Safety and Reliability – Safe Societies in a Changing World – Haugen et al. (Eds)

Time-dependent reliability in flood protection decision making in The Netherlands

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ABSTRACT: Since 2017 Dutch flood protection standards are defined as target flood probabilities that all primary flood defences have to comply with by 2050. Explicitly accounting for uncertainties in probability distributions of load and resistance is an integral part of estimating the actual flood probability. Based on such estimates, many flood defences will be reinforced in the coming years, for design lifetimes that are generally 25–100 years. Therefore it is important that we correctly take into account time-dependence of both load and resistance during the lifetime. Loads are typically uncorrelated from year to year, whereas strength parameters exhibit significant correlation over time. This correlation over time of strength parameters can significantly reduce the failure rate and increase the lifetime reliability of a flood protection structure. In this paper we show the implications of time-dependent reliability for a set of illustrative cases. We consider the effect of different degrees of temporal dependence on reliability, lifetime and relative cost savings. The cases show that for common configurations, the inclusion of time-dependent effects, especially the correlation in time of strength variables, can increase the lifetime of a flood protection structure by up to 50%.

1 INTRODUCTION

Since January 2017 the Dutch primary flood defences have to satisfy new risk-based safety standards. Based on economic risk analysis, analysis of societal risk (risk of large numbers of casualties) and individual risk (risk of dying due to a flood), allowable (i.e. target) probabilities of failure for all major flood defences have been derived (Kok et al. 2017). The failure criterion is herein defined as the loss of flood retention capacity resulting in flood of a (defined) neighborhood with an average depth of > 0.2 meters. Safety standards are generalized into main categories with annual allowable failure probabilities 1/300, 1/1000, 1/3000, 1/10000 etcetera.

These safety standards are based upon a Bayesian interpretation of probability, meaning that the failure probability should be interpreted as a state of belief. A change in the magnitude of uncertainties due to e.g. new knowledge or measurements will then cause a change in the estimated failure probability. Hence, when the safety standard is not met, reducing dominant uncertainties can be a very relevant measure.

The new failure probability requirements for flood defences are formulated as annual probabilities, implicating that for each separate year the failure probability has to satisfy the defined standard. In design this is often interpreted as that the failure probability at the end of the design life has to equal the maximum allowable probability of flooding. This is different from for instance the failure probabilities in the Eurocode, where the design criterion is expressed as both an annual target reliability and a reliability for a lifetime, e.g. 50 years (CEN 2002). Depending on the exact definition of the failure probability, the difference between annual and lifetime reliability, which will be discussed in the next section, can have significant implications.

The actual reliability of a flood protection structure can be assessed by doing probabilistic computations using probability distributions of both load and strength variables. Historically, the tools for design and assessment that were used in the Netherlands are based on a semi-probabilistic approach. Slomp et al. (2016) gives a thorough overview of the current safety assessment tools. The current assessment and design tools allow for both probabilistic and semi-probabilistic assessment and are based on an explicit coupling between (old) semi-probabilistic tools and the new probabilistic safety standards.

The quantitative assessment of failure probabilities also enables accounting for time-dependent reliability effects. Next to time-dependent (uncertain) deterioration and uncertain changes in climate, also correlations between years can be taken into account explicitly. Loads are typically independent from year to year (the maximum water level in year i is typically not conditional on the maximum water level in year $i-1$). However, the strength variables are typically correlated from year to year as these uncertainties are merely caused by spatial variability in combination with limited knowledge. Incorporating this correlation could have significant impact on the assessment of reliability during the lifetime.

Currently there is little attention for the actual source of the uncertainty, which poses a problem when using concepts of time-dependent reliability. For instance for hydraulic load models, model uncertainties are used for water level, wave height and wave period, but the source of these uncertainties is not immediately clear. Also in the distributions for strength parameters, there can be significant uncertainty, especially for geotechnical failure mechanisms. For instance, the failure probability for piping is dominated by uncertainty in permeability and grain size (Jongejan and Maaskant 2015). Strength uncertainties can typically consist of natural variability, measurement uncertainty, transformation uncertainty or model uncertainty (Phoon and Retief 2016). The source of the uncertainty is important for two main reasons:

- It determines the optimal method for uncertainty reduction: some methods might have the same source of uncertainty. These will not (efficiently) increase the quality of available data and hence not reduce uncertainty nor improve the reliability estimate;
- It determines the amount of time dependence of subsequent years as some uncertainties might be (fully) correlated in time and others may not.

In this paper we explore different definitions of reliability that can be used for flood defences. Using illustrative cases with different degrees of uncertainties we illustrate the influence of these definitions and how that translates to the lifetime of flood defences and their life cycle costs.

