

RAMSSHEEP analysis: a tool for risk-driven maintenance

Applied for primary flood defence systems in the Netherlands

By Wesley Wagner

RAMSSHEEP analysis: a tool for risk-driven maintenance for primary flood defence systems in the Netherlands

Master thesis

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“Be a yardstick of quality. Some people aren't used to an environment where excellence is expected.”

Steve Jobs (US computer engineer & industrialist, 1955 – 2011)

Colophon

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Preface

This report has been written as graduation project for the Delft University of Technology, Civil Engineering within the section of Hydraulic Engineering of probabilistic design and flood risks. The graduation project contains an elaboration of a probabilistic approach for maintenance of primary flood defence systems in the Netherlands. The conclusion of the research can be drawn by the comparison of the current existing VNK method and the relatively new RAMSSHEEP method. I'm of the opinion that the result can make a significant difference in approaching several tender projects of RWS in the Netherlands. Moreover, I think this graduation report can be seen as a standard kind of manual for determining the most economic beneficial solution of a technical and societal problem.

I would like to thank my graduation committee for their supervision, support and enthusiasm during the last half year:

Prof.dr.ir. J.K. Vrijling	Delft University of Technology (Chairman)
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Furthermore there are some people I want to thank individually. To start with Reinder Peterse and Pieter van Gelder for their help and support during the phases of my graduation process. Their technical abilities always helped me to increase the quality of my report and resulted most of the time in new challenges. Subsequently, I want to thank Friso Mosterman, Michel Verlaan, and Michiel Zoons for their reviews and their independent advices on technical contents. I also want to thank Floris Hei for putting his fate in me by proposing further collaboration in the coming future. The rest of the team of DPI, I want to thank them for the nice and educational walks during the lunches in Haarlem.

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And last but not least I want to thank my girlfriend Tina Wit for her mental support during my graduation period; she always helps me to put things in perspective on moments the process was not going as I planned this to happen.

I hope my results are valuable for future tender projects.

Alkmaar, September 2012
Wesley Wagner

Abstract

Introduction

Floods are a threat to millions of people who live in lowlands like the Netherlands. Therefore the Dutch government has been given requirements for primary flood defence systems like dikes. The current condition of most of these dikes does not fulfill to these requirements based on flood risk analyses of RWS. The most logical step is to take measures by constructing new structures (dike, lock, water defence structures, and etcetera) or planning regular maintenance activities over time (maintenance intervals). It is most commonly used to focus on economical most beneficial measures and maintenance intervals for a primary flood defence system like the Afsluitdijk.

Objective

RWS aims to launch a certain risk-driven maintenance concept named RAMSSHEEP which should be developed by the current market. The objective is to assess whether or not RAMSSHEEP can be applied as a risk-driven maintenance tool for primary flood defence systems based on the results of the existing method of Probabilistic Approach. By describing the two approaches of a flooding problem more insight will be gained in the advantages and disadvantages of RAMSSHEEP.

Methodology

A general flooding problem, like the Afsluitdijk (case study), has been approached by analyzing the system on its main functions: retaining water, navigation, surplus water discharge and a road connection. Subsequently, a probabilistic analysis has been made by estimating the possible damage and the annual probability of occurrence which together lead to the monetary risk. To decrease this probability of occurrence some measures can be taken which cost a certain amount of money. These investments can be extra road lanes, an extra navigation lock and discharge sluice, and increasing the crest height of the dike. Eventually these investments will be analyzed whether or not this amount is lower than the expected level of damage in the old situation; so is the criterion of gaining safety larger than the improvement costs met?

Moreover, it is not always profitable to take such investments in an existing system. Therefore maintenance can be applied to decrease the probability of failure temporarily. Dominant failure events have been approached by describing scientific deterioration models which illustrates decrease of strength over time. A maintenance optimization describes the most economical beneficial time intervals in which maintenance should be applied based on the deterioration model (decrease of strength/increase of probability of failure).

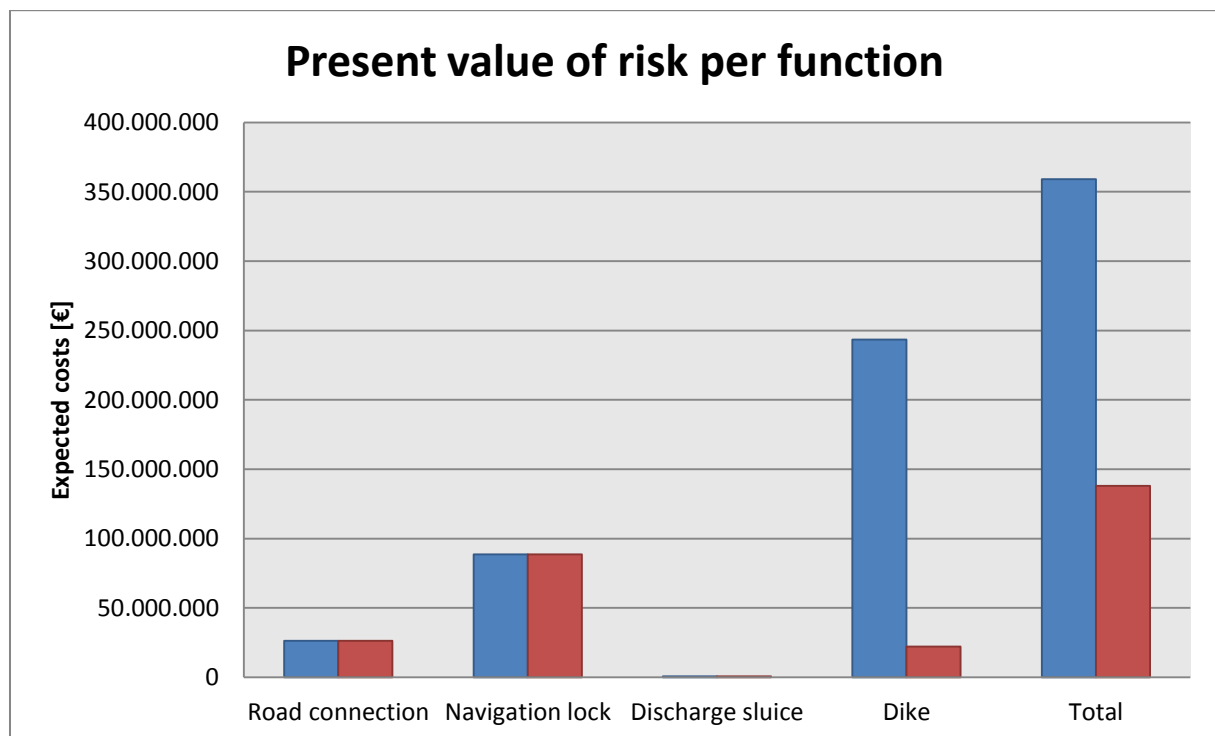
Eventually these results form the basis for the translation to RAMSSHEEP requirements. This result illustrates which requirements have been used and which not. This comparison (Probabilistic Approach vs. RAMSSHEEP) gives more grip on the assessment of the correctness of RAMSSHEEP as a risk-driven maintenance tool in the hydraulic engineering.

Results

The economical optimization of the investments resulted in only one measurement: heightening the crest level of the dike. This investment leads to almost a decrease of the monetary risk of 91% (in absolute numbers: € 221 million) which can be seen as an enormous amount. The other investments were not profitable with respect to the present value over the life time of the functions.

Function	Investment	Expected cost before investment [€]	Expected cost after investment [€]	Part of total [%]
Road connection	-	26.280.000	26.280.000	19,1
Navigation lock	-	88.572.000	88.572.000	64,2
Discharge sluice	-	800.000	800.000	0,6
Dike	Heightening crest level	243.480.000	22.288.280	16,3
Total		359.132.000	137.940.280	100

The figure below shows the result of the investments that have been determined (and calculated); the blue and red charts represent respectively the current monetary risk and the monetary risk after the investment.



In the table below an overview of the maintenance activities has been given including the corresponding maintenance intervals. These aspects have been translated to a reliability and availability number of the certain functions for the economical most beneficial maintenance intervals for preventive maintenance.

Function	Reliability [%]	Availability [%]	Maintenance interval [years]	Maintenance
Road connection	47,6	97,6	18	Renewing of asphalt pavement
Navigation lock	91,1	98,5	14	Repairing bed protection/scour hole
Discharge sluice	99,9	99,9	-	Vertical cut-off wall
Dike	99,9	99,9	-	Heightening crest level

Conclusions and recommendations

A Probabilistic Approach (PA) of a technical problem has been developed by order of RWS and is generally accepted by the market, which can be seen as an advantage. However, RAMSSHEEP is relatively new in the hydraulic engineering field, because it has been translated from other parts of the technique.

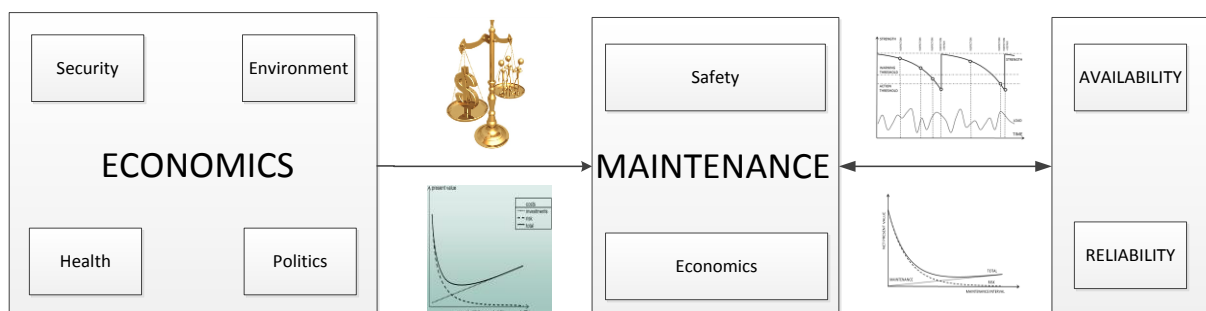
- Both methods form a basis for a maintenance plan/strategy;
- Both methods use more or less the same aspects in the analysis;
- PA uses the bottom-up approach by starting at the problem and RAMSSHEEP uses the top-down approach by starting at the standard acronym format (not adapted to the problem);
- PA approaches the problem on a general and uniform level and RAMSSHEEP aspects cover more than one levels (from system to element);
- PA translates all aspects to costs and RAMSSHEEP uses its aspects as individual system requirements and not expressed in money;
- PA acts on economical optimization and RAMSSHEEP aims on economical optimization by using set requirements based on high contentment of its users, which are contrary definitions;
- PA is sensitive for uncertainties because of the large amount of input numbers and RAMSSHEEP has less input and therefore is more robust;
- PA does only form a basis for maintenance plan, but it has not been implemented in its method and RAMSSHEEP considers investments as well as the maintenance;
- PA is a straight-forward working method and RAMSSHEEP is often broad and vague in describing the required steps.

The conclusions can be summarized in the table below which indicates the differences between PA and RAMSSHEEP.

Aspects	PA	RAMSSHEEP
<u>Reliability</u>	Amount of time functional [%]	Amount of time functional [%]
<u>Availability</u>	Amount of time available for its users beside planned and unplanned maintenance [%]	Amount of time available for its users beside planned and unplanned maintenance [%]
<u>Maintainability</u>	Mean Time To Repair (MTTR)	Measures to ease the maintenance on the system
<u>Safety</u>	Costs of unsafe/danger situation of the system	Using and maintaining the system according to the Safety manual

Aspects	PA	RAMSSHEEP
<u>Security</u>	-	Safe system with respect to vandalism, terrorism and human errors (including all kinds of sabotage of the system)
<u>Health</u>	Casualties have been translated into possible damage to the system	Minimization of the casualties due to function failure.
<u>Environment</u>	Pollution, contamination, and etcetera have been translated into possible damage to the system	To meet certain requirements which have been secured in Environmental Acts one suffices the rules of a good and clean environment.
<u>Economics</u>	Decision will be made by this main driver. The cost-benefit balance aims at the most optimal situation.	A serious reflection in terms of a Cost-Benefit Analysis must be made to provide more insight for an economical choice.
<u>Politics</u>	Politics gives the level of strategy on which the cost-benefit analysis will be executed: micro-, macro economy or political science.	A rational decision has to be made.

The recommendation comes forth from these conclusions and can best be given by the illustration below.



- (1) Choose just one political strategy for the cost-benefit analysis: economic optimization or contentment of its users. Hereby the economic optimization will be recommended.
- (2) Using PA for making decisions of measurement/investments to increase the safety of the system bases on the economic most beneficial solution.
- (3) An optimization of the maintenance can be based on physical deterioration models (verified on its current situation) which leads to a maintenance plan.
- (4) Results of PA can be translated to an optimal reliability and availability of the system which can be used as a level of intervention.

KEYWORDS: *PROBABILISTIC APPROACH, RAMSSHEEP, PRIMARY FLOOD DEFENCE SYSTEM, AFSLUITDIJK, ECONOMICAL OPTIMIZATION, MAINTENANCE STRATEGIES, RELIABILITY AND AVAILABILITY.*

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List of Symbols

Symbol	Description	Unit
α_L	Dustbin parameter	[-]
β	Reliability index	[-]
β	Curvature parameter	[-]
β	Slope angle of the scour hole	[°]
γ_{clay}	Volumetric mass of clay	[kN/m ²]
γ_p	Volume weight of sand grains under water	[kN/m ³]
γ_w	Volume weight of water	[kN/m ³]
γ_{water}	Volumetric mass of water	[kN/m ²]
Δ	Relative density	[-]
ΔH	Hydraulic head over the flood defence	[m]
ΔH_c	Maximum permissible gradient	[m]
Δh_{crest}	Net increased height of the crest of the dike (new height – previous height)	[m]
$\varepsilon(t)$	Consolidation on t	[m]
ε_e	Consolidation at end of consolidation	[m]
η	Drag force factor	[-]
θ	Rolling resistance angle of the sand grains	[°]
κ	Intrinsic permeability of the sand layer	[m ²]
λ	Rate of failure	[-/s]
μ	Time parameter	[yr]
μ_L	Expected value of the life span	[s]
σ'	Original soil stress of the subsoil at depth z	[Pa]
σ_1'	New soil stress of the subsoil at depth z	[Pa]
σ_R	Standard deviation of the resistance	[-]
σ_S	Standard deviation of the load	[-]
ϕ	Standard normal distribution	[-]
a	Dispersion factor	[-]
$A(t)$	Function of availability	[-]
b	Mode	[m]
B	Width of the dike body	[m]
c	Costs per unit of strength	[€/R]
C	'Smoothness' coefficient according to Chézy	[m ^{1/2} /s]
$C(t)$	Function of the maintenance costs	[€]
C_1	Direct and indirect economic damage per kilometer per hour	[€/km/min]
C_2	Direct and indirect economic damage per ship per hour	[€/ship]
C_3	Direct and indirect economic damage due to failure of the discharge sluice	[€]
C_{bridge}	Cost of constructing a bridge	[€]
C_{creep}	Bligh's 'creep' factor	[-]
C_{crest}	Cost of increasing the crest height per kilometer per 1 meter	[€/km/m]
C_f	Consequence of failure	[-]

Symbol	Description	Unit
C_i	Costs per maintenance activity	[€]
C_p	Primary consolidation coefficient	[m ⁻¹]
C_{PM}	Costs of planned maintenance of renewing asphalt layer	[€]
$C_{road\ lane}$	Cost of constructing a road lane per kilometer	[€/km]
C_s	Secondary consolidation coefficient	[m ⁻¹]
$C_{total\ damage}$	Total damage due to congestion/unavailability of the road in the current situation	[€]
C_{UM}	Costs of unplanned maintenance of renewing asphalt	[€]
c_v	Consolidation coefficient	[m ² /s]
$C_{w,creep}$	Lane's 'creep' factor	[-]
D	Height of the sill of the lock	[m]
D	Thickness of the compressible subsoil	[m]
D	Thickness of the sand layer	[m]
d_{70}	70 per cent value of the grain distribution	[m]
$D_{congestion}$	Costs of congestion due to maintenance	[€]
D_{Dike}	Damage due to failure of the dike	[€]
$D_{Discharge\ sluice}$	Damage due to failure of the discharge sluice	[€]
D_i	Damage due to functional failure	[€]
$D_{logistic}$	Logistic damage due to failure	[€]
d_{n50}	Grain size of the sand	[mm]
$D_{navigation\ lock}$	Damage due to failure of the navigation lock	[€]
D_{road}	Damage due to failure of the road	[€]
D_{SLS}	Direct and indirect economic damage due to SLS failure of the dike	[€]
D_{ULS}	Direct and indirect economic damage due to ULS failure of the dike	[€]
$E(H_0-H(t))$	Expected value of the need to increase the dike	[€]
$E(L_{SP}-L(t))$	Expected value of the need to repair the bed protection	[€]
$E(PV)$	Expected value of the present value	[€]
$E(t)$	Expected value of the life span	[s]
$F(t)$	Function of unreliability	[-]
$f(\underline{x})$	Function with a vector of stochastic variables	[-]
$f(X)$	Probability density function	[-]
f_c	Smooth protection	[-]
F_L	Probability distribution of the life span	[-]
f_L	Probability density of the life span	[s ⁻¹]
F_W	Annual probability of failure	[-]
$F_W(t)$	Function of annual probability that an element has failed before number of load repetitions n (Weibull distribution)	[-]
g	Acceleration of gravity	[m/s ²]
h	Thickness of the asphalt layer	[mm]
$H(t)$	Function of the strength during the consolidation	[m]
h_0	Undisturbed water depth of the lock	[m]

Symbol	Description	Unit
H_0	Original strength	[m]
h_s	Maximum depth in the scour hole developed for clear-water scour behind a bed protection	[m]
i	Rank number of the maximum occurrences in decreasing order	[-]
$I(n)$	Total estimated cost of constructing an extra road lane	[€]
$I(P_f)$	Investment in the structure or system	[€]
$I(\Delta h_{crest})$	Total estimated cost of increasing the crest of the dike	[€]
I_p	Solution of the integral	[-]
k_r	Equivalent roughness	[mm]
L	Length of the Afsluitdijk	[km]
L	Total considered time frame	[yr]
L	Minimum seepage length	[m]
L	Length of the seepage line	[m]
L_h	Horizontal seepage length	[m]
L_n	Slope of the scour protection	[-]
L_{SP}	Length of the bed protection	[m]
L_v	Vertical seepage length	[m]
L_β	Slope of the scour hole	[-]
$MTTF$	Mean Time To Failure	[d]
$MTTR_{pl}$	Mean Time To Repair of planned maintenance	[d]
$MTTR_{unpl}$	Mean Time To Repair of unplanned maintenance	[d]
N	Number of failure mechanisms according to which the section can collapse	[-]
N	Sample size of water levels	[-]
n	Total number of years of observations	[-]
N	Congestion length by the functional failure	[km]
N	Amount of ships of congestion by the functional failure	[ship]
n	Amount of extra road lanes	[-]
n	Total amount of maintenance activities within the given time period	[-]
$P(t)$	Function of condition of the asphalt	[-]
P_{Dike}	Annual probability of failure due to failure of the dike	[-]
$P_{Discharge\ sluice}$	Annual probability of failure due to failure of the discharge sluice	[-]
P_f	Annual probability of failure	[-]
P_f	Vector of design variables	[-]
$P_f(X)$	Cumulative distribution function	[-]
P_{flood}	The annual flooding probability of the area	[-]
P_i	Probability of collapse in case of collapse according to failure mechanism i	[-]
P_i	Annual probability of occurrence of functional failure	[-]
$P_{navigation\ lock}$	Annual probability of failure due to failure of the navigation lock	[-]
P_{road}	Annual probability of failure due to failure of the road	[-]
$P_{section}$	Total annual probability of collapse of the section	[-]

Symbol	Description	Unit
P_{SLS}	Annual probability of failure due to SLS failure of the dike	[-]
P_{ULS}	Annual probability of failure due to SLS failure of the dike	[-]
q	Probability of non-exceedance	[-]
R	Risk	[-]
r	Annual discount rate	[-]
R	Hydraulic radius	[m]
$R(n)$	Total annual economic cost due to congestion/unavailability of the road	[€]
$R(P_f, X)$	Monetary risk	[€]
$r(t)$	Rate of failure	[s ⁻¹]
$R(t)$	Function of reliability	[-]
$R(\tau)$	Resistance side of a limit state function	[-]
r_0	Turbulence	[-]
R_{Dike}	Costs expectation of the risks of failure of the dike	[€]
$R_{Discharge\ sluice}$	Costs expectation of the risks of failure of the discharge sluice	[€]
$R_{Navigation\ lock}$	Costs expectation of the risks of failure of the navigation lock	[€]
R_{Road}	Costs expectation of the risks of failure of the road	[€]
R_{total}	Total risk due to system failure	[€]
S	The monetary value of all inventory of the area	[€]
s	Standard deviation of the samples	[m]
$S(\tau)$	Loading side of a limit state function	[-]
S_T	Maximum achievable amount of cracks	[-]
s_y	Standard deviation of the reduced variate	[-]
S_τ	Amount of cracks at τ	[-]
t	Time	[s]
T	Return period	[s]
T	Amount of congestion time by the functional failure	[min]
t	Moment of time when maintenance will be executed	[yr]
t	Exposure time	[d]
t_e	End of the consolidation activities	[d]
$U(t)$	Function of unavailability	[-]
$U(t)$	Degree of consolidation	[-]
u_0	Flow velocity on top of the sill	[m/s]
u_c	Critical flow	[m/s]
$U_{pl}(t)$	Function of unavailability due to planned maintenance	[-]
u_s	Average flow velocity at the end of the scour protection	[m/s]
$U_{unpl}(t)$	Function of unavailability due to unplanned maintenance	[-]
X	Vector of random variables	[-]
X	Water level sample	[m]
X_m	Mean of X	[m]
y	Reduced variate	[-]
y_m	Mean of the reduced variate	[-]
Z	Limit state function	[-]

List of Abbreviations

Abbreviations	Description
BUA	Bottom-Up Approach
CBA	Cost-Benefit Analysis
DISK	Data Informatie Systeem Kunstwerken (Data Information System of Structures)
DUT	Delft University of Technology
EIA	Environmental Impact Assessment
EMAR	Economics, Maintenance, Availability and Reliability
FM	Failure Mechanism
FMECA	Failure Mode, Effects and Criticality Analysis
FORM	First Order Reliability Method
KNMI	Koninklijk Nederlands Meteorologisch Instituut (Royal Netherlands Meteorological Institute, Ministry of Infrastructure and the Environment)
LCC	Life-Cycle Costing
LEM	Lifetime-Extending Maintenance
LSF	Limit State Function
MCS	Monte Carlo Simulation
MSL	Mean Sea Level
MTTR	Mean Time To Repair
MV	Mean Value approach
NAP	Normaal Amsterdams Peil (Dutch reference water level)
NI	Numerical Integration
PA	Probabilistic Approach
PDCA	Plan-Do-Check-Act
PDF	Probability Density Function
PMM	Probabilistic Management and Maintenance
PV	Present Value
RAMSSHEEP	Reliability, Availability, Maintainability, Safety, Security, Health, Environment, Economics and Politics.
RWS	Rijkswaterstaat
SD	Standard Deviation
SLS	Serviceability Limit State
SORM	Second Order Reliability Method
TDA	Top-Down Approach
ULS	Ultimate Limit State
VNK	Veiligheid Nederland in Kaart (Flood risks in the Netherlands)

1. Introduction

This chapter contains the RAMSSHEEP aspects and their direct mutual relations which will be explained from the basis. Moreover, the history of the roots of the RAMS analysis will be analyzed and the development over time of the expansion to RAMSSHEEP. This has been especially applied on the hydraulic constructions. At last, the maintenance procedure of RWS will be explained whereby the RAMS analysis has been implemented in the Deming circle. Eventually this should lead to the most optimal maintenance strategy against the lowest costs.

1.1 RAMSSHEEP in general

The world has become a more complex system than it already was in the past and therefore the reliability of installations and processes has become more important due to the high-technical mechanization systems. The systems and processes in society will be indicated as reliable and robust until system failure occurs. Many questions will arise like:

- What is the reason for this system failure?
- Who is responsible?; and
- How can one prevent this system failure from happening again?

More and more (large) companies have been made an inventory of their process system and qualified and quantified the possible risks which may endanger an optimal reliable performance system. The main purpose of this risk analysis will be based on controlling or even avoiding the risks during the operating phase. Therefore maintenance to a system will play a crucial role in ensuring that the availability of the system is as high as possible. So this means that by applying the concept of risk management it is possible to get an indication of the reliability of the system and also which possible and low-cost actions can be taken to ensure the highest possible condition of the system.

A well-known analysis to get an indication of the performance reliability (quality) of the functioning of a system can be described by the acronym: RAMS analysis. Since a year or so, this analysis has been expanded to several other aspects which eventually led to the new expanded acronym: RAMSSHEEP.

The RAMSSHEEP acronym stands for:

- **R**eliability
- **A**vailability
- **M**aintainability
- **S**afety
- **S**ecurity
- **H**ealth
- **E**nvironment

- Economics
- Politics

The RAMSSHEEP will be applied to get insight in the risks to the system both qualitatively and quantitatively. It is possible to make an optimization of the system.

1.2 Hydraulic engineering

1.2.1 Original purpose of RAMSSHEEP

In the seventies of last century the American Defense industry, in corporation with the civil aircraft industry has been developed a structural analysis whereby the reliability a crucial role plays in the system. In the late seventies a risk method has been developed called Failure Mode and Effect Analysis (FMEA) to analyze the Reliability, Availability and Maintainability of a technical system. The official document '*Military Standard 1629a*', which has been published in 1980, has been one of the most important sources for the RAMSSHEEP theory nowadays (Van Gestel, Bouwman & Reijnen, 2004).

In the late eighties the European Commission decided to divide the properties in the rail sector: the infrastructure was managed by the nation and the exploitation of the trains was managed by the private sector. This led directly to a response in the rail sector to create a method which considers the performance of a system and how this can be tested. Therefore the RAMS analysis has been used which has been developed in the US. Over the years this analysis has been expanded and further developed based on the technique in the rail sector. This method has been taken over by the other infrastructural sector, i.e. hydraulic infrastructure. In the late zeroes the RAMS analysis has been expanded to more aspects which should be considered by the determination of the reliability of a system: the so-called RAMSSHEEP analysis.

1.2.2 Goals in hydraulic work field

The Dutch live and work below sea level which has been protected by the primary flood defence system. The primary flood defence system exists of dams, flood surge barriers, and dikes and protects the Dutch population against flooding. Rijkswaterstaat and other organizations (water boards, municipals, provinces, and etcetera) take care of this system and that the country is as safe as possible.

The primary flood defence system must be adapted to the higher requirements due to i.e. the climate change and the sea level rise. The Dutch government makes use of very strict safety requirements for the height of the water and the impact of the waves which have to be resisted by the primary flood defence system. These safety requirements have been documented in the Dutch water law (Waterwet, 2009). This Act is a very important instrument to check, to test, and to maintain the primary flood defence system. Every six years this system will thoroughly be checked if it still fulfills the required safety level.

Since the flooding in Zeeland (1953) the KNMI¹ has made daily expectations (based on models) of the tide, precipitation and wind velocities. During storm condition the governmental organizations (i.e. Rijkswaterstaat) monitors the development of the current situation with the expectation results. When the situations occurs that extreme and dangerous water levels and waves (significant wave height and period) are present, a warning signal will be given to the manager of the primary flood defence system. This manager can take appropriate actions to prevent system failure.

The water boards and Rijkswaterstaat are planning to increase the strength of the primary flood defence system in the coming years. This is necessary because numerous parts of the system (i.e. the Afsluitdijk) do not fulfill the required safety level of 1/10.000 per year. This safety requirement means that in one human life (assume that an average Dutch inhabitant lives about 80 years) the change of flooding will be about 0,8%² which can be seen as a very acceptable value (Safety level in hydraulic work field, 2012).

The VNK project³ (and VNK2 project) has been developed to calculate the current probability of flooding in the Netherlands based on innovative calculations methods. This project also relates the probability of flooding to the expected consequences expressed in financial damage and the total loss of life. The innovative calculation methods has been based on making a complete analysis of the diked area and testing the highest risks which would lead to a breach of a part of the dike (i.e. height of the dike is too low, dike unstable, piping, and etcetera). The information from the VNK can be helpful for governmental organizations to take specific actions to prevent the Netherlands from flooding (Rijkswaterstaat, 2005).

At last the goals of the Dutch government in the hydraulic work field can be summarized as:

- 1) Preventing the Netherlands from flooding by maintaining the primary flood defence system to the required safety levels (performs well);
- 2) Protect next human generations against high waters (keeps performing well); and
- 3) Take care of enough fresh water which has been documented by the Delta Program.

1.3 Applying RAMSSHEEP in maintenance phase

Rijkswaterstaat is responsible for the condition states of all hydraulic constructions in the Netherlands. Thereby maintenance is necessary to guarantee the safety and mobility of the infrastructural waterways. RWS is outsourcing maintenance activities on the engineering market to achieve higher quality against lower costs.

Therefore the organisation makes use of a management and information system called DISK (Data Informatie Systeem Kunstwerken⁴) which contains all sorts of data of hydraulic

¹ KNMI = Koninklijke Nederlands Meteorologisch Instituut – Ministerie van Infrastructuur en Milieu (Royal Netherlands Meteorological Institute – Ministry of Infrastructure and the Environment)

² $P_f = \left(1 - \frac{1}{10.000}\right)^{80} = 0,00797$.

³ VNK = Veiligheid Nederland in Kaart (Flood Risks in the Netherlands)

⁴ Translated in English: Data Information System of Structures

structures (bridges, tunnels, aqueducts, locks, dams and dikes). The requirements to an infrastructural system have become more strict (higher performance) in the years while degradation of the construction is resulting in lower overall resistance.

The main goals of DISK are:

- To make a sufficient prediction of the functional and financial lifetime of a specific construction or system;
- To actualize the database of known data;

Nowadays RWS is using the Lifetime-Extending Maintenance (LEM) model which has been developed by the Civil Engineering Division of RWS (Lifetime Extending Maintenance model (LEM), 2012). The LEM model is a program for Life-Cycle Costing (LCC) with which cost-optimal maintenance decisions can be determined under uncertainty:

- Through lifetime extension, the deterioration can be delayed as such that failure is postponed and the lifetime of a component is extended.
- Through replacement, the condition of a component can be restored to its original condition.

This model can be used to optimize the maintenance costs in the operating phase; the cost of preventive maintenance (lifetime extension and preventive replacement) can be optimally balanced against the cost of corrective maintenance (corrective replacement and failure). The cost-based criterion of the expected discounted costs over an unbounded time-horizon (Net Present Value) is used to compare different maintenance strategies.

Since the last decade the RAMS analysis and the FMECA have become more popular during the maintenance phase. RWS is mainly using the Deming circle (Plan, Do, Check, and Act (PDCA)) to determine if a system fulfills to the required RAMS aspects.

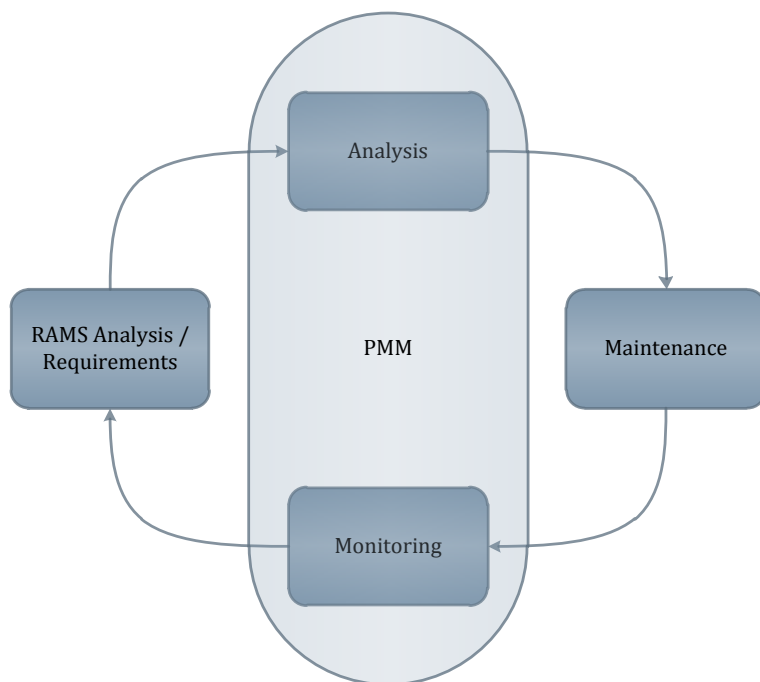


Figure 1-1: Schematic illustration of the Deming circle which relates maintenance to the RAMS aspects.

The Probabilistic Management and Maintenance (PMM) is the direct connection between the RAMS aspects and the maintenance activities. The RAMS aspects will be expressed by parameters like:

- Failure frequency;
- Interval times;
- Mean time to repair (MTTR);
- Inspection and maintenance durations.

The monitoring (connection between maintenance activities and the RAMS analysis) forms the basis to the trend analysis of a certain data set. This analysis can be seen as an update of the current RAMS aspects (see above).

NB. This analysis also considers the difference between redundant failure and failure due to human actions. Eventually RWS uses the LEM model to optimize the maintenance activities.

1.4 Outline of the research

In this graduation project the methods of Probabilistic Approach and RAMSSHEEP is compared with each other. On the basis of the structure of the methods and by using them on a case study the applicability of these methods is determined and the strong and weak aspects have to be found. In the scheme below the rough framework of the report is presented.

Part	Chapter
Introduction and research methodology	1 and 2
Literature study	3, 4, 5, 6, and 7
Case study: Afsluitdijk	8 and 9
Conclusions and recommendations	10

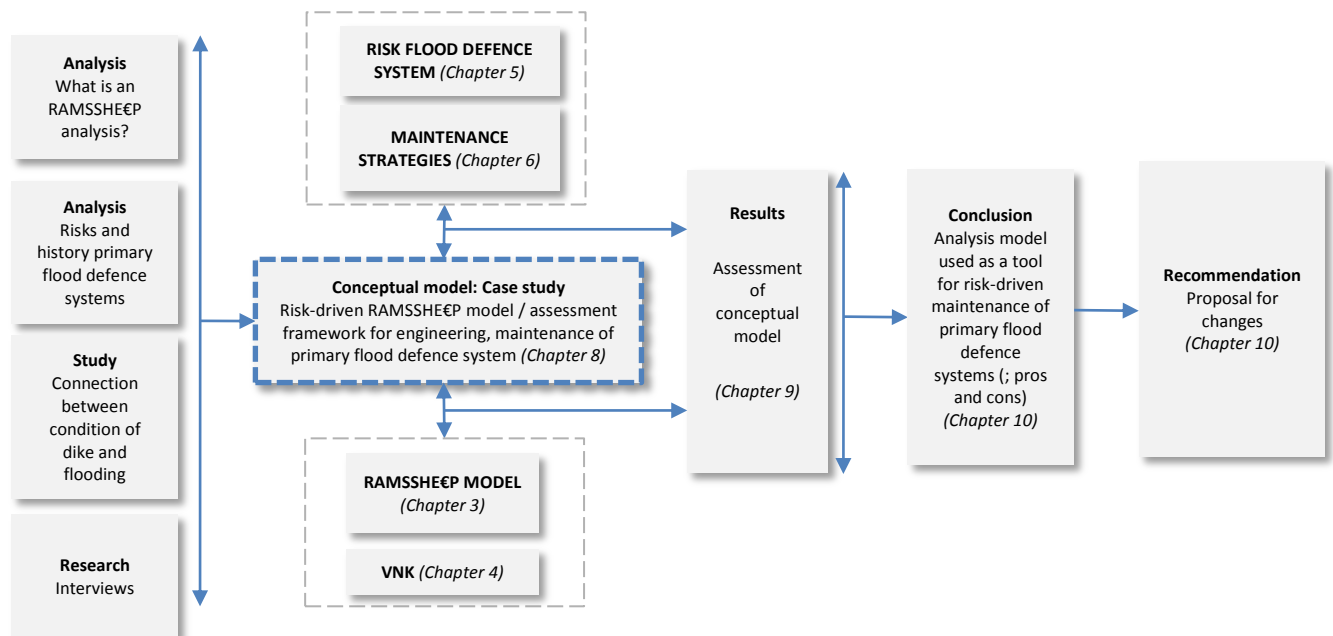
In the first two chapters an introduction of the subject has been given including the original purpose and the current goals of RAMSSHEEP in the hydraulic engineering. Subsequently, the line up of the research shows the goals, research model and questions based on the problem analysis.

The foundation of the case study will be given based on the literature information starting in chapter 3. Here, an elaborated part of the RAMSSHEEP analysis has been described in which RWS determines the definitions of the individual aspects. At the end of this chapter some remarks have been given (discussion) about the technical content of these definitions and examples. In chapter 4 the method of the VNK analysis has been described. Together with chapter 3 this forms the main (theoretical) part of the case study. Subsequently, chapter 5 gives an overview of the most relevant failure mechanisms of a dike (presented in a fault tree). Moreover, an elaborated flood frequency analysis has been given to calculate the probability of a dike breach. The basis of the maintenance has been presented in chapter 6 in which also the optimization of the maintenance intervals will be elaborated on the

maintenance costs and strategy. Chapter 7 illustrates the direct connection between the theoretical parts with the practical part of the maintenance process.

The literature study from chapter 3 until 7 forms the basis for the execution of the case study of the Afsluitdijk in chapter 8. This case study has been applied to assess whether or not the RAMSSHEEP analysis results in the same as the Probabilistic Approach ('Delftse methode') according to the economical optimization (bottom-up approach). Here, a probabilistic approach has been given of the determination of the expected costs (monetary risk) and the possible and most feasible investments (to reduce the probability of failure) will lead to an economical optimization. This forms directly the basis for the optimization of the maintenance activities over time (intervals). In chapter 9 both the RAMSSHEEP (top-down approach) as well as the Probabilistic Approach (bottom-up approach) have been elaborated over the individual requirements.

Finally, chapter 10 presents the conclusion from the case study in an objective way and elaborates a recommendation for the most efficient and effective approach of such a problem. To conclude, less significant recommendations have been given with respect to this report.



2. Research

The best way to explain the motive for this research is to analyse the changes over time and then especially based on hydraulic loads and the set requirements. First, the history of the IJsselmeer area will be considered and the changes over time towards the current situation. This area forms the main reason for this assignment, because the safety levels of the dikes (Afsluitdijk and Houtribdijk) are below standard and will therefore be considered directly related to the problem. The problem analysis will show the critical situation and give the core of the problem. This core must be carefully defined and this will be done to determine the scope of the research. The safety levels of primary flood defence systems has been guaranteed in the Dutch water law (Waterwet) and will therefore be used as the leading direction to define the core of the problem. To conclude an overview has been given of the goals and motive of this research.

2.1 History and current situation

The history of the IJsselmeer area originates from the era of the Romans when the Netherlands was one enclosed country with just one big lake in the middle (called the Flevomeer (1)). After a couple of centuries this lake had been connected to the Nord Sea due to the increasing numbers of tidal waves from sea. The soil near the Wadden area could no longer resist the loads and eventually the erosion resulted in several channels from sea to the lake which thereby transformed to a lagoon; the lagoon was called the Almere (2) which means 'the big lake'. In the late Middle Ages the tidal waves had become more and more extreme which resulted in disappearance of the soil (most of the soil existed of peat). After some time the lagoon became bigger and eventually expanded to the scope of the former Zuider Sea (3). The lake does no longer exist of only fresh water, but due to the direct connection to the Nord Sea it became brackish (fresh/salt).



Figure 2-1: The development over time of the Netherland (History of the Zuider Sea works, 2012).

Since the Zuider Sea had become jeopardized for the inhabitants of the ambient grounds many hydraulic engineers started thinking to create a solution to this problem. A hydraulic engineer named Hendric Stevin came with the idea to construct a dike from North-Holland via the Wadden islands to sea dikes of Groningen (north-east of the Netherlands). However, they did not have any experience with this type of engineering and decided that it was not

feasible to execute this design. It took approximately more than two centuries before another engineer copied the idea. The name of this engineer was Dr. Ir. Cornelis Lely and he used the original design of Stevin to construct a dike. Contrary to Stevin Lely designed an enclosure dam between North-Holland and Friesland (2) and the Zuider Sea (1) would become a lake again. Beside this enclosure dam also the idea had been arising of land reclamation and winning back the soil which was lost over the centuries by the force of the sea. This plan originates from 1881 and the Dutch government accepted the plan of the enclosure of the Northern part of the Netherlands and land reclamation within the new created lake in 1918. The reason for this delay in decision-making was the flood in 1916 in the Northern part of the Netherlands. The construction of the Afsluitdijk (enclosure dam) took more than 12 years to complete; from 1920 to 1932. After this enormous achievement of the Dutch engineers the land reclamation started and resulted eventually in a new polder area, named Flevopolder (3) and its main purposes are to improve flood protection and create additional land for i.e. agriculture. One decided to continue the land reclamation and therefore constructed a new dam (Houtribdijk) from North-Holland to the new Flevopolder (4). This dam should form the northern boundary of the Markerpolder, but unfortunately this plan has been delayed until 2003 when the government finally decided to cancel the plan for land reclamation in the Marker area (History of the Zuider Sea works, 2012).

