soils. As soon as the process of creep becomes dominant over the consolidation process, or the settlement rate is very slow, the Asaoka method stops functioning properly. This problem can be easily solved by using a linear extrapolation on a logarithmic timescale on the last measurements. Secondly, a tool is developed that allows the quantification of the accuracy of a given settlement prediction. An accurate settlement prediction is essential in selecting the optimum method of rehabilitation. Based on other research at CROW and CUR, still in progress at the time of writing, a flow

chart is created showing step by step all actions needed in setting up an accurate settlement prediction. By using this flow chart the information that is needed in a given situation can be identified. A scoring system is set up next to the flow chart in such a way that the route taken in the flow chart determines the end score and thus the accuracy of the settlement prediction. This method is called the 'DOS-method'. The difference with other methods, like the method in development at CROW at this moment, is that this method takes in account the specifics of the location. For a project at a site with a more homogeneous subsurface getting a high score will be easier than for a project at a site with a more heterogeneous subsurface. This DOS-method was validated in one case study. It is recommended to validate the method in at least four other projects. Finally a model is presented to select the most appropriate settlement theory. Four settlement theories which are commonly used when raising infrastructure in The Netherlands were selected. These four models are the Terzaghi method, the NEN method, the Koppejan method and the Isotach method. An analysis of these theories showed that the Koppejan and the Isotach model are most useful in these situations, the NEN method and the Terzaghi method are not suitable for these situations. Theoretically the isotach model is more suitable for predicting the time-settlement behaviour after the raising of infrastructure, but there is a large experience base for the Koppejan model and a general inexperience with the Isotach model. This means that making a definite choice of one model over the other is not possible at this time. A table is presented which can be used to select the Koppejan or Isotach model to predict the time-settlement relation in a given infrastructure rehabilitation project.

14. Molendijk and Dykstra [2003]

Author(s): W.O. Molendijk & C.J. Dykstra
Title: 'Restzettingen na oplevering: Het belang van een verbeterde voorspellingskracht. Casus Betuweroute Sliedrecht-Gorinchem'

The Waardse Alliantie (A partnership of the project organization Betuweroute) was responsible for the design and construction of the part of the Betuweroute between Sliedrecht and Gorinchem. A couple of substantial cost optimizations was realised as a direct consequence of the improved prediction of settlements. It is concluded in the article that this approach already in an early stage of the project leads to more reliable prognoses and that these prognoses can be used as a steering element within the project. Two geotechnical risks were relevant for the design of the earthen structure, the first one was a low rate of levelling up the structure and the second one was the large sensitivity on creep. The subsoil at the location is made of a thick package (i.e. 10 m) of weak soil layers like 'Hollandveen' and 'Gorkum licht'. These type of soil is known for its combination of large compressibility and very low permeability that leads to extremely low consolidation coefficients. Due to the low permeability the rate of levelling up the structure was limited, however to constrain the residual settlements it is beneficial to level up as fast as possible. A method that supports a faster consolidation is vertical drainage. This method is also applied in the KIS project together with horizontal drainage. The geotechnical risk of the large amount of expected creep is enhanced with the reliability of predicted settlement calculated with the Buisman-Koppejan model, therefore the a,b,c-isotachenmodel was chosen in the project to get more reliable predictions about residual settlements. For the purpose of determining the volumes of sand and the development of the settlement in time, settlement plates were installed along the considered stretch. The distance between the settlement plates was 50 m. With the measurements an extrapolation of the residual settlement prediction was obtained. A question in the project was how to deal with the heterogeneity of the subsoil in longitudinal direction, because only every 50 m a settlement plate was installed. Therefore Hölscher Hölscher [2003] carried out calculations for indicative sections with help of Monte Carlo Simulation. Concluded from this research is that the results were especially sensitive to the consolidation parameter c_v. See the report of Hölscher for a detailed explanation of the Monte Carlo Simulation. Finally a probability of exceedance of the residual settlement criterium of 0,2% was obtained, which is quite an acceptable number.

15. Den Haan and Molendijk [2002]

Author(s): E.J. den Haan & W.O. Molendijk
Title: 'Voorspelling restzettingen met het a,b,c isotachenmodel Betuweroute, km 16.7 en km 11.7'

In 2002 a research was done to investigate if the predictions for settlements of the embankment of the Betuweroute could be made more reliable. Soil investigation, like CPT's, were done at virgin terrain to estimate the limit stress of the subsoil. The a,b,c-isotachenmodel was used in combination with monitoring data and the settlement plate fit function in MSettle (i.e. a former version of DSettlement). The eventual fit was closely to the measured data. The optimal fit, however, requires a substantial increase of the limit stress,a moderate increase in the creep factor and a significant reduction of the permeability. Especially due to the increase of the creep factor, the residual settlements conducted with the fit are larger than with the numbers used in Koppejan and NEN methods. From this research it follows that the 'fit opportunities' of the Isotachenmodel after the limit stress are much better than the 'fit opportunities' with Koppejan and NEN models. With the Isotachenmodel it is possible to fit each loading step and unloading step neatly on the measured values. Furthermore there is a quite detailed description of the determination of the soil parameters. Also the results are presented and the method of carrying out the fit on settlement predictions with measurements is extensively discussed.

16. Sipkema [2006]

Author(s): D. Sipkema
Title: 'a,b,c Isotachen: Van a,b,c tot zetting'
Sipkema's MSc thesis is about the a,b,c-isotachenmodel and how to make it understandable for practical purposes. With an extensive description of the isotachenmodel and simple examples the working of the model is explained. Also the limitations of the model are shown and the full derivation is given. The coupling to the theory of Terzaghi, Koppejan and Buisman is made. Definitions as "Aging', 'Intrinsic time', 'Consolidation', 'Direct strain' and 'Secular strain' are given. For each parameter the equations are given to determine the values of these parameters. When there is full consolidation the change of effective stress in time approaches zero and therefore the direct strain becomes zero after the hydrodynamic period. It must be noted that full consolidation is never reached because of asymptotic behaviour. As a practical choice it is assumed that the end of the hydrodynamic period is reached when having 99,4% of consolidation. Furthermore the a,b,c-isotachenmodel is in a very detailed way described, for questions regarding the model this thesis is a good reference.