2 METHODOLOGY

2.1 *Time-dependent reliability*

Flood defences are generally constructed for design periods of 25–100 years which, in the context of annual failure probability, implicates that in any given year in such a period the reliability should be higher than the requirement. In reliability engineering in general, concepts such as the survival time, failure rate and hazard function are used to characterize the temporal reliability (Kottegoda and Rosso 2008). Especially the hazard function is of interest, as this provides the failure rate of the system, this is conceptually shown in Figure 1. Here three phases are distinguished for the hazard rate of a system:

- The inception phase: here the hazard rate decreases as due to first experiences and quality control errors are corrected. One could say that at $t_0 = 0$ the constructed system is accepted.
- The phase where neither initial errors, nor deterioration play a role. In Kottegoda and Rosso (2008) this is denoted the *useful life*.
- The deterioration phase where deterioration of the system causes the hazard rate to increase significantly.

The inception phase for a flood defence has two major aspects: first of all there is the experience from initial performance, mainly during construction, that improves the reliability as instantaneous repairs are carried out. In this paper we consider flood defences that have just been delivered, so this phase is not considered. Secondly there is the dependence of failures on preceding years, meaning that if a dike doesn't fail and there is any kind of correlation between the years it yields some information on its performance. This is an effect that will be relevant during the entire life-cycle.

When also considering the other two phases, the distinction between the three phases doesn't fit that well for flood defences. Most of the deterioration processes are gradual and play a role during the entire life-cycle (see e.g. Buijs et al. (2009),

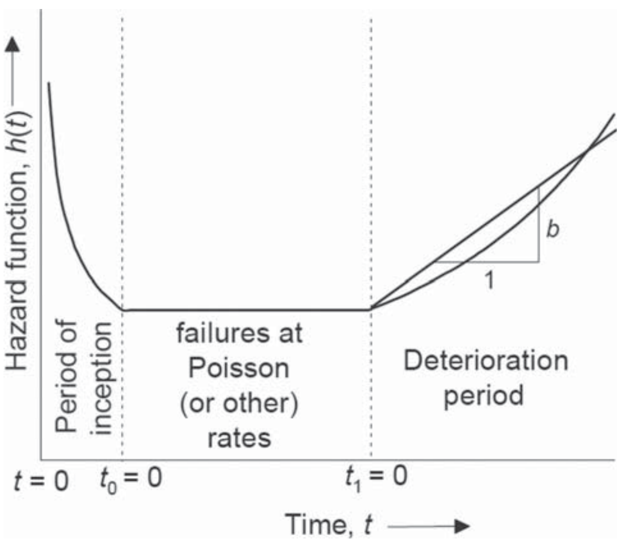

Figure 1. Hazard rate with distinction of three phases (Kottegoda and Rosso 2008).

Speijker et al. (2000)). In practice this means that there is no second phase ("failures at Poisson (or other) rates"), but that after delivery the reliability simultaneously decreases due to deterioration and increases due to information from non-failures. Whether and how these processes are taken into account depends on the definition of the failure probability that is used. For the failure probability in year t $P_f(t)$ the three main ones are:

1. $P_f(t)$ which means that the failure probability in year t is independent from the failure probability in other years.
2. $P(f_t \cap \overline{f}_{1..t-1})$ which denotes the probability of failure in year t and no failures occurred in the previous year.
3. $P(f_t \mid \overline{f}_{1..t-1})$ which denotes the probability of failure in year t given that no failures occurred in the previous year.

where f_t denotes failure at t and $\overline{f}_{1...t-1}$ denotes no failure in the period $1 \ldots t-1$. It has to be noted that most failure rates considered in literature assume a constant failure rate, which is in fact comparable to the first definition. The second and third are best compared to a description of a Decreasing Failure Rate (DFR) as described by Finkelstein (2008). However in order to better connect to current flood defence reliability practice a slightly different description is chosen here.

The choice of definition is dependent on the application and on the specifics of the situation. For cases where either the correlation between $P_f(t)$ and $P_f(t-1)$ is small and/or $P_f(t)$ is small equation 1 holds, and there is little difference in the three definitions.

$$P_f(t) \approx P\left(f_t \cap \overline{f}_{1..t-1}\right) \approx P(f_t \mid \overline{f}_{1..t-1}) \tag{1}$$

For all other cases the first definition is conservative.