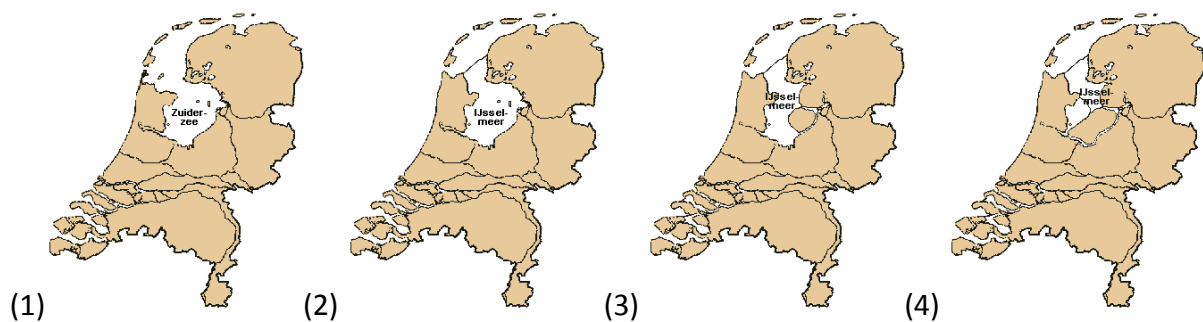


Figure 2-2: The construction of the Afsluitdijk, the land reclamation of the Flevopolder and the construction of the Houtribdijk in the middle of the IJsselmeer (History of the Zuider Sea works, 2012).

Afsluitdijk (Den Oever (NH) – Zurich (Fr))

The Afsluitdijk has a length of 32 kilometres and changed the Zuider Sea into the IJsselmeer and forms nowadays a part of the primary Dutch flood defence system and an important road connection between North-Holland and Friesland. Rijkswaterstaat (RWS) continuously manages the condition of the dike. Due to the rapid climate change the drain capacity will be enlarged to guarantee the safety of the Netherlands. The separation of the Zuider Sea from the North Sea has several functions for the Netherlands (Rijkswaterstaat, 2009):

- Safety

The Afsluitdijk is a part of the primary Dutch flood defence system and its main function is to protect the hinterland from flooding. Therefore, the dike must have sufficient strength and height to resist extreme circumstances of the sea. Also the climate change and sea level rise will be a future topic what will lead to adaptations of the dike.

- Fresh water

The IJsselmeer is one of the most important fresh water reservoirs in the Netherlands. This water can be used for several purposes, like drinking water, irrigation water, and etcetera. Besides, this lake forms an important ecological area for fish- and bird species. However, the Afsluitdijk also forms an obstacle for fish species that are moving from fresh to salt water.

- Economical values

The high road on the Afsluitdijk (A7) connects Friesland to North-Holland and approximately nineteen-thousand motorists are passing this road every day. Therefore, the A7 is a very important connection and creates a lot of economic benefits in the Northern part of the Netherlands. Besides this road, also a navigation lock is present for shipping. At last, the Afsluitdijk is a special form of art and tourists all over the world come to visit the dike.

- Cultural

The Afsluitdijk has some cultural values and a strong amenity. This comes forth from the history of the Netherlands and the identity of the Zuider Sea forms an important role in the creation of the dike. Therefore, the Afsluitdijk is one of the most famous dikes in the world.



Figure 2-3: The former Zuider Sea with an open connection with the Wadden sea and North sea (left); and the new IJsselmeer has been created by closing the gap between Den Oever (North-Holland) and Zurich (Friesland) (right) (CMO EUForum, 2010).

Houtribdijk (Enkhuizen (NH) – Lelystad (FI))

The Houtribdijk separates the IJsselmeer and the Markermeer (see Figure 2-3) and had been constructed in the period of 1963 until 1976. This dam has a length of 26 kilometres and its original function was to form the northern boundary during the land reclamation of the Markermeer. This decision has been cancelled in 2003 and its function nowadays is only a

road connection N302 between Enkhuizen (North-Holland) and Lelystad (Flevoland). The daily intensity of this dike is about 8.500 motorists.

The condition of the dikes is below the standard and some maintenance activities are necessary. At this moment the Houtribdijk has been renovated and RWS is repairing its locks – the lock doors, the lock buildings and the bridges nearby the drainage complex. Also the technical installation will be replaced. The locks of the Houtribdijk are part of an important navigation route between Amsterdam and the northern part of the Netherlands. This renovation will secure an optimal navigation in the coming future and the labour men will be finished in the summer of 2012 (Rijkswaterstaat, 2009).

2.2 Problem analysis

The Afsluitdijk has three main functions (enumeration 1 up to and including 3) and a couple of values (or qualities) (enumeration 4 up to and including 6):

1. Safety;
2. Water management; and
3. Mobility.

4. Environmental values;
5. Cultural and historical values; and
6. Natural values.

Safety

The test on primary flood defence system in the Netherlands, which was executed by the Ministry of Infrastructure and the Environment, based on the Dutch water law (Waterwet) noted in 2006 that the Afsluitdijk in its current condition, including its locks and drain facilities, was not sufficient to the standard for primary flood defence systems. This standard represents a guarantee of safety of water levels which will appear 1/10.000 per year (read: ones in the ten-thousand years). This is what one calls the probability of appearance and represents the highest safety standard for primary flood defence systems.

In 1960 this sort of test was executed by the former Delta commission what resulted in a probability of exceedance of 1/1.430 per year. At that moment the Delta commission was satisfied with that result. Over time the standard has been updated and renewed in 2006 to its current safety level of 1/10.000 per year. A new test has been executed in 2006 with its adapted standards and the condition of the Afsluitdijk has been assessed as insufficient. This judgement was on beforehand not surprisingly and that was not only due to the higher standards. The resistance of erosion of the crest and the inner slope of the some parts of the Afsluitdijk were assessed as insufficient. Beside this, also all the engineering structures (four locks) are assessed on the height of the structure, the reliability of the navigation part, the stability of the structure and soil, and the resistance and strength of the construction parts. The final result was again insufficient to the standard (CPB, 2011).

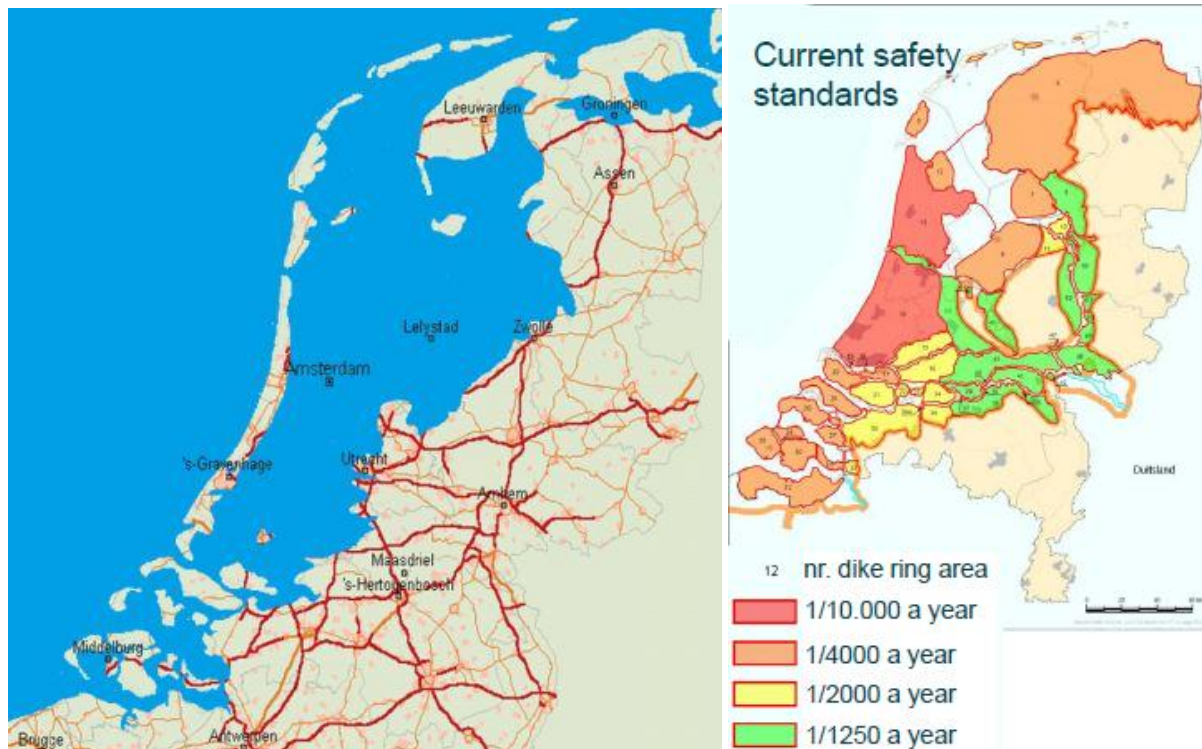


Figure 2-4: An illustration of the Dutch ground which lays below sea level (Left) (<http://www.grotescheur.nl/wp6/wp-content/uploads/2011/01/Nederland-026-onder-NAP.png>) and the safety standards for primary flood defence systems (right) (Rijkswaterstaat, 2011).

The consequences of a flooding due to failing of the functions of the Afsluitdijk will result a water level rise of several decimetres. That means that the surrounding dikes of IJsselmeer area must be resistant to this high hydraulic load. Also, these dikes have been tested and assessed as insufficient to the same standard. If this situation appears, consequences are enormous and incalculable for the Netherlands:

- Many people will drown;
- About one quarter of the Netherlands will disappear below water level and millions of inhabitants will lose their homes;
- Enormous financial damage;
- Decrease of health level;
- Political consequences;
- Dutch engineers losing the title of 'Water Specialists';
- ...etcetera

Water management

The function of water management can be explained by the fact that the IJsselmeer is a reservoir of fresh water and therefore has a very important function for the wellbeing of the Netherlands. The lake has been fed by the rivers (the Rhine/IJssel, the Vecht and the Eem) and also by the discharge of the nearby polder areas. The surplus of the fresh water from the rivers and polders into the IJsselmeer can be discharged by using the locks during ebb-tide,

because of the tidal force in the Wadden Sea. One is expecting that the possibility to discharge this surplus of fresh water due to the future climate change and the continuous sea level rise will become more difficult. With the assumption of an unchanged lake water level the time to discharge from the IJsselmeer to the Wadden Sea will become shorter, what may result in problems for the dikes around the lake.

Another direct problem of the climate change will result in more extremes in the weather patterns in the Netherlands. In winter periods the precipitation will increase and in summer periods the drought will play a larger role. This means that the variation of the lake water level will have more fluctuations what result in problem to control the water (CPB, 2011).

Mobility

Every day approximately 19.000 vehicles are using the Afsluitdijk to move from North-Holland to Friesland. The expectation is that this amount of vehicles will grow to more than 26.000 vehicles per day in 2040. This growth will not lead to any congestion on the dike. Beside the high way there are also two locks in the Afsluitdijk (Den Oever: Stevinluis and Kornwerderzand: Lorentzsluis). The locks can only operate when the bridges are opened which means that the traffic has to wait for several minutes. In the period of May to September about 80.000 ships are passing the locks and 90% of this amount is for recreational purposes. To conclude the function of mobility will not lead to any direct problems in the future (CPB, 2011).

Budget

The budget has been inadequate to make a decision in 2012. The ministry set apart about 300 million euros to repair the safety standard of the dike *and* the locks. According some rough calculations only the reparation of safety of the dike will cost this amount of money. So, one may conclude that the ministry must invest more money in the safety of the Netherlands.

Due to the fact that the safety level of the dikes and the locks are way too low, the damage of a severe storm will eventually lead to loss of the flood function. This endangers availability of the locks and the salt water from the North Sea will be able to pour in the IJsselmeer area; this will lead to financial and ecological losses (Rijkswaterstaat, 2011).

Problem

The core of the problem can be divided over two sections: (1) safety level of the dike is below standard; and (2) the budget is inadequate to repair all the necessary aspects to realize a safety level according to the standard.

Because of all the above aspects the problem definition can be described as the lack of knowledge and insight of which maintenance activities are the most efficient and effective with respect to the available budget and required safety level. However, because of the lack of information on the economical aspect (budget to improve the safety level of the dike) the

research will only focus on the safety level of the dike which is below standard. The scope will determine aspects on how the engineers in the Netherlands can develop a risk-driven maintenance plan based on an RAMSSHEEP⁵ analysis (the driver). This acronym has been chosen to implement as main driver, because of the fact that RWS will make a standard of this analysis for the coming tender projects.

2.3 Scope

The problem analysis gives a wide overview of problems for the primary flood defence system in the IJsselmeer area. The problem analysis will only be applied on the technical part of the primary flood defence system. This directly led to the requirements of this system which has been documented in the so-called 'Legger'⁶. This means that the scope has already been defined to just one part of the Dutch water law (Waterwet). Below an overview has been given of the top-down method based on this law which originally is a sort of reverse engineering.

The scope is based on the Dutch water law and will work its way downwards to operation level. This top-down method has been used, because this ensures that the criteria of the law for primary flood defence systems are met. This law has been introduced in December 2009 and represents an aggregation of eight laws. The Act consists of management regulations between groundwater and open water and also improves the consistency of water management and spatial planning. This can be summarized by three basic components (Waterwet, 2009):

- Water Decision;
 - National order for water deficit, national Water Plan and Management plan.
- Water Regulation;
 - Organization of Water Management including its boundaries of open waters.
- Water Permit;
 - Modification and requests for water permits.

By analyzing the complete Dutch water law one can be ensured that the criteria for primary flood defence systems satisfy the law. It gives a general overview of which documents must be present for the Dutch water system. Many of these documents will not be considered, because the scope has been set on the technical aspects of the Dutch water law (Waterwet). This means that permits, regulation documents, specific laws, and etcetera, will be eliminated. Documents that are interesting for this topic are basically technological documents, like:

⁵ RAMSSHEEP stands for Reliability, Availability, Maintenance, Safety, Security, Health, Environment, Economy and Politics (see also chapter 3).

⁶ In English: Object specific dossier. This dossier describes the zones and gives an overview of the minimum requirements of a primary flood defence system. Beside this, it will give information on the physical parts of the dike (i.e. height) displayed in several technical drawings and the safety aspects which are applicable within the system.

- Technical Guidance – Overview of design, maintenance and management requirements on primary flood defence systems;
- General hydraulic condition of primary flood defence systems – General hydraulic conditions will be reported every six years;
- Report of measures applied on primary flood defence systems – Measurements executed by the manager as result of the report of the general hydraulic condition;
- Management plans – The maintenance strategy to improve the condition of the dike and when this will be executed (time line);
- Object specific dossier - Information on the location (technical overview of the different protection zones), design and dimensions of the construction;

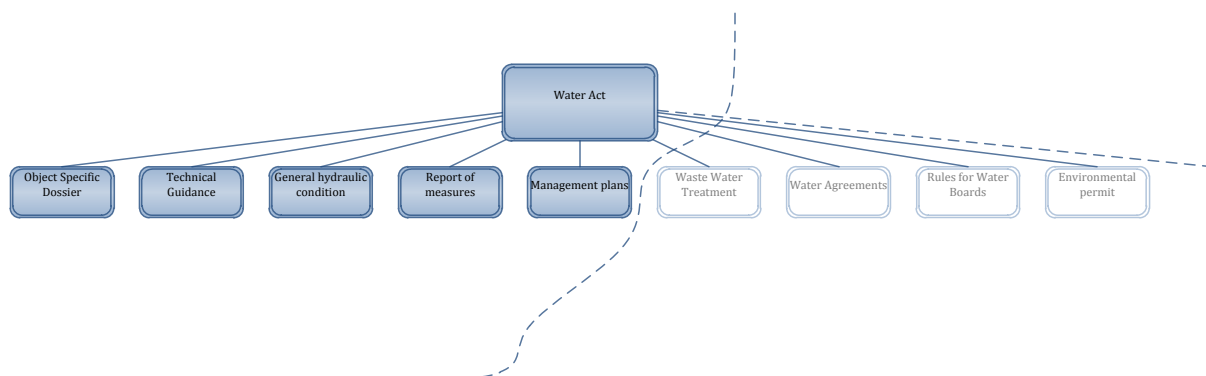


Figure 2-5: Defining of the relevant documents for the technical aspects according to the Dutch water law (Waterwet).

The focus of the research must be pointed to the RAMS aspects of these documents, because the maintenance activities will be applied on the physical part of the primary flood defence system. Beside this focus only the function safety of the primary flood defence system will be considered, because the water management and mobility part present another maintenance strategy and has been used in a different working field. Therefore, the selected five technical documents derivative from the Dutch water law (Waterwet) only the 'legger' will be used and analyzed on RAMSSHEEP aspects for primary flood defence systems.

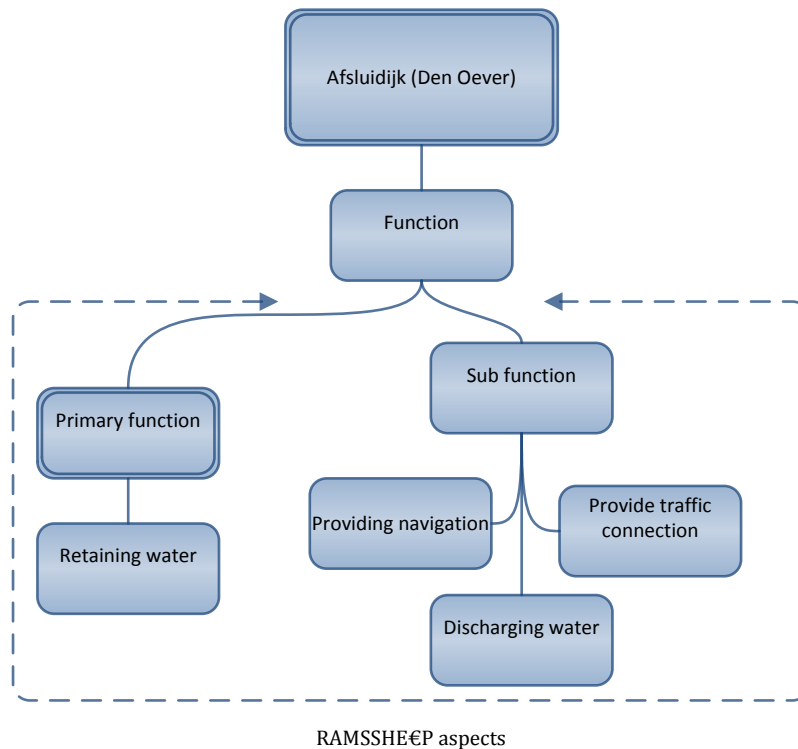


Figure 2-6: Ledger of the Afsluitdijk derivative from the Dutch water law (Waterwet) with an analysis of the RAMSSHEEP aspects.

For this research the primary function and the sub functions will be assessed. This decision defines the technical safety scope which only can be applied on the functions.

The RAMS analyses will be based on the primary flood defence system of the IJsselmeer area and then especially applied on the Afsluitdijk⁷. This dike has been categorized in the primary flood defence system of the Netherlands (category b). This means that the ledger will be applied on this dike.

The ledger of the Afsluitdijk describes an area of several zones (core, protection and outer protection zone) and gives an overview of the minimal requirements of the primary flood defence system. This does not only involve the height of the dikes, but also the safety aspects (of the use of the system) which are applied for these dikes.

⁷ The reason why only the Afsluitdijk has been chosen and not in combination with the Houtribdijk is because the Afsluitdijk can function without the Houtribdijk, but the Houtribdijk cannot function without the Afsluitdijk. On top of that the Afsluitdijk is more important as a primary flood defence system due to its direct connection with the sea.

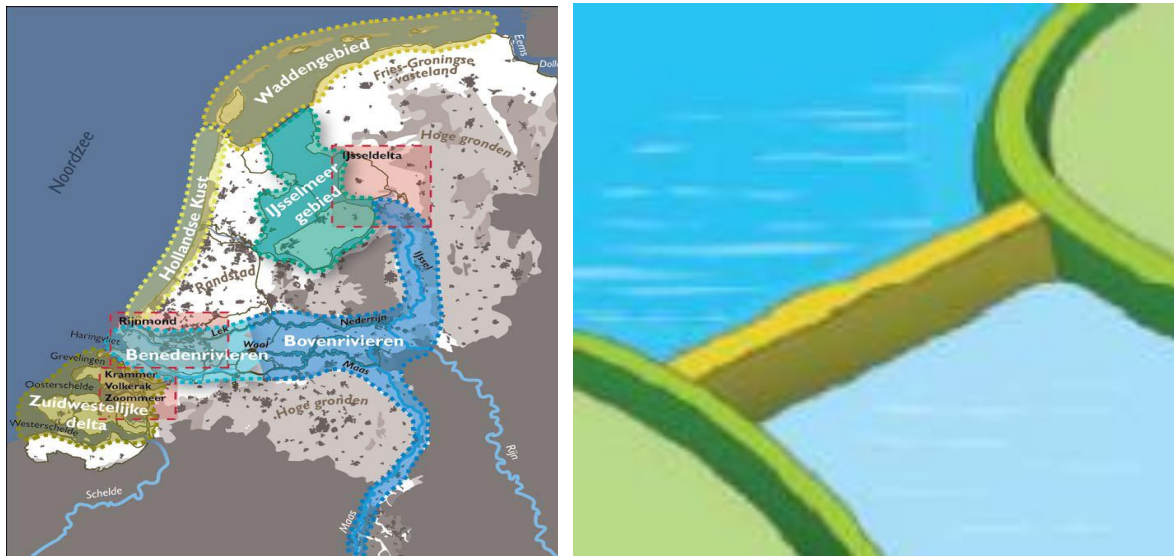


Figure 2-7: The regions in the IJsselmeer area (Left) (Rijkswaterstaat, 2012) and the assessment category b 'connecting dams' of primary flood defence systems (Right) (Rijkswaterstaat, 2012).

2.4 Goal and concepts

The main goal of this research study is to create an RAMSSHEEP analysis format for primary flood defences in the Netherlands. This format can be used to receive more insight in the most efficient and effective investment strategy with respect to maintenance activities. This means that this research contribute to the solution of the problem and will not lead directly to the complete solution. Besides, the budget is always too short and therefore it is very valuable to know which maintenance activities must be executed to create an optimal safety level of the primary flood defences as long as the new situation will ensure the given safety level.

The research results will help RWS to obtain more insight in the primary flood defence system and especially applied on the Afsluitdijk. This is the target of this research.

Looking at the coming future the weather conditions are becoming more extreme, and in combination of the sea level rise and the land subsidence on the Western part of the Netherlands, the primary flood defence system will play an even more crucial role than it already does at this moment.

These weather changes combined with the financial crisis in the world (2007 – nowadays) will play an important role in the basic aspects of the conceptual model, because these two facets are the main reason for the increasing risks of a primary flood defence system; the

weather changes lead to higher loads and the financial limitation lead to less possible measures that can be taken to reduce the risks⁸.

In addition to this main goal the interest of the problem can be separated over DPI and RWS and at last also Delft University of Technology.

- (DPI) Development of a theoretical model which forms a standard for an RAMSSHEEP analysis for the aspects on maintenance for wet as well as dry infrastructure;
- (DPI) Enrichment of risk based maintenance activities in the work field of hydraulic engineering;
- (DPI) The model can be used to create more value to the client RWS and therefore may result in benefits in comparison with the rest of the competition;
- (RWS) Scientific and independent research results in an effective maintenance strategy based on the results of the safety level of the Afsluitdijk which is already determined by RWS.
- (RWS) (Possible) practical use of results for elsewhere in the Dutch flood defence system.
- (DUT) To set up an independent report (on scientific level) where a fully risk analysis is given by using the methods of fault trees and RAMS (-SHEEP) and also to give an advice on maintenance aspects for dikes.

All these goals will lead to a final product which will represent my graduation work. On beforehand, this graduation work will exist of a risk based approach where an FMECA forms the base of the qualification of the risks for the flood defence system. Subsequently, an RAMS-analysis will follow to quantify these qualifications into a fault tree. Also the elaboration part of SHEEP (Security, Health, Environment, Economy and Politics) will be analysed. Besides these results, an advice can be given to RWS with respect to the maintenance aspects for the dikes.

⁸ Nevertheless, these two reasons will not have any connection with each other. The weather condition changes are not something one is able to adapt or even to control and can therefore be seen as an exogenous process. However, this weather change can be modeled and named in the recommendations of the research. In contrast to the weather change the budget is always a direct (political) choice of the manager/government but cannot be influenced.

2.5 Research Model

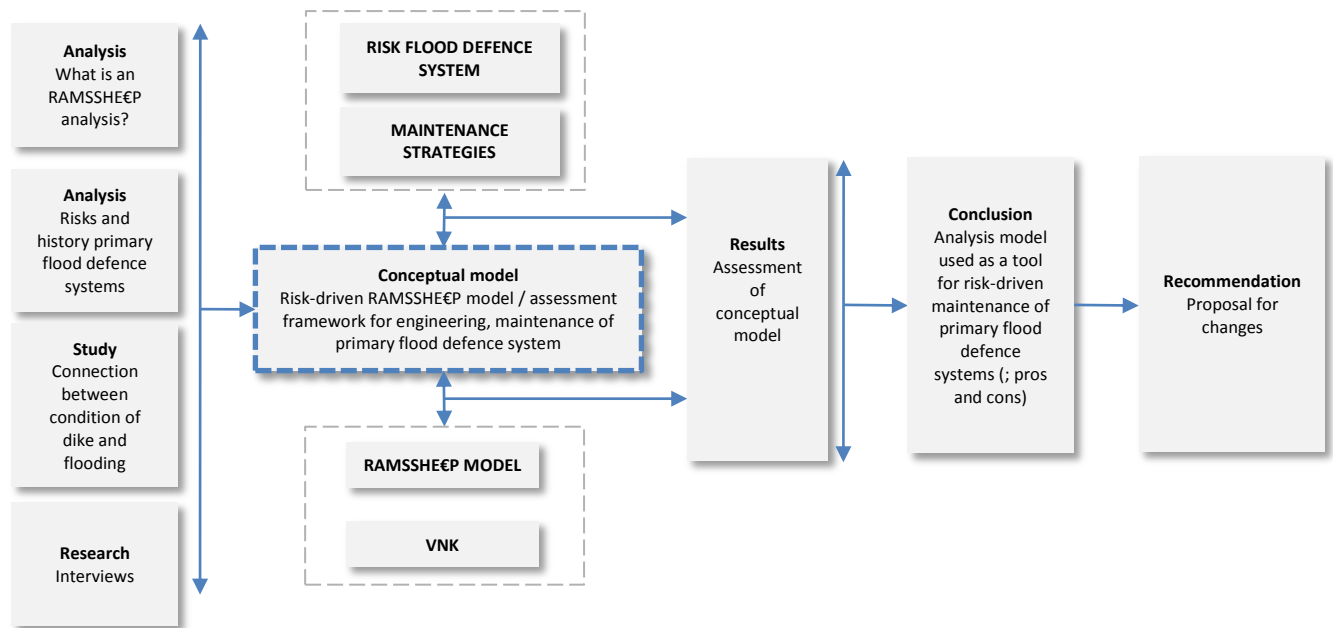


Figure 2-8: Research model applied for a risk-driven RAMSSHEEP model (Verschuuren and Doorewaard, 2005).

The research model has been based on the model of Verschuuren and Doorewaard. This model describes a translation from theory to a conceptual model and conclusion.

The theoretical framework (left column of the research model) will form the fundamental of the research and is also leading in the literature study. This framework exists of some theoretical analyses like the background of the RAMS analysis and in addition to that the SHEEP aspects. Also the history and current situation and condition of the primary flood defences will give more insight to the safety level. The current condition of the dike can be interesting, because this can say something more on which risk mechanisms will occur first.

The conceptual model will describe a risk-driven RAMSSHEEP assessment framework for engineering and maintenance of primary flood defence system. This risk-driven model will be based on the theoretical framework and be tested to the risk which endangers the functioning and the set requirements a flood defence system must suffice to. On the other hand the conceptual model will implement the RAMSSHEEP and VNK model to form the correct format for a risk-driven model. Eventually the conceptual model can be tested to risks and requirements on the one hand and the risk models RAMSSHEEP and VNK on the other hand. Subsequently this conceptual model will be filled in with the known data on primary flood defence systems.

Finally, a conclusion is drawn by testing the practice to the conceptual model what may result in an advice on the maintenance activities to a primary flood defence system. This advice will suffice to the Dutch water law and because it is risk-driven it will give which maintenance activities are the most efficient.

2.6 Research Questions

The problem analysis in combination with the research model results in a research question:

“What can RAMSSHEEP, as a tool for risk-driven maintenance on a primary flood defence system, contribute to improve its safety level and give more insight to guarantee that the dike will fulfill its main functions?”

The solution to this research question can be realized by answering sub questions on aspects like inventory of the risks of the dike, analysing and quantifying these risks and conclusions based on these previous aspects.

- **Inventory phase:**
 - I. What is RAMSSHEEP?
 - II. Which failure mechanisms can be recognized?
 - III. Which maintenance activities (and strategy) have been used?
 - IV. What are the requirements of a dike?
 - V. Which risks must be assessed to these requirements?

- **Conceptual model phase:**
 - VI. How can the crucial risks be qualitatively related to the functional aspects (RAMS-analysis)?
 - VII. What is level of quality of the function of a primary flood defence system with respect to the security, health, environment, economic and political aspects be considered (SHEEP)?
 - VIII. What are the failure mechanisms of the dikes (VNK)?

- **Analysing model phase:**
 - IX. Which failure mechanisms are related to the crucial function (retaining water) of the dikes (Fault tree)?
 - X. Can the combination of the input data (requirements of the condition of dikes and the risks to be assessed) and the models (VNK and RAMSSHEEP) form an assessment framework for engineering, maintenance of primary flood defence systems?
 - XI. Which measurements can be taken to decrease the monetary risk of the functions of the Afsluitdijk based on the economical most beneficial approach (cost/benefit)?

- **Conclusion phase:**

- XII. Does the Afsluitdijk suffice according to the risk-driven (conceptual) RAMS-model?
- XIII. Are there quick wins that can be implemented in the planning to increase the security level to the standard?

2.7 Prospect

I have aspirations that my result of the graduation work will:

- Make a significant contribution to the safety knowledge for RWS, particularly in the IJsselmeer area – Afsluitdijk.
- Produce and use technical science to increase knowledge which improves the quality of decision-making and management of maintenance activities to dikes in the area.
- Sketch a new method whereby the costs will be approached as the main driver to reduce the annual costs of RWS on the Dutch flood defence system and its infrastructure.

3. Acronym: RAMSSHEEP

3.1 What is RAMSSHEEP?

The RAMS analysis can be seen as a risk concept that describes the primary performance of all the functions of a system, like i.e. a primary flood defence system. An RAMS analysis can be used in every stage of the life cycle for the entire infrastructure in the Netherlands: road network, major waterways and main waters. This analysis can be used for a complete network but also for small components within a network. Crucial information for RWS as management for the Dutch infrastructure is:

- What is the condition of a primary flood defence system?
- Is the primary flood defence system safe to use and to maintain?
- Does the primary flood defence system fulfill its purpose (function)?
- When is maintenance needed to the primary flood defence systems?

It may be obvious that these questions will be made comprehensible by executing an RAMS analysis: consistency of Reliability, Availability, Maintainability and Safety. The RAMS analysis is an unambiguous method to estimate the risks to a system and it may result in several measures that must be taken to fulfill the system to its requirements. Therefore this analysis can be used by many users in the system, i.e. the client, contractor, and etcetera.

Developments in the risk concept of RAMS analysis lead to the expansion on more societal aspects. Eventually this development led to the RAMSSHEEP definitions. The acronym of RAMSSHEEP stands for:

Reliability

The probability that a system/structure will fulfill its function under certain circumstances and during a specific time interval.

Availability

The probability that a system/structure can fulfill its function at any random moment under certain circumstances.

Maintainability

The probability that a system/structure fulfills its function under certain circumstances during maintenance within the established time frame.

Safety

The absence of unacceptable risks in the system/structure in terms of human injuries.

Security

The guarantee of a safe system/structure with respect to vandalism, terrorism and human errors (including all kinds of sabotage of the system).

Health

The feeling of good health with respect to the physical, mental and societal views. This does not implement if an individual is feeling well or not (subjective argument).

Environment

To meet certain requirements which have been secured in Environmental Acts one suffices the rules of a good and clean environment. The environment can be seen as a physical environment wherein human life is even possible.

Economics

The Cost-Benefit will form a central position in the aspect of Economy. The increase the performance of the RAMS aspects will lead also to an increase of the direct costs. A serious reflection in terms of a Cost-Benefit Analysis must be made to provide more insight for an economical choice.

Politics

A rational decision has to be made based on the aspects above, including also some political aspects.

The goal of this RAMSSHEEP is a protraction of the relation among the reliability, availability, maintainability, safety of the primary flood defence system (Rijkswaterstaat, 2012). The information of this analysis can be translated in RAMS aspects: the performance of the primary flood defence system network, to control the risks in the system and to take measures to improve the performance. This goal of RAMSSHEEP gives a good solution method to the problem which has been formulated.

The RAMS analysis is mainly about the integrality between the definitions subscribed by the individual letters. The RAMS aspects describe what the quality rate of a certain system is when it comes to fulfilling its function. This analysis shows the functional rate of a system, see example below.

A primary flood defence system has a very important main function: the system protects the Netherlands from flooding. Therefore the requirements of the performance of the water retaining function are very strict. When the primary flood defence system suffices to these requirements, it may be assumed that the probability of failure of the system is low enough. This includes that the amount of interruptions during the functioning is limited. When a certain interruption occurs, it must be given that this can be solved immediately or as soon as possible. Only that ensures the safety of the inhabitants and environment.

Especially a *Failure Mode, Effect and Criticality Analysis* (FMECA) is an often used method to create more insight in the essential RAMS aspects and performances. Also, the way of failing of a system can be analyzed by using the FMECA.

3.2 Decomposition and integration of RAMS aspects

The primary flood defence system can be divided in several subsystems. In other words, the required RAMS aspects of a system can be divided in a decomposition of required sub performances for the concerning subsystem. The other way around, the integration of the sub performances of the subsystems, is also possible for the verification of the total system.

The decomposition and integration of the RAMS aspects can be seen in Figure 3-1. The subsystems have different assignments and working levels which must be executed by several parties.

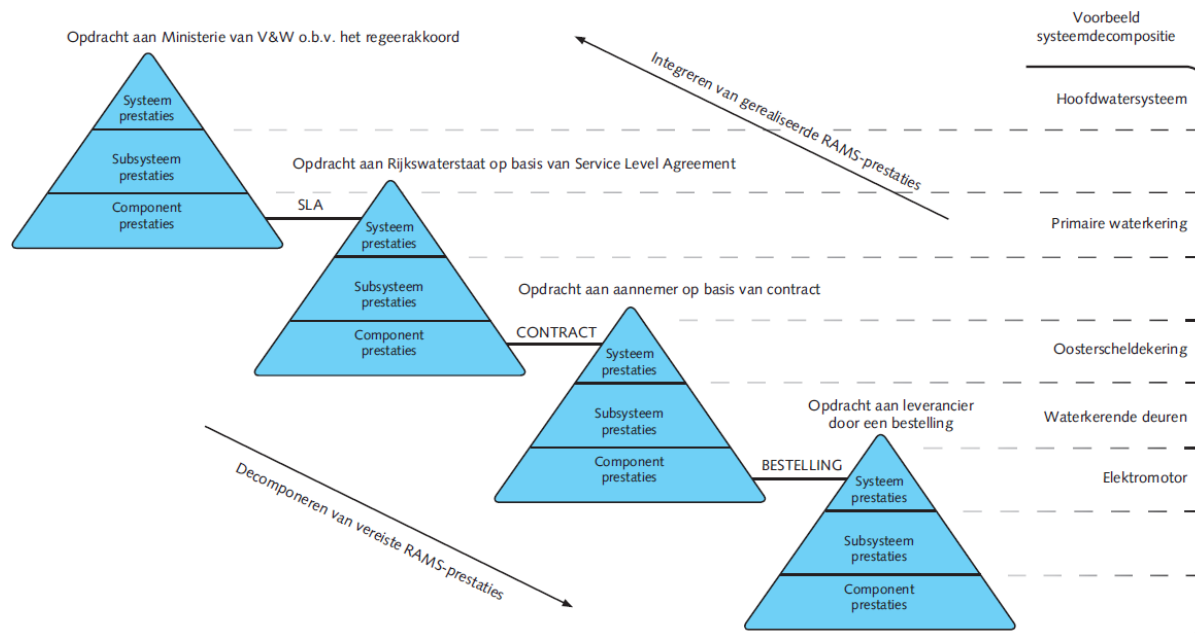


Figure 3-1: The decomposition and integration of the RAMS aspects of a certain system (RWS, 2010).

The RAMS aspects of a system depend on the performances of the subsystems. Also, several parties are involved in the RAMS performances during the lifecycle of a system. The Ministry of Infrastructure and Environment have to prevent the Netherlands from flooding. Therefore Rijkswaterstaat must act according to the following Dutch water law (Waterwet) to ensure a maximum value of the chance of failing. The Afsluitdijk is only one component in the complete system of primary flood defences. Although, one can see in the illustration of decomposition and integration, the lower subsystems will be considered by the executive parties of the total system.

3.3 Functional Failure

The functional failing of a certain system will result in inability to accomplish its predetermined function or fulfill its function on the wrong way. A system has several ways of not functioning (or in other words: failing), like:

- Noticeable failure vs. unnoticeable failure;
- Behaviour of failure over time:
 - Failure due to teething troubles (i.e. initial strength too low);
 - Failure by randomness (i.e. extreme high water levels);
 - Failure by old age (i.e. consolidation of a dike).

The definition of failing can be described as:

A system loses its functionality (or a part of it) due to an event or a combination of several events.

NB. There is a difference between failure and fault of a system. Failure is the occurrence of a basic component failure which will lead to no further breakdown (is possible). Fault is the occurrence of an undesired state for a component/element, subsystem or system. It operates correctly, but at the wrong moment. All failures are faults but not all faults are failures.

Noticeable failure vs. unnoticeable failure

The failure of functionality can be divided in two categories. The first category can be described as the noticeable failure; functional failing of a component which can be directly seen or noticed at the moment the failure is occurring. For example, the failing of the component directly lead to a process stop. The second category can be described as the unnoticeable failure; functional failure of a component which cannot be directly seen or noticed at the moment at the moment the failure is occurring. For example, a certain function is only active at some periods of time. So, that means that an event must occur before a component is failing to fulfill its function. It may be obvious that the unnoticeable failure is far more dangerous than the noticeable failure. The consequences may be the same, but the unnoticeable failure does not make one know there is something wrong what leads to unpreparedness. There are two possibilities to notice these unnoticeable failure moments:

- 1) A functionality test of a specific component; or
- 2) Making use of a specific component to fulfil the function;

As long as the component will not be used or tested, one will not notice a possible failure. An example can be described for a primary flood defence system, see below.

A primary flood defence system is permanently functional. Some parts of the system will only be used when high waters come up. This phenomenon will only occur a couple of times a year or even less. Some parts of the system will be tested by this high water, like the upper part of the revetment of a dike. In combination with extreme waves this revetment can fail due to several happenings, like large wave impact or deformation of the soil below the revetment.

This may lead to functional failure, because the water can pass the dike and endanger the hinterland. The retaining function has been lost.

Unnoticeable failure is very dangerous, because it will happen unexpectedly. So, this phenomenon must be detected, before the system will actually fail, to take necessary measures to prevent this from happening (risk management).

Behaviour of failure over time

In the operation phase of a primary flood defence system the frequency of failure may not be assumed as constant. This frequency will differ over time and lead thereby to different causes to the failure mechanisms.