Also it has to be noted that dike reinforcements generally do not entail a complete renewal, but rather an improvement of an existing flood defence. This means that part of the flood defence has already passed the inception period (as well as the other periods) and has to some degree proven itself. For this paper we consider a completely new flood defence and do not take that consideration into account although it can be very important if part of the dominant uncertainty is in a part of the dike body that has existed and survived for multiple decades or centuries.

2.2 *Temporal dependence in life-cycle reliability*

We consider a simple reliability problem where the limit state function at time t is given by:

$$Z(t) = R(t) - S(t) \tag{2}$$

with R the resistance and S the strength. In such a case, if we assume the limit state function can be approximated as a linearized hyperplane, the limit state function can be written as:

$$Z(t) = \beta(t) - \alpha_R(t)u_R(t) - \alpha_S(t)u_S(t) \tag{3}$$

where $\beta = \Phi(1 - P_f)$, where $\Phi(\cdot)$ is the inverse standard normal distribution. α_R and α_S are the influence coefficients of the random variables, indicating the respective contribution of their uncertainty towards the failure probability. u_R and u_S are random variables.

If we want to calculate the temporal reliability according to definitions 2 and 3 in the previous section, we need to take into account the correlation between subsequent years. The correlation of a component of the system in equation 2 is defined by:

$$\rho(Z_t, Z_{t-1}) = \alpha_{R,t}\alpha_{R,t-1}\rho_R + \alpha_{S,t}\alpha_{S,t-1}\rho_S \tag{4}$$

where ρ_R and ρ_S is the autocorrelation for the random variables of strength and load. For the strength, provided that there is no deterioration it could be argued that $\rho_R = 1$, the load is independent each year so $\rho_S = 0$.

For combining correlated components one could use numerical integration or probabilistic techniques such as Monte Carlo, but a very fast and efficient method is the Equivalent Planes Method, which is extensively described by Roscoe et al. (2015). In Roscoe et al. (2015) it is shown that this method is accurate for most cases, although some accuracy is lost for very large systems and for very strong correlations. However as the load is uncorrelated and values for α_R^2 are typically at most 0.7, such high values for the correlation will not be encountered when studying temporal reliability of flood defences.

The Dutch flood defense act allows for all three definitions of the previous section to be applied. The Equivalent Planes method therefore provides a fast and reliable method for evaluating the second and third definition of the annual failure probability. For the assessment of existing structures the third definition is most sensible, as in such cases it would be desirable to take into account that the structure didn't fail in the previous years, as has been done in for instance Schweckendiek (2014) and Schweckendiek et al. (2017). The third definition fits best with that. The second definition can be used for design purposes, as it is sensible to not account for the probability of failure in year t when the built structure has already failed in year $t-1$.

It has to be noted that for small probabilities of failure the second and third definition are almost the same as it follows from the definition of conditional probability that the difference between the two definitions for year t equals $1/\left(\prod_{t=1}^{n} 1-P_{f,t}\right)$. This indicates that for small P_f the difference will be negligible.

In order to combine different years, it is important that correlations in time between the limit state function from year $i-1$ to year i are correctly estimated. In many cases the reliability problem will not be so easy as the previously described problem, but will consist of many random variables that are (partially) correlated in time. In order to properly determine the temporal correlation of parameters and uncertainties it is important to classify uncertainties based on their original source, as only then a reliable classification can be made. This is further explained in the following section.

2.3 *Uncertainty in flood defence reliability*

There are various sources of uncertainty in flood defence reliability assessments. These are categorized by Gelder (2000) as inherent (aleatory) in time and space and knowledge (epistemic) uncertainty due to model and statistical uncertainty.Other categories can be used as well, e.g. Walker et al. (2003) distinguishes between different levels of uncertainty, and how these influence a decision problem. In general a distinction is often made between reducible and irreducible uncertainty as these influence the optimal action to deal with unacceptable failure probabilities (see e.g. Slijkhuis et al. (1997) and Schweckendiek (2014)). Inherent uncertainties are typically considered irreducible, where as knowledge uncertainty is considered reducible. This framework works well for typical loads on flood defences (a better model reduces model uncertainty but inherent natural variability in annual maxima of river discharges remains irreducible), but is less trivial for strength uncertainties. The strength uncertainty of flood defences mainly arises due to heterogeneity of the subsoil and dike body combined with limited knowledge of this subsoil, in combination with imperfect models describing the strength of the flood defence. In here, most uncertainty could theoretically be reduced but the question is more whether it is economically feasible to do so than whether it is technically possible (Schweckendiek 2014). It is therefore more applicable to use the classification of Phoon and Retief (2016) where geotechnical strength uncertainties are split into natural variability, measurement uncertainty, transformation uncertainty and model uncertainty. All of these uncertainties can be reduced to some extent, but each requires a different measure. For instance, if the source of uncertainty in Pre-Overburden Pressure (POP) is mainly natural variability, more measurements could be applied. However, if the source is a old and inaccurate measurement method, a more accurate method should be applied as there will also be a lot of measurement uncertainty. Hence it is important to systematically distinguish the main uncertainties based on their original source.

When doing a time-dependent reliability analysis this becomes even more important, as some uncertainties (mainly epistemic strength uncertainties and model uncertainties) will be correlated in time, whereas aleatory uncertainties are not. In order to correctly apply the notion of non-failure in preceding years, these uncertainties should be clearly distinguished. In this paper we will focus on the influence of temporal correlation on time-dependent reliability: it has to be noted that also spatial correlation can be used as information. For instance if the same model is used for different dike sections, and model uncertainty is the dominant parameter, failures and non-failures at location A might provide information on the reliability at location B. However this is out of the scope of this paper.

3 RESULTS

3.1 *Case description*

In order to investigate the effects of different formulations of temporal reliability and the influence of different values of uncertainty and correlation for different values of the reliability index we use fragility curves to describe the strength of a flood defence. This is a broadly used method of aggregating failure probabilities from more complex failure models (see e.g. Bachmann et al. (2013) and Schweckendiek et al. (2017)). The fragility curve expresses the critical height h_c which is an integration of the joint probability of the strength given a certain water level, resulting in the following limit state function

$$Z = h_c - h \tag{5}$$

where h is the water level and h_c the critical height. This approach is sound as long as the water level is (strongly correlated to) the dominant load for the mechanism. In this case we consider flood defence reliability described by the aforementioned limit state function where it holds that h_c and h are normally distributed. The Equivalent Planes method requires information about the influence coefficients (α_{ij}) of all i random variables per year j, reliability indices (β_j) for each year j and, as autocorrelations are constant in time, a correlation matrix with dimensions $i*i$. Using this method we can then combine the non-failure and failure events for subsequent years.

3.2 *Example 1: Life-cycle reliability of a dike without deterioration*

First we investigate the life-cycle reliability of a dike without deterioration, so constant h_c. In this case the value for $P_f(1) = ... = P_f(t-1) = P_f(t)$. For the temporal autocorrelation it holds that the strength is fully correlated $(\rho_{h_c} = 1)$, which is typical for many strength parameters of flood defences. The loads are uncorrelated from year to year $(\rho_h = 0)$, as the maximum water level in year i is typically independent of the maximum in year $i-1$. In the examples we will only consider the second definition for annual reliability index which is $\beta(f_t \cap \overline{f}_{1..t-1})$, as the difference with $\beta(f_t \mid \overline{f}_{1..t-1})$ is very small. For instance, if we assume that $\beta = 3$ and $\alpha_{h_c}^2 = \alpha_h^2 = 0.5$, the relative difference in β after 100 years is only 0.5%.

As h_c is assumed to be fully correlated in time, and h is fully uncorrelated, the influence coefficients will have a significant influence on the difference between $\beta(t)$ and $\beta(f_t \cap \overline{f}_{1..t-1})$.

Figure 2 shows the relative change in reliability index β for different values of α_{h_c} for $\beta = 3$. As expected it is observed that for higher values of α_{h_c} the difference is larger. Typical values for the influence coefficient of the strength for failure due to overflow are very small (order of 0.1 or 0.2), but for geotechnical failures these are often in the order $\alpha_{h_c} = 0.75$, meaning that for a lifetime of 50 years the various definitions of the reliability yield a difference in resulting reliability index of 10%. In terms of failure probability this is approximately a factor 3, which is equal to the difference in safety standard for two subsequent categories as defined in the law (e.g. 1/300 to 1/1000). Another important fact is that the reducing effect diminishes over time, which can be explained from the change in α over time, see Figure 3. The fact that the influence coefficient of the correlated variable reduces in time makes intuitive sense as more of the same information will result in increasingly less new insight.

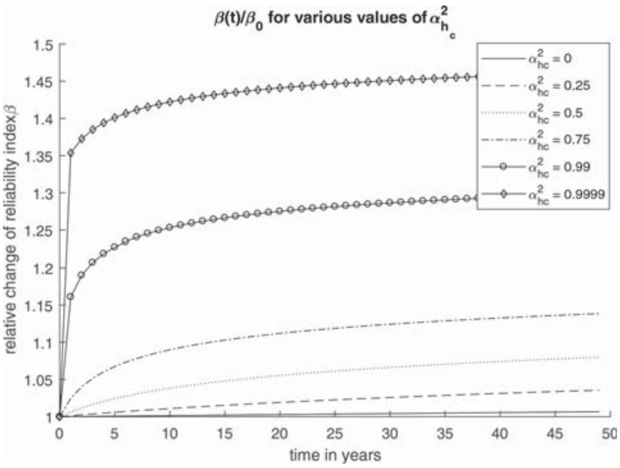

Figure 2. Relative change in $\beta(t)$ for various values of the time correlated α_{h_c} and $\beta(t = 0) = 3$.