In general, both the loads and the strengths are functions of time. It is therefore typically of little use to speak of a probability of failure without mentioning the period, to which this refers. In case of only load variations in time, it is wise to define the normative load for the considered period. If the normative load equals the maximum load during the considered period, the probability distribution can be determined by using the theory for extreme values. If fatigue problems are concerned, the normative load must be established by addition of the loads over the period. The calculation of the probability of failure can be carried out with level III methods, after defining the normative load. If both the resistance strength and the load are time dependent, the normative strength and load have to be defined carefully. After all, it is possible that the maximum value of the load does not coincide with the minimum value of the strength (see Figure 3-2 below). In such a case the instantaneous distributions of the strength and the loads have to be assumed.

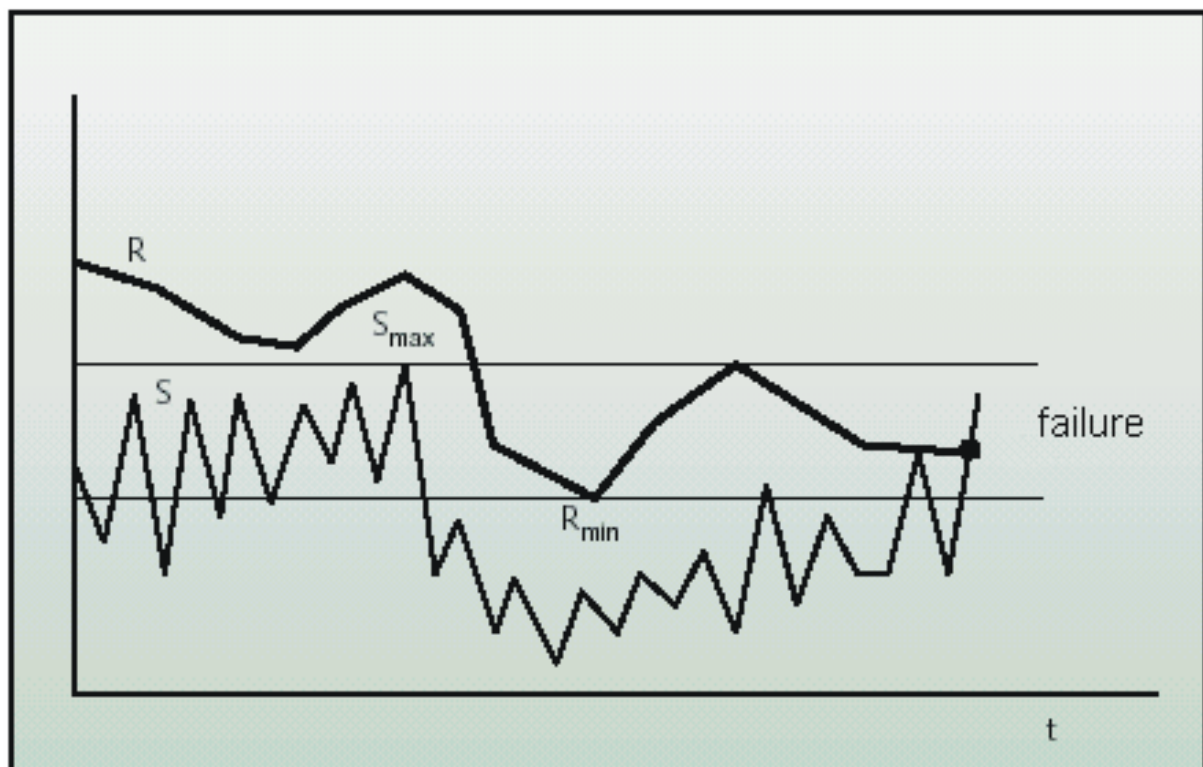


Figure 3-2: Strength and load varying in time (Vrijling, J.K., et al (2002)).

The probability of failure in the time span $(0, t)$ is equal to the complement of the probability that no failure occurs in the interval. Or in formula form:

$$P_f(t) = 1 - P((R(\tau) > S(\tau)) \text{ for all } \tau \in (0, t))$$

In which:

$R(\tau)$ = strength at time τ ;

$S(\tau)$ = load at time τ .

A rough division can be made of three periods which has a dominating cause of failure, see also Figure 3-3:

- Period 1: Failure due to teething troubles;
- Period 2: Failure by randomness;
- Period 3: Failure by old age.

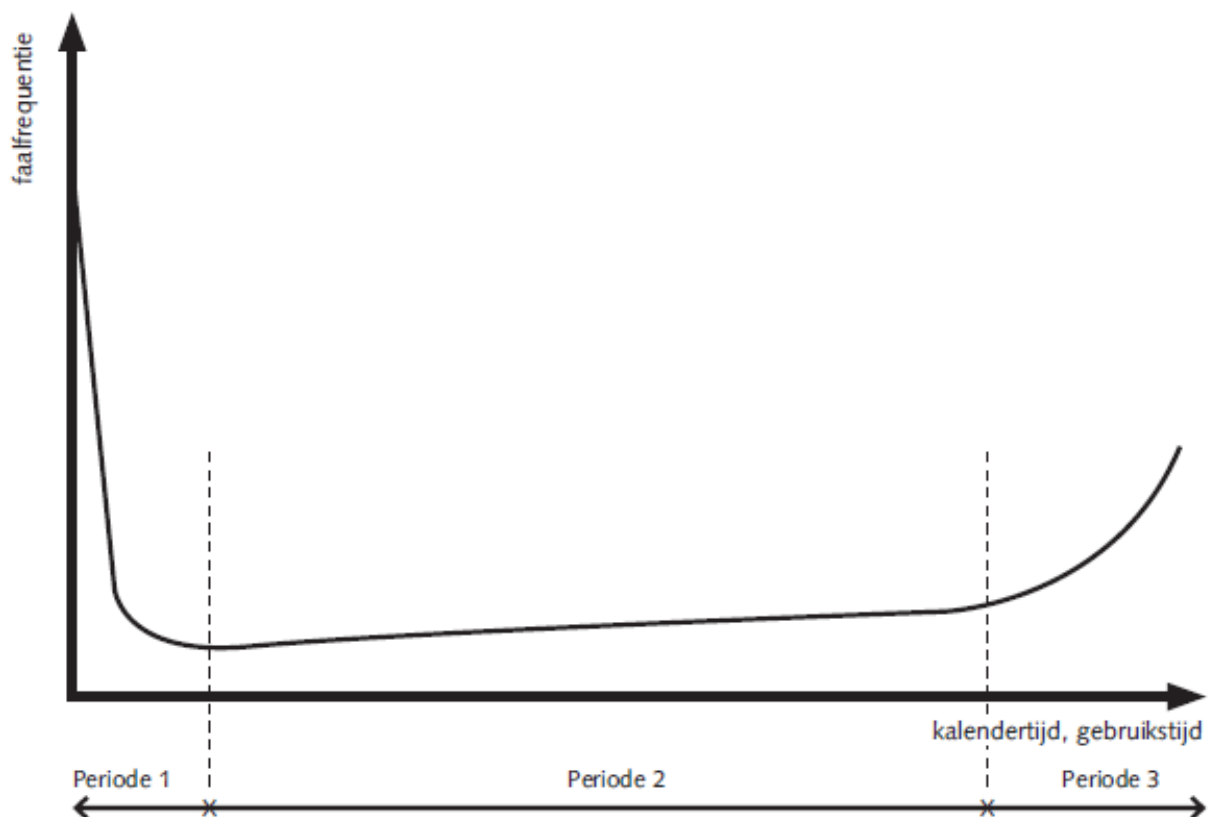


Figure 3-3: A rough illustration of the three periods over time related to the frequency of failure of a certain system (Rijkswaterstaat, 2010).

NB. A remark can be placed by Figure 3-3: this figure represents the behaviour of the frequency of failure of a compiled system and not for individual elements from that system. There are no components which will behave in the beginning and at the end upwards.

Period 1: Failure due to teething troubles

Some components will fail more often when it just has been installed and constructed. These failures are called teething troubles. Most of the time, these failures can be caused by

mistakes in the design of the construction system. Small adaptations or changes must be made to repair construction elements of components to preserve the system from failing again. It may be clear that this type of failure is not applicable for a primary flood defence system like a dike, because this period of time has been passed for many years. Although, when the Dutch government decides to construct a new primary flood defence structure (i.e. Maeslantkering – flood storm barrier) it must be considered.

NB. This period can be characterized by that the standard deviation of the strength (resistance) is way larger than the standard deviation of the load: $\sigma_R \gg \sigma_S$.

Period 2: Failure by randomness

The second period of time does not involve teething troubles or ageing failures, but involve more failure by randomness which does not have anything to do with condition or age. These system failures will mostly occur by external factors and effects, like lightening stroke, extreme loads, and etcetera. This period of time is still applicable for a primary flood defence system like a dike. The external events may cause severe damage to the system and even lead to direct failure of its function. However, the importance of this type of failure is limited to the failure by old age of the elements (see Period 3).

NB. This period can be characterized by that the standard deviation of the strength (resistance) is way smaller than the standard deviation of the load: $\sigma_S \gg \sigma_R$.

In engineering the rate of failure is important. Often a constant rate of failure can be assumed. The probability distribution of the life span can then be denoted as:

$$F_L(t) = 1 - e^{-\lambda t}$$

In which:

$\lambda = r(t)^9 = \text{constant failure rate}$

In this case, the probability density of the life span is:

$$f_L(t) = \lambda e^{-\lambda t}$$

The expected value of the life span is known as the Mean Time To Failure (MTTF) and is determined by integration:

$$\mu_L = E(t) = \int_0^{\infty} t \lambda e^{-\lambda t} dt = \frac{1}{\lambda}$$

The MTTF plays an important part in the determination of non-availability of elements with a constant rate of failure and in planning of maintenance and inspections. If the rate of failure is not a constant in time, this simplification is not possible.

Period 3: Failure by old age

⁹ The term of $r(t)$ can be described as the conditional rate of failure, also known as the “hazard function”.

After a certain time the components and construction elements become older which involves the condition of the system and therefore depends on the behaviour of failing. The frequency of failure and the probability of occurrence over time will increase which also means that the risk level increases. This degradation process will take a more dominant role in the failure process of the system. This risk can be decreased by taking maintenance activities (measures) to that certain element and therefore control the exceedance of probability of failure a bit more. This type of failure is the most important for the primary flood defence system, because most of the elements will fail due to degradation. The focus of this research will be mostly determined by this period of time where maintenance activities are necessary to ensure the set safety level by the Dutch water law (Waterwet).

NB. This period cannot be characterized by a ratio of the standard deviations of the strength and loads, but this is a function of the time (S(t) and R(t)).

3.4 Reliability

The reliability of a system is the probability of non-failure during a given period of time. Therefore the frequency of failure can be seen in direct relation to the reliability. The definition 'probability' will generally be expressed in a chance. The reliability gives the chance that a system can fulfill its function during a given period without any failures. The definition of the reliability can be given by (Rijkswaterstaat, 2012):

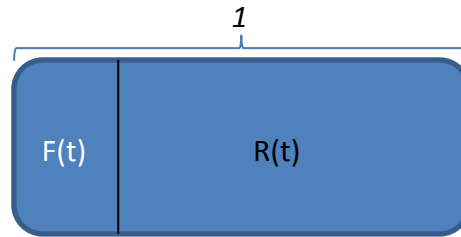
The probability that a system will fulfill its function under certain circumstances and during a specific time interval.

For example, the reliability of a primary flood defence system can be expressed by the chance that the system is able to function without any technical trouble. In the Netherlands this regulatory chance is given by the Dutch water law (Waterwet) which has a safety level ones in the ten thousand years (1/10.000 per year). The functional period will often be expressed in the unit of time. The period between failure events (Mean Time Between Failure (MTBR)) is an important rate for the reliability and has a direct relation; the longer the mean time between two failure events, the more the reliability increases.

However, in some cases the unreliability will be used instead of the reliability. The reliability and the probability of failure can be seen as the opposite. In case of the primary flood defence system (continuous system) the MTBR is inversely proportional to the frequency of failure, i.e. if the frequency of failure decreases, therefore the MTBR and the reliability increases. However, the time periods between two future failure events cannot be determined with exact numbers, and will therefore be expressed in a certain chance.

The relation between the reliability and probability of failure (unreliability) has been formulated as:

$$R(t) = 1 - F(t)$$



Notification of the parameters:

- $R(t)$ = function of the reliability
- $F(t)$ = function of cumulative probability of failure

In Figure 3-3 the probability of failure has been illustrated what can directly be translated to the reliability of the system by creating the inverse function of this probability of failure.

The function of the reliability will be based on two definitions:

- 1) What is the definition of system failure? (see sub paragraph 3.3)
- 2) Determine the time definition (i.e. calendar time, amount of cycles, operating time, and etcetera).

It is important to formulate these definition carefully, because there is a strong mutually relation. The time definition for primary flood defence systems has been given in calendar years which also can be used for the definition of failure.

The reliability is changing over time and therefore also depends on new elements in the system or on maintenance activities/replacements of a certain element. In line with this, the reliability will decrease over time due to usage of the system.

NB. Looking back at the previous paragraph and Figure 3-3 it may be obvious that this graph has a direct link to the reliability graph above. The first period 'teething troubles' of Figure 3-3 indicates that the frequency of failure rapidly decreases over time what can be translated to the figure above from a relative larger section of $F(t)$ to a smaller one. In the second period 'failure by randomness', where the frequency of failure is more or less constant, can be seen as a constant distribution between $F(t)$ and $R(t)$. In the last period 'wear' the frequency of failure increase more rapidly than before due to degradation of the elements in the system what therefore also indicate an increase of $F(t)$.

Reliability Analysis (calculation methods)

The reliability of a single element can be calculated by several methods expressed by the probability of failure, like:

- First Order Reliability Method (FORM);
- Second Order Reliability Method (SORM);
- Monte Carlo (MC);
- Numerical Integration (NI).

The limit state function will be used as a probabilistic calculation for the failure mechanism model (Steenbergen *et al*, 2004). This limit state function can be formulated as:

$$Z = f(\underline{x})$$

Notification of the parameters:

- $Z < 0$: failure
- $Z = 0$: limit state
- $Z > 0$: non-failure
- \underline{x} = vector of stochastic variables

This limit state function needs hydraulic loading models and can be formed by several elements, like statistics of the stochastic variables, correlation between these variables and load parameters. The global stochastic variables will exist of water levels, wind directions, wind speed, and etcetera. The primary flood defence system can be considered as a series system¹⁰ and the lower bound of the probability of failure of the system is equal to the maximum probability of failure of the different sections. However, the upper bound of the probability of failure of the system is the sum of all individual probabilities of failure.

Therefore the probability of failure can be formulated as:

$$P_f = P[g(\underline{x}) \leq 0] = \int_{g(\underline{x}) \leq 0} f_x(\xi) d\xi$$

Notification of the parameters:

- P_f = probability of failure
- $f_x(\xi)$ = joint probability density function of \underline{x}

Usually the reliability index β is used, because this has the advantage that it is easier in use and proportional related to the safety level. The reliability index is related to the probability of failure by:

$$\beta = \phi^{-1}(P_f)$$

Notification of the parameters:

- β = reliability index
- ϕ = standard normal distribution
- P_f = probability of failure

¹⁰ A series system of several failure mechanisms is considered to have failed if any of the failure mechanism fails. The probability of the union can be determined as: $P_f = P[\bigcup_{j=1}^m E_{fj}]$ with E_{fj} = the event that a certain failure mechanism has failed.

3.5 Availability

The availability is a theoretical rate of time of which the system can be functional. This availability can be expressed in a chance which will generally be given by a percentage of the time. The definition of the availability can be given by (Rijkswaterstaat, 2012):

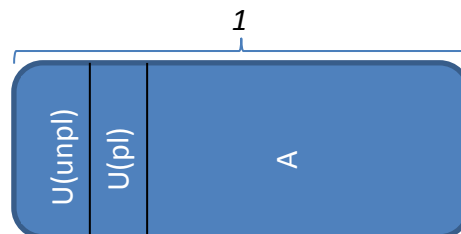
The probability that a system can fulfill its function at any random moment under certain circumstances.

The definitions of the availability and unavailability can be seen as the opposite; the time period when a system is available is inversely proportional to the time period when the system is unavailable. The focus lays most of the time (logically) on the unavailability. This unavailability can be divided over several causes:

- Unavailability due to planned maintenance activities;
Planned or foreseen causes of unavailability fall within the scope of planned testing of the condition or repair/maintenance activities.
- Unavailability due to failure events;
Unplanned or unforeseen causes of unavailability are functional failure, interruptions or unexpected replacements of several components. Also, extreme weather circumstances can introduce direct failure (insufficient sight: ship collision; extreme wind and water level: wave overtopping/overflow).

The relation between the availability and unavailability has been formulated as:

$$A = 1 - U = 1 - (U_{unpl} + U_{pl})$$



Notification of the parameters:

- A = availability;
- U = unavailability;
- U_{unpl} = unavailability due to failure events;
- U_{pl} = unavailability due to planned maintenance activities.

There is a direct relation between the planned and unplanned unavailability of a system. For example, the planned unavailability (preventive maintenance, inspection, and etcetera) may lead to less unavailability due to failure events and vice versa. In other words, a good management and preventive maintenance activities on a system can lead to less interruptions and therefore less unplanned failure causes, which may result in a more controllable and predictable performance level of the system.

Example of availability

A primary flood defence system can be considered as a continuous operative system and therefore the availability will be expressed by a mean value (percentage of the operating time). The duration of a system which is not able to fulfill its function is important to know and because the primary flood defence system can be assumed as a continuous system, this is the same as the duration of the unavailability of that system.

A primary flood defence system has an annually unavailability which can be divided over 10 days unavailable due to planned maintenance activities and 9 days due to unexpected failure. The remaining time of the year the system is fully functional and no interruptions will occur. The annual availability of the primary flood defence system can be calculated by:

$$A = 1 - U = 1 - (U_{pl} + U_{unpl}) = 1 - \left(\frac{10}{365} + \frac{9}{365} \right) = 1 - \frac{19}{365} = 0,95$$

Or in other words, the annual availability of this particular system can be read as 95%.

3.6 Maintainability

The maintainability indicates the ease to maintain a system (1) to prevent the system from functional failing (planned unavailability) and (2) the time to repair the system due to functional failure of the system (unplanned unavailability). It also indicates the easiness to execute the maintenance activities on the system and thereby also at which moments maintenance can be executed during operation phase. Nevertheless, maintainability does not only gets affected by the technical factors of the system, but can also be influenced by the Health and Safety at Work Act (USA 1997) and the availability of sufficient trained personnel. The definition of the maintainability can be given by (Rijkswaterstaat, 2012):

The ease to maintain a system to decrease the probability of failure under certain circumstances within the established time frame.

The maintainability can be seen as a quality of the system; the system is unmaintainable when one is not able to maintain the system within the determined requirements. The theoretical RAMS analysis approaches the maintainability as a probability which will be used in practice as a quantified chance.

It is important to realize that maintainability does not implement anything about the maintenance as in planned activities or measures that must be taken. The maintenance activities on the system must be executed according the maintainability requirements during the operation phase. In some cases when one determines that the maintainability cannot meet the requirements, the design of the system can be adapted to a situation where it will meet the requirements.

3.7 Safety

The safety indicates the danger for humans due to the presence of the primary flood defence system. The definition of the safety can be given by (Rijkswaterstaat, 2012):

The absence of unacceptable risks in the system in terms of human injuries.

An overview of the possible safety themes has been given in the Dutch manual 'Leidraad Integrale Veiligheid (2009)'. The several safety themes can be determined for a primary flood defence system. This analysis gives more information and insight on the safety aspects of the system. To determine the safety of a specific system it is important that the system can be divided in the seriousness of possible harm and the type of safety; for example one differentiates the user's safety from the health and safety of labour men. Also, the safety function of the system must be analysed on the effects of wrongly executed safety functions (like activation of the sprinkler system in a tunnel). The road traffic safety has been split up in traffic safety (open air) and tunnel safety (closed air). At last, external safety differentiates risk on a specific place and group risks.

The RAMS analysis will sometimes be analysed one by one, but mostly the aspects will have many overlaps. Especially when a primary flood defence system (function of increasing the safety of society) has been analysed: the safety analysis will mostly exists of reliability analysis of its safety function. The safety may mathematically be assumed equally to the reliability, but one has to realize that the consequence will lead to loss of life instead of failing its primary function. Beside the safety analysis of the system, one also has to consider the safety of the system from a user's and environmental point of view at moments when the safety function cannot be executed. This means the safety of the labour men during construction, testing and maintaining of the system.

3.8 Integration of RAMS aspects

The previous paragraphs describe the four aspects as an individual analysis which is not the original idea of the RAMS analysis. A certain analysis acts and reacts on another analysis. However, all the individual analysis indicates the performance reliability of a functional system. The RAMS analysis is a special one, because the direct relations to the other three aspects are very strong: an adaption in one analysis will influence the other and vice versa. The RAMS analysis is often divided over three analyses (Rijkswaterstaat, 2012):

- Performance reliability analysis (RA-analysis);
- Maintenance analysis (M-analysis);
- Safety analysis (S-analysis).

Performance reliability analysis (RA-analysis)

The performance reliability analysis indicates the amount of system failure, or in other words the failure frequency, and the mean time to repair this system failure, or in other words the unavailability. Therefore, this analysis has been based on the RA-aspects. The system failure can be caused by technique, processes, human errors or surroundings or a combination of these possible causes, and it gives more insight in the unavailability of the system due to system failure of unplanned causes.

Maintenance analysis (M-analysis)

The performance reliability analysis will not be sufficient to determine the unavailability. Beside the unavailability due to system failure of unplanned causes, there is also unavailability of the system due to planned maintenance activities. Therefore, a maintenance analysis will be done. This analysis gives more insight in future maintenance activities that must be executed and the frequency and total maintenance time of these actions. This may form the base for planned maintenance actions at the most favourable moments, like evenings, weekends, holidays, low tide, and etcetera.

This maintenance analysis does not only indicates the technical factors, but also considers the Health and Safety at Work Act (USA 1997) and if the system will be maintained and managed by competent personnel.

The power of the RAMS analysis can only fully be developed by combining the four analyses together and not to approach each analysis on its own. This is often the procedure, because many aspects of the RAMS analysis will overlap, which automatically result in a lot of cohesion among the four analyses.

With the results of individual analysis an optimal design can be realized. The four aspects of RAMS have an interactive character which has been illustrated in Figure 3-4.

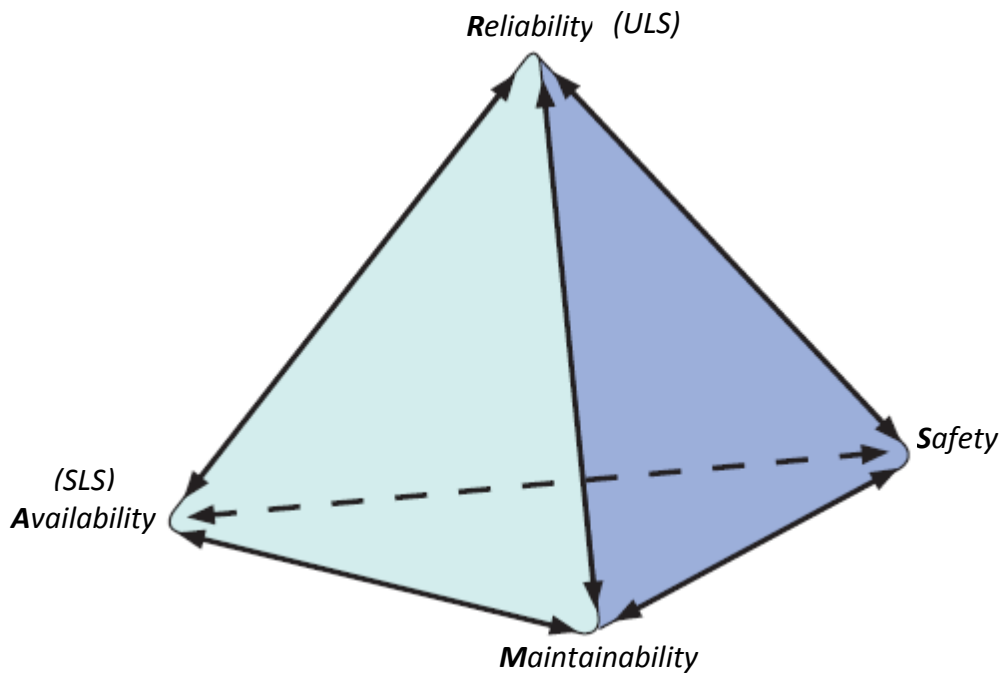


Figure 3-4: The direct relations among the four RAMS aspects (Rijkswaterstaat, 2012).

The RAMS aspects act on a common phenomenon: the system does not fulfill its function the way it should function (interruption, damage, and etcetera):

- The safety risks will increase due to this not functioning.
- The availability of a function depends on the amount of interruptions and the mean time to repair (MTTR) this interruption.
- The frequency of interruptions is directly related to the reliability of the system.
- The maintainability will be influenced by the time of repair or replace.

The reliability and availability have a very strong relation and during the analysis these two aspects will be considered at the same time. The reliability indicates the chance of a functional system over a certain time period. However, the availability indicates if a system can be functional on a random moment in time. On top of these two the maintainability is also playing an important role. The reparation time (MTTR) influences the maintainable system: a small MTTR has a positive influence on the availability. Nevertheless, the reliability cannot be influenced, because one failure event in a certain time period is governing.

(1) From reliability to availability and maintainability

A primary flood defence system generally fails ones in the ten years due to the many extreme weather moments (tornados, and alike). This knowledge is an indication for the reliability of the system, but it is still not possible to say anything about the availability. Some extra information must be required; how long is the flood defence system not functional due to the interruption. This time can be calculated by the sum of the time until the failure has been noticed and the time which is needed to repair the system (availability). The unavailability increases linear when the reparation time (MTTR) has been increased.

The example above gives a clear relation between the reliability and the availability. Here becomes clear that it might seem that if a system has a high availability, it should also have a high reliability. However, this is not the case. The reliability does not account for any maintenance activities (like repair or replace actions) that may take place. It only accounts for the time that it will take the system to fail while it is operating. Therefore it does not reflect how long the system will take to get the unit under repair back into working condition.

Nevertheless, the availability can be seen as a function of reliability and also as a function of maintainability. Below an overview has been given of the possible reactions of the availability of a system, see Table 3-1.

Table 3-1: Relationship among reliability, maintainability and availability.

Reliability	Maintainability	Availability
Increases	Increases	Increases
Increases	Constant	Increases
Increases	Decreases	Depends
Constant	Increases	Increases
Constant	Constant	Constant
Constant	Decreases	Decreases
Decreases	Increases	Depends
Decreases	Constant	Decreases
Decreases	Decreases	Decreases

As one can see from Table 3-1, if the reliability is held constant, even at a high value, this does not directly imply a high availability. As the time to repair increases (so the maintainability decreases), the availability decreases as well. Even a system with a low reliability could have a relative high availability if time to repair (MTTR) is short (which has a direct relation with the maintainability).

(2) From availability to reliability and maintainability

The availability of a primary flood defence system depends on the amount of system failures, which means that the system cannot fulfill its primary function. Logically, therefore if the system is not safe to use due to the many system failures, the system must be repaired or replaced. The failure frequency (reliability) and the mean time to repair the system (maintainability) eventually determine the availability. This failure frequency can be decreased by planned and preventive maintenance to the system. However, for some maintenance activities the system will not be available which also has a negative influence on the system. But the management can choose a particular moment when the damage (economical/trouble of users/etcetera) will be as low as possible, which lead eventually to a higher net availability.

The system is not only unavailable due to failure of the system, but also due to the planned and preventive maintenance activities. The impact of this unavailability is clearly less than due to failure.

In case of a primary flood defence system, which can be seen as a continuous operation system, the reliability and also the availability must both be high. The amount of

interruptions or repairs must be as small as possible (reliability) and the mean time to repair must be as short as possible (maintainability). This combination results in a high availability, see also Table 3-1. Therefore one could say to make only requirements based on the availability, but this can be misleading. For example, a continuous system that will fail 10 times and takes just 1 hour of reparation cannot be assumed equally as 1 fail event of 10 hours of reparation. The difference can be expressed in extra reparation time, costs and/or perception. This implies that the reliability does play an important role and cannot be implemented in the availability. Moreover, system failure will also lead to unsafe situations which may form a direct reason to immediate maintenance/repair activities.

3.9 SHEEP Analysis VS. Sustainability

In the previous paragraphs the original RAMS aspects and their direct and indirect relations have been analysed and elaborated which lead to the next part of the RAMSSHEEP analysis: SHEEP aspects. The consequences of an interruption of the desired execution of the function are not always the same. These consequences are often divided over several categories:

- **Security;**
- **Health (working condition);**
- **Environment;**
- **Economy (money);**
- **Politics (image).**

It also may be clear that it is possible that some consequences can be considered in more than one category. Moreover, one should always make decisions based on economic backgrounds.

The SHEEP analysis has many connections to the sustainability, which also considers the results of social, ecology and economy aspects. A sustainable system is constantly balancing among the three P's (Van Breughel & Fraaij, 2007):

- **People (social quality, health, liveability, freedom, freedom of choice, safety)**
This social result considers people with any connection with the system. Questions will arise like: How are the working condition for maintenance workers? What relation does the system have with the environment? Which people do have a direct or indirect relation with the system?
- **Planet (energy, water, material, mobility, waste, purity)**
This environmental result considers the effects (as well as positive and negative) on the environment. Questions will arise like: What are the consequences to the direct environment due to system failure? Will maintenance activities influence the direct environment?
- **Profit (economic quality, profit, pay ability, transparency, honesty)**
This economical result considers financial efficiency of a system. Questions will arise like: What is financial the most economical point for maintenance?

What is the price to decrease the risk? Is it economical more favourable to take risk or to decrease this risk?

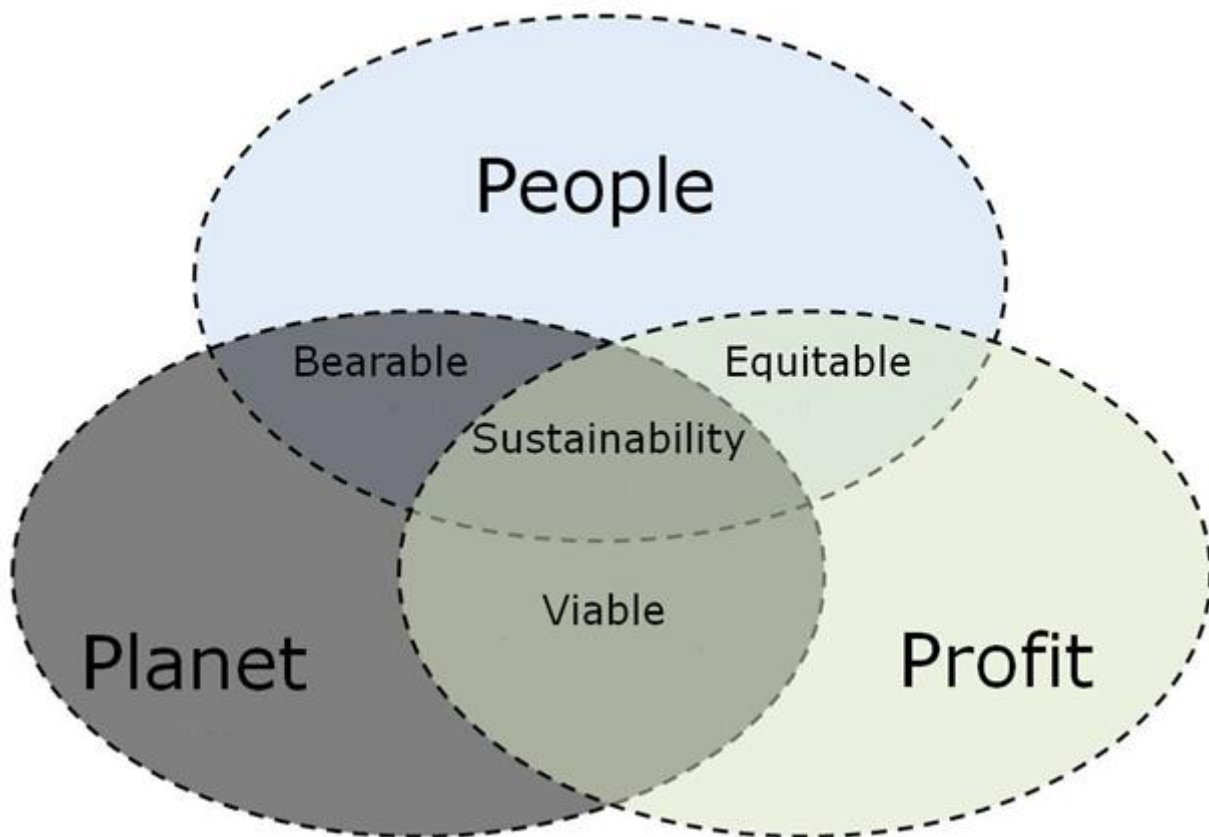


Figure 3-5: (Meta-model) Triple P: People, Planet and Profit connected to the sustainability (http://www.gertjanschop.com/-/modellen/people_planet_profit.html).¹¹

The triple P concept has obvious overlap with the SHEEP analysis. The security and politic aspect have overlap with the triple P concept, but in a lesser extent. The triple P brings some sustainable definitions to a higher level which can be applied for a primary flood defence system:

- Ecological environment
The relation between atmosphere, earth, water and noise will influence the wellbeing of flora, fauna and humans (or in other words: the direct natural surroundings)
- Usage quality
A syntheses of the function, shape and technique within the given requirements like time, money and legislation and regulations.

¹¹ Sustainable development is development that fulfils our current requirements without to jeopardize the individual needs of the future generation. (*Brundlant, former premier of Norway*)

- Health
The condition of physical, mental and societal wellbeing and not only the absence of disease and other physical damage.
- Future value – sustainable development
Consideration of the needs of the future generation during the development of the current requirements.
- Safety
The absence of possible jeopardy on the condition of an individual or a group.

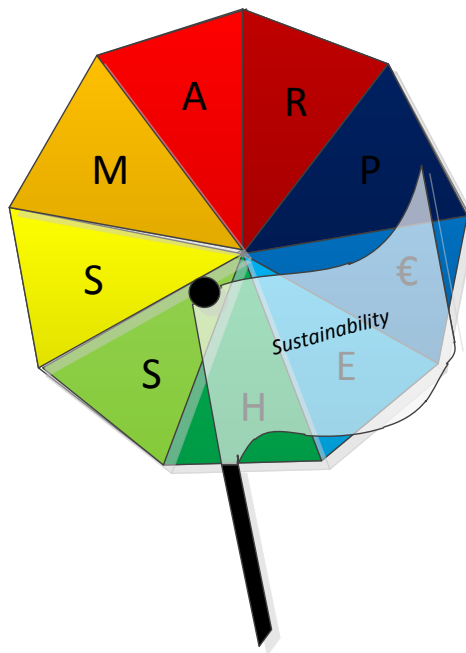


Figure 3-6: The sustainability in RAMSSHEEP.

3.10 Security

The security indicates whether a system can be assumed safe based on the security issues like vandalism and unreasonable human behaviour. This also includes terrorism and other way of sabotage to the system. The definition of a secure system can be given by (Rijkswaterstaat, 2012):

The guarantee of a safe system with respect to vandalism, terrorism and human errors (including all kinds of sabotage of the system).

The distinction between the safety and security may be confusing but is necessary to implement different risk angles to the system. An important reason for this distinction is the fact that security related risks are hard to quantify, mostly because of lack of data of these risks. A possible second reason is that these risks can be assumed to have an intentional character which means that it is human's intention to damage the system. Originally this fundamentally deviates from the other risks based in the RAMS analysis.

At this moment violence, vandalisms and terrorism are hot topics in the world. Therefore, for the primary flood defence system it is strongly recommended to execute a security analysis, because of the serious consequences of system failure. Moreover, managers must have a reliable way of estimating risk to help them decide how much security is needed at their system. A security analysis can be made using a methodology which is based on the traditional risk equation (Sandia Corporation, 2000):

$$R = P_{attack} \cdot (1 - P_{effectiveness}) \cdot C$$

Notification of the parameters:

- R = risk due to a certain security threat
- P_{attack} = likelihood of adversary attack
- $P_{effectiveness}$ = security system effectiveness – so $(1 - P_{effectiveness})$ = Adversary success
- C = consequence of loss to the attack

The first step in a security analysis is to characterize system operating states and condition. This can only be done if a thorough description of the system itself has been made (location, site boundaries, construction locations, floor plans, access points, and etcetera). This also means the location of pumping stations, lock and bridge management buildings, and etcetera. Beside this, identification must be made of any existing physical protection features, like alarms, fences, and etcetera.

The system failure will occur due to site specific undesired events which will result in undesired consequences, like adverse impacts on public health and safety, the environment, functions, and publicity. A fault tree can be used to identify these critical components for prevention of the undesired events. The goal of this fault tree is to estimate the relative consequence value. The undesired events which lead to system failure can be listed as:

- disruption of operations (vandalism);
- theft of assets (theft);
- crime against persons or traffic (crime);
- destruction of property (terrorism);
- negative publicity or embarrassment;
- ...and etcetera.

The consequences will be considered in categories like amount of deaths/injuries/illness, functional loss and environmental damage.

3.11 Health

The health analysis mainly indicates the wellbeing of an individual as well as the society. This wellbeing will be determined on aspects like physically, mentally and in societal view. The definition of the health can be given by (Rijkswaterstaat, 2012):

The good health with respect to the physical, mental and societal views. This does not implement if an individual is feeling well or not (subjective argument).

One has to make an objective judgment on these aspects, and not like if an individual is feeling well (because that is a subjective judgment). An objective judgment will be based on the general health data of being well, because every individual will judge differently about being physically and mentally healthy or not. This general health data has been gathered in Health and Safety at Work Act (USA, 1997).

The health aspect in the SHEEP analysis forms the main part of the health and safety at work. That part has also been discussed in the safety analysis (S-analysis), so this is a bit redundant. These risks will be determined by making a risk inventory and evaluation.

3.12 Environment

The environmental analysis indicates the condition of the physical surroundings of the system whereby human life is possible and takes place. The environment can be seen as a social good and due to the fact that it is not possible to put a price to contamination of the environment, a regulation has been made of the allowed amount of contamination. The definition of the environment can be given by (Rijkswaterstaat, 2012):

To meet certain requirements which have been secured in Environmental Acts one suffices the rules of a good and clean environment. The environment can be seen as a physical environment wherein human life is even possible.

The influence on the environment due to the present of the system can be indicated by the environmental impact assessment (EIA). This procedure has been determined in the Environmental Management Act. This procedure of EIA can be used for the verification (or optimization) of environmental requirements of a primary flood defence system.

The Netherlands is a subsiding country and combined with rising sea level, this will form a direct reason to strengthen the primary flood defence system. Similarly, the population and the economy will grow which leads to larger villages near the coast. These two situations have opposite goals; free space near the coast must be preserved for strengthening the primary flood defence system, but will also be used due to increase the population. This needed space must be reserved for strengthening the dikes, because otherwise the strengthening cannot be done further inland. Therefore a certain zone must be reserved to ensure the future plans.

The core of the EIA will be formed by an environmental document that consists of environmental effects of the system, as well as positive and negative. The environmental

effects can be seen as consequences of nature, landscape, recreation, cultural heritage and morphology. Beside these effects also alternatives must be described of spatial zones which should be secured including their environmental effects. The EIA exists of several elements that must be considered (Gerrits, Groen & Schippers, 2006):

- a description of system;
- a description of the current environmental condition and the future changes due to population, weather conditions, and etcetera;
- a comparison of the possible consequences of future changes of the expected developments to the system with respect to the environment.

To complete the procedure of the EIA, stakeholders like governmental institutes, provinces, municipals, water boards, and etcetera. Also, the scope and level of detail of the EIA must be determined.

The goal of the EIA is to create a base document for the spatial zones that must be ensured for possible strengthening of the primary flood defence system. The ensured spatial zones must ensure the safety against flooding of the coming 200 years. Therefore it is necessary to analyze the expected changes.

3.13 Economics

The economic analysis indicates the direct relation between the costs and the values of a certain system. It may be clear that the economic aspects have a very strong relation with the other analyses of the RAMSSHEEP, because if one is increasing or decreasing the RAMSSHEEP performances this will result in consequences to the costs within the system. Therefore this topic can be seen as the driving force behind every other possible aspect. The definition of the economics can be given by (Rijkswaterstaat, 2012):

The Cost-Benefit will form a central position in the aspect of Economics. The increase the performance of the RAMS aspects will lead also to an increase of the direct costs. A serious reflection in terms of a Cost-Benefit Analysis must be made to provide more insight for an economical choice.