A last important investigation of this simple case is the level of correlation. As was argued in the preceding sections it is important to distinguish different parameters with different uncertainties and different temporal correlations. However,

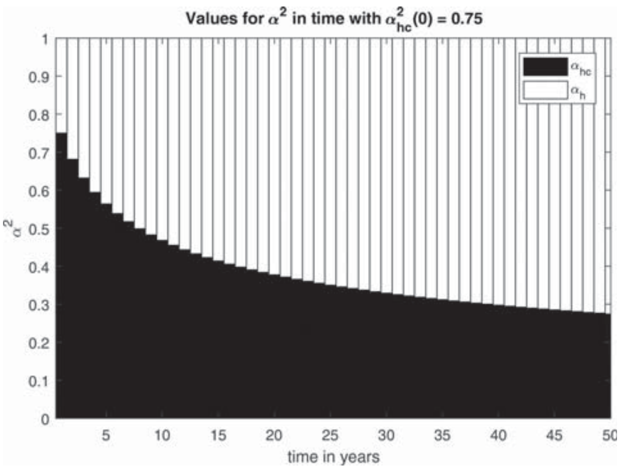

Figure 3. Change in α^2-values over time for a case with $\beta(t = 0) = 3$.

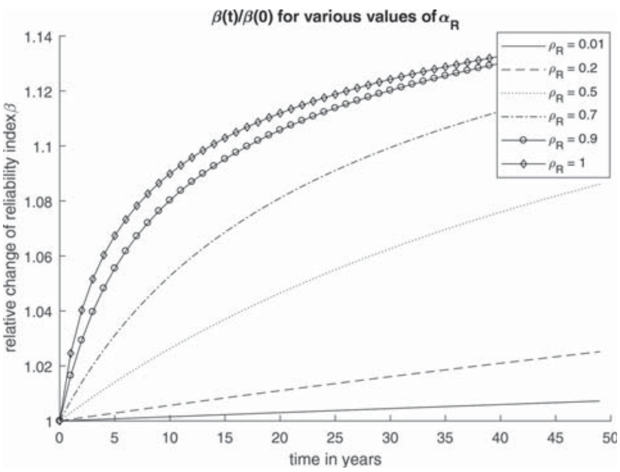

Figure 4. Relative change in $\beta(t)$ for various values of the correlation coefficient ρ_{h_c} and $\beta(t = 0) = 3$.

Table 1. Parameters for Example 2.

Variables	Distribution	Parameters	
h_c	$N(\mu, \sigma)$	4.5	1.05
H	$N(\mu, \sigma)$	0	0.6
Δh_c	$\Gamma(\eta, \delta)$	$\frac{1}{var_{\Delta h_c}^2}$	$\frac{1}{\mu_{\Delta h_c} * var_{\Delta h_c}^2}$

Figure 4 shows us that the influence of having a ρ_{h_c} that is slightly smaller than 1 is not that influential on the $\beta(t)$: even for a ρ_{h_c} of 0.7 values close to the ones for full correlation are found. So even if there is a small error due to e.g. combining two (uncertainty) parameters with different time correlations, the influence on the result will often be relatively small.

3.3 *Example 2: Life-cycle reliability of a dike with deterioration*

In practice it will not occur that a dike will remain the same for 50 years. The most common deterioration mechanism for dikes is settlement, which can be described by parametric models (see e.g. Buijs et al. (2009)) or stochastic process models such as the Gamma Process (Pandey and van Noortwijk 2004)). Here we use such a Gamma process. We introduce a new random variable Δh_c which denotes the change in critical height compared to the first year. For the sake of the example we make an important simplification here as we assume that the critical height is fully dominated by the initial crest height and its settlement. For many (geotechnical) failure mechanisms this is not the case, and other types of deterioration will be more dominant. We assume that the average annual settlement is 2 cm with a coefficient of variation of 30%. By splitting the variables we can maintain that $\rho_{h_c} = 1$. We assume $\rho_{\Delta h_c} = 0$ as it is a random process, this leads to the following distributions for the random variables:

The choice of the distributions is such that for the initial situation $\alpha_{h_c} \approx 0.75$, comparable to the first example. The initial $\beta \approx 3.7$. It has to be noted that while we attribute the temporal change to settlement in this case (i.e. decrease in strength), it could also be attributed to an increase in load, for instance due to climate change. The behaviour of such a parameter would be similar: increasing in time with increasing uncertainty.

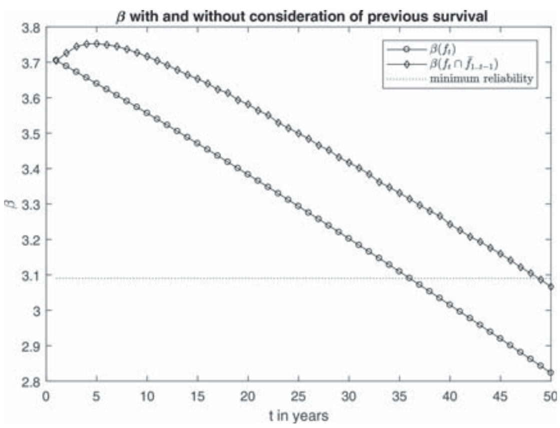

Figure 5. Values for different definitions of β with deterioration

Figure 5 shows the results for the time-dependent reliability. Here it can be seen that for this case the influence of the definition is rather large: when we compare to a minimum required reliability $\beta = 3.09\left(P_f = 1/1000\ yr^{-1}\right)$, the expected extended life when taking into account survival is approximately 15 years (or: an extension of the lifetime by almost 50%).

This amount of lifetime extension however is dependent on the rate of deterioration, and especially the uncertainty in deterioration. Figure 6 shows the α^2-values for two rates of deterioration, on the left is the same as used in Figure 5, the right is a distribution with higher variation and slightly lower mean, such that $\beta(f_{50})$ is equal for both cases. However for the deterioration with high variation $\beta\left(f_{50} \cap \overline{f}_{1..49}\right)$ is significantly smaller than for the case with smaller variation. This can be explained by a smaller $\alpha^2_{h_c}$ in the design point, meaning that the influence of that uncertainty on the reliability is smaller, resulting in less valuable non-failure information.

3.4 *Economic implications of time dependent reliability*

Generally the goal of flood defence management is to maintain flood defences at a desired level of reliability, against acceptable costs. In many cases Life Cycle Costing (LCC) is used to evaluate costs in time, for which the principles were first reported by Samuelson (1937). In an LCC analysis the Net Present Value, which denotes the value in current day prices, is calculated using the following formula:

$$NPV = \sum_{i=1}^{t} \frac{C_i}{(1+r)^i} \tag{6}$$

where, C_i is the total cost in year i, r is the discount rate and t is the evaluation period. One of the major implications of this economic theory is that postponing an investment yields significant benefits. For instance: if we postpone an investment by 10 years, assuming a discount rate of 3%, the current stand-

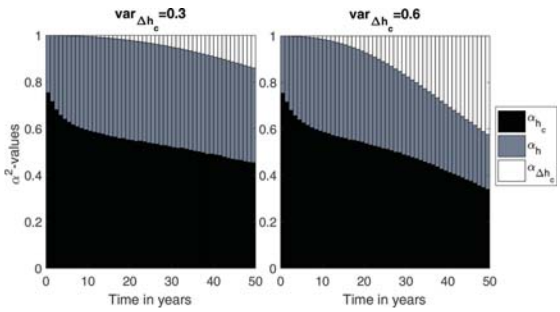

Figure 6. Change of α^2 in time for different rates of deterioration.

ard in the Netherlands (Werkgroep Discontovoet 2015), the cost after ten years is only 75% of the cost, expressed in present day prices. A disadvantage of using LCC is that it is slightly harder to compare investments with different lifetimes. For such comparisons the NPV can be expressed as Equivalent Annual Cost (EAC), which is calculated using the following formulas (Schoemaker et al. 2016):

$$EAC = \frac{NPV}{A_{t,r}} \tag{7}$$

$$A_{t,r} = \frac{1-(1+r)^{-t}}{r} \tag{8}$$

where $A_{t,r}$ is the Annuity factor for year t and discount rate r, which denotes the sum of the discount factors compared to $t = 0$.