Figure 3-7: Schematic illustration of the cost-benefit analysis.

To make the most optimal choice among the RAMSSHEEP elements it is advisable to work with the method of the Cost-Benefit Analysis (CBA) over a certain time period. This method describes a systematic process for calculating and comparing costs and benefits of a certain system change to see how it compares with alternate solutions and therefore other different values among the RAMSSHEEP aspects; in other words one is ranking the alternatives. It involves comparing the total expected costs of each solution possibility against the total expected benefits. Therefore both costs as well as benefits are expressed in money terms, which also need to be adjusted for the time value of money; all flows of costs and benefits over time are expressed on a common basis in terms of their *present value*, using a suitable discount rate.

The valuation of costs is most of the time not a problem, because this term can be determined by costs of construction of the solution with process costs added to that value. These values are most of the time already given in a financial unit (€, \$, etcetera). Nevertheless the valuation of benefits is not so straight forward. The overall benefits are often evaluated in terms of the public's willingness to pay for them, minus their willingness to pay to avoid any adverse effects. So environmental safety can be measured in terms of 'cost per population saved' or 'cost per prevented environmental damage', without placing a financial value to the life and damage itself. Besides, most of the time it is not feasible to put a decent price to i.e. damaging (eroding, polluting, and etcetera) a unique landscape. Moreover, the CBA indicates the future benefits with large uncertainties to the future real situation. So, it is not possible to express ratio costs versus benefits in just one number. Uncertainties must lead to certain (controllable) boundaries.

With respect to a primary flood defence system this Cost-Benefit Analysis will primarily be used during adaptations to the design due to increasing safety levels. Above all, CBA can also be very effective during maintenance and management activities. To ensure the performance quality of the system some investments must be done over a certain time period. It can be profitable to compare different moments of investment to the system resulting in the lowest costs in the systems lifecycle, but also guarantee the required benefit

level¹². Costs and benefits can be expressed in several ways, like money or on a more qualitatively basis. This qualitatively basis can help making decisions on alternatives, uncertainties and the dividing ratio of several effects (Eijgenraam *et al*, 2000).

Some examples can be given based on costs of a maintenance action on a primary flood defence system:

Direct cost effects

- Investments to fulfill its function at the initial time moment;
- Maintenance to ensure the quality of the system compared with the new initial moment.

Direct benefit effects

- Safety level will be met.

Indirect benefit effects

- Security of weak spots can be improved;
- Environmental improvements for flora and fauna;
- Feeling safe below sea level.

So the Benefits minus costs give an indication number between two boundaries; lower boundary and upper boundary. The decisions of effects which cannot be expressed in a given value will be determined by politicians.

3.14 Politics

In some cases the decisions will not be made based on performance aspects of a certain system, but on political grounds. This means that not all decisions will be made on rational reasons of other aspects, but also some political aspects will be involved. The definition of politics can be given by (Rijkswaterstaat, 2012):

A rational decision has to be made based on the other aspects of acronym RAMSSHEEP, including also some political aspects.

The term political represents political management and societal aspects, but also the more general administrative aspects based on developments and maintenance and management of the system. These political aspects do not make decisions based on models and calculation like in RAMS. It is advisable to determine whether a decision will be made based on RAMS aspect or based on political grounds.

¹² Asset management is useful to indicate when to invest to keep the total costs at the end as low as possible.

3.15 Discussion

The discussion part will mainly discuss individual aspects of RAMSSHEEP. The discussion about individual aspects will be analyzed individually and in combination with other aspects (integrality). This can therefore be seen as a critical remark on the current definition of the RAMSSHEEP aspects.

The theory of RAMSSHEEP is still very young and has also some teething troubles. Therefore, the analysis will show weak links within this theory and be helpful to define the RAMSSHEEP better and more concrete. Besides, this involves also the extra value of the theory in comparison with other known models.

The discussion subjects are:

- 1) The RAMSSHEEP aspects do not act on the same operating level; therefore it is hard to control the system.
- 2) The RAMSSHEEP analysis has been developed for all life-cycle phases of the system.
- 3) The reliability will be influenced by all the other aspects and therefore can be expressed in a certain value which decides whether the system suffices or not.
- 4) The availability of a dike is not applicable because this always has to be 100%. However, it is applicable for other parts of the system, like a lock or road connection.
- 5) Maintainability of a system which already has been constructed, is not more important than the maintenance activities itself.
- 6) The security of a primary flood defence system based on terrorism and sabotage is limited important in comparison with the security of other soft targets (high potential opportunities).
- 7) The health aspect indicates the well-being of humans which are relevant to the system; this implies a health ratio before, during, and after function failure of system.

The seven discussion subjects from above will be elaborated:

- 1) The RAMSSHEEP aspects do not act on the same level and is therefore not the most optimal way to use these aspects to control the system. The individual aspects do have (of course) many (in)direct relations with each other, but it does not guarantee the same operating level. For example, one should take a look at the Politics aspect; this aspect mainly can be applied on the highest level of the system. This highest level of a primary flood defence system is a governmental organization, like Rijkswaterstaat. If one is applying RAMSSHEEP on an single element or even a larger part of a sub system the Politics aspect will not be applicable, because it only considers the final decisions to construct/maintain the system. However, aspects which can be decomposed to the lowest possible level (a single element) are the Reliability, Availability, Maintainability, Safety, and Economics (RAMSE). The other individual aspects (Security, Health, and Environment (SHE)) mostly consider a part of the sub system which acts on a lower level than the Politics aspect, but on a higher level than the RAMSE aspects.

The origin of RAMSSHEEP implies that these aspects can be used to consider a complete system (top level) and works its way downwards (decomposition of the system). The

original purpose of RAMSSHEEP cannot be achieved according its own definitions of the individual aspects.

- 2) The RAMSSHEEP analysis has been developed for the complete life-cycle; development, design, construction, maintenance, and demolition phase. This can be seen as an advantage. One is able to control a system from initial to demolition phase. In comparison with other methods, RAMSSHEEP will lead to a uniform way of documenting and a complete overview over a certain period of time. Although, it is necessary to make a side note, because not all the individual aspects have the same influence in the different phases (see discussion point 1).
- 3) The reliability of the primary flood defence system forms one of the most important information. There is a direct connection with other aspects which will influence the reliability aspect. Therefore, it is necessary to test this reliability to a given requirement which must be expressed in an absolute value (i.e. percentage). This is the only way to create a result which indicates that the system suffices or not. The reliability directly depends on the availability and maintainability (integrality of RAMS aspects). Moreover, the system must also suffice to the SHEEP aspects of the analysis. These aspects are mostly expressed in a qualitatively way and not by absolute numbers. This makes it hard to base a respectable judgment on it. The reliability will be influenced by all the other aspects (which do have both qualitatively (no hard numbers) and concrete requirements) and therefore can be expressed in a certain value which decides whether the system suffices or not.
To conclude, the reliability (and availability) aspect is relatively more important than other aspects. Although, the RAMSSHEEP analysis does not indicate this distinction and put all aspects on the same level of importance.
- 4) The availability is not applicable for a primary flood defence system, because the primary function (retaining water to flow land inwards) must always be fulfilled. This can be translated to an availability number of 100%. However, most primary flood defence systems have more sub functions beside primary function. For example the Afsluitdijk, beside retaining water also functions as a direct road connection between Noord-Holland and Friesland, fresh water basin used for a source of drinking water, navigation passage, and etcetera. For these sub functions the availability is in fact necessary to analyze. It will usually be expressed in a percentage of time in which the system has to be functional. This percentage only indicates the total time and not the frequency of availability of the system. This can be a crucial detail with respect to the economic consequences. For example, the economic consequences for a lock complex with an unavailability of one day are larger than an unavailability of six times four hours. The unavailability of six times four hours, which can be seen as delay, can be 'repaired' over the day, but the unavailability of one day may lead to less customers which eventually lead to higher costs and lower benefits.

So, the availability should be more than just one number which indicates the availability over a certain time period (like it is implemented by RAMSSHEEP) and therefore add a requirement of the maximum amount of time for just one moment of unavailability.

- 5) The maintainability in RAMSSHEEP for a primary flood defence system, which already has been constructed, is based on the ratio of easiness and safety of maintaining the system. This aspect will generally be considered during the development and design of the system, but not when it has already been built. In these situations the maintenance is more interesting to consider. The management is mostly interested in when, how and which elements must be maintained. Therefore, the maintenance activities can be planned over time based on current conditions. This condition can be indicated by using results of planned inspection, continuous monitoring and make use of prediction deterioration models based on historical data of the system.

Although, this aspect of RAMSSHEEP is complicated because of the direct relation between maintainability and maintenance to the system. Maintainability can be changed over time, because of several adaptations of requirements or to total design and due to maintenance activities. However, maintainability can be useful for managers. When an element has a low maintainability (which means that it is difficult to maintain or replace) it is advisable to prevent it from failing. So the intervention level of this element will probably be set on a high level which means that it is regularly maintained. This prevents the element from total failure which is hard to repair and will lead to higher unavailability and lower reliability in comparison with planned maintenance.

So, maintenance to a structure has same importance as the maintainability as indicated in the acronym of RAMSSHEEP. It depends on the phase which of the two needs more attention, but be aware that the maintenance itself is certainly not less important than maintainability.

- 6) The security in RAMSSHEEP for a primary flood defence system cannot be seen as a relevant aspect in comparison with other systems. The security is mainly about terrorism and sabotage of the system which may lead to loss of function. For example, the only form of sabotage to the Afsluitdijk can be done by disabling locks or creating chaos/blockage on the road. Also, terrorism to the Afsluitdijk is also not very probable when one is comparing the effect of terrorism to other systems. The definition of terrorism can be given by: *'A terroristic event which uses violence and threats to create a state of fear, to intimidate or coerce, especially for political purposes'*. To create chaos on the Afsluitdijk by using terrorism it is necessary that the water level is extremely high which may lead to possible flooding of the hinterland. This means that a terrorist depends on weather conditions and many other variables and the chance of success is not certain. It will not create enough fear among the inhabitant of the Netherlands when other targets are more effective and easier to reach. These efficient targets will not depend on uncontrollable variables and have preferably a large effect on humans (loss of life) or on the economy. Historical terroristic events show mainly soft targets based on larger numbers of humans and economic damage, like a bomb attack on the subway network in Minsk (2011), on the airport of Moscow (2011), on the subway network in

London (2005), and etcetera. Other soft spots for a terroristic attack can be football stadiums, music events, and etcetera.

To conclude, the security aspect in RAMSSHEEP for primary flood defence system is of minor importance with respect to other aspects of the acronym.

- 7) The health in RAMSSHEEP for a primary flood defence system considers if a group of people is feeling well (good health) with respect to the physical, mental and societal views. This 'feeling well' term can be approach on a very broad view. The health aspect indicates if the society feels safe by the fact they live below sea level and are only protected by the primary flood defence system. This includes the times when a severe storm is harassing the system. So this can be expressed in a certain stress ratio; a stress ratio of zero means that nobody feels stressed about a possible flooding due to a random reason and a stress ratio of 1 means that the country is in chaos. Beside this, the health level will also be modeled/predicted when a flooding is actually happening. Logically, many people will die when the flooding comes unexpectedly and inhabitants did not have enough time to plan an evacuation. This number will be expressed in the loss of life based on the analysis of Jonkman *et al.* (2008). This theory does not count for consequences afterward the flooding. Many people will be evacuated and what will happen to the health of this group. Moreover, what will happen to the health level in the other parts of the country and is the sewer system still functional. The health aspect of RAMSSHEEP must indicate the level of feeling well before, during and after a flooding occurred and not only the number of loss of life due to this flood (this can be seen as an important value of RAMSSHEEP). Although, there is not yet a model developed which indicates a certain stress ratio. This means that health aspects can only be based on a qualitatively basis and unfortunately not be given by a hard number.

So, the definition of health in RAMSSHEEP has a broad and complete way of analyzing the total loss of life. However, this ambience is not realistic and not (yet) feasible to calculate or even to determine a estimation. Therefore, the result will be too subjective and not measureable/concrete to translate to monetary risk (value of someone's life).

4. Veiligheid Nederland in Kaart

The project 'Veiligheid Nederland in Kaart' (VНК)¹³ has started in 2001. The objective of the project VНК is to obtain more insight into the probability of inundation in the Netherlands, into the consequences of inundations and into the uncertainties that play a part in the determination of probabilities and consequences. As a result the inundation risks of the dike ring areas in the Netherlands are obtained and moreover there will be more insight into the weakest links of flood defences. To realise this objective four tracks are followed:

- 1) The probabilities of inundation are determined for 16 dike ring areas.
- 2) Give more insight into the problems of structures
- 3) Give more insight into the possible consequences of inundations
- 4) Visualise the extent of different uncertainties and how to deal with those uncertainties.

The Netherlands are divided in 53 ring dike areas (including the dike ring areas along the river Meuse there are 99 areas). For the first part of VНК only 16 dike rings are analysed.

Last year the follow up VНК II has been finished. The inundation probabilities and consequences of an inundation of the remaining dike rings are calculated.

4.1 Introduction

The project VНК researches the risks of large flooding's. To make an indication of these risks the probability of failure and their consequences must be determined. The definition of a risk can be seen as the probability of failure of a scenario multiplied by the consequence of that scenario:

$$R = P_f \cdot C_f$$

The protection against large flooding events has been documented in the Dutch water law (Waterwet). The required safety levels of several flood defence constructions has been given in this law which will be expressed in a probability of failure, i.e. the Afsluitdijk has a safety level of the probability of failure of ones in the ten thousand years (1/10.000 per year). This safety level has been based on several failure mechanisms (overflow, piping, instability, and etcetera) and not just on high water level; so this means it contains rules for both height and the strength of the flood defence system.

¹³ English: Flood Risks in the Netherlands.

4.2 Probability of flooding - Reliability Analysis

The probability of flooding of the primary flood defence system gives the chance of flooding due to system failure by a failure mechanism or a combination of failure mechanisms. This probability of flooding will be calculated by using the program PC-Ring (Vrouwenvelde *et al*, 2003). The needed data result from the model that will be gathered based on the current situation.

The VNK made a choice for not applying all twelve failure mechanisms which has been documented in Leidraad Grondslagen Waterkeren (Technische Adviescommissie voor de Waterkeringen, 1998). The VNK uses just four mechanisms which have been based on the most common used failure mechanisms. Moreover, not all the failure mechanisms do immediately lead to system failure and therefore flooding (i.e. settlement of a dike, softening of the dike, and etcetera). This means that the probability of flooding can be in reality even larger. The VNK considers the following four failure mechanisms (Rijkswaterstaat, 2005):

- Overflow and wave overtopping;
- Piping;
- Erosion outer slope;
- Sliding inner slope.

When the total probability of failure of the system has been calculated the weak spots can be determined to give an indication where and which actions must be taken to decrease the probability of failure. The identification of the weak spots within the system can be determined by comparing the current safety level with the required safety level (which has been given by the law). This boundary level gives an indication based on relative weak spots.

Besides, the costs of these actions must be globally calculated (order of magnitude) which may give an overview of the amount of money that is needed to increase safety level of the system. The costs of the actions will not be calculated based on the most optimal point where a balance has been given of costs and benefits. In other words, the actions for the weak spots will be not based on a Cost-Benefit Analysis.

4.3 Consequences of a flooding

To determine the consequences of a flooding the project VNK is applying two methods to define the flooding scenarios: 'global' and 'detailed'. The global flooding scenario has been defined as the worst case scenario and can easily be used to determine the expected damage due to the flooding. Although, for this scenario one assumes that there is enough water to fill the complete hinterland which often is not completely realistic. The detailed flooding scenario will be modelled by using a hydrodynamic model (SOBEK 1D – 2D (WL, 2003) which has been developed by WL|Delft Hydraulics) to calculate the patterns of flooding.

4.3.1 Health Analysis

The total loss of life due to a flooding will also be calculated based on the flooding scenarios. To calculate the total loss of life one should consider two steps: (1) Evacuation analysis; and (2) Estimate the amount of victims in the area.

The total loss of life can be calculated by the relation between flooding characteristics (water depth of flooding and velocity of water level rise) and the amount of victims. This relation has been determined by the historical data of the flooding in 1953 and the international literature of flooding. This often result in an estimation of the loss of life by 0,1% to 1,0% of present humans in the flooding area (Jonkman, Kok & Vrijling, 2008). However, this is strongly depended on the flooding characteristics: high water level rising velocity and large water depth will directly lead to a higher number of loss of life.

The probability of flooding (risk = consequence x probability) is not the only important aspect in a risk analysis. The risk perception has also a significant impact on the society. This perception is different for every individual and therefore can be seen as a very subjective definition. One should analyze aspects like current perception of flood risks, importance of communication about flood risks, how to quantify the subjective risk perception of flood risks, and etcetera.

This risk perception subject is still a bit vague. This is not different when comparing this risk perception of flood risk to other forms of risks (smoking, traffic, flying, and etcetera). All these forms are present every day, but not always noticeable. Most people living or working below sea level do not realize flood risks, and taxes that are paid to minimize these risks.

4.3.2 Environmental Analysis

A flooding has a large impact on the quality of environmental aspects. One should think of roughly four quality aspects (Rijkswaterstaat, 2005):

- Landscape (geographical aspects, landscape ecological aspects, culture historical aspects, scale characteristics and land usage);
- Nature (flora, vegetation and fauna);
- Culture history (archaeological aspects, historical architecture aspects and historical geographical aspects);
- Environment (damage of fresh water system, environmental damage aspects, dissipation of dangerous goods).

These consequences of environmental aspects depend strongly on water depth, salinity of the water (sea) and the duration of the flooding.

4.3.1 Economics Analysis

The results of the flooding scenarios have been used to calculate economic damage due to a certain flooding. The depth of the water during this kind of flooding is an important parameter which indicates expected damage. At every location within the system damage

will be determined based on usage of the land (indicated by GIS¹⁴) and the damage function. The calculation of economic damage will be based on three damage categories: (1) direct material damage; (2) direct process damage; and (3) indirect damage.

A tool which can be applied in economic analysis is the Cost-Benefit Analysis (CBA). Costs must be put on a balance against benefits of increased safety. The costs and benefits contain not only the costs and benefits of the actions that must be taken (financially), but also costs and benefits with respect to nature, environment, spatial quality, and etcetera. Benefits are mostly defined as the reduction ratio of the probability of annual flood damage. The optimal safety level in economical way can be achieved by implementation of total costs of actions and the expected flood damage will thereby be minimalized (Rijkswaterstaat, 2005).

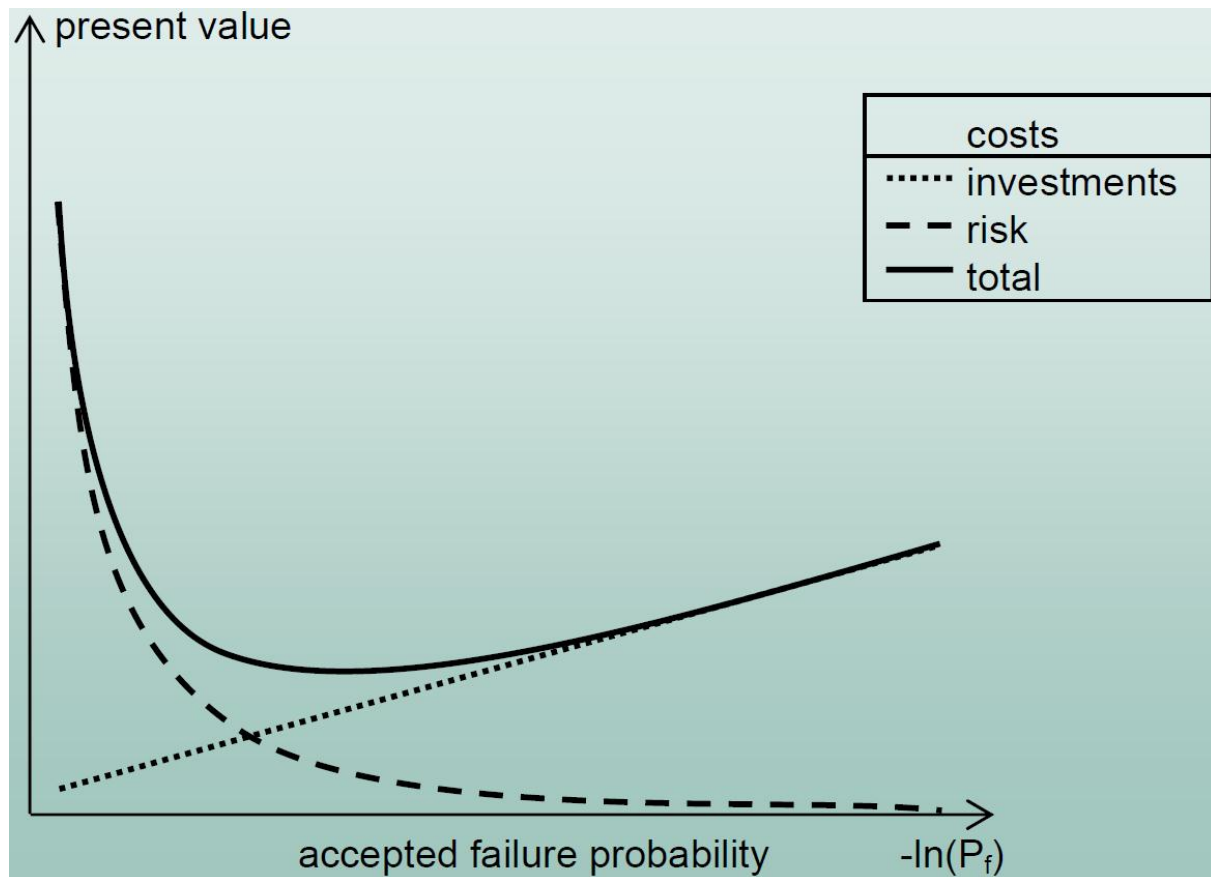


Figure 4-1: Schematic illustration of optimal safety in an (simplified) economical way based on the CBA (Vrijling, *et al* (2002)).

The benefits of a maintenance action can be seen as reduction of decrease of loss of life as well as the damage. The damage has already been expressed in a financial unit, but not the loss of life. This will result in the question if a life can be expressed in an economical value. This subject is still quite vague and therefore the immaterial damage will mostly be given by a certain value.

The risk-based optimisation is adopted as the main principle on which to base an analysis of acceptable flooding risk. The type of optimisation has been applied successfully in several

¹⁴ Geographic Information System

earlier studies in hydraulic engineering. Reference is made to van Dantzig (1956), Burcharth et al (1996), Vrijling et al (1998) and Voortman et al (1998). The basic form of risk-based optimisation is economic optimisation that is aimed at minimisation of the lifetime cost of the flood-defence system:

$$C_{life}(P_f, X) = I(P_f) + R(P_f, X)$$

$$R(P_f, X) = P_{flood}(P_f, X) \cdot S$$

With parameters:

P_f = vector of design variables

X = vector of random variables

$I(P_f)$ = investment in the structure or system

$R(P_f, X)$ = monetary risk

P_{flood} = flooding probability of the area

S = monetary value of all inventory of the area

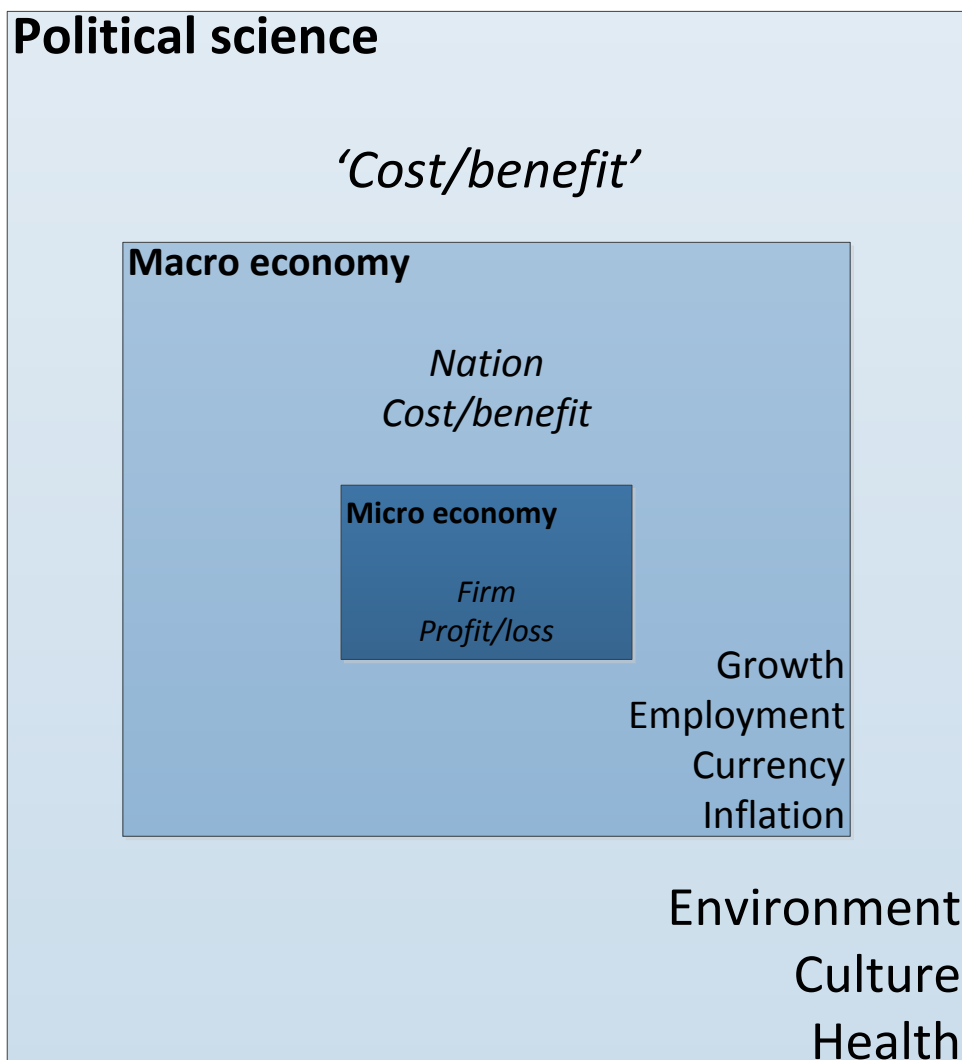


Figure 4-2: The levels on which economic decisions have been based on.

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The cost benefit analysis can be calculated on different levels. Mostly, there are three dividing categories:

- 1) Micro economy (firm);
- 2) Macro economy (nation);
- 3) Political science.

Each category has its own focus. Starting at the smallest part 'micro economy', a cost benefit analysis of a firm is relatively simple. The analysis contains an optimization of the process/improvement costs and the gains of the firm. Finally, this should result in maximization of the profit (or minimization of the losses).

The next level of macro economy is a bit more complex. The cost benefit analysis is acting on a higher level of society and this especially relate to the benefits. The cost of a certain activity will be more or less the same, but the benefits of that activity can be low what result in a loss. In macro economy (nation) one may decide to continue this activity when it seems that there are other benefits like growth, employment, currency, inflation, and etcetera. So, the cost benefit analysis has been analysed on the bigger picture instead of only the activities on the firm.

The highest level is political science which is based on more immaterial aspects. The costs of a certain activity will however be the same, but benefits can be analysed by considering more than only activity benefits like environment, culture, health, and etcetera. These benefits can be classified as intangible. The politicians may decide to accept the losses of a certain activity by considering the intangible aspects worth this loss.

5. Risks and Failure Mechanisms

5.1 Overview of Risks and Failure Mechanisms on Dikes

The systems are designed to satisfy civil engineering needs. A system usually consists of a number of structures, i.e. the Afsluitdijk consists of a dike, sluices, road connection, and etcetera. These structures are designed and constructed to fulfil one or several needs (i.e. retaining water, navigation of ships, traffic, and etcetera). A structure fails or collapses when these functions can no longer be carried out. The way a structure fails or collapses is called a failure mode (or failure mechanism). A failure mode occurs when a limit state is exceeded, i.e. when the load exceeds the strength.

5.1.1 Fault Tree

To assess to what extent the occurrence of a failure mode leads to the failure of a certain structure (dike, sluice) and whether the failure of the structure leads to the failure of the system, risk analytical methods have been developed which has been called fault trees in probabilistic design. The fault trees are particularly suitable for the illustration of cause and consequence chains which lead to an unwanted top-event if one cause has only two consequences which can be clearly distinguished (failure or non-failure). Only the negative consequences (failure) are included in the fault tree.

To efficiently present chains of modes, which could lead to failure of a system in a fault tree, a function analysis of the considered system is necessary. Which loss of function is selected as the top event must be well defined. This is usually clear for safety evaluation of water defences, like for the Afsluitdijk (Vrijling & Van Gelder, 2006):

*The salt water is no longer kept out of the IJsselmeer area
(And lead to inundation of hinterland)*

NB. The fault tree in Figure 5-1 is a schematic and basic illustration of a typical fault tree applied on a random dike based on its structures within the system (sea dike, dune, and sluice). The top-bottom approach shows at a certain level that the failure mechanisms will be applied and form eventually the basis to the safety of the system. This fault tree must also be applied on every single section of the system, thus this means that the illustration below will just indicate the probability of failure of one section.

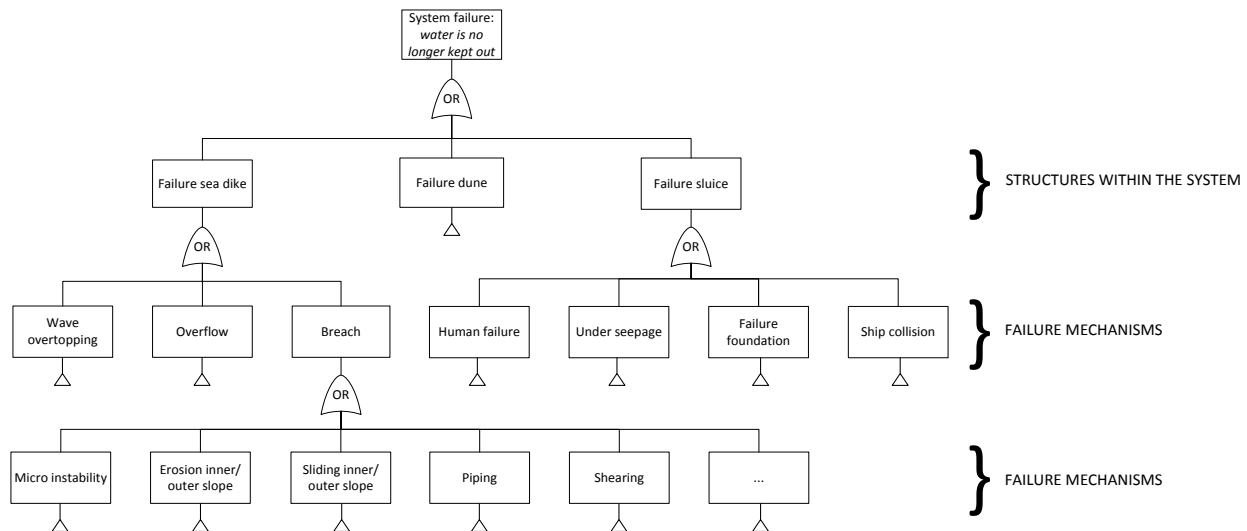


Figure 5-1: A schematic (and basic) illustration of a fault tree applied on just one cross-section of the Afsluitdijk with the failure mechanisms (Vrijling & Van Gelder, 2006).

5.1.2 Failure mechanisms

For every section, characterized by one cross-section, a probability of failure can be calculated by determining the probabilities of all relevant failure mechanisms for that cross-section according to level III: fully probabilistic (or level II: probabilistic with approximations FORM) calculations. Every cross-section is expected to concern a whole system. Failure is considered in a cross-section (dike) or per structure (sluice) and if the water level in a cross-section is higher than the defending height, the system is considered failing over the entire length. The approach is characteristic for the analysis of a serial system. An upper limit of the probabilities of failure of the system of a dike is acquired by adding the probability of failure of all sections. The greatest failure probability serves as a lower limit of the probability of failure for the system. The upper limit of the failure probability can be established, because several failure mechanisms can be determined by the same parameters. Sections can be correlated, because i.e. the storm surge level burdens all of them (Vrijling & Van Gelder, 2006).

- *Lower bound*¹⁵: $\max\{P_i\}$ – Fully correlated/dependent
- *Upper bound*¹⁶: $\sum_{i=1}^n P_i$ – Failure of one component excludes the failure of other components/mutually exclusive.
- *Summarized*: $\text{MAX}\{P_i\} \leq P_{\text{section}} \leq \sum_{i=1}^N P_i$

¹⁵ The lower limit is valid if there is one failure mechanism that includes all the others. For example: erosion, wave overtopping and overflow.

¹⁶ The upper limit is applicable if all failure mechanisms exclude each other. By taking correlation between the failure mechanisms into account the upper limit can be decreased.

In which:

N = number of failure mechanisms according to which the section can collapse

P_i = probability of collapse in case of collapse according to failure mechanism i

P_{section} = total probability of collapse of the section

An overview of risks and failure mechanisms on dikes has been extracted in Figure 5-2 (see below) and shows which failure mechanisms are the most common for hard defences, like dikes and dams.

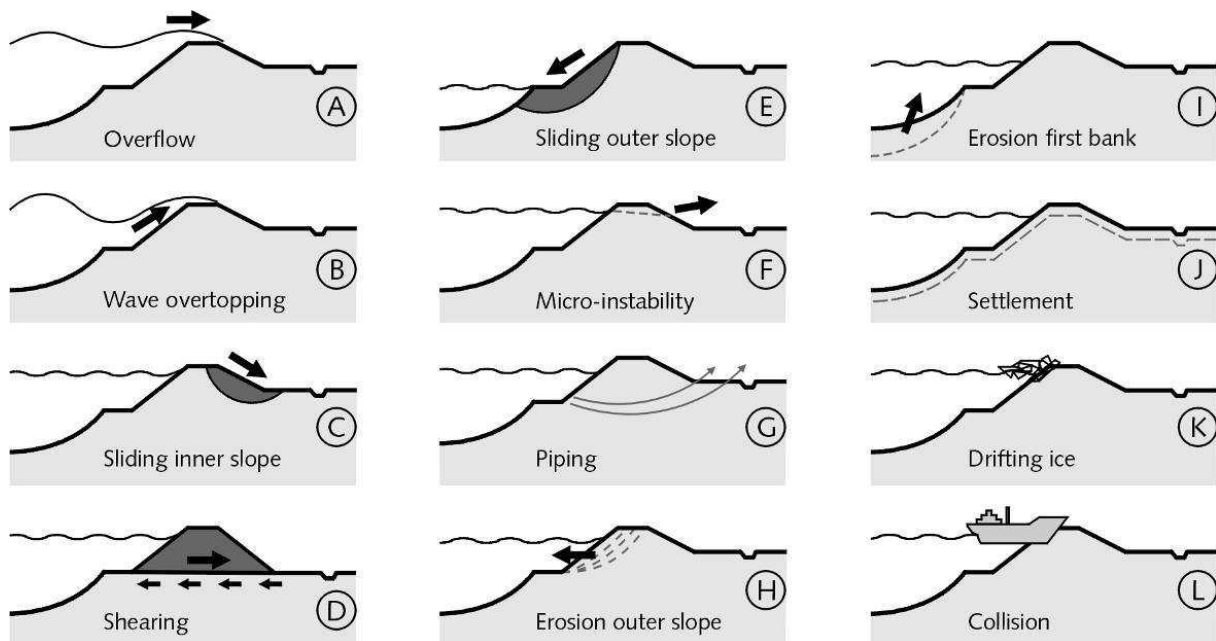


Figure 5-2: Average flood defence failure mechanisms for hard defences like dikes and dams (Weijers, J. & Tonneijk, M., 2011).

A. Overflow

- A dike must be able to withstand a water level that will be exceeded with a normative probability (in this case 1/10.000 per year).

B. Wave overtopping

- The amount of water the inner slope can withstand by wave overtopping which is related to the water level (just like the overflow). It can be different for adjacent sections of a dike and it is very well possible that the stability of the dike is not jeopardised if this limit has been exceeded.

C. Sliding inner slope

- Overall stability of a dike will be in jeopardy if sliding of an inner slope in combination with overtopping of water or waves will occur during a period of high water. However, the water retaining function of the dike could still be intact. The stability factor (load over resistance) based on a calculation with the Bishop method is defined as a limit state.

D. Shearing

- The shearing will be treated as horizontal sliding of the dike.

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E. Sliding outer slope

- If water in front of the dike drops very fast and water inside the soil mass cannot adjust to this sudden change, the pressure of the water inside the dike will cause the dike to slide towards the water (sliding of the outer slope). The next period of high water can cause direct problems.

F. Micro-instability

- Seepage water that reaches the inner slope of a dike and will erode the smaller particles (micro) due to water pressure from the inside. The strength will be substantially reduced and the integrity of the whole dike is in jeopardy despite the minor damage what can be observed on the outside of the dike.

G. Piping

- Erosion will occur underneath a cohesive layer in the subsoil and is very similar to micro-instability. If the exit gradient of seepage water is high enough soil particles will erode from underneath the dike and form a meandering cavity (a pipe).

H. Erosion outer slope

- A top layer that provides resistance of the dike is often protecting erosion process and therefore the start of a collapse of the dike. Some examples of top layers are grass cover on top of a clay layer (small loads) or asphalt, natural stones, concrete blocks, etcetera (high loads). If there is severe damage to this top layer the erosion process will cause problems.

I. Erosion first bank

- A steep under water slope in front of the dike can start this process and cause instability of the outer slope. Especially if the sand in the subsoil is loose packed.

J. Settlement

- This geotechnical process will occur over many years and causes deformations of the dike.

K. Drifting ice

- Relative small magnitude of formation of ice dams but can cause instability of the dikes.

L. Collision

- Vessel collision could theoretically damage a dike and start the overall collapse. However, this is not very plausible (this is more probable for destruction of locks).

Note that some of the above mentioned failure mechanisms do not directly result in flooding i.e. there is still some but not quantified reserve strength present.

A closed system of water defences does not usually consist of a single defence structure (dike, sluice) which does not usually consist of one cross-section. This is a serious complication, because it makes it virtually impossible to calculate the probability of collapse of a vaguely realistic system of water defence structures. As one can earlier in this chapter, a number of upper and lower limit approximations for 'simple' serial systems can be found, which can be worked out numerically. However, the knowledge to calculate these limits for more realistic systems of water defence systems is lacking.

Moreover, several failure mechanisms in one cross-section are not totally independent either. A limited number of parts of a dike ensure the defending function and the stability of the core, the revetment and the subsoil. If a part is insufficiently strong (assumed), it raises the probability of failure for more than one failure mechanism. A relation scheme for parts and failure mechanisms is given in Figure 5-3. The dependence of failure mechanisms in one cross-section due to a certain load, which is involved in more than one failure mechanism, and the coherence of functional parts and failure mechanisms are still very vague.

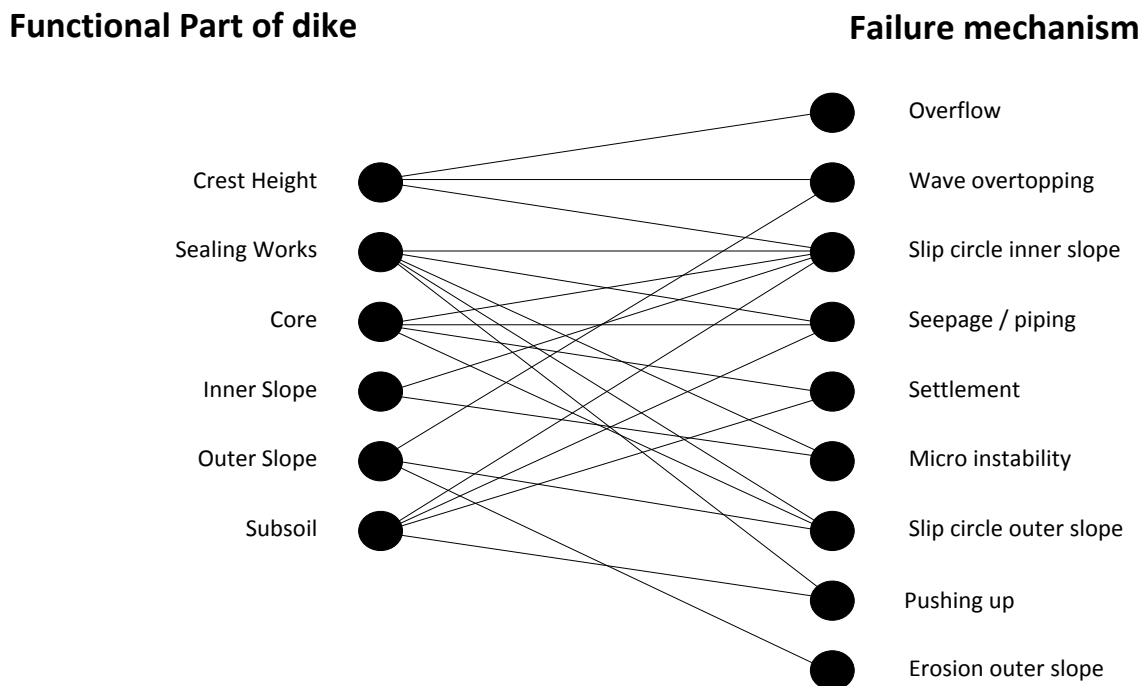


Figure 5-3: A relation scheme between functional parts of a dike and the possible failure mechanisms (Vrijling & Van Gelder, 2006).