In Figure 5 it was shown that the definition of time-dependent reliability can have a significant influence on the lifetime of a structure. To further investigate this we consider 4 situations, and generate distributions for the initial strength h_c corresponding to a wide range of α-values. The 4 cases contain 3 cases (1, 2 and 3) with different reliability requirements and 1 case (case 4) with an adapted uncertainty for the settlement (similar to the comparison of different deterioration rates in the previous section). The different cases are summarized in Table 2. It is expected that the case with low target reliability has the largest life extension, as here a non-failure is more relevant than for the case with a very high reliability. Also, based on the α-plots in Figure 6, where we saw a more rapid decrease in α_{h_c} for a higher variation of the settlement, we would expect the increase in lifetime for the case with small variation in settlement to be larger.

Figure 7 shows the extension of the lifetime in years for the 4 considered cases. Here it can indeed be seen that for a lower reliability larger extensions are gained, and that for lower uncertainty in deterioration the effect of non-failures in preceding years is also larger. It has to be noted that the lines are a bit wobbly, which is due to the fact that the reliability is determined per year, resulting in discrete steps and therefore small wobbles. However the lifetime extension doesn't directly translate into financial benefits. Therefore in Figure 8 the relative savings following from the postponement of a new reinforcement are shown for a discount rate of 3%. Here it follows that in the most extreme case (high $\alpha^2_{h_c}$ for Case 2) the relative savings can amount up to a factor 3. These relative savings are independent of other investments during the life cycle, and also independent of the actual reference year as the relative savings are linear in time (due to the exponential character of the discount rate). For instance for assessments of existing structures this would be a relevant value, as the reference year wouldn't be 0 but somewhere between 0 and the end-of-life. However in design decision making the change in Equivalent Annual Costs is more relevant, as this denotes the economic yearly cost for a design option. If we assume that at t = 0 a reinforcement is made for a lifetime of 50 years, for a discount rate of 3% a reduction in Equivalent Annual Cost of approximately 12.5% is realized for a lifetime extension to 70 years, which is in accordance with Figure 8. Obviously this will not hold for all flood defences in the Netherlands, but mainly in the

Table 2. Cases for analysis of lifetime extension.

Case	$\beta(P_f)$	$var_{\Delta h_c}$
1	$2.326(10^{-2})$	0.3
2	$3.090(10^{-3})$	0.3
3	$3.719(10^{-4})$	0.3
4	$3.090(10^{-3})$	0.05

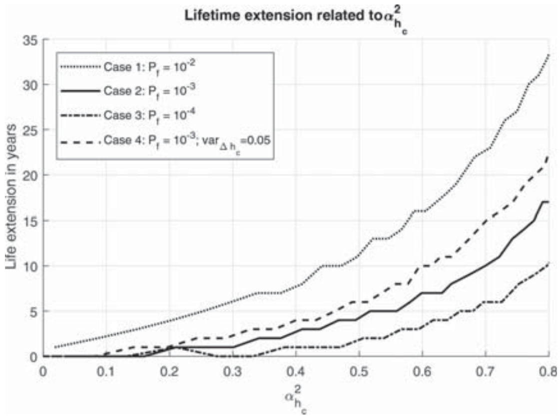

Figure 7. Life extension in years for different α_{h_c}-values for the 4 considered cases.

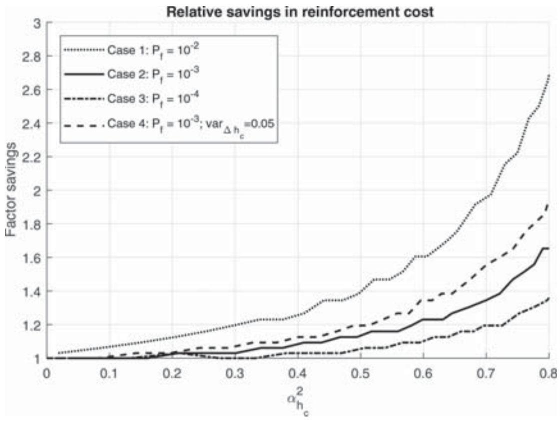

Figure 8. Factor of relative savings for the reinforcement cost for the 4 considered cases

riverine area failure probabilities are dominated by geotechnical failure mechanisms, meaning that such lifetime extensions will not be uncommon.