5.2 Cross-section and top views of a dike

The general build-up of a dike can be based on the necessary protection of primary flood defence system. The highest protection will be maintained at the core of the dike and will reduce to the outer section of the protection area. The core zone must guarantee the set safety level for the hinterland. Therefore the core zone shows many construction limitations. On both sides of the core zone the (outer) protection zone will take care of the damming capacity and the stability of the flood defence system. These zones have fewer limitations than count in the core zone.

The general build-up of the dike can be divided in three zones (see also Figure 5-4):

- Core zone (pink area)

- Protection zone (blue area)
 - Sea side; and
 - Land side.
- Outer protection zone (green area)
 - Sea side; and
 - Land side.

This can be translated to the situation of the Afsluitdijk:

- Core zone (CZ)
- Protection zone
 - Wadden sea side (PZWS)
 - IJsselmeer side (PZIJ)
- Outer protection zone
 - Wadden sea side (OPZWS)
 - IJsselmeer side (OPZIJ)

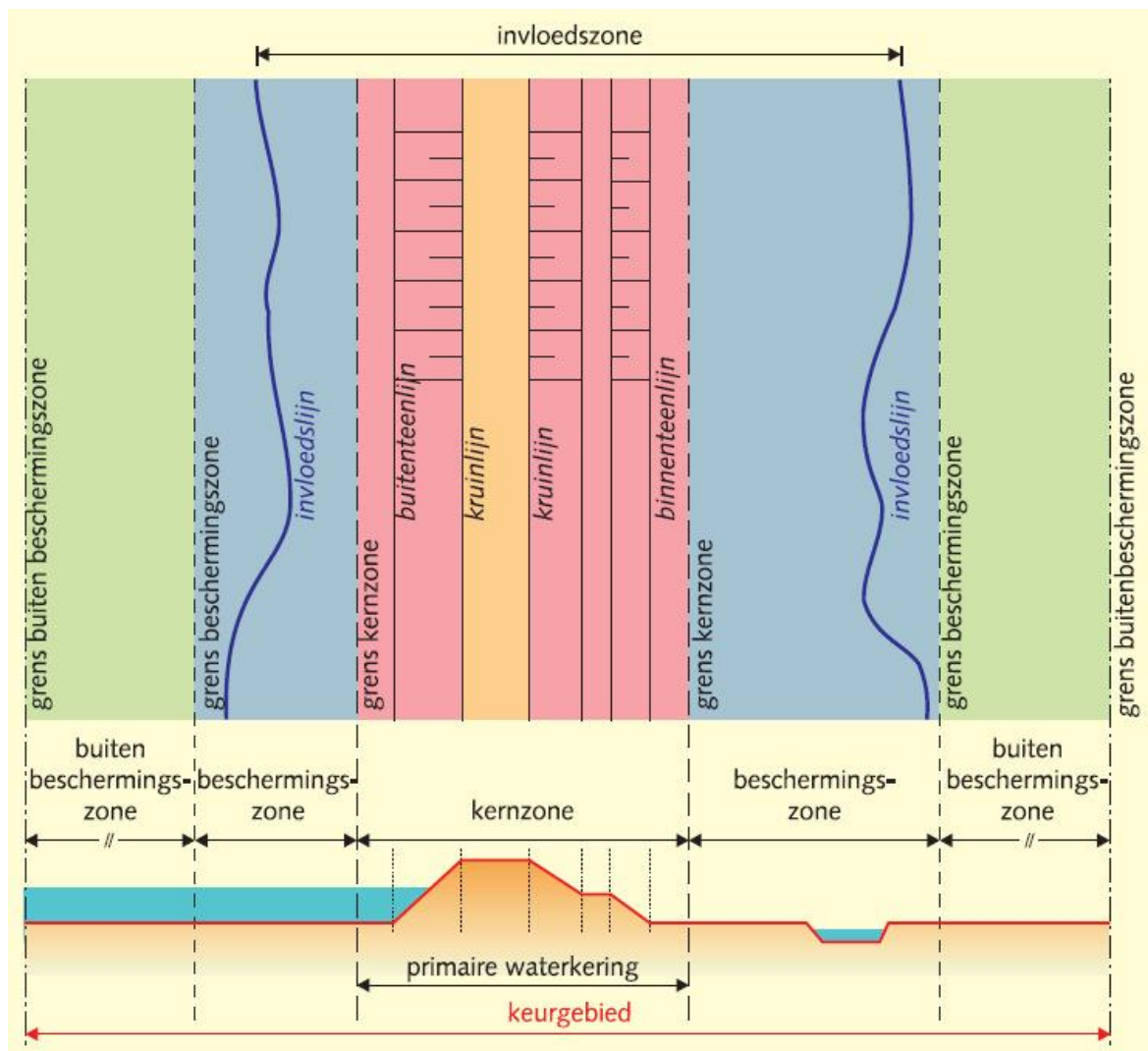


Figure 5-4: Schematic view of different primary flood defence zones (management limits) for a dike (Rijkswaterstaat, 2009).

There is no uniformity of the dike in the longitudinal direction. The variations are noticeable in:

- Soil composition;
- Morphology;
- Hydrological assumptions;
- Surface height;
- Type of flood defence system;
- Location of channel passage;
- Bottom protection.

Therefore boundaries of different zones are variable which is not very useful for the management and may lead to vagueness of a uniform boundary. To eliminate this variability the management team will choose a conservative distance of the zones what also can be seen in Figure 5-4.

5.2.1 Core zone

Dimensions (height, width, and etcetera) of the core zone must be chosen on such a way that the primary flood defence system will fulfil its functions. Therefore the boundary of the core zone will be determined by the most outer construction element of the dike.

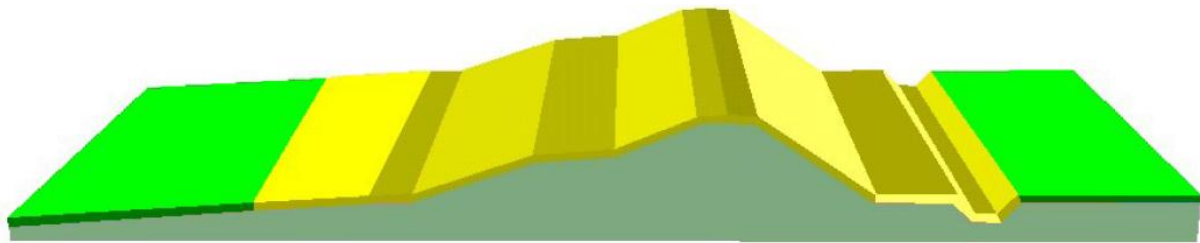


Figure 5-5: Core zone of the dike (yellow) (Rijkswaterstaat, 2009).

On both sides of the Afsluitdijk is water and boundaries will be set by the outer toe of the dike, the wave screen or the foreland.

5.2.2 Protection zone

The dimensions of the protection zone have been based on a strip whereby the stability of the dike will be guaranteed. This strip will also function as available space for future adaptations to the flood defence system for the coming 200 years. This involves sea level rise and the climate change in the area; increasing loads of the waters.

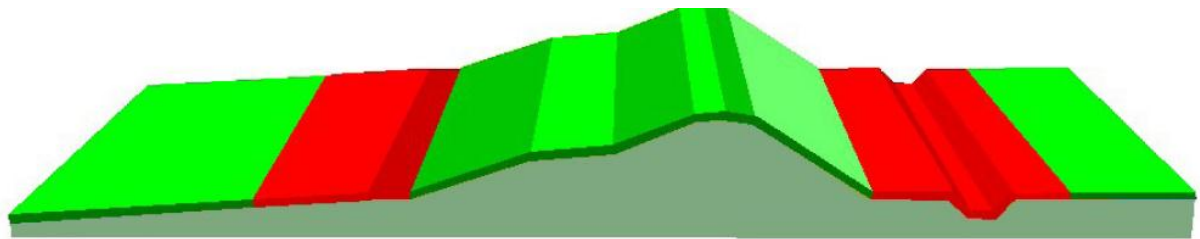


Figure 5-6: Protection zone of the dike (red) (Rijkswaterstaat, 2009).

The protection zone guarantees geotechnical stability of the dike and will be influenced by the location and depth and slope of the surface. The dimension of the protection zone will be based on technical criteria (Rijkswaterstaat, 2009):

- Piping length: $18 \times$ water retaining height measured from the inner toe;
- Stability dike: $5 \times$ depth of channel passage measured from outer toe;
- Stability foreland: $(2 + A) \times$ depth of channel passage measured from outer toe;
 - A = profile of the slope:
 - A = 6 for assessment on shearing;
 - A = 15 for assessment on settlement and depth of channel passage < 40 metres.
 - A = 20 for assessment on settlement and depth of channel passage > 40 metres.
- Boundary of falling apron;
- Open space for future strengthening (now + 200 years) due to increasing hydraulic loads.

5.2.3 Outer protection zone

The outer protection zone can be seen as the expanding zone of the protection zone. The purpose of this zone is to guarantee stability of the protection zone. The boundary of the outer protection zone is at maximum 50 metres from the protection zone.

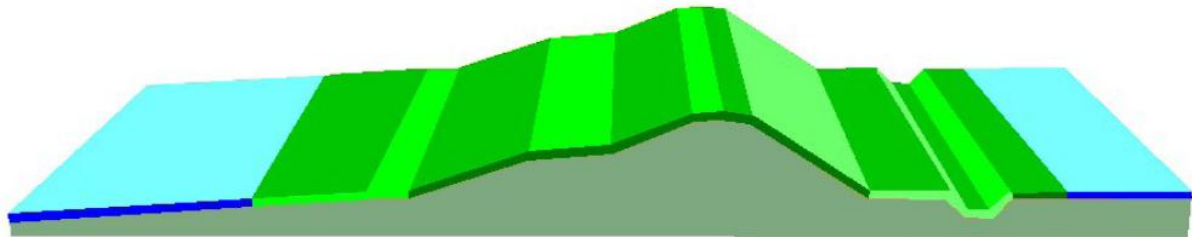


Figure 5-7: Outer protection zone of the dike (blue) (Rijkswaterstaat, 2009).

5.3 Flood frequency analysis

The design height of the dike must be based on the safety level of 1/10.000 per year what will guarantee safety of the hinterland. The height of the dike only represents failure mechanisms *overflow* and *wave overtopping* and other mechanisms which will result in a breach (instability, piping, and etcetera) will not directly be considered. Water levels of the locations of the locks at the Afsluitdijk have been analyzed from 1932 until 2012. The data analysis will show which the high water level the dike must resist according to the frequency of ones in ten thousand years. The analysis has been executed for the locations *Den Oever (NH)* and *Kornwederzand (Fr)* (see Figure 5-8).

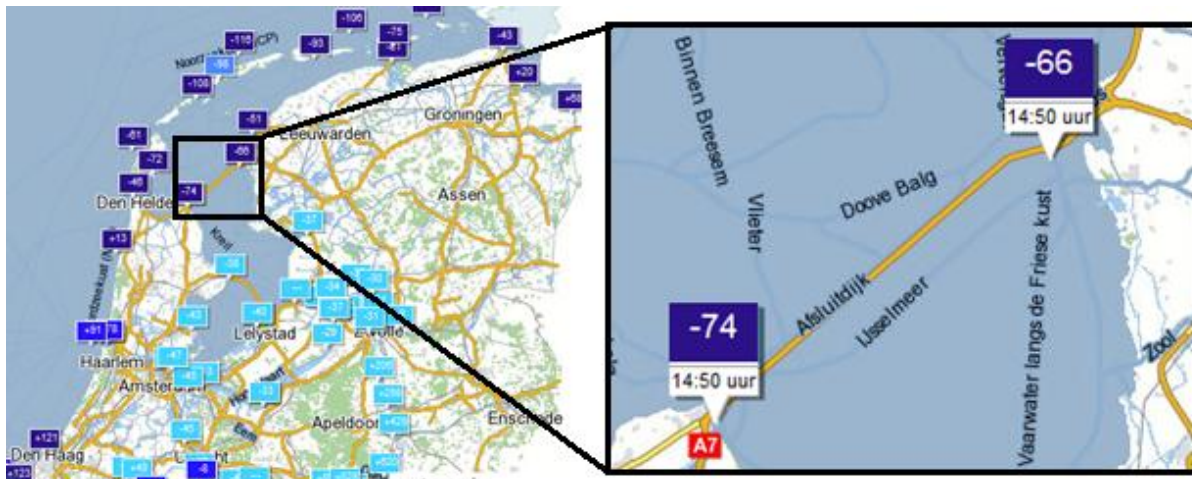


Figure 5-8: Locations in the Netherlands where water level measure devices are located (Left). Locations of Den Oever (NH) (left) and Kornwederzand (Fr) (right) with their current water level (real time) (Right) (Rijkswaterstaat, 2012).

Apart from normal flow frequency, scientists are also interested in the occurrence of extreme events. For this purpose water level frequency may be derived which yield the probability that a certain annual maximum water level is exceeded.

Depending on the phenomenon, different probability distributions are recommended. The Gumbel type I distribution applied to flood levels is presented. In the case of floods, where possible, extreme analysis should be done on water levels. However, if water level records do not exist, or if a rating curve has not yet been established, then we may not have another choice than to analyze water levels. This is, however, risky particular if we want to maintain flood protection.

If a river has a flood plain where we consider building a dike, then the water levels in the original situation (without a dike) will be less high during a flood than in the case where the dike is present. Hence, a dike elevation, which is based on the recorded flood levels, will underestimate the flood level after the dike has been build. In the following, the Gumbel type I distribution is presented for the analysis of flood levels.

Gumbel type I

In 1941, Gumbel developed the Extreme Value Distribution. This distribution has been used with success to describe many flood events. As applied to extreme values, the fundamental theorem can be stated:

If $X_1, X_2, X_3, \dots, X_n$ are independent extreme values observed in n samples of equal size N (i.e. years), and if X is an unlimited exponentially-distributed variable, then as n and N approach infinity, the cumulative probability q that any of the extremes will be less than a given value X_i is given by:

$$q = \exp(-\exp(-y))$$

In which:

q = probability of non-exceedance

y = reduced variate

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If the probability that X will be exceedance is defined as $p = 1 - q$, then the equation yields:

$$y = -\ln(-\ln(1 - p)) = -\ln\left(-\ln\left(1 - \frac{1}{T}\right)\right)$$

In which:

T = return period measured in sample sizes N

According to Gumbel, the reduced variate is defined as a linear function of X:

$$y = a(X - b)$$

In which:

a = dispersion factor

b = mode

This reduced variate is much like the reduced variate of the Gaussian probability distribution: $t = \frac{x - \mu}{\sigma}$.

If the sample is finite, which it always is, the coefficients a and b are adjusted according to the following:

$$b = X_m - s \frac{y_m}{s_y}$$

$$a = \frac{s_y}{s}$$

In which

X_m = mean of X

s = standard deviation of the sample

The values of s_y (the standard deviation of the reduced variate) and y_m (the mean of the reduced variate) as a function of n are tabulated.

Result

From water level data (Appendix 12.A) mean and standard deviation can be determined:

$$X_m = 2,71 \text{ m}$$

$$s = 0,42 \text{ m}$$

The theoretical for the mean y_m and standard deviation s_y of the reduced variate from the standard table:

$$s_y = 1,193 \text{ m}$$

$$y_m = 0,557$$

The coefficients a and b are adjusted according to the following:

$$b = 2,71 - 0,42 \cdot \frac{0,557}{1,193} = 2,52 \text{ m}$$

$$a = \frac{1,193}{0,42} = 2,85$$

According to Gumbel, the reduced variate is defined as a linear function of X:

$$y = 2,85(X - 2,52)$$

The probability of non-exceedance:

$$q = \exp(-\exp(-2,85(X - 2,52)))$$

And logically the probability of exceedance will therefore be:

$$P_f(X) = 1 - q = 1 - \exp(-\exp(-2,85(X - 2,52)))^{17}$$

A probability density function (or density of a continuous random variable) is a function that describes the relative likelihood for this random variable to take on a given value. The probability for the random variable to fall within a particular region is given by the integral of this variable's density over the region. The probability density function (PDF) is non-negative everywhere, and its integral over the entire space is equal to one. A PDF is most commonly associated with absolutely continuous univariate distribution. If P_f is the cumulative distribution function of X, then:

$$P_f(X) = \int_{-\infty}^X f(u) du$$

And (if f is continuous at X):

$$f(X) = \frac{d}{dX} P_f(X) = 2,85 \cdot \exp(-\exp(-2,85(X - 2,52))) \cdot \exp(-2,85(X - 2,52))$$

Intuitively, one can think of $f(X)$ as being the probability of X falling within the infinitesimal interval $[x, x+dx]$.

¹⁷ After some algebra rewriting: $X(P_f) = 2,52 - \frac{\ln(-\ln(1-P_f))}{2,85} [m]$.

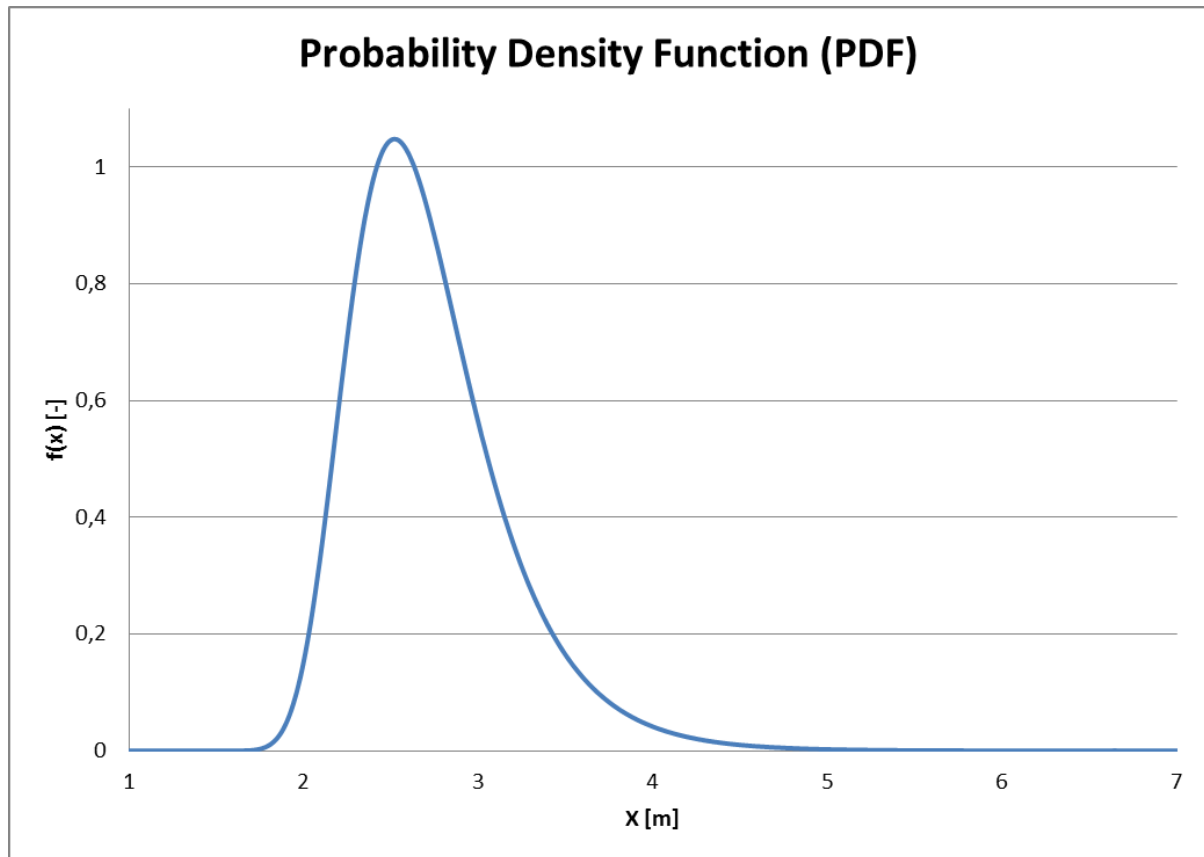


Figure 5-9: The probability density function of the extreme annual water levels.

On probability paper where the horizontal axis is logarithmic, equation of the probability of exceedance plots a straight line. To plot the data points on the horizontal axis a – so called – plotting position, or estimator, of the probability of exceedance p is required. The following plotting position is used:

$$p = \frac{i}{n + 1}$$

Where i is the rank number of the maximum occurrences in decreasing order and n is the total number of years of observations. In Figure 5-10 the Gumbel probability distribution and the original data points of annual maximum flood levels in the Wadden Sea at the Afsluitdijk have been plotted.

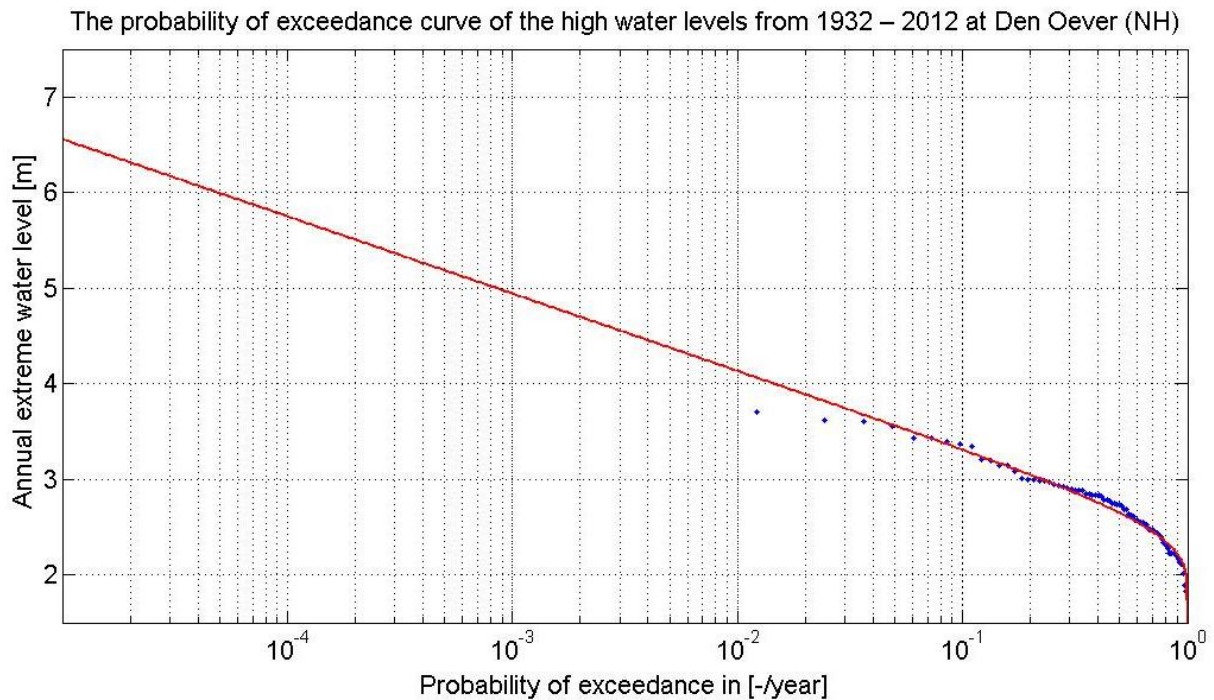


Figure 5-10: Gumbel probability distribution and the original data points of annual maximum flood levels in the Wadden Sea at the Afsluitdijk.

Table 5-1: The MatLab (R2010a) program code which forms the basis for the result in Figure 5-10.

```
clf; %Clear figure
clear all; %Removes all functions and MEX-files, and variables and global variables from the
base workspace

%% Determination of variables
X=xlsread('Annual Extreme Water levels.xlsx','A2:A82'); %The rank number of the maximum
occurrences in decreasing order of the dataset water levels at Den Oever from RWS
Y=xlsread('Annual Extreme Water levels.xlsx','D2:D82'); %Annual extreme water levels [m] based
on dataset water levels at Den Oever from RWS.
N=length(X); %The total number of years of observations.

%% Visualization of the dataset
i=1.5:0.01:7.5;
semilogx(X./(N+1),Y, '.', 1-exp(-exp(-2.85*(i-2.52))),i,'r-')
axis([10E-6 10E-1 1.50 7.50])
xlabel('Probability of exceedance in [-/year]');
ylabel('Annual extreme water level [m]');
title('The probability of exceedance curve of the high water levels from 1932 - 2012 at Den
Oever (NH)');
grid
```

With the help of MatLab (R2010a) the dataset of the maximum annual water levels has been plotted on half-logarithmic paper (x-axis) and has been displayed by the blue dots. The x-axis shows the probability of exceedance in [-/year] of a certain water level above +NAP in [m] which has been showed on the y-axis.

Beside data of maximum annual water levels also the Gumbel distribution has been plotted. As one can see in Figure 5-10 this Gumbel distribution gives a good illustration for a trend line through the original data points. Therefore this distribution will be used for the calculation of the probability of failure for a certain height of the dike.

Table 5-2: The water level (+ NAP) in [m] according to the determined safety level derived from graphical calculated probability of exceedance in Figure 5-10.

Probability of exceedance	Water level (+NAP) in [m]
10^0	1,301
10^{-1}	3,310
10^{-2}	4,134
10^{-3}	4,944
10^{-4}	5,752
10^{-5}	6,560

A remark can be made with respect to the tables and figure above. The dataset of water levels (Rijkswaterstaat, 2012) from time period 1932 – 2012 lead to a frequency analysis and a graph of the probability of occurrence (Figure 5-10). When one plots a trend line through the given water levels, a probability of exceedance (based on a Gumbel distribution) for certain safety levels can be determined. This can be seen in Table 5-2 which has been measured from the data of the trend line. According to this new data of water levels related to a certain probability of exceedance it can be noticed that Afsluitdijk does not suffice to the determined safety level. Take for example the probability of 1/10.000 per year:

- According to the trend line of the data set the water level related to a probability of exceedance of 1/10.000 per year is + 5,752 m +NAP.
- The current height of the Afsluitdijk has a height of 5,20 m +NAP which corresponds with a safety level of 1/2.080 per year.

One can conclude that the Afsluitdijk does not suffice (by almost a factor 5) to required safety levels which have been given in the Dutch water law (Waterwet).

6. Maintenance

6.1 Lifetime of a structure without maintenance

The life-span of a structure is the time which passes between realisation of the structure and the failure of the structure. In Figure 6-1 this is clearly marked for time dependent strength and load, for which the exact values are known for every point in time. The intersection of strength and load determines the moment of failure of the structure.

Figure 6-1: Life-span for an exactly known course of strength and load in time (Vrijling & Van Gelder, 2006).

In the case above, both strength and load are deterministic. The life-span is simple to determine, in that case. This is less so, when load and/or strength are random variables, because the life-span is also a random variable. The definition of the probability distribution of the life-span is:

$$F_{L(t)} = \Pr\{L < t\} = 1 - \Pr\{R > S\} \text{ for every } \tau \text{ in the interval } (0, t)$$

For the consideration of probability of failure the load has to be defined as the dominating load in the period (0, t). As time increases the average of the load in the interval (0, t) will also increase. This way dependence on duration of the dominating load is incorporated in probability distribution of the load. If the strength is also time dependent a new problem arises, namely the determination of normative strength for the period (0, t).

6.2 Deterioration models

The relation between strength and time is given by a deterioration model. The relation can be linear, exponential, logarithmic, and etcetera. The deterioration model thus determines strength at every point in time. The model is an approximation of reality. The input required for the model is the starting strength and usually a number of parameters which describe characteristics of the material or the structure. The parameters which serve as input for the deterioration model are usually determined from tests or observations. They rarely have a certain value and can usually be best described by a random variable. This means that strength at a certain time is a function of random variables and is thus a random variable itself.

The deterioration models give a stochastic description over the process of strength over time. This describes the structures condition on a certain moment of time. This means that it does not only gives information for determining risks over that period, but also the expected value for the amount of repair on the structure. The deterioration model forms therefore an important position in the maintenance planning of a structure. However, this piece of information of the behaviour of the structure is often not known. A mathematical description of the strength process over time has to be found by measurements of the structure and by physical research. The model research will be based on a certain time interval what forms the basis for the deterioration model by using curve fitting and extrapolation techniques.

This also introduces model uncertainty. The input parameter in the model will also have some uncertainties (numbers based on tests, experience or intuition). These parameters can therefore be described as stochastic variables with each an own probability density function.

The deterioration process can best be modelled as a stochastic process. The result gives an average value and deviation of the structures strength for each moment of time. The deviation will increase when the uncertainties (in the mathematical model) of the input parameters are larger.

In general, the deterioration process is complex and has a lot of unknown variables. Therefore, it is essential to schematise the process to a mathematical model in which most important factors will be implemented. Basically, deterioration model exists of three crucial components:

- 1) Loads;
- 2) Resistance of the structure;
- 3) Relation between the decrease of the strength of the structure per unit of time and the load and resistance.

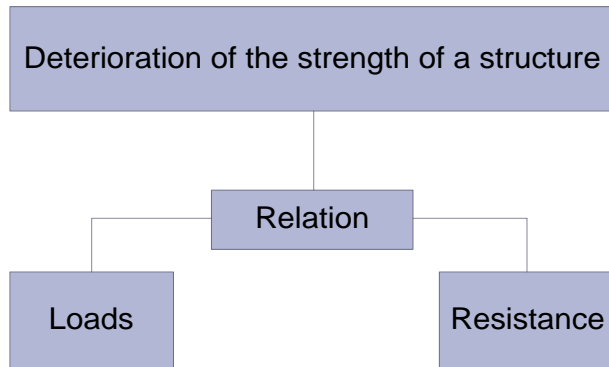


Figure 6-2: Three crucial components in a deterioration model.

6.3 Probability of failure

Failure is generally defined as non-performance of what is requested or expected. The limit state is a state, where strength of and load on the construction are equal. Two types of limit states can be distinguished, namely:

- 1) ultimate Limit State (ULS); and
- 2) serviceability Limit State (SLS).

When the ULS is exceeded, failure occurs as a result of collapse of the structure under extreme loads. Examples of ULS are i.e. collapse of an earth body, deflection of the structure, and etcetera. When the SLS is exceeded, the functional demands can no longer be met (for a certain moment of time). Deflections of a floor, cracking in reinforced concrete, waves which are too high behind a breakwater and so on, are serviceable limit states.

Generally, failure can be schematized as exceeding the load over the strength. The state of a structure can be described using a limit state function:

$$Z = R - S$$

In which:

R = strength

S = load

If the strength and/or the load are described with random variables, Z is also a random variable. If $Z < 0$ the structure fails. The probability of failure is:

$$\Pr\{Z < 0\} = \int_{-\infty}^0 f_z(\xi) d\xi$$

In which:

f_z = probability density of Z

ξ = realisation of Z

The difficulty with the determination of the failure probability is the fact that the distribution function of Z usually cannot be determined exactly. Only in a few cases, i.e. where all variables are normally distributed, the distribution of Z can be determined. However, there are techniques that make it possible to calculate the probability of failure or to approximate it.

NB. These techniques will not be implemented in the research, because this would not add extra value to the comparing part of the risk models.

6.4 Maintenance strategies

The maintenance activities have certain goals that must be fulfilled. But on beforehand one must define the real function of maintenance within the system. The primary flood defence system has been degraded over time and therefore safety level will not be met. Due to this degradation through use, the system has to be permanently restored. This is because functional outcomes of the main and the sub functions, in relation to the original levels (here: safety level requirements of 1/10.000 per year), are maintained and this system should continually fulfill its functions.

The core of maintenance has always been based on as well as technical and economical drivers. Eventually one must find a balance between technical and economic aspects to make a good maintenance plan. Many companies do have a profit-driven maintenance plan, which describes a desire to make as high profit as possible. Other strategies of the maintenance plans are availability-driven, reliability-driven, comfort-driven, and etcetera. These strategies are not applicable in the system where the government ensures the safety of the inhabitants of the Netherlands. Therefore, there is a different driver needed. The most applicable driver with respect to its main function of the Afsluitdijk is a maintenance concepts based on risks, or in other words the system is risk-driven. The direct profit to ensure this performance is the safety of the Dutch people and the use of land to stimulate the economy.

First of all, it is proper to define the term maintenance in just one sentence (Vrijling & Van Gelder, 2006):

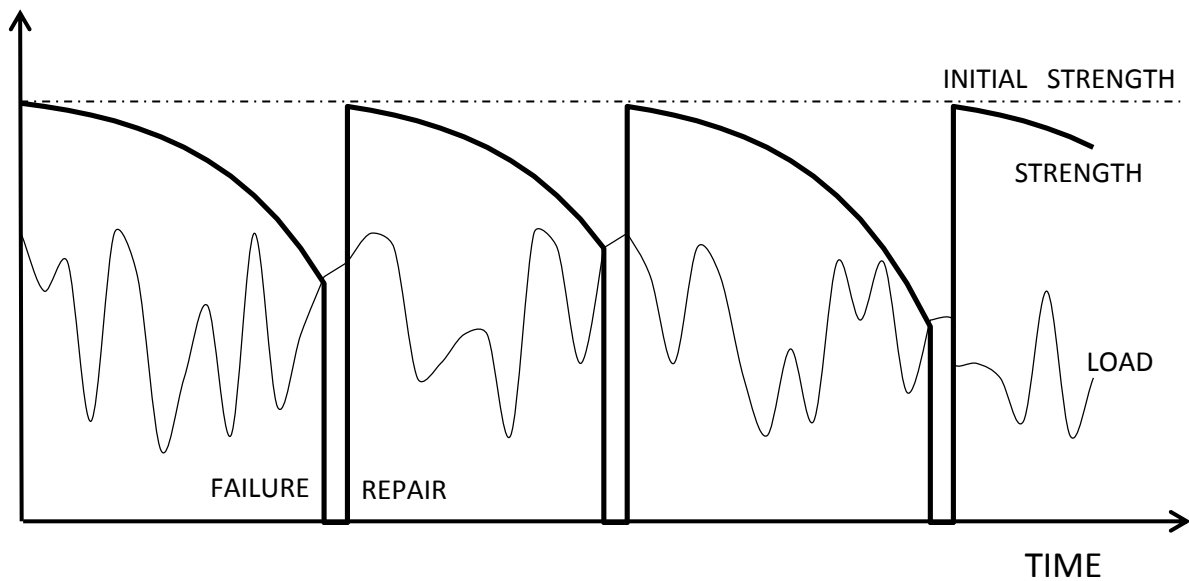
All activities aimed at retaining a structure's technical state or at reverting it back to this state, which is considered a necessary condition for the structure to carry out its function.

These activities include both the repair of the structural strength, back to the starting level, and several inspections. The cost of maintenance of civil engineering structures amounts to approximately 1% of the founding costs per year. For a life-span of i.e. 100 years this means that the maintenance costs are of the same magnitude as the construction costs. Taking into account the decline in new housing development projects, maintenance costs are clearly becoming an increasingly greater share of the expenses.

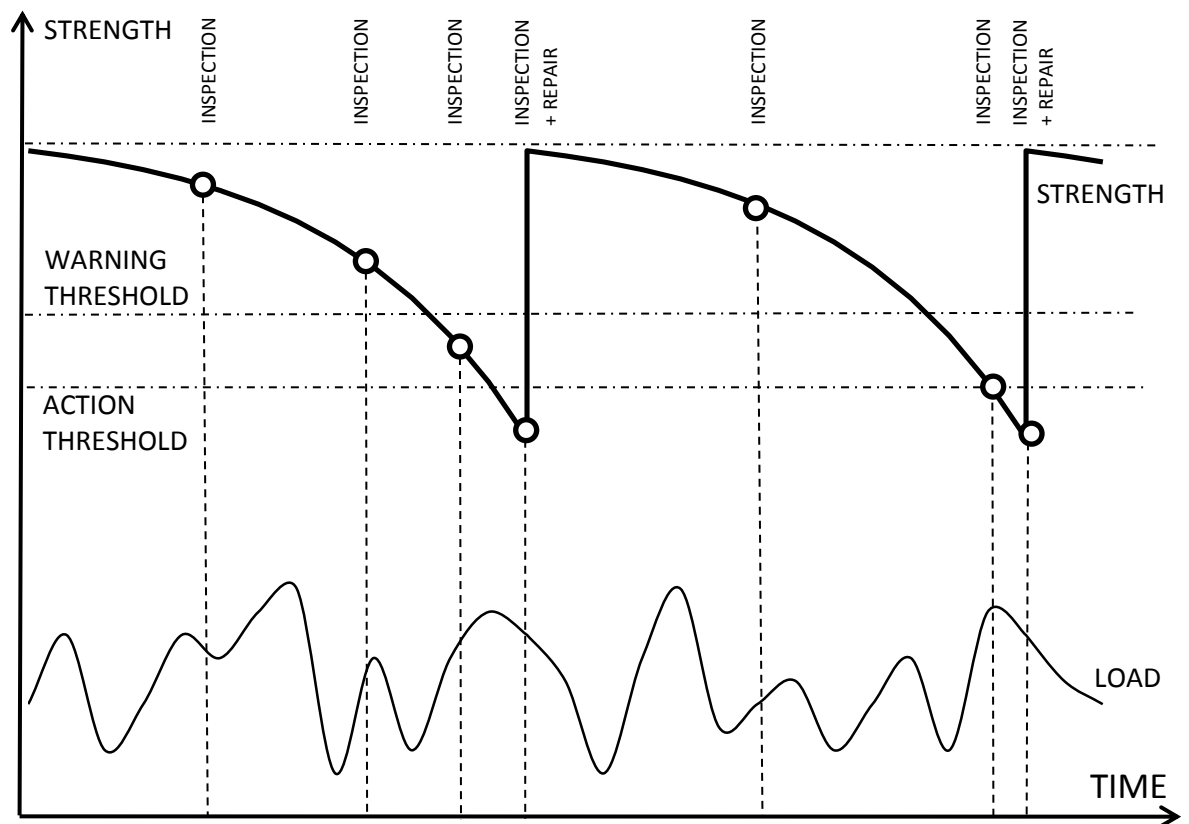
A direct consequence is the desire to minimize maintenance costs. In order to realize this, the optimal maintenance strategy has to be sought. From the mechanical engineering

maintenance theory, the following classification in strategies are known (Vrijling & Van Gelder, 2006):

1) Curative maintenance (failure dependent maintenance);



2) Preventive maintenance (state dependent maintenance).



In the first case of fault dependent maintenance, a structure is repaired or replaced when it can no longer fulfill its function. Thus, repair takes place after failure; therefore a failure norm is involved. The life of a structure is fully exploited. A structure's failure (and the associated costs) is accepted. In hydraulic engineering this type of maintenance is usually not acceptable because, generally, the accepted probability of failure is limited. This type of maintenance can, however, be applied to non-integral construction parts (parts which do not contribute to the stability of the entire structure), with modest consequences of failure (provided reparation or replacement is not postponed for too long).

In the second case with condition dependent maintenance the state of the structure is determined at set intervals, by means of inspections. The decision whether or not to carry out repairs is based on observations. The inspection intervals can be regular or dependent on the condition of the structure. In the latter case condition parameters, indicating the condition of the structure, have to be visible. The probability of failure in a period between two inspections has to be sufficiently small. Generally, the life time of the structure can be better exploited than with usage dependent maintenance, but the costs of the inspections do have to be taken into account. This type of maintenance also involves drawing up norms. These norms concern:

- 1) a limit state which leads to an increase of inspection frequency (warning threshold);
- 2) a limit state which leads to carrying out repair works (action threshold).

In fact this concerns strength norms. These norms result from an optimization of maintenance correspond to a socially accepted failure probability in a year.