4 CONCLUSIONS AND RECOMMENDATIONS

In this paper we have explored various aspects of temporal dependence in reliability of flood defences. As flood defence reliability is often determined by highly uncertain but temporally correlated strength parameters, non-failures in previous years constitute information that can improve the estimate of the strength, especially when the estimated reliability is low. In this paper we've explored some parametric cases where it is shown that the savings due to accounting for non-failures rather than the commonly used conservative approach of disregarding temporal dependence in reliability can be significant (up to 20 years in lifetime extension). This is of relevance for many aspects of flood defence management such as design guidelines as well as assessment of existing flood defences where often low reliabilities are found from models. It has to be further investigated for different decision problems in flood defence management what the consequences of accounting for this temporal dependence are, especially by looking in more detail into the sources of uncertainty of actual dike reliability analyses. However, based on the considered cases it is expected that it can significantly improve reliability estimates in assessment and design, especially when there is large uncertainty on strength parameters that are correlated in time. It has to be noted that in such cases it might be necessary to improve the knowledge on strength parameters through obtaining additional information. Due to the high strength uncertainties that are often encountered, the consideration of the value of improved information is one of the major decision problems to be studied for flood defence management.

ACKNOWLEDGEMENTS

This work is part of the Perspectief research programme All-Risk with project number P15–21, which is (partly) financed by NWO Domain Applied and Engineering Sciences.

REFERENCES

Bachmann, D., N.P. Huber, G. Johann, & H. Schüttrumpf (2013). Fragility curves in operational dike reliability assessment. *Georisk 7*(1), 49–60.

Buijs, F., J. Hall, P. Sayers, & P. Van Gelder (2009). Timedependent reliability analysis of flood defences. *Reliability Engineering & System Safety 94*(12), 1942–1953.

CEN (2002). Eurocode—Basis of structural design. *En 3*(December 2008), 89.

Finkelstein, M. (2008). *Failure Rate Modelling for Reliability and Risk* (Springer S ed.), Volume 2008. Springer London.

Jongejan, R.B. & B. Maaskant (2015). Quantifying flood risks in the Netherlands. *Risk analysis: an official publication of the Society for Risk Analysis 35*(2), 252–64.

Kok, M., R. Jongejan, M. Nieuwjaar, & I. Tanczos (2017). Fundamentals of Flood Protection.

Kottegoda, N. & R. Rosso (2008). *Applied statistics for civil and environmental engineers* (2nd ed.). Blackwell Publishing Ltd.

Pandey, M.D. & J.M. van Noortwijk (2004). Gamma process model for time-dependent structural reliability analysis. In E. Watanabe, D.M. Frangopol, and T. Utsunomiya (Eds.), *Bridge maintenance, safety, management and cost,* pp. 1–8.

Leiden: A.A. Balkema. Phoon, K. & J. Retief (2016). *Reliability of Geotechnical Structures in ISO2394.* Number September.

Roscoe, K., F. Diermanse, & T. Vrouwenvelder (2015). System reliability with correlated components: Accuracy of the Equivalent Planes method. *Structural Safety 57,* 53–64.

Samuelson, P.A. (1937). A Note on Measurement of Utility. *The Review of Economic Studies 4*(2), 155–161.

Schoemaker, M.A., J.G. Verlaan, R. Vos, & M. Kok (2016). The use of equivalent annual cost for cost-benefit analyses in flood risk reduction strategies. In *FLOODRisk 2016,* Volume 20005.

Schweckendiek, T. (2014). On reducing piping uncertainties: A Bayesian decision approach.

Schweckendiek, T., M.G. van der Krogt, A. Teixeira, W. Kanning, R. Brinkman, & K. Rippi (2017). Reliability Updating with Survival Information for Dike Slope Stability Using Fragility Curves. In *Geo-Risk 2017,* Reston, VA, pp. 494–503. American Society of Civil Engineers.

Slijkhuis, K., P.H.A.J.M. van Gelder, & J. Vrijling (1997). Optimal dike height under statistical-, construction- and damage uncertainty. In Y.W.N. Shiraishi, M. Shinozuka (Ed.), *Structural Safety and Reliability,* Volume 7, Kyoto, Japan, pp. 1137–1140.

Slomp, R., H. Knoeff, A. Bizzarri, M. Bottema, & W. de Vries (2016). Probabilistic Flood Defence Assessment Tools. *E3S Web of Conferences 7,* 03015.

Speijker, L., J.M. van Noortwijk, M. Kok, & R.M. Cooke (2000). Optimal maintenance decisions for dikes. *Probability in the Engineering and Informational Sciences 14*(1), 101–121.

Van Gelder, P. (2000). Statistical methods for the risk-based design of civil structures.

Walker, W., P. Harremoës, J. Rotmans, J. van der Sluijs, M. van Asselt, P. Janssen, & M. Krayer von Krauss (2003). Defining Uncertainty: A Conceptual Basis for Uncertainty Management in Model-Based Decision Support. *Integrated Assessment 4*(1), 5–17.

Werkgroep Discontovoet (2015). Rapport werkgroep discontovoet 2015. pp. 95.