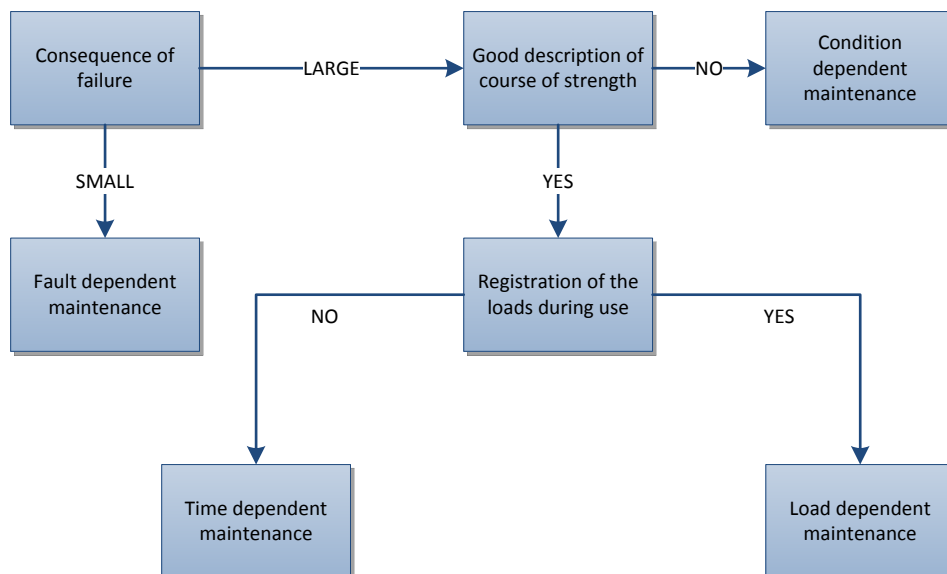


Figure 6-3: Global selection of the maintenance strategy.

The maintenance strategy can be determined by using Figure 6-3. The starting point of the maintenance strategy figure is in the upper right corner: consequence of failure. The consequence of failure in most hydraulic systems is great, take for example the Measlandt barrier, Oosterschelde barrier, and of course also the Afsluitdijk. The next step indicates whether there is a good description of the course of strength of the structure or not. Completely condition dependent maintenance will be carried out if it is not possible to make a

prognosis of the strength in the course of time or if inspection is very simple and therefore inexpensive. Moreover, an important aspect of the condition dependent maintenance is collecting data concerning strength in the course of time. This allows better planning of maintenance or inspections.

6.5 Maintenance costs

In general, strength of the structures will decrease over time and what eventually will lead to maintenance of the structure. In paragraph 6.4, maintenance has been defined as to keep its original technical state or revert it back to this state. This refurbishment to this original state will be assumed as fully recovery of the structure. Therefore, decrease of strength over time will be exactly equal as the amount of maintenance that must be taken at a certain moment. Because decrease of strength on a certain moment of time is not exactly known, as well as the amount of maintenance. The maintenance costs of the dike depend on the decrease of strength and can therefore be seen as a function of the amount of maintenance:

$$C_{\text{maintenance}} = g(\Delta R(t))$$

The present value of the costs on the moment of maintenance can be calculated by:

$$PV_0^t = g(\Delta R(t)) \cdot \exp(-r \cdot t)$$

And the expected net present value of the maintenance costs will be:

$$E(PV_0^t) = \mu_{g(\Delta R(t))} \cdot \exp(-r \cdot t)$$

The expected net present value of the maintenance costs is in general a decreasing function of t. By using the simplest form of the maintenance function:

$$C(t) = g(\Delta R(t)) = C_i + c \cdot \Delta R(t)$$

In which:

$C(t)$ = maintenance costs at moment t [€]

C_i = costs per maintenance activity [€]

c = costs per unit of strength [€/R]

$C_{\text{maintenance}}$ = maintenance costs of the dike [€]

$g(\Delta R(t))$ = function g of the amount of maintenance dependent of change in strength [€]

PV_0^t = present value of the costs on the moment of maintenance activities [€]

r = failure rate [yr^{-1}]

t = time [yr]

$E(PV_0^t)$ = expected net present value of the maintenance costs [€]

$\mu_{g(\Delta R(t))}$ = expected mean present value of the maintenance costs [€]

6.6 Optimization of maintenance intervals

The net expected present value of the total cost during the maintenance phase depends on time intervals of planned maintenance. In Figure 6-4 a general illustration has been given of the costs over time. The present value of the maintenance costs decreases over time as a function of the discount rate and the risk costs increases over time, because generally the strength of the structure decreases over time which lead automatically to a higher risk. The total cost is a summation of the maintenance and risk costs.

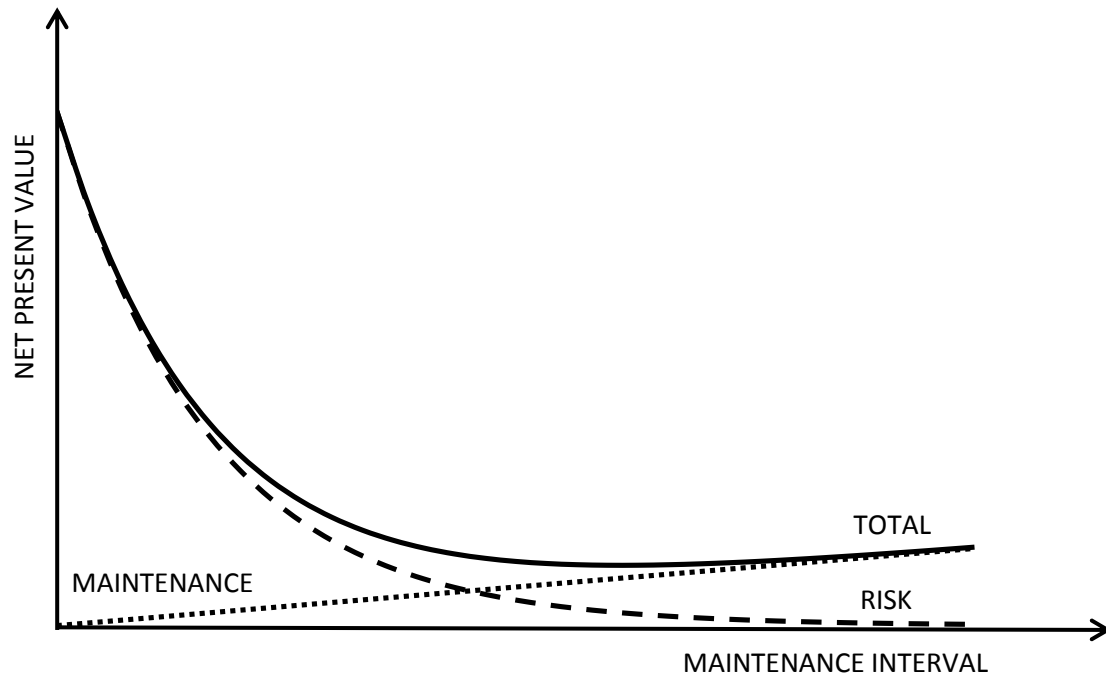


Figure 6-4: Present value of the (maintenance, risk and total) cost dependant on the maintenance time interval (Vrijling & Van Gelder, 2006).

The optimization of maintenance can be determined by searching for the minimum of the expected net present value of the total cost. There are two kinds of optimization of the maintenance:

- 1) Invariable deterioration process;
- 2) Variable deterioration process.

The invariable deterioration process describes maintenance intervals of the same length. The expected net value of the present value of the total costs depends of just one single variable; the maintenance interval:

$$E(PV_{tot}) = f(t)$$

This function has a minimum for a certain value of t in which:

$$f'(t) = 0 \text{ and } f''(t) > 0$$

The variable deterioration process describes a couple of maintenance intervals with different lengths. This total present value depends then on n variables:

$$E(PV_{tot}) = f(t_1, t_2, \dots, t_n)$$

It is not possible to illustrate more than two variables in a graph of the present value. To find the minimum of the present value of a function with more than one variable, one uses the same method as the invariable deterioration process. Only in this case one is using the partial derivative of the function $f(t_1, t_2, \dots, t_n)$ and set this equal to zero:

$$\left(\frac{\partial}{\partial t_1} f(t_1, t_2, \dots, t_n), \frac{\partial}{\partial t_2} f(t_1, t_2, \dots, t_n), \dots, \frac{\partial}{\partial t_n} f(t_1, t_2, \dots, t_n) \right) = (0, 0, \dots, 0)$$

$$\text{Or in other a more simple notation: } \nabla f(\bar{x}) = 0$$

These two optimization methods will both be used in the hydraulic engineering. For example, maintenance on a road will describe an invariable deterioration process whereby all the parameters stay constant and only the maintenance interval is variable. The expected net present value will result in a constant maintenance interval. However, this is only true when the reconditioning will be done with the same properties as the original condition.

An example of variable deterioration process can be illustrated by increasing the height of a dike, because heightening of the crest level by sand will lead to a higher soil pressure whereby the subsoil consolidates. When consolidation is in process it is the challenge to find the most economical beneficial point in time to increase the crest level again to its original height. After this new heightening of the dike the consolidation process of the subsoil will now move a lot slower, because of the fact that the subsoil has become more stiff what lead to a longer consolidation period. Heightening of the dike will now be better to do this in a longer period of time than the first maintenance interval. This will continue until the point has come that it is more beneficial to not increase the top level anymore, because the present value will not have any minimum point and approaches the present value of the risk (asymptotically).

7. Theory VS. Practice

The VNK gives a prioritization of the largest risks of the considered system. The result from this analysis has been given in a list of risks ordered on highest criticality (high product of the combination of damage and probability of occurrence) which has been determined based on the method elaborated in paragraph 0.

The list with risks will form the base for RAMSSHEEP what eventually will lead to a decent maintenance plan for the primary flood defence system. In the Figure 7-1 below a schematic illustration has been given of the process for the conservation of the system.

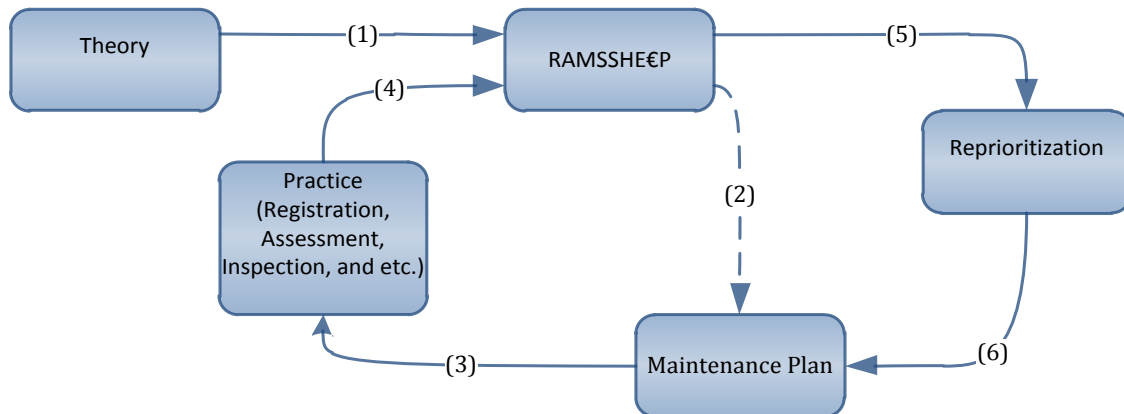


Figure 7-1: Schematic illustration of the RAMSSHEEP scheme which describes an infinite cycle between theory and practice.

The illustration above describes the circle between the theory and the practice of the conservation of a certain system. The theory of the qualitatively and quantization methods can be used to make an RAMSSHEEP analysis (1). The analysis will determine which numbers and requirements in which the system should be fulfilled.

The results from this analysis will form the base for maintenance plans of the primary flood defence system (2). This maintenance plan exists of a time line which represents maintenance activities in the coming years. Some of these activities can be executed annually and some of them will be executed once in the lifetime (i.e. full reparation) of the system.

The maintenance plan must be put into practice to ensure the RAMSSHEEP aspects of the system (3). The practice can be divided in several tasks in the field, i.e. registration of the loads, assessment of the conditions, inspection of the structures, and etcetera.

Due to the performed maintenance activities a new situation of the condition can be assessed. Some risks have been reduced due to these maintenance activities, but also some risks have become larger over time. So, a new RAMSSHEEP analysis can be made based on the new situation of the primary flood defence system (4). This analysis will result in a new list of critical risks within the system and therefore RAMSSHEEP. Eventually this will lead to a reprioritization the failure mechanisms (5) and therefore lead to an adaption of the maintenance plan (6).

This cycle (RAMSSHEEP – Reprioritization – Maintenance Plan – Practice) goes over and over again to ensure that the system will fulfil its functions as good (with an economic point of view) as possible.

8. Case Study: RAMSSHEEP on Afsluitdijk

8.1 Introduction

The Dutch water law (Waterwet) makes it obligatory to check the condition of the primary flood defence system to its original requirements. This check has to be done every six years and the results will be presented by Rijkwaterstaat. Therefore the Afsluitdijk must also be checked whether the current situation will meet the required performance level.

The Afsluitdijk can be seen as a part of the Dutch primary flood defence system and exists of the dike construction, two lock complexes and a road connection between North Holland and Friesland. There are also a couple of moveable bridges which passes the navigation locks and the discharge sluice should secure the hinterland from flooding by draining the surplus water (from the rivers) to the Wadden Sea. The locks are important for navigation and discharge and therefore the availability of the locks are important to analyse.

The system of the Afsluitdijk is globally known when one analyses its functions, physical structure elements and possible failure mechanisms. These aspects are not always fully known or the data is not always present. Therefore this data must be gathered and analysed which can be done by Probabilistic Management and Maintenance (PMM) based on the Reliability Centered Maintenance (RCM). This document has a dynamic character and therefore also an iterative connection whereby several experiences will be used. Iterative connections exist of production-, maintenance- and application experience, but also the changes in requirements and system boundaries will be implemented.

For this case study a part of the Afsluitdijk has been analysed. A risk-based optimization has been adopted over time as the main principle on which to base an analysis of acceptable flooding risks. The basic form of risk-based optimization is economic optimization that is aimed at minimization of the lifetime cost of the total system. The final solution has been translated to RAMSSHEEP requirements if that is even possible.

8.2 Why this case study?

This case study of the Afsluitdijk has mainly been executed to test which aspects of the RAMSSHEEP will be used in the maintenance phase of a construction. This result should make RAMSSHEEP more transparent and understandable during maintenance activities and which aspects actually should be used.

The problem of this case study is that the current condition of the Afsluitdijk is not sufficient to requirements of the Dutch water law. Calculations has been made to estimate the monetary risk of the current situation and the most plausible and feasible solutions to decrease these risks. By using the principle of minimisation of lifetime cost of the system will result in a solution. This final solution will eventually be translated to RAMSSHEEP aspects that are determined in its own descriptions. It may be obvious that not all aspects will be

filled in, what means that it has not been used in the solution that has been given by the optimisation method.

This way of determining a solution can also be illustrated by Figure 8-1. In this figure the optimisation method describes a bottom up approach: solving the problem in a feasible way (green field) by approaching possible engineering and social solutions. Eventually the most economical solution will be chosen what subsequently will be translated to the original acronym: RAMSSHEEP.

From the acronym RAMSSHEEP to the solution (top-down approach) can only be found by using a simple model and a lot of assumptions. It already has been set in a standard form which can be not formed to the problem you are facing. Therefore this top-down approach will never lead to the economical most favourable solution.

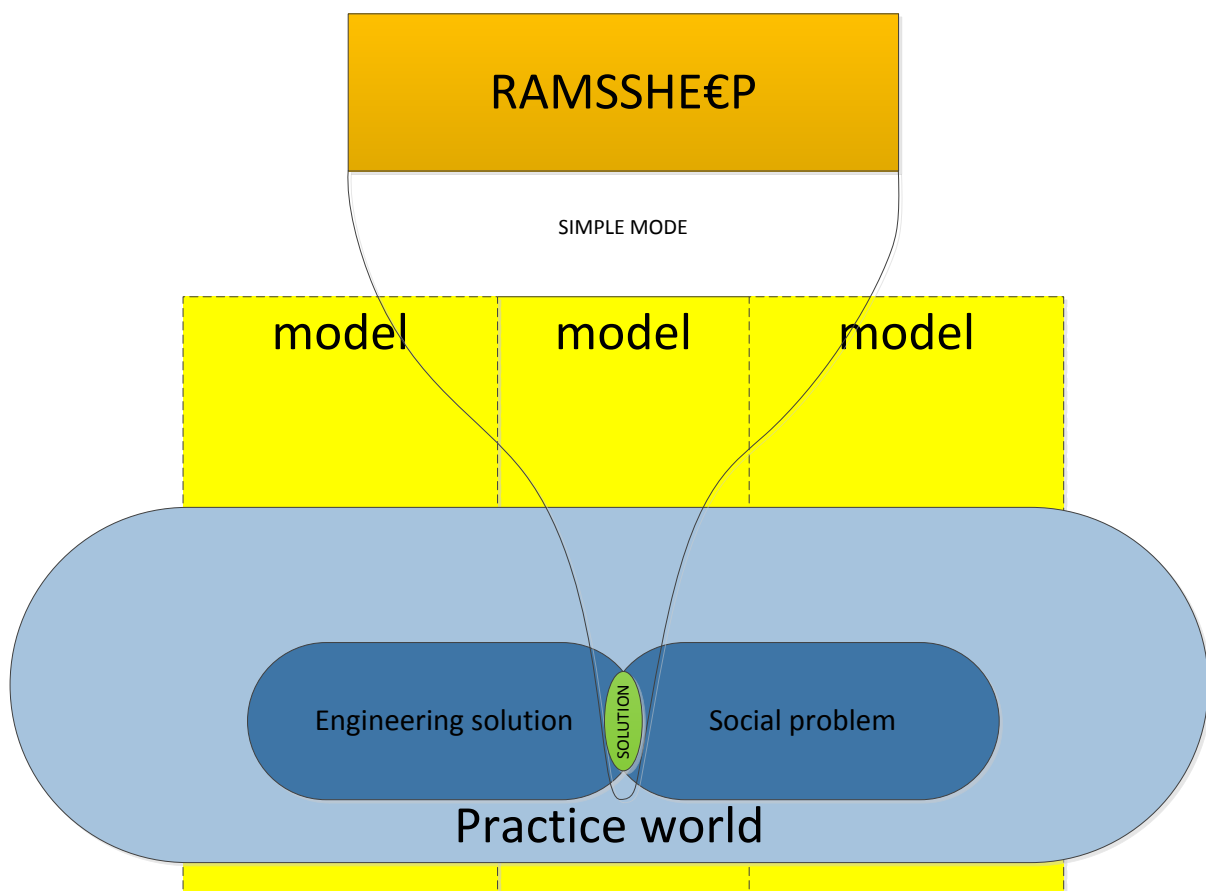


Figure 8-1: Schematic illustration of possible solution field.

8.3 Function analysis

After the introduction of the system it is necessary to determine which functions are applicable for further analysis. The RAMSSHEEP aspects can be related and connected to this required performance. A decomposition of the functions will result in an overview of which performances lead the largest impact of good functioning of the system. The most important functions of the Afsluitdijk are:

- retaining of water (high/storm level);
- discharging (Q) of surplus water from the rivers;
- navigating ships through locks; and
- road connection over the dike.

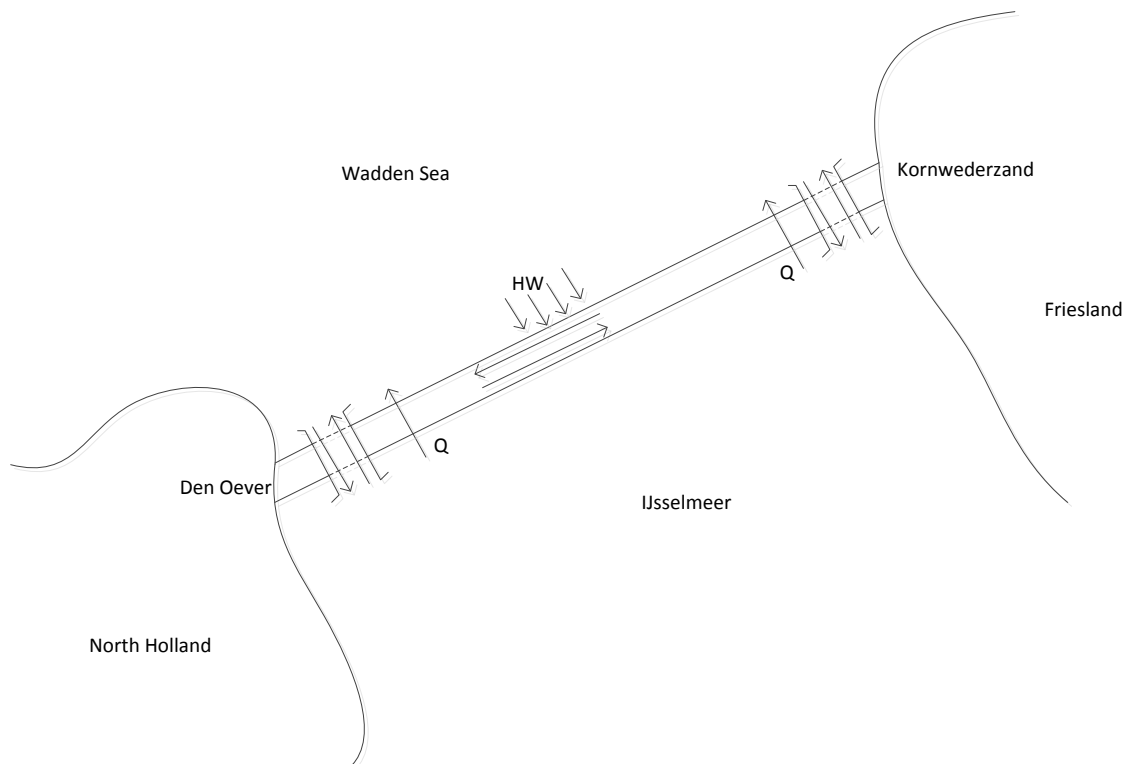


Figure 8-2: Schematic overview of the four function in the system of the Afsluitdijk.

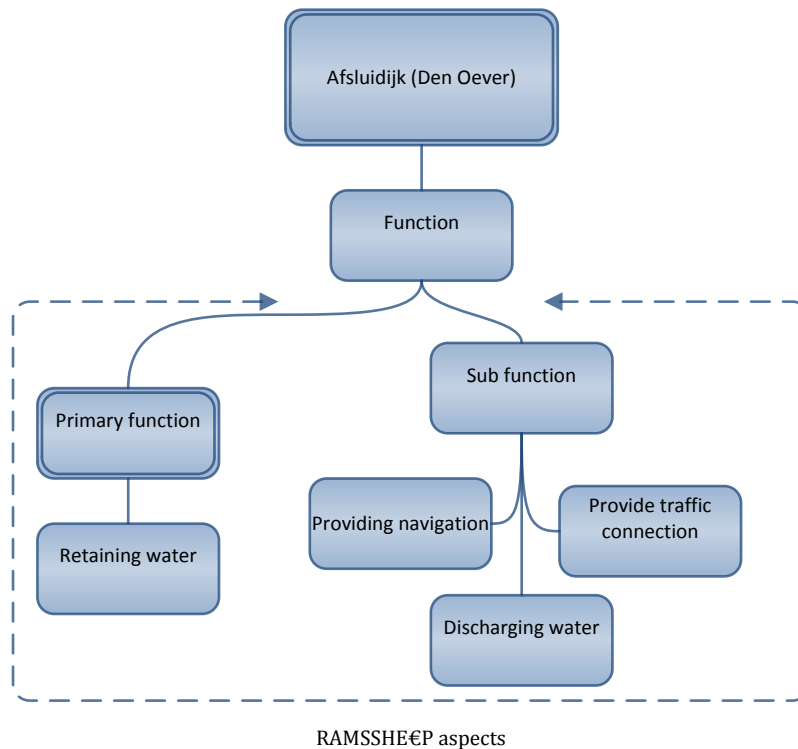


Figure 8-3: A schematic view of the functions of the Afsluitdijk at location Den Oever.

Retaining of water (high/storm level)

This function describes the retaining function of high waters (storm water levels) to ensure the safety against flooding of the hinterland of the Netherlands. This retaining function is present in both directions, which depends on the orientation of the danger of flooding. The safety level of failure frequency of the Afsluitdijk has been determined on 1:10.000 (with respect to the primary danger of the Wadden Sea). The danger from the IJsselmeer has a safety level of failure frequency of 1:4.000.

Drainage of surplus water from the rivers

This function describes the discharging of the surplus water from the IJsselmeer to the Wadden Sea, mainly to regulate the water quantity and the water quality. This discharge function has been important to preserve the system from failing, like:

- safety against flooding;
- water discharge;
- drainage of ice or sediments;
- navigation;
- recreation;
- swim water;
- nature/ecology fishery;

- regional water supply;
- cooling- and drinking water;
- quality of water soil; and
- quality of water.

The locks always have to be able to open or close, but the term 'always' is not concrete and has no expressed unit. This means the phrase has to be redefined to:

'The availability of the locks has to be determined as high as possible, based on the highest ratio of the total costs and benefits'

Based on this new statement the failure frequency of the discharge function has been assumed on 1:100 per use. This requisite can be given by a concrete requirement of the discharge or a certain water level.

Navigating ships through locks

The ships should as fast as possible pass the locks (so this means without any delay from the normal time to pass the locks). The Dutch water law (Waterwet) determines that the professional navigation (approximately 45.000 ships a year) in 75% of the cases will pass the locks within the given maximum passing time. For the locks of the Afsluitdijk this number has been determined on 40 minutes.

Road connection over the dike

This function describes interaction of road traffic and the traffic on water by a provincial road over the dike and locks. This system forms a part of the connection between Den Oever (NH) and Zurich (Fr). This road connection is important for the people who are going to work and using this dike as a shortcut. Therefore, availability of the road connection and movable bridges has a large societal benefit factor due to the large time of the detour.

8.4 Specifications of the objects

The chosen location at Den Oever describes a 1,6 kilometer long dike where other functions has been implemented in the scope (see Figure 8-4). This object has been classified to the category of *shores and dikes* of the primary flood defence system in the Netherlands. The part of the Afsluitdijk at Den Oever had been constructed in 1927 and is therefore the first part of the dike which has been constructed.

Based on Figure 8-4 four functions of the Afsluitdijk can be represented by four different block shapes:

- Red: navigation lock;
- Blue: discharge sluice;
- Green: dike; and
- Yellow: road connection A7.



Figure 8-4: A bird view of the current situation of Den Oever which can be divided in a lock complex Stevinsluis (red), discharge sluices (blue), dike (green) and the road connection (yellow) (Source: Google Maps).

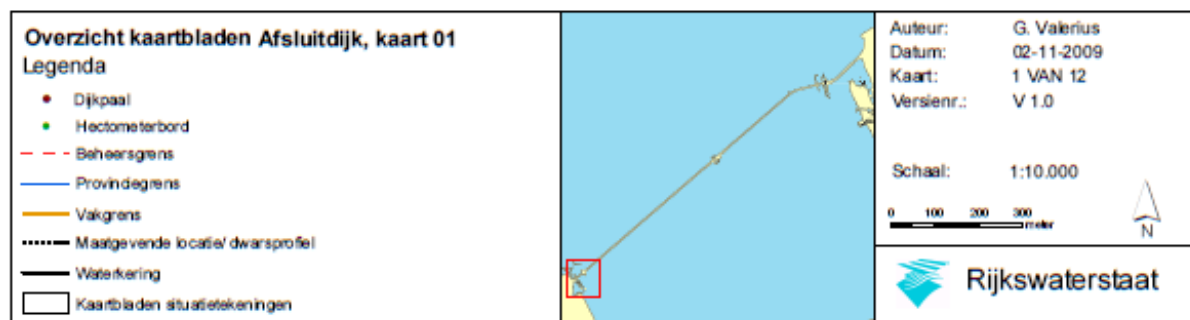
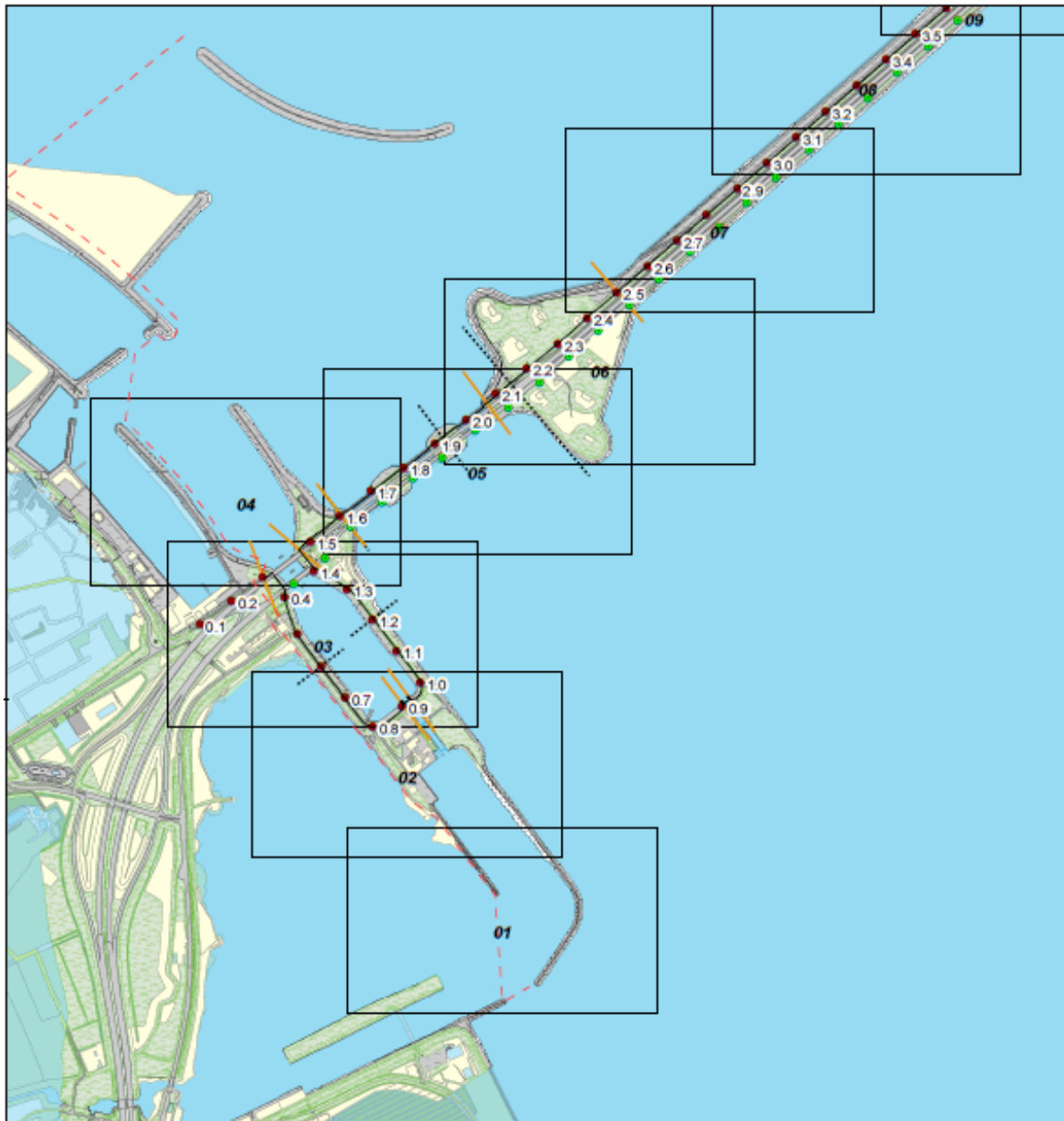


Figure 8-5: Overview of the location Den Oever (Ibelings, 2012).

September 18, 2012

Delft University of Technology < > DPI Consultancy

Navigation lock – Stevin (Den Oever)

To maintain the soil profiles in front and behind the lock against propeller turbulences, it is necessary to create a stable bed protection below water level. This is the most efficient method to prevent the structure from failing (instability of the subsoil). Also some electricity, phone cables and water pipes are present in the near dike compartment. The soil profiles should therefore be inspected and maintained on a regularly basis, because the lock has beside the function of navigation also the function retaining water (Ibelings, 2012).

Before the ships will arrive at the actual lock it first has to pass a rotating bridge, which allows the A7 road connection from Den Oever travel to the other side of the IJsselmeer. The water depth decreases towards the lock and therefore leads to lower required navigation speed.

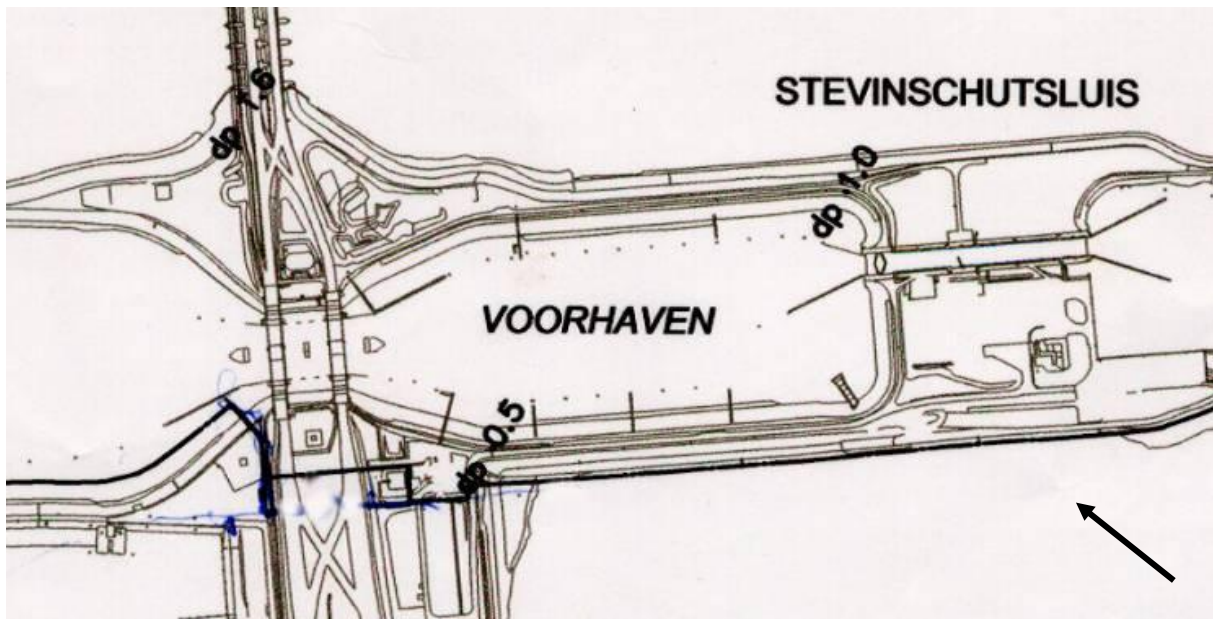


Figure 8-6: Technical view of the lock complex (Ibelings, 2012).

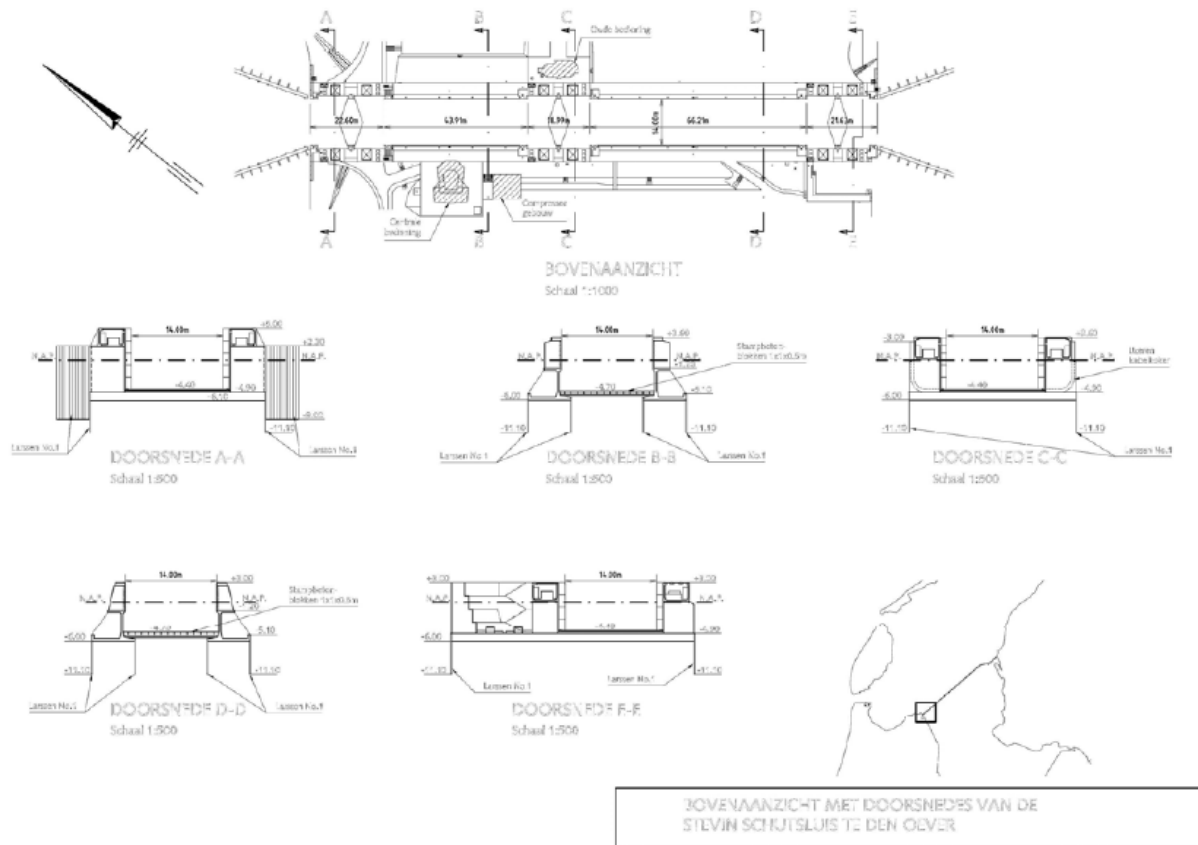


Figure 8-7: Technical design drawings of the navigation lock of Den Oever (Ibelings, 2012).

The dikes of the outer harbour are also a part of the primary flood defence system, because technically these dikes are in direct connection with the Wadden Sea. The outer harbour has the function of channel guide way for the navigation. The main material of this part of the dike exists of a sand body which has been covered with a thick layer of boulder clay.

The navigation lock has only one lock chamber which can be summarized in the table below.

Table 8-1: Dimensions of Stevin lock complex (Ibelings, 2012).

Detail	Length [m]	Width [m]	Depth [m]	Retaining height [m]
Dimensions of lock chamber	120	14	4,40	NAP + 4,88
Maximum dimension of ship	110	13	4,20	-

The working length of the lock chamber is 120 meters with a working width of 14 meters. The ground sill of the chamber has a level of NAP -4,40 meter and the maximum lock level is NAP + 1,80 meter. Ships with maximum weights of 6.000 ton will fit in the Stevin lock complex, which is functional 20 hours a day for professional navigation (annually ± 3.000 ships) as well as recreational navigation (annually ± 33.500 boats) (data from 2007) (Ibelings, 2012).

Discharge sluices

The blue part of Figure 8-4 indicates discharge sluices at Den Oever of the Afsluitdijk system. This discharge complex exists of three drainage compartments. The A7 road connection passes this drainage facility on top of its structure which exists of two traffic lanes. Water level in front of the sluice has a depth of -5,0 NAP and exists mainly of sand and boulder clay. At the south side of the discharge sluices (IJsselmeer side) water can move freely towards the drainage facility. However, at the north side a scouring basin has been constructed and levees lead drained water to the Wadden Sea. These levees are also important for the water retaining function, because this significantly reduces wave height.

Table 8-2: Dimensions of the three similar discharge sluices (Ibelings, 2012).

Detail	Compartments	Width [m]	Height [m]
Discharge sluice	5	12,0	6,9

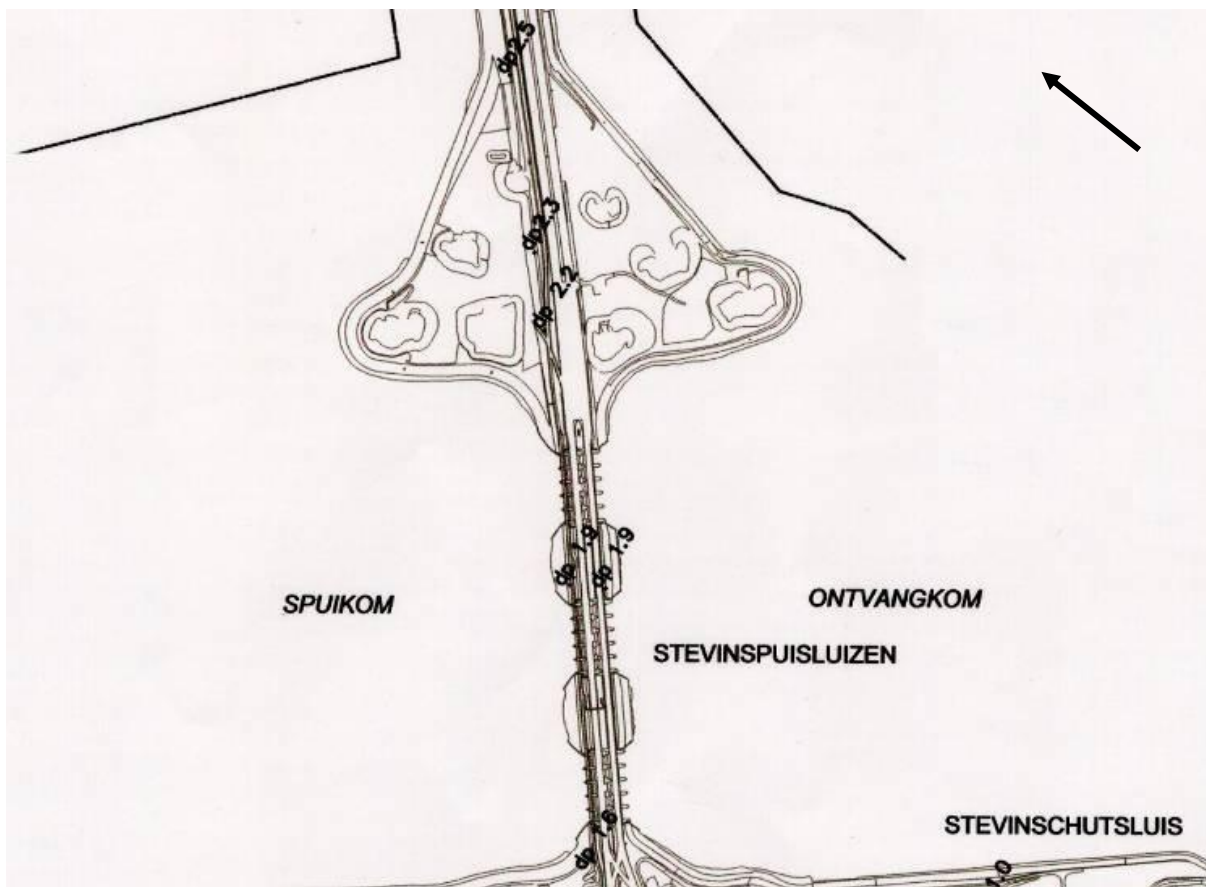


Figure 8-8: Technical view of the discharge sluice (Ibelings, 2012).

On average about 290 m³ water per second is being drained from the IJsselmeer to the Wadden Sea. This can be determined from the graph below, see Figure 8-9. In the winter period more water is being drained in comparison with summer periods.

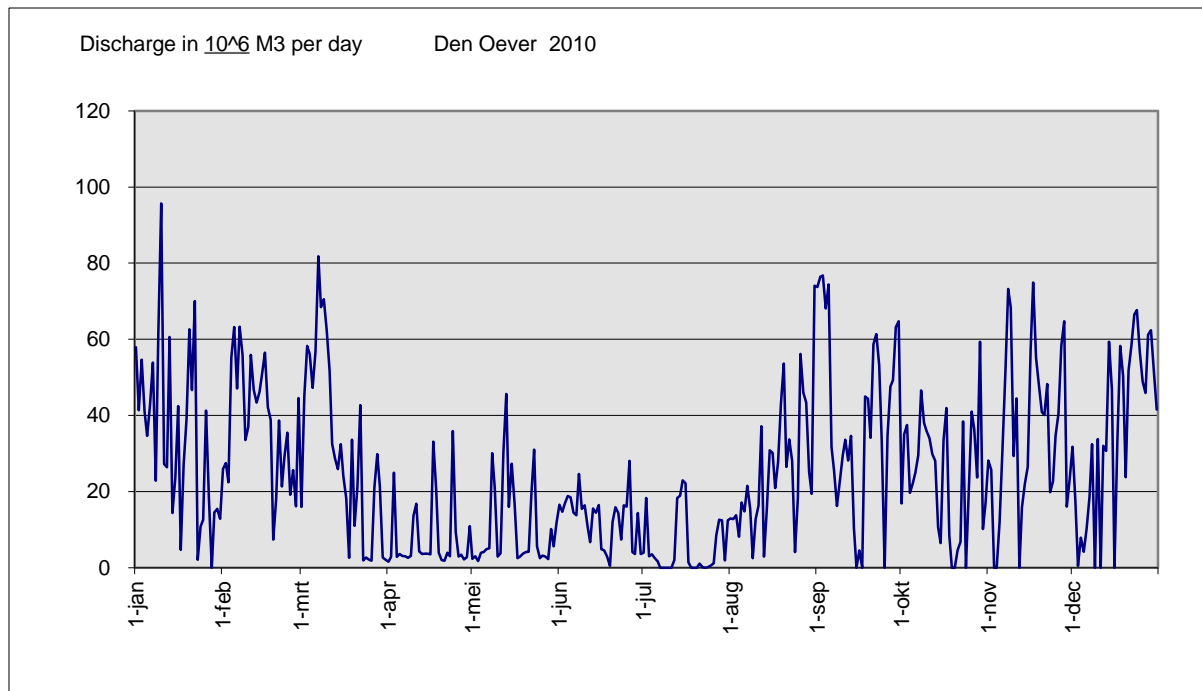


Figure 8-9: Discharge of the discharge sluice per month at Den Oever (RWS, 2010).

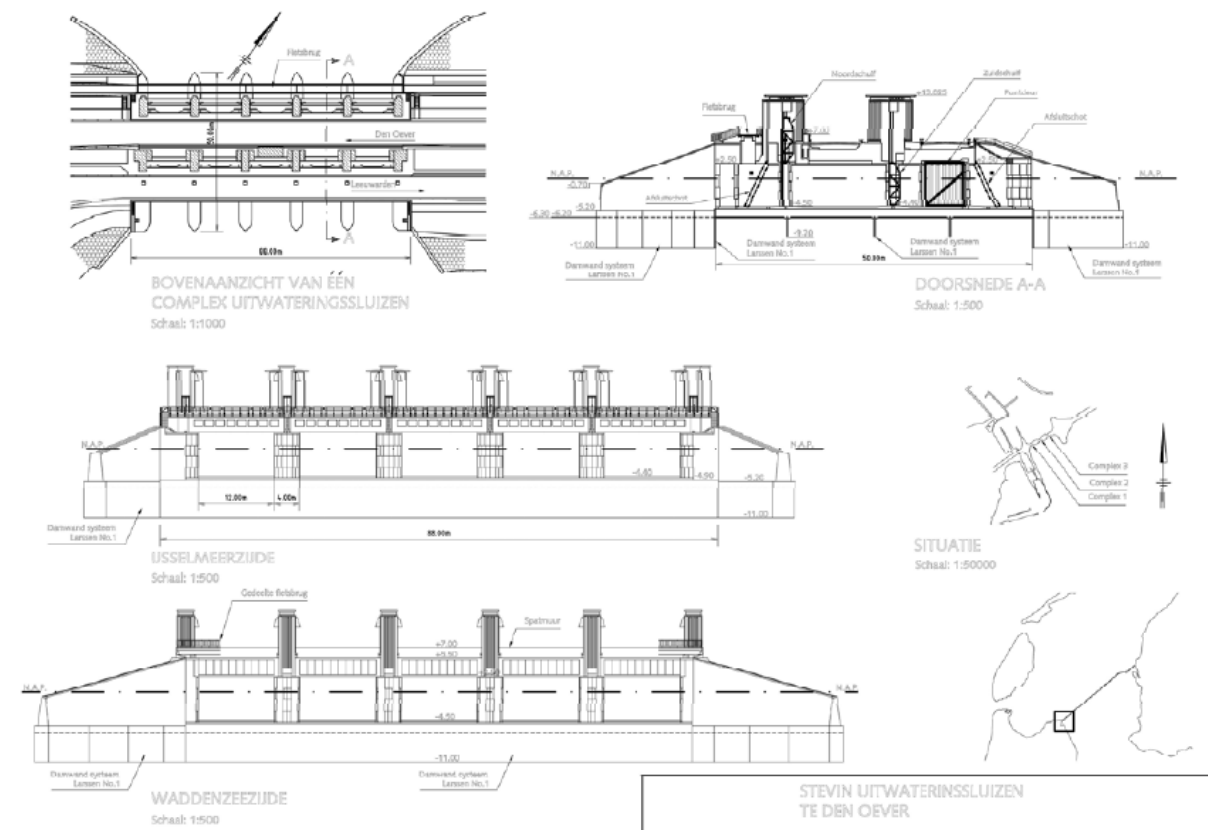


Figure 8-10: Technical design drawings of the discharge sluice of Den Oever (Ibelings, 2012).

Dike

The Afsluitdijk protects areas, which border to the IJsselmeer, against flooding from the Wadden Sea. This dike forms an important part of the primary flood defence system and defends a significant part of the Netherlands from flooding. The diked area 6 (Friesland and Groningen) and diked area 12 (North-Holland) are connected by the Afsluitdijk and the Dutch water law (Waterwet) classified the connecting flood defence dike with a probability of flooding of one in ten thousand years (1:10.000).

The profile of the dike can be assumed constant over the total length and the only irregular dike profiles are present at Den Oever and Kornwederzand (navigation lock and discharge sluices, bridges). Within the dike many cables are present which take care of the electricity, phone cables and road signals. The water depth in front of the dike at the Wadden Sea side is regularly and is assumed to be constant over the first 100 meters.

Every primary flood defence system has to fulfill to the set requirements which has been documented in the TAW Richtlijnen. This document gives an overview of elements that should be checked in terms of strength and safety requirements that has been based on expert judgment data. Elements that should be checked are:

- Outer slope below water level;
- Outer slope revetment;
- Grass coating;
- Inner slope revetment;
- Connection points between the constructions (locks, sluice, dike, and road).

To ensure the safety of the Afsluitdijk it is obligatory for the manager of the system to do a check-up of the current condition which has to be made every six years. This statement has been documented in the Dutch water law.

The dike, which has been indicated by the green block in Figure 8-4, has a diversity of soil composition, morphology, hydraulic requirements, sea/lake water height, type of system, rock filling, and bottom protection nearby constructions. These aspects are for every dike section different and therefore form an unsteady profile. This is the reason to assume a uniform dike profile in the length direction for several dike section of a determined distance (here: 100 meters).

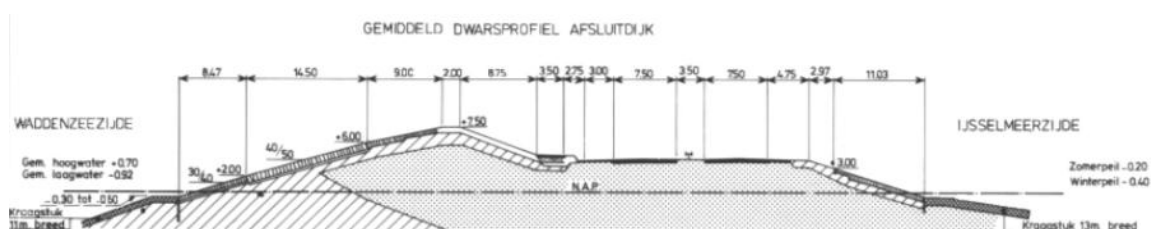


Figure 8-11: Average current cross-section of the Afsluitdijk (Ibelings, 2012).

On the side of the Wadden Sea the sand body of the dike reaches NAP + 5,50 meter which has been covered with 1,0 meter of boulder clay. The slope below water level has a gradient of 1:4. The other side of the dike the sand body is covered from NAP – 1,00 meter with 0,75 meter boulder clay.

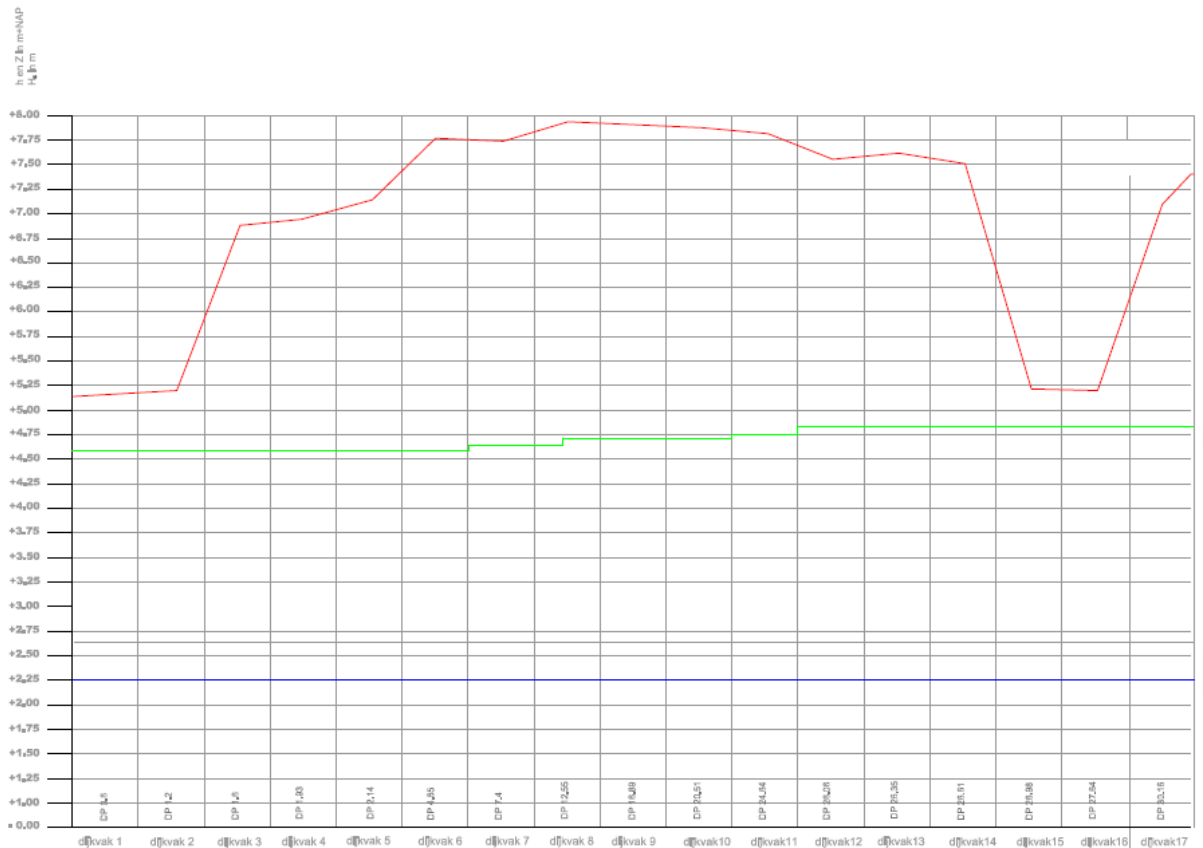


Figure 8-12: Length profile of the Afsluitdijk with respect to the crest height (red), test level (green) and significant wave height H_s (blue) (Ibelings, 2012).

Figure 8-12 illustrates the crest height over the length profile of the Afsluitdijk. By observing values of the red line, it is remarkable to see that the crest height between 'Dijkvak 1' and the other varies with about 2,80 meters crest level. Even while driving over the Afsluitdijk this difference will not be observed.

The reason for this difference cannot be explained, and RWS only gave this profile of the crest height, but cannot give a reason for this particular difference. Reasons that can be dedicated to this difference are i.e. specific located consolidation of the subsoil, or the crest height has never been that high because of the structures in that particular area (navigation lock, drainage sluice, bridge), and etcetera.

8.5 Probabilistic approach

8.5.1 Economic damage due to system failure

The first step in the process of making a probabilistic approach to solve the problem is to make a rough calculation of possible economic damage that will occur when the system fails. In this calculation one should interpret system failure in a situation if one of four functions does not work the way it is supposed to. For example, one of two road lanes has been blocked (immediate failure, planned maintenance, and etcetera) what causes congestion during evening jam in both ways. This can be translated to indirect damage to the cars, trucks and other vehicles. Or in a more extreme way both lanes are blocked what will result in a detour.

The total annual risk due to system failure can be calculated by composing the four economic damages multiplied by the annual probability of occurrence and sum it up:

$$R_{total} = D_{road} \cdot P_{road} + D_{navigation\ lock} \cdot P_{navigation\ lock} + D_{discharge\ sluice} \cdot P_{discharge\ sluice} + D_{dike} \cdot P_{dike}$$

With:

R_{total} = total annual cost expectation (monetary risk) due to functional failure [€]

D_i = damage due to functional failure [€]

P_i = annual probability of occurrence of functional failure [-]

All these individual damage parameters will be elaborated and eventually calculated for different scenarios. This will be elaborated in the coming paragraphs.

8.5.2 Road connection

The road connection A7 is a highway from Den Oever (North Holland) to Kornwederzand (Friesland) and forms an important passage way for foreigners, truckers and recreational people. On an average day almost 40.000 vehicles are passing the Afsluitdijk in both directions. The direct and indirect damage will be calculated based on two scenarios:

- 1) The first scenario describes congestion duration T on the road and involves an average congestion length L due to any possible reason; think of a car incident, failure of asphalt, and etcetera.
- 2) The second scenario describes blocking duration T of the road and involves an average congestion length L due to any possible reason; think of bulge of the road, big accident, and etcetera.

NB. The following numbers that have been determined are based on an average of these two scenarios.

The causes of congestion are most of the time caused by a structural lack of capacity of the high way (84%), accidents (12%) and maintenance activities (4%) (Rijkswaterstaat, 2006).

The direct damage describes damage based on lost time of the people who are bothered by the functional failure. Moreover, the indirect damage describes the logistic damage of the

companies which are supplied by trucks which are transporting the goods over the Afsluitdijk.

The annual cost expectation can be translated in formula¹⁸ form:

$$R_{road} = D_{road} \cdot P_{road} = C_1 \cdot L \cdot T$$

With:

C_1 = economic damage (direct and indirect) per kilometer per minute [€/km/min]

L = congestion length due to the functional failure [km]

T = congestion duration by the functional failure [min]

NB. The congestion duration parameter T can show some vagueness, because one may assume several definitions for the explanation of this parameter. Here, the congestion duration T is the average duration of the presence of the congestion. So, this means that the total congestion of the traffic will be approached as the main driver. This also means that this analysis will not use individual vehicles and their individual loss time during this congestion. The loss time of a vehicle is way less than the duration of the congestion itself and because it does not matter who has damage due to the congestion (because it will be summed up anyway), it is better to analyze the congestion itself (and its duration) than to analyze an individual vehicle. To conclude, the congestion duration T is the duration of the congestion itself and does not depend on who is in the congestion.

Determine C_1 -value

A consultancy agency calculated that the direct and indirect damage to the Dutch economy is approximately 500 million euros in one year due to congestion. In this calculation the agency did not count on extra fuel usage which is about 200 million euros. This makes a total of 700 million euros damage due to congestion in one year in the Netherlands (Damage due to road congestion, 2012).

Rijkswaterstaat monitors every day congestion numbers of the highways in the Netherlands. One of the most important numbers measured is the weight of congestion which has been defined by the average congestion distance multiplied by the duration of the congestion. In 2011 Rijkswaterstaat determined that this weight of congestion is about 9,9 million kilometer minutes (Rijkswaterstaat, 2011).

From the information above the average C_1 -factor can be calculated in euros per kilometer per minute of congestion:

$$C_1 = \frac{700.000.000 \text{ [€]}}{9.900.000 \text{ [km} \cdot \text{min]}} = 60,34 \cong 60 \text{ €/km/min}$$

¹⁸ Assumption: The costs of congestion on the road occurs every year on average or in other words; the probability of occurrence has been determined on 1. By using average costs and data of the location this will lead to the implementation of the probability.

The uncertainty of this number is hard to quantify. The direct and indirect costs have been determined by a special consultancy agency in traffic and one may assume that this company has many reliable numbers. However the agency had to make some assumptions to come to this amount of damage and therefore result in an average uncertainty.

Determine L-values and T-values

The total amount of kilometers has been registered over the years by several traffic institutes and be published on internet divided over all Dutch highways. The road connection over the Afsluitdijk at the location Den Oever has been classified to the highway number A7. The historical data of congestion at A7 Den Oever shows that the average congestion is about 1 kilometer per day (Average congestion at Den Oever, 2012):

$$N = 1 \text{ km}$$

Beside the total amount of kilometers also the total amount of congestion time has been determined by RWS. The road connection over the Afsluitdijk at the location Den Oever has not been determined and therefore will be calculated based on data of a similar highway with a similar amount of kilometer of congestion. From a data list of RWS of the top 50 highest congestion (not including the highway A7) resulted in average daily congestion duration of 30 minutes what can be used in the calculation of congestion time (Congestion time at Den Oever, 2012):

$$T = 30 \text{ min}$$

The annual cost expectation can be calculated by:

$$\begin{aligned} R_{road} &= C_1 \cdot N \cdot T \\ &= 60 \text{ [€/km/min]} \cdot \frac{1[\text{km}] \cdot 30[\text{min}]}{1 [\text{d}]} \cdot 365 [\text{d/yr}] = \text{€ } 657.000^{19} \end{aligned}$$

8.5.3 Navigation lock

The navigation lock forms a passage way for the professional and recreational navigation from the IJsselmeer to the Wadden Sea and vice versa according to the waterway number 302 which represents navigational route of Den Oever/Texel/Den Helder/North Sea. Each year almost 1.000 lock actions have been executed at the Stevin lock complex in both ways. In this total amount of lock actions ships are present on a professional basis, so this means that the recreational part has been eliminated from damage analysis. The Stevin lock is able to transport ships with a maximum classification of CEMT Vb and is available 24 hours per day and 7 days a week over one year. The direct and indirect damage will be calculated on a basis of two scenarios:

¹⁹ By assessing this result, the number is quite high. The annual damage due to congestion on the Afsluitdijk forms a serious amount of money that will be lost.

- 1) The first scenario describes congestion duration T during the lock process and involves N professional ships due to any possible reason; think of difficult weather conditions, failure of critical lock elements, and etcetera.
- 2) The second scenario describes blocking duration T of the navigation lock and involves an average amount of N professional ships due to any possible reason; think of long lasting technical problems, constructional failure of a lock door, and etcetera.

NB. The following numbers that have been determined are based on an average of these two scenarios.

The navigation is equally divided over the year what lead to an average amount of ships of 3 per day. The average lock time is approximately 35 minutes per ship and congestion will occur when the passage takes longer than 60 minutes.

The direct damage describes damage based on lost time of navigation that are bothered by functional failure and waiting time during bridge passing. Moreover, indirect damage describes logistic damage of companies which are supplied by ships which are transporting goods along the Afsluitdijk.

The annual cost expectation can be translated in formula form:

$$R_{navigation\ lock} = D_{navigation\ lock} \cdot P_{navigation\ lock} = C_2 \cdot N$$

With:

C_2 = economic damage (direct and indirect) per ship [€/ship]

N = annual amount of ships which has congestion due to the functional failure [ship]

Determine C_2 -values

The congestion costs (hard numbers) for navigation is hard to determine, because there is not much representable information available. The research institute TNO calculated waiting costs of navigation per hour per ship making use of prefix. The waiting costs of navigation have been calculated on €1.940 per ship based on data from 2006. The background data, what lead to this number, is not clear and known (Vellinga & de Jong, 2010)

From the information above the average C-factor can be given by:

$$C_2 = 1.940 \text{ €/ship}$$

The uncertainty of this number is hard to quantify. The direct and indirect costs have been determined by a research institute (TNO) and one may assume that this company has many reliable numbers. However the company had to make some assumptions to come to this amount of damage and therefore result in an average uncertainty.

Determine N -values

The total amount of navigation ships (only professional) has been registered over the years by RWS and be divided over the different navigation routes. The historical data of congestion at navigation route 302 shows that daily 3 ships are passing the lock complex.

$$N = 3 \text{ ship/d}$$

This number can be analyzed with a low uncertainty, because of the annual measurements of RWS (and thus a lot of data) (Rijkswaterstaat, 2012).

The annual cost expectation can be calculated by:

$$R_{navigation\ lock} = C_2 \cdot N$$

$$= 1.940 \text{ [€/ship]} \cdot 3 \text{ [ship/d]} \cdot 365 \text{ [d/yr]} = \text{€ } 2.124.300^{20}$$

8.5.4 Discharge sluice

The discharge sluice at Den Oever is a regulated installation for draining fresh water from the IJsselmeer to the Wadden Sea. The surplus of water from the rivers will be stored in the IJsselmeer what later on will be used for drinking water for inhabitants of the Netherlands. On an average day almost 25 million m³ fresh water is being drained from three sluices. The maximum discharge capacity is about 3.000 m³ per second (by a water drop of 2 meters). The direct and indirect damage will be calculated based on one scenario:

- The scenario describes failure of all the sluices of the discharge complex due to any possible reason; think of mechanical failure, power loss, and etcetera.

The direct damage describes damage based on environmental damage (flora and fauna), damage to pumping station, and water damage to surrounding polder areas. Moreover, indirect damage describes economic consequences after the failure has occurred.

The economic risk can be translated in formula form:

$$R_{discharge\ sluice} = D_{discharge\ sluice} \cdot P_{discharge\ sluice} = C_3 \cdot P_{discharge\ sluice}$$

With:

C_3 = economic damage (direct and indirect) [€]

$P_{discharge\ sluice}$ = annual probability of occurrence of functional failure of all the sluices at Den Oever [-]

Determine C_3 -value

First of all, the water level of the IJsselmeer has been set on the highest water level ever occurred in the history: 1,55m above the average water level.

By assuming an 'extreme' water level in the IJsselmeer it may be obvious that the damage will not be very big. The dikes (levees) around the IJsselmeer can resist this height of water and therefore does not lead to flooding in surrounding polders. The damage due to high water level will come from small harbors (boats, jetties, and etcetera) and some problems with the pumping stations which pumps the water from the polders to the IJsselmeer.

The damage number has been estimated on 20 million euros to the surrounding of the IJsselmeer during a water level of 1,55m. Hereby data has been used of references of a

²⁰ By assessing this result, the number is even higher than the damage of congestion on the road (almost a factor 4). The annual damage due to congestion forms a serious amount of money that will be lost.

severe storm that causes damage to the whole Netherlands of 175 million euros (Causes of high water surplus, 2012).

The uncertainty of this C-factor is hard to quantify. The direct and indirect costs have been determined based on numbers and assumptions from assurance companies. However these companies also had to make some assumptions to come to this amount of damage and therefore result in a high uncertainty.

Determine P-value

The probability of occurrence forms an important indication of how often this situation will occur. Most of the time this number can be determined based on historical data of failure events of the installation. The object manager, in this case RWS, should control and monitor the discharge sluice but does not have any numbers present about failure events. This means that the annual probability of occurrence has to be determined based on Expert Judgment (EJ) by a group of experts²¹ with experience of the object. The P-value has been given by:

$$P_{sluice} = 0,10$$

$$P_{discharge\ sluices} = P_{sluice}^3 = 0,10^3 = 1,0 \cdot 10^{-3}$$

The decision of 10% failure probability of just one sluice is definitely an upper boundary what has been chosen during the consultation. The discharge sluices exists of 3 identical sluices what result in a total probability of failure of the discharge sluice of 0,1%.

The annual monetary risk can be calculated by:

$$R_{discharge\ sluice} = C_3 \cdot P_{discharge\ sluice}$$

$$= 20.000.000 \text{ [€]} \cdot 0,001 [-] = \text{€ } 20.000 \text{ }^{22}$$

8.5.5 Dike

The dike forms the mayor part of the Afsluitdijk and retaining water is the primary function of the system. Materials that has been used to construct the Afsluitdijk is mainly sand and boulder clay what eventually forms the top of the dike at a level of 5,25 m + NAP at location Den Oever (the weakest point of the total dike). And according to Dutch water law (Waterwet) the dike should fulfill the required safety level of one in the ten thousand years (1/10.000 per year). Direct and indirect damage and probability of occurrence will be calculated based on two scenarios:

- 1) The first scenario describes functional failure based on the Serviceability Limit State (SLS) whereby the road connection, navigation and drainage functions are not available due to wave overtopping.

²¹ Experienced group of people of RWS: E. Pompert, T. Zwanenbeek, H. Molenaar (2012).

²² By assessing this result, the number does not have (m)any effect on the total amount. This amount forms just a fraction of the other annual damages.

- 2) The second scenario describes function failure based on the Ultimate Limit State (ULS) whereby the complete dike collapses, the water level rises and salt water will enter the lake.

NB. The following numbers that have been determined will be determined separately for both scenarios.

Direct damage describes damage based on broken structures (levees, buildings, roads, and etcetera) due to the incident. Moreover, the indirect damage describes the losses of lives, environmental damage, image of specialists in hydraulic engineering will be heavily damaged and the economic consequences over time after the incident.

The annual economic risk can be translated in formula form:

$$R_{dike} = D_{dike} \cdot P_{dike} = D_{SLS} \cdot P_{SLS} + D_{ULS} \cdot P_{ULS}$$

With:

D_{SLS} = estimated (direct and indirect) economic damage based on scenario 1 [€]

D_{ULS} = estimated (direct and indirect) economic damage based on scenario 2 [€]

P_{SLS} = annual probability of occurrence of scenario 1 [-]

P_{ULS} = annual probability of occurrence of scenario 2 [-]

Determine D_i -values and P_i -values

The damage of the first scenario will be determined by summation of several damages. When the Afsluitdijk is not functional for one day (24 hour western storm) over the year the road connection, navigation and discharge activities will not be available to execute.

The probability of occurrence can be determined from the dataset from the water levels over the last 80 years. The functional loss of the Afsluitdijk occurs when the water level comes above 3,00m +NAP what can be translated to a frequency of occurrence of 3 times per year and here has been assumed that this unavailability takes one day.

So the damage of unavailability of the other functions is:

$$\begin{aligned} D_{unavailable} &= D_{traffic} + D_{navigation/ships} + D_{discharging} \\ &= \frac{3}{365} \cdot (657.000 + 2.214.300 + 200.000) \cong \text{€}24.000 \end{aligned}$$

Beside these damages also damage of the dike itself is important. A severe storm will cause serious damage to the revetment, stability, erosion, and etcetera. Critical elements should be checked after such an event. Referring to a storm from 2008 the repair damage in total was about €280.000 (Zwanenveld, 2012). So that makes a total (average) damage according to scenario 1 of:

$$D_{SLS} = 3 \cdot 280.000 + 24.000 = \text{€}864.000$$

The annual probability of occurrence is in this case not exactly 1, because there is always the change of no occurrence. Therefore, the method of Poisson has been used to calculate the annual probability of occurrence:

$$P_{SLs} = 1 - e^{-3} = 0,9502^{23}$$

The damage of the second scenario will be indicated in a different way, because such an incident has never occurred since the Afsluitdijk has been constructed. For this indication a study from CPB has been used as example of damage due to breaching of the dike. The damage due to breaching of the dike has been estimated on (Grevers & Zwaneveld, 2011):

$$D_{ULs} = \text{€}11.000 \text{ million}$$

The probability of occurrence will be estimated on a water level of exactly the minimum dike height of 5,20m + NAP at Den Oever:

$$P_{ULs} = 1 - \exp(-\exp(-2,85(5,20 - 2,52))) = 0,00047$$

The probability of exceedance of 1/2.100 is of course way below the standard from the Dutch water law (Waterwet) of 1/10.000 per year which can be translated to a minimum dike height of 5,75m + NAP.

The annual monetary risk can be calculated by:

$$\begin{aligned} R_{dike} &= D_{SLs} \cdot P_{SLs} + D_{ULs} \cdot P_{ULs} \\ &= 864.000 \cdot 0,9502 + 11.000.000.000 \cdot 0,00047 \cong \text{€} 6.087.000 \end{aligned}$$

²³ This probability of failure can be described by a one-dimensional Poisson process. The difference in occurrence can be given by the formula: $Q_1 = X_i - X_{i-1}$. Here we define $Q_1 = X_1$, the amount of annual occurrence (here: 3 times per year). To determine the probability distribution of Q_1 , we observe that the event $\{Q_1 > q\}$ that the annual occurrence more than 3 times per year is the same as the event $\{N_q = 0\}$ that no occurrence will be measured in $[0, t]$. But this implies that $P(Q_1 \leq q) = 1 - P(Q_1 > q) = 1 - P(N_q = 0) = 1 - e^{-\lambda t}$. Therefore Q_1 has an exponential distribution with parameter λ .

8.5.6 Recapitulate annual monetary risks per function

On beforehand one expects that the dike is the most important element and it may be obvious that the largest risk can be pointed to the primary function of the Afsluitdijk: retaining water. This has been summarized in the graph below, see Figure 8-13.

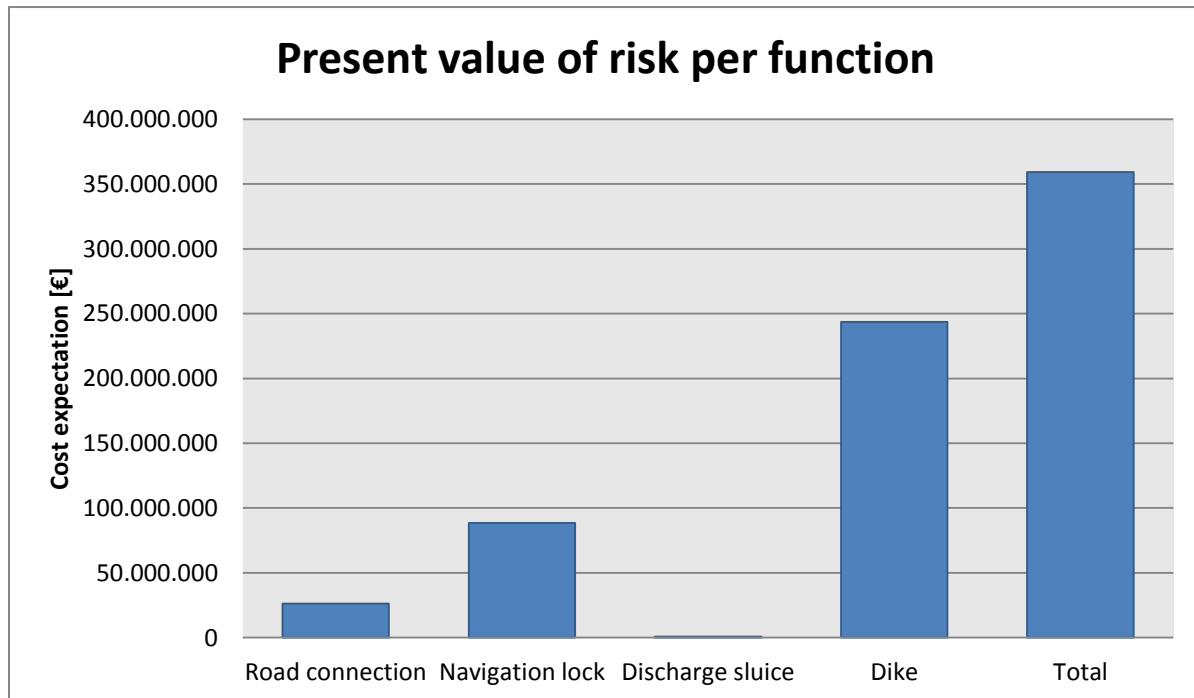


Figure 8-13: Schematically view of the contribution of the functions to the total monetary risks (expected damage) within the system.

The risk which has been determined above, gives the annual value per function. When this damage will be calculated over a certain time line, one is able to assess and compare this outcome with an increase of the crest of the dike. The present value (PV) of the damage due to flooding has been calculated for a situation in which this risk annually occurs and will be discounted to the current situation. This can be calculated with the following formula:

$$PV = C \cdot \left(1 - \frac{1}{(1 + r)^n}\right) \cdot \frac{1}{r}$$

With:

PV = present value [€]

C = annual damage [€]

r = annual discount rate [-]

n = time line (lifespan of the system) [-]

The annual discount rate of 2,5% is applicable whereby the inflation has not been taken account for. In the table below the present value for an r = 2,5% and a lifespan of infinity (n = ∞) has been calculated.

Table 8-3: Overview of the division of the functional risk.

Function	Annual monetary risk [€]	Economical risk with $n=\infty$ [€]	Part of total [%]
Road connection	657.000	26.280.000	7,32
Navigation lock	2.214.300	88.572.000	24,66
Discharge sluice	20.000	800.000	0,00
Dike	6.087.000	243.480.000	67,80
Total	8.978.300	359.132.000	100

Classification of uncertainties

The determined numbers of the damages has been based on several sources, like papers, articles, internet website, and etcetera. This obviously leads to a certain uncertainty in these numbers, because the damages have been determined on reference numbers which are not exactly applicable for this situation.

Although the costs items in a normal budget estimate of a project become increasingly clear in the course of time, and the estimate becomes more accurate, many causes of uncertainty will remain as long as the project is not finished. With the necessary changes made, this can be applied to time-planning. The degree of uncertainty can be classified as follows:

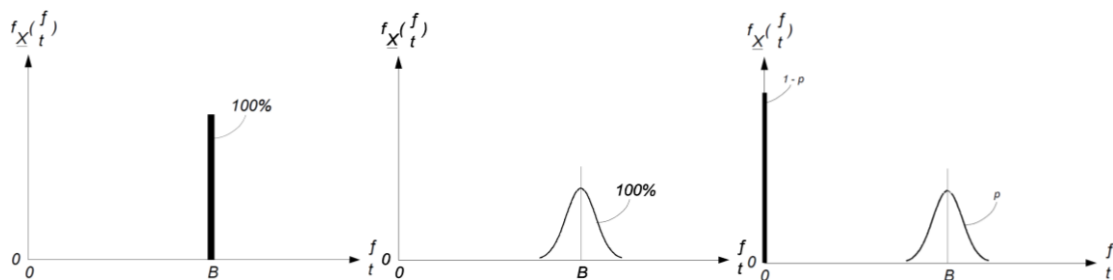


Figure 8-14: Uncertainty related to events: no uncertainty (left), consequence uncertainty (middle), and probability and consequence uncertainty (right) (Vrijling & Van Gelder, 2006).

- 1) There is a no cause of uncertainty. The item concerned is deterministic. This concerns costs items that are known exactly in size. If, for example, the contract settling the purchase of land that has been signed, this amount of money is known exactly. The probability density can then like in the figure above (left).
- 2) Often costs are not so uniquely determined and one is uncertain about the duration or an activity. By using the example of (1), when the negotiations are still in progress, there is a notion about how much money the land will cost, but one cannot be certain. The probability density can then like in the figure above (right).
- 3) Often another type of uncertainty plays a role with the evaluation of costs of a project, namely uncertainty caused by the unforeseen or by special events (mainly calamities). Two criteria characterize a special event: 1. it is not meant to occur and 2. the occurrence is not likely. The probability of occurrence, p , is small (less than 0.5), but if the event occurs, consequence (damage) is large. The probability of no occurrence (and accordingly: no damage) is $1-p$. In a 'classical' estimate of the budget such events are seldom taken into account. Contractors insure against such events,

associated with small probabilities but with large consequences. In a statistically estimate, the probabilities and consequences are a subject to uncertainty.

Table 8-4: The statistical properties to the 3 cases of degree of uncertainty.

Case	Mean	Standard deviation	Description
(1)	B	0	A deterministic amount of money, B , expressed in units of money.
(2)	B	σ_B	A stochastic amount of money, with mean B , and some spreading, σ_B .
(3)	$p \times B$	$\sqrt{p \cdot \{(1 - p) \cdot B^2 + \sigma_B^2\}}$	An event with probability of occurrence p that has a statistic consequence with mean B and some spreading, expressed by σ_B .

The spreading for an item of the estimate increases with the related uncertainty. In case (1) one is absolutely certain about the size of the sum, B . The standard deviation equals zero. In the second case there is some uncertainty. The spreading, σ_B , is smaller than the expected value, B , because otherwise, the estimate of the item was of no significance. It then suggested that there is not the vaguest idea of the size of B . In case 3, one is not certain if there will be costs (damage) at all. The probability that there will be costs is p ($p < 1$). There is a greater probability $(1-p)$ that there will be no costs. In fact the monetary risk²⁴ is estimated. But if such a moment occurs, the estimated amount of money ($p \times B$) is not nearly enough to cover the costs (B).

It may be obvious that the degree of uncertainty of (1) is not applicable here and degree of uncertainty of (2) will be used. However, some numbers are more precise than others, so the degree of uncertainty of (2) will be divided in a small, normal and high uncertainty (see Figure 8-15). In Table 8-5 an overview has been given of relatively and subjective classification of the damage numbers.

Table 8-5: Classification of uncertainty of the damage numbers.

Damage	Degree of uncertainty	Uncertainty	Colour in Figure 8-15
$D_{\text{road connection}} \sim C_1$	(2)	Small	Green
$D_{\text{navigation lock}} \sim C_2$	(2)	High	Red
$D_{\text{discharge sluice}} \sim C_3$	(3)	High	Red
D_{dike}	(3)	Normal	Blue

As one can see is that uncertainties in de largest parts of the risk analysis (navigation lock and dike) have respectively a high and normal uncertainty distribution. This means that the chosen numbers should be more thoroughly determined, because this forms more than 90% of the total number.

²⁴ From a mathematical point of view the estimates of case (1) and (2) are estimates of the risks as well. The probability of occurrence is these cases are 1 (or 100% certainty). The definition of risk here is: probability \times consequence = $p \times B$ (Vrijling & Vrouwenvelder, 1984).

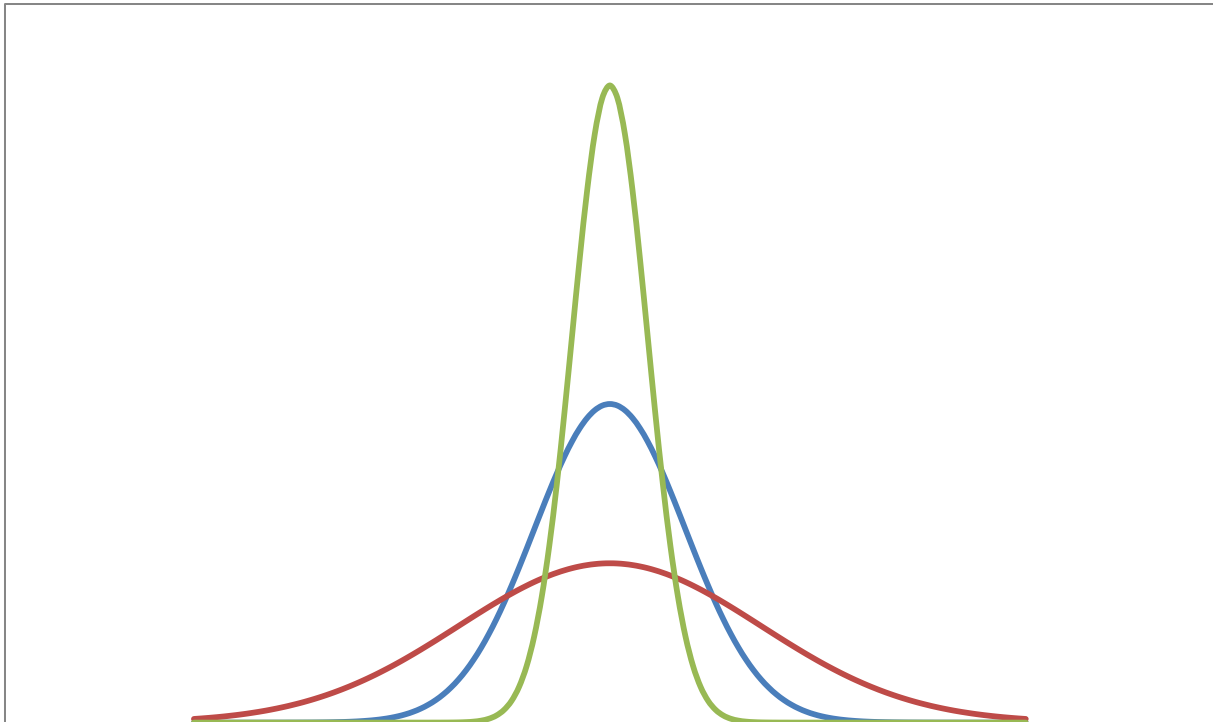


Figure 8-15: Uncertainty contribution shapes: high uncertainty (red), average uncertainty (blue) and small uncertainty (green).

8.6 Investment in risk reduction

The risks determined in the previous paragraph, should be reduced in the most efficient and economical way. In other words, some measurements should be taken to reduce the probability of occurrence of a certain failure. These investments in the system can be modeled on several parameters, like an extra road lane, higher crest of the dike, and etcetera. On every investment measurement a cost-benefit analysis will be made, to receive more insight in the influence of the probability of occurrence on the most efficient measure.

For every function just one measure has been analyzed to prevent (or just to decrease probability of occurrence) the system from failing. The following measures will be analyzed:

- Road connection: extra road lane;
- Navigation lock: extra navigation lock;
- Discharge sluice: extra discharge sluice;
- Dike: higher dike crest.

All these measurements actions have reducing influence on the probability of occurrence and not on the damage numbers. In the coming paragraph the measurements and their effects will be further elaborated in an overview of costs and benefits.

8.6.1 Road connection

The reduction of the annual probability of occurrence for the road connection can be done in several ways, like an extra road lane, a tunnel, a new dike with a road on it, and etcetera. It may be obvious that not all these possible solution are feasible and therefore the most efficient solution is the construction of an extra road lane on the (already existing) Afsluitdijk. Hereby the congestion length will be decreased what lead to a decrease of the annual cost expectation of the system. By constructing an extra road lane the damage costs will decrease with a certain factor. The present value (PV) for an infinite lifespan is calculated on €26.280.000 (see table Table 8-3), which is equal to the maximum benefits.

Before a project starts an estimation of the costs need to be made and therefore RWS uses 'a rule of thumb' for the construction of a road. This rule of thumb indicate the construction of one road lane on €3,5 million per kilometer including soil preparation, streetlights, taxes, and etcetera. An extra bridge element for passing the navigation route will be estimated on about €20 million. These numbers can be seen as the average value for the construction of a road lane (Average costs for construction of a road lane, 2012).

Beside the construction of the road also the dike must be extended to implement the road in the design. A very rough calculation estimates that 35 m³ ground per meter dike is needed. The average price per m³ ground is about €7,50 (Average costs for construction ground, 2012).

Also the revetment on the inner side of the dike must be replaced which leads to even more costs. Because of the lack of reference numbers for this activity a percentage of the total costs has been determined on 20%.

The total cost of constructing an extra road lane on the Afsluitdijk can be formulated as:

$$I(n) = C_{road\ lane} \cdot L \cdot n + 2 \cdot C_{bridge}$$

With:

$I(n)$ = total estimated cost of constructing an extra road lane [€]

$C_{road\ lane}$ = cost of constructing a road lane per kilometer [€/km]

L = length of the Afsluitdijk (which is 32 kilometers) [km]

n = amount of extra road lanes [-]

C_{bridge} = cost of constructing a bridge [€]

The $C_{road\ lane}$ -value (cost of constructing a road lane per kilometer) can be calculated by using the road costs, ground costs and revetment costs:

$$\begin{aligned} C_{road\ lane} &= C_{road} + C_{ground} + C_{revetment} \\ &= (\text{€}3,75 \text{ million} + 35m^3 \cdot \text{€}7,50/m^3/m^1 \cdot 1.000m) \cdot 1,20 = \text{€}4.815.000 \end{aligned}$$

By filling in the numbers:

$$\begin{aligned} I(n) &= \text{€}4,815 \text{ million/km} \cdot 32\text{km} \cdot n + 2 \text{ bridge} \cdot \text{€}20 \text{ million/bridge} \\ &= \text{€}154 \text{ million} \cdot n + \text{€}40 \text{ million} \end{aligned}$$

Table 8-6: Total costs per road lane.

Extra road lanes: n [-]	Total costs: $C_{extra\ road\ lane}$ [€]
0	0
1	194.080.000
2	348.160.000
3	502.240.000
4	656.320.000
5	810.400.000
6	964.480.000

These numbers do not involve maintenance costs over the years, but this is significant less than the total amount of construction costs so that will not influence the total costs.

In the previous paragraph (0) the current situation gave a present value of €26.280.000. To calculate the present value in the new possible situations a linear relation will implemented, or in other words the damage cost will decrease proportional. This has been given by the following formula:

$$R(n) = C_{total\ damage} \cdot \frac{2}{n + 2}$$

With:

$R(n)$ = present value of the total economic cost due to congestion/unavailability of the road [€]

$C_{total\ damage}$ = present value of the total damage due to congestion/unavailability of the road in the current situation [€]

n = amount of extra road lanes [-]

By filling in the numbers:

$$R(n) = \text{€}26,3 \text{ million} \cdot \frac{2}{n+2}$$

8.6.2 Navigation lock

The reduction of the annual probability of occurrence and the damage cost for the navigation lock can be by constructing an extra navigation lock aside the current lock. Hereby the congestion time will be decreased and the annual probability of failure of this function will also decrease what lead together to a decrease of the annual monetary risk of the system. By constructing an extra navigation lock the damage costs will decrease with a certain factor. The present value for an infinite lifespan is calculated on €88.572.000 (see table Table 8-3), which is equal to the maximum benefits.

To construct an extra navigation lock will cost a lot of money. Because of the fact that the current navigation lock cannot be used as a reference to this situation, a similar lock must be used. When the costs of a single lock complex are being presented it shows high costs for the construction of a certain navigation lock. The most simple lock construction is about €200 million and this number is even without dredging and all rest costs. The construction of a new lock is obviously way more than the present value in the current situation and can therefore be seen as not feasible (Bonnes, 2011).

As a recommendation to this analysis to take a look at the highest failure mechanisms (lock gate failure, soil protection, and etcetera). To analyze the highest annual probability of failure per element can form a basis for specific maintenance activities to decrease the annual probability of failure.

8.6.3 Discharge sluice

The reduction of the annual probability of occurrence and the damage cost for the discharge sluice can be done by constructing an extra drainage gate aside the current complex. Hereby the annual probability of failure will be decreased what lead to a decrease of the annual monetary risk of the system. The present value for an infinite lifespan is calculated on €800.000 (see table Table 8-3), which is equal to the maximum benefits.

To construct an extra discharge sluice will cost a lot of money. Because of the fact that the current discharge sluice cannot be used as a reference to this situation, a similar sluice should be used. The most simple discharge sluice construction is about €250 million. The construction of a new sluice is obviously way more than the present value in the current situation and can therefore be seen as not feasible (Hoogenboom, Gründemann, Muntinga, Laane, 2005)).

8.6.4 Dike

The reduction of the annual probability of occurrence can be done by increasing the height of the crest of the dike. Hereby the safety level of the Afsluitdijk will be increased what lead

to a decrease of the annual monetary risk of the system. The Dutch water law (Waterwet) gives a minimum safety level of 1/10.000 per year what corresponds with a crest height of 5,75 meters. By increasing the crest height the annual probability of occurrence will decrease with a certain factor. The present value (PV) for an infinite lifespan is calculated on more than €243.480.000 (see table Table 8-3), which is equal to the maximum benefits (the dike breach will never occur).

To increase the crest height of the dike will cost a lot of money. In earlier research of increasing the crest of a dike one used the following costs:

- €1,5 million for a crest increasing of 1,0 meter.

This number had been determined by the total amount of sand what is needed to increase the dike. However, beside the costs of the sand also the revetment will lead to a certain amount of costs.

The cost for 1 meter increasing the height of the Afsluitdijk over its full length will therefore be estimated on €2,0 million (+25%):

$$I(\Delta h_{crest}) = C_{crest} \cdot \Delta h_{crest}$$

With:

$I(\Delta h_{crest})$ = total estimated cost of increasing the crest of the dike with 1 meter [€]

C_{crest} = cost of increasing the crest height per 1 meter [€/m]

Δh_{crest} = net increased height of the crest of the dike (new height – previous height) [m]

By filling in the numbers:

$$I(\Delta h_{crest}) = €2 \text{ million} \cdot \Delta h_{crest}$$

From the perspective of the VNK (2005) it is interesting to increase only the weak spots or in other words what is the dominant failure mechanism. According to the VNK (2005) the following numbers has been presented:

Table 8-7: Contribution of annual probability of flooding of the Afsluitdijk per failure mechanism (source: VNK, 2005).

Type defense system	Failure mechanism	P_f [-]
Dike	1. Overflow and overtopping	1/2.100
	2. Cracking and piping	$<10^{-6}$
	3. Damaging revetment	1/15.000
	4. Instability inner slope	$<10^{-6}$
	TOTAL: 1/1.840 ($5,429 \cdot 10^{-4}$)	

The elementary limits of the probability of collapse of the Afsluitdijk are:

$$\text{MAX}_{i=1}^4 \{P_i\} \leq P_{dike} \leq \sum_{i=1}^N P_i$$

Inserting numerical values from the table above:

$$4,762 \cdot 10^{-4} \leq P_{dike} \leq 5,429 \cdot 10^{-4}$$

It may be obvious that the weakest spot here is the overflow and overtopping. This means that increasing the crest height is the most efficient way to increase the safety level.

The water level analysis from paragraph 5.3 gives a safety level of 1/2.100 for the current situation. From this graph a formula can be derived which depends on the height of (Bakker, 2009). The formula can be given by:

$$R(P_f) = C_{total\ damage} \cdot P_f$$

$$P_f(\Delta h_{crest}) = 1 - \exp(-\exp(-2,85(\Delta h_{crest} - 2,52)))$$

With:

$R(P_f)$ = total annual economic cost due to breaching of the dike [€]

$C_{total\ damage}$ = total damage due to breaching of the dike [€]

P_f = annual probability of flooding dependent on the extra crest height of the dike [-]

Δh_{crest} = the net increased height of the crest of the dike (new height – previous height) [m]

By filling in the numbers:

$$R(\Delta h_{crest}) = \text{€}11.000 \text{ million} \cdot (1 - e^{-e^{-2,85(\Delta h_{crest} - 2,52)}})$$

8.7 Economical optimization

In a research project conducted at Delft University of Technology, risk-based optimization is adopted as main principle on which to base an analysis of acceptable flooding risk. This type of optimization has been applied successfully in several earlier studies in mechanical and coastal engineering. The basic form of risk-based optimization is economic optimization that is aimed at minimization of lifetime cost of the flood defense system:

$$C_{life}(\mathbf{p}, \mathbf{x}) = I(\mathbf{p}) + R(\mathbf{p}, \mathbf{x})$$

With:

\mathbf{p} = Vector of design variables

\mathbf{x} = Vector of random variables

$I(\mathbf{p})$ = Investment in the system

$R(\mathbf{p}, \mathbf{x})$ = Monetary risk

The monetary risk in its basic form is given as the expected value of monetary damage in case of flooding:

$$R(\mathbf{p}, \mathbf{x}) = P_{flood}(\mathbf{p}, \mathbf{x}) \cdot S$$

With:

P_{flood} = The flooding probability of the system

S = The monetary value of all inventory of the system

The probability of flooding of an area may be evaluated by using system reliability theory, calculating reliability of all components first, followed by an evaluation of the system probability of failure. Extending reliability evaluation with optimization can be done in two ways:

- 1) Top-down approach;
- 2) Bottom-up approach.

In the top-down approach, lifetime costs of the system are defined in the space of the design variables of the components. Well-known optimization methods may then be used to find the minimum of lifetime costs. In practice, this approach leads to a high-dimensional optimization problem; number of design variables in order of 100.

Since the tool is ultimately aimed at supporting decision-making for the definition of acceptable flooding risk levels, it is important that the tool is transparent for decision-makers and design engineers. It is the opinion of the authors that the top-down approach is too much a black box approach.

An alternative is the bottom-up approach. In this approach lifetime costs of the system are defined in the space of failure probabilities of individual components of the system; or given in the formula of:

$$C_{life}(\mathbf{P}_f) = I(\mathbf{P}_f) + R(\mathbf{P}_f) = I(\mathbf{P}_f) + P_{flood}(\mathbf{P}_f, \rho) \cdot S$$

With:

\mathbf{P}_f = Vector of component failure probabilities

ρ = Correlation matrix, providing the correlation between components

In this case, the number of dimensions of the optimization problem is reduced of the number of individual components in the system. Generally, this means a reduction of the number of dimensions in system optimization by a factor 10.

To obtain the optimization in this form, the functions of the formula above have to be found. System reliability theory provides flooding probability as a function of failure probabilities of components and the correlation matrix. The investment function and the correlation matrix have to be found by a closer analysis of every component.

The result of the optimization should be independent of the strategy used. It can be proven that the minimum of the lifetime costs can be determined, if and only if the investment function is defined as the minimum investment in the component for a given failure probability. Therefore, the investment function can be found by minimization of the investments in the section for a number of prescribed failure probabilities (Voortman & Vrijling, 2001).

This theory above has been applied in the previous two chapters (paragraph 0 and 0). The bottom-up optimization is applied to find the economically optimal system failure probabilities. This also can be illustrated in a fault tree schematic with the top event of 'system failure' (see Figure 8-16).

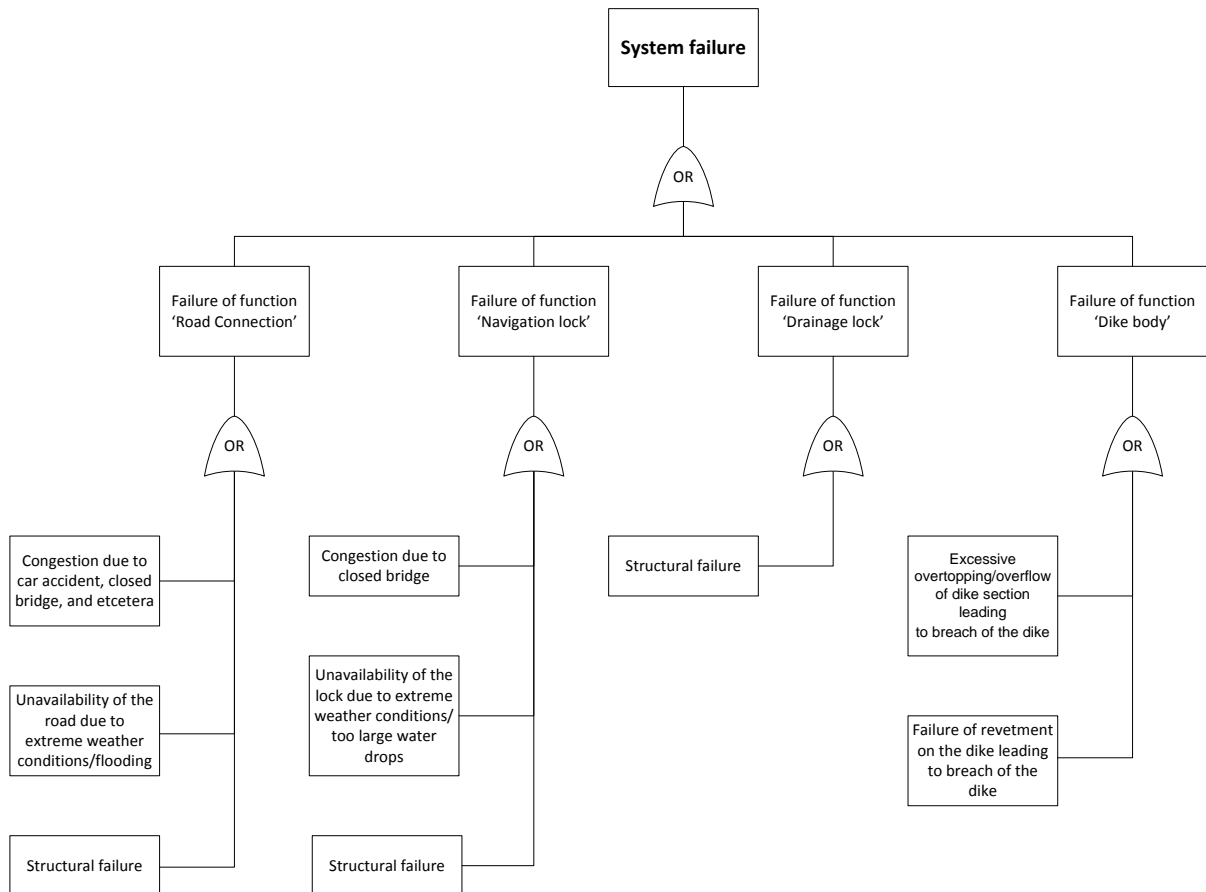


Figure 8-16: Fault tree schematic with the top event of 'system failure'.

8.7.1 Road connection

The optimization of the function 'traffic connection' has been based on the current monetary risks and the investment by constructing extra road lanes. These formulas have been determined in respectively paragraph 8.5.2 and 8.6.1:

$$\text{Monetary risk (Red): } R(n) = C_1 \cdot N \cdot T \cdot \frac{2}{n+2}$$

$$R(n) = C_1 \cdot N \cdot T \cdot \frac{2}{n+2} = \text{€ } 26,3 \text{ million} \cdot \frac{2}{n+2}$$

$$\text{Investment (Blue): } I(n)$$

$$I(n) = \text{€ } 154 \text{ million} \cdot n + \text{€ } 40 \text{ million}$$

$$\text{Lifetime costs (Green): } C_{life}(n) = I(n) + R(n) = I(n) + C_1 \cdot N \cdot T \cdot \frac{2}{n+2}$$

$$C_{life}(n) = (\text{€ } 154 \text{ million} \cdot n + \text{€ } 40 \text{ million}) + \text{€ } 26,3 \text{ million} \cdot \frac{2}{n+2}$$

The optimization will result in the economically best extra road lanes: n [-].

Table 8-8: Optimization of parameter extra road lanes on the Afsluitdijk.

n [-]	Investment [€]	Present value of monetary risk [€]	Lifetime costs [€]
0	0	26.280.000	26.280.000
1	194.000.000	17.520.000	211.520.000
2	348.000.000	13.140.000	361.140.000
3	502.000.000	10.512.000	512.512.000
4	656.000.000	8.760.000	664.760.000
5	810.000.000	7.508.571	817.508.571
6	964.000.000	6.570.000	1.170.570.000

The Table 8-9 above has been illustrated in Figure 8-18 below.

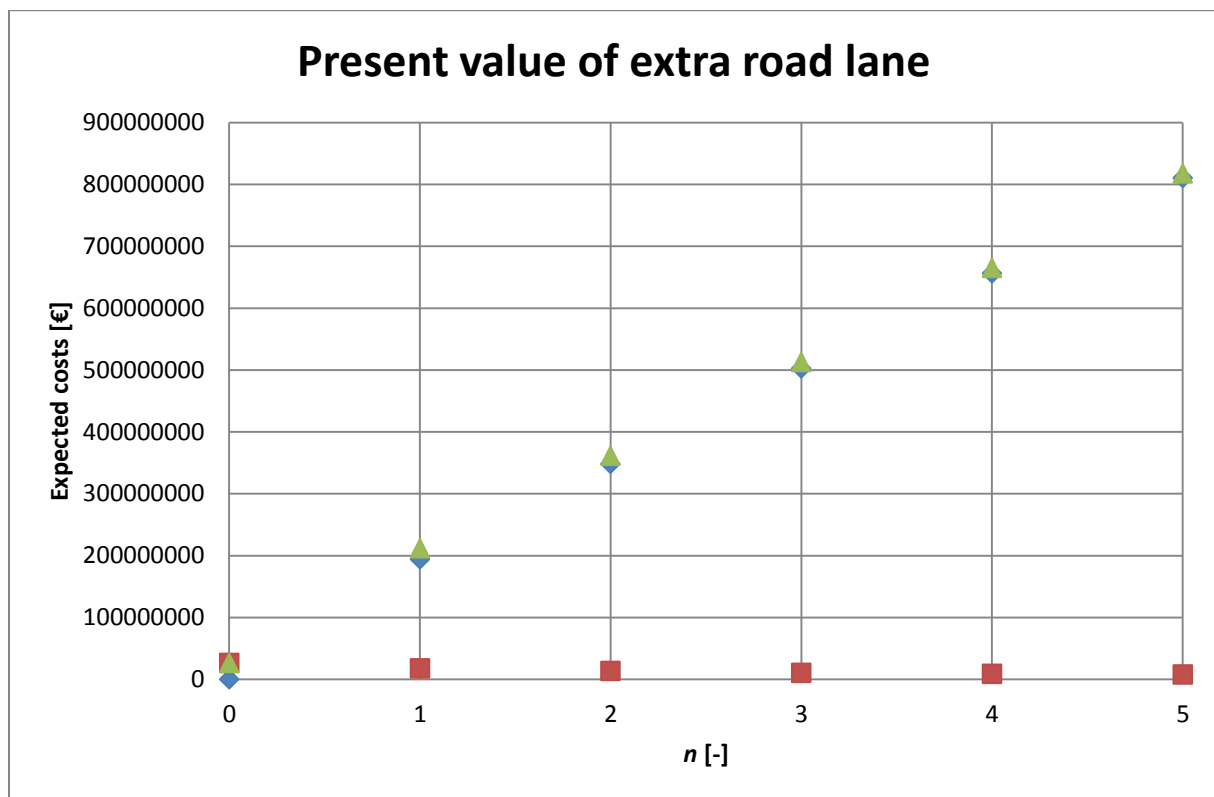


Figure 8-17: Optimization of the lifetime costs for the road connection.

The most economical point is zero extra road lanes on the dike. The total road lanes therefore stay on 2 x 1 road lanes.

8.7.2 Navigation lock and discharge sluice

The optimization of the navigation lock leads to the action 'do nothing'. The current situation with its risks is economically better than taking any measurements (see paragraph 8.6.2 and 8.6.3). However, it is recommended to do a research on the lock components itself. Specific analyses on components may result in elements that form the weakest spot what eventually result in certain maintenance activities to these elements.

8.7.3 Dike

The optimization of the function 'retaining water' has been based on the current monetary risks and the investment by increasing the dike height. These formulas have been determined in respectively paragraph 8.5.5 and 8.6.4:

$$\text{Monetary risk (Red): } R(\Delta h_{crest}) = P_{flood}(\Delta h_{crest}) \cdot S$$

$$R(\Delta h_{crest}) = \text{€}11.000 \text{ million} \cdot \left(1 - e^{-e^{-2,85(\Delta h_{crest}-2,52)}}\right)$$

$$\text{Investment (Blue): } I(\Delta h_{crest})$$

$$I(\Delta h_{crest}) = \text{€}2 \text{ million} \cdot \Delta h_{crest}$$

$$\text{Lifetime costs (Green): } C_{life}(\Delta h_{crest}) = I(\Delta h_{crest}) + R(\Delta h_{crest})$$

$$= I(\Delta h_{crest}) + P_{flood}(\Delta h_{crest}, \rho) \cdot S$$

$$C_{life}(\Delta h_{crest}) = \text{€}2 \text{ million} \cdot \Delta h_{crest} + \text{€}11.000 \text{ million} \cdot \left(1 - e^{-e^{-2,85(\Delta h_{crest}-2,52)}}\right)$$

The optimization will result in the economically best extra height of the crest: Δh_{crest} [m]. The minimum of lifetime cost is found by presuming the derivative with respect to the parameter of dike increasing equal to zero:

$$\frac{\partial C_{life}}{\partial \Delta h_{crest}} = 0$$

Table 8-9: Optimization of parameter increasing crest height of the Afsluitdijk.

Δh_{crest} [m]	Investment [€]	Annual monetary risk [€]	Lifetime costs [€]
0,65	975.000	831.198	1.806.198
0,70	1.050.000	720.808	1.770.808
0,75	1.125.000	625.079	1.750.079
0,80	1.200.000	542.062	1.742.062
0,81	1.215.000	526.832	1.741.832
0,85	1.275.000	470.071	1.745.071
0,90	1.350.000	407.641	1.757.641
0,95	1.425.000	353.502	1.778.502

Δh_{crest} [m]	Investment [€]	Annual monetary risk [€]	Lifetime costs [€]
1,00	1.500.000	306.553	1.806.553
1,05	1.575.000	265.840	1.840.840

The Table 8-9 above has been illustrated in Figure 8-18 below.

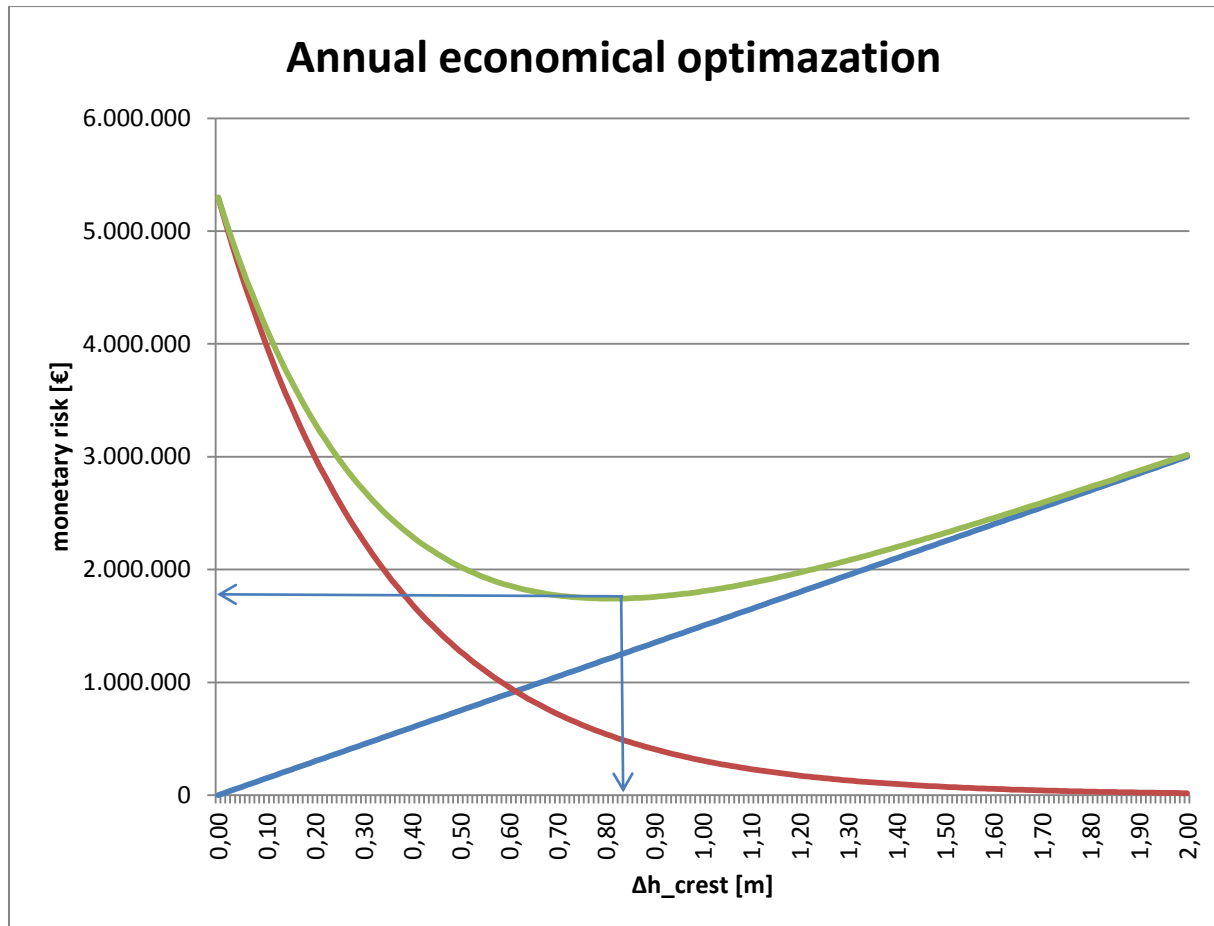


Figure 8-18: Optimization of the lifetime costs for the dike.

It must be established whether the sum of the costs at the optimal height is lower than the expected level of damage in the old situation (that is the existing state before raising the dike). Only if this last condition:

$$\text{gain safety} > \text{costs dike improvement}$$

is met, the dike improvement is profitable. Here, it is obviously true in this example. The initial annual monetary risk is €5,3 million and the annual monetary risk after taking dike improvements is almost €1,74 million, so this is way below the initial costs and is therefore also profitable.

The most economical optimal point is 0,81m of increasing the dike height. The total height of the dike will then be 6,01m which corresponds with a safety level of:

$$P_f = 1 - \exp(-\exp(-2,85(6,01 - 2,52))) = 0,0000479 \cong 1/20.900$$

The safety level will then be 1/20.900 per year what is more than the required safety level of 1/10.000 per year. Therefore it may be clear the most beneficial point also fulfills to requirements of the Dutch water law.

8.7.4 Recapitulate economical optimization

An overview will be represented of the economical most beneficial situation based on present value of the costs (/annual monetary risk) and expected present value of the costs after investment what lead to decrease the probability of occurrence, see Table 8-1.

Table 8-10: Investments that lead to lower expected costs given by the present value.

Function	Investment	Expected cost before investment [€]	Expected cost after investment [€]	Part of total [%]
Road connection	-	26.280.000	26.280.000	19,1
Navigation lock	-	88.572.000	88.572.000	64,2
Discharge sluice	-	800.000	800.000	0,6
Dike	Heightening crest level	243.480.000	22.288.280	16,3
Total		359.132.000	137.940.280	100

The current situation makes it clear that the dike had the highest monetary risk and was dominant in the analysis. After the investment analysis has been made, the choice of executing this investment based on the economical most beneficial value changed this dominance a bit. If an investment has been made, the expected costs should be decreased. The new situation describes a new overview of the monetary risk, see Figure 8-13.

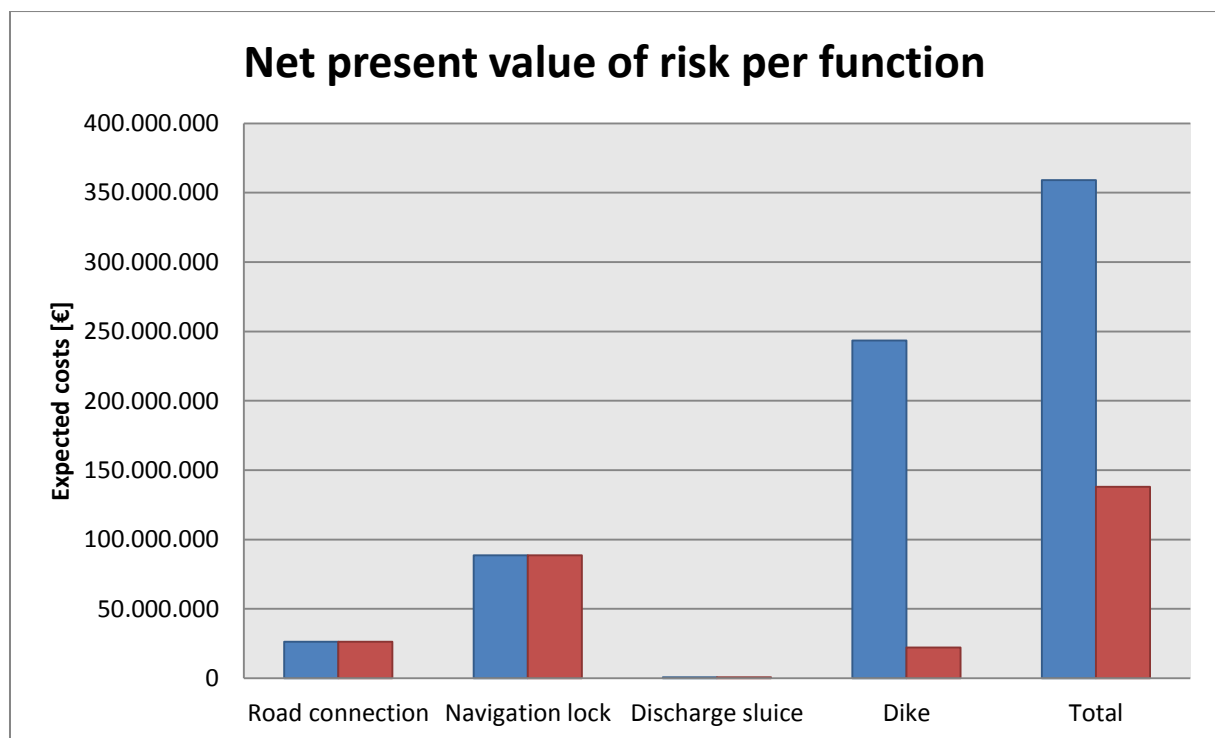


Figure 8-19: Schematically view of the contribution of the functions to the total monetary risks within the system.

8.8 Maintenance optimization

8.8.1 Road connection

The deterioration model that will be used for the road connection will focus on the condition of the asphalt of the road. In order to be able to make predictions about the type and amount of maintenance that is needed in the future, predictions on the future condition of the asphalt pavement should be made. The models that are required to make such predictions are called performance models.

Because a large number of factors that influence pavement performance, such as climate and traffic, are very difficult to estimate and to predict and because large variation occur over the length of the road in the quality of the materials used and the applied layer thicknesses, accurate performance predictions are difficult to make. Each road segment has in fact its own performance model. In spite of this, research carried out that it is possible to derive general applicable models using data obtained from individual road sections. Although, models were derived for the most important damage types, one should realize that it will never be possible to model all the damages one observes in practice.

Furthermore, one should realize that in many cases damage types do not occur independently from each other. For example, the interaction between damage types is the interaction between rutting and cracking that occurs on asphalt pavements with a thin later asphalt layer and an unbound base. Due to the traffic loads the asphalt layer will crack. Because of the cracking, the asphalt layer loses some of its original bending stiffness which results in higher stress levels in the unbound base which could cause permanent deformations in the base. The situation becomes even worse when water is moving through the cracks which certainly cause in a loss of strength of the base due to a loss of cohesion and a decrease of the angle of internal friction. The higher stresses can then not be taken by the base material and permanent deformation or even shear failure might occur. In both cases the deformation of the base will show at the pavement surface as rutting.

There are many mechanisms that explain initiation and progression of some damage types, like:

- 1) Fatigue cracking in asphalt pavements due to traffic;
- 2) Permanent deformation or rutting in asphalt layers;
- 3) Permanent deformation in granular bases, sub bases, sands and soils;
- 4) Transverse cracking in asphalt pavements with cement treated base.

Because of the large diversity of these mechanisms, here has been chosen to elaborate only the first damage type: fatigue cracking in asphalt pavements due to traffic. This because of the fact that it is the damage type with the largest impact and it depends on the amount of traffic that passes.

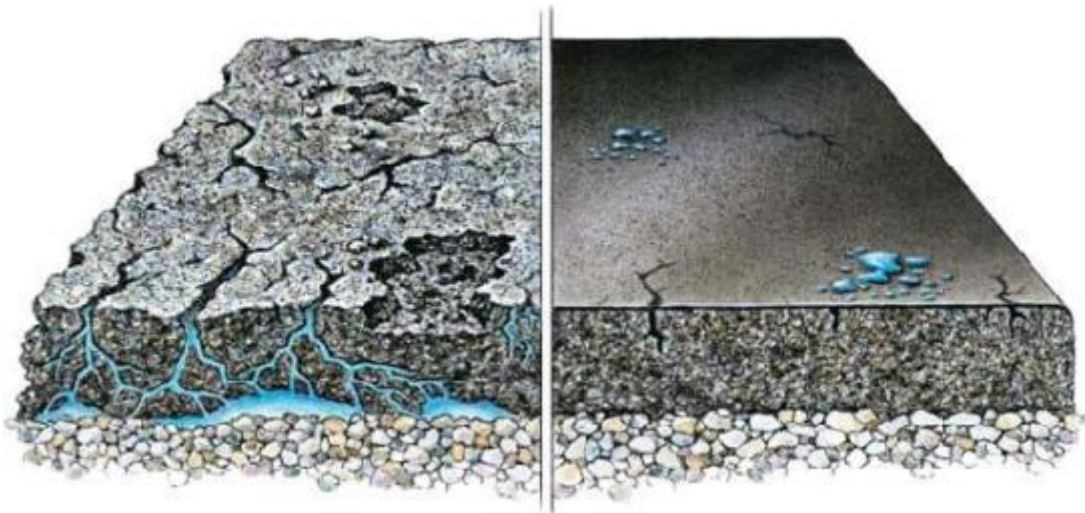


Figure 8-20: Example of cracks in an asphalt layer of a road: asphalt layer on its end of its life cycle (left); and an asphalt layer what starts to crack (right) (source: http://westrocksealing.com/images/asealer/Fixes_test_04.jpg)

Fatigue cracking in asphalt pavements due to traffic

It is a well-known fact that the tensile strain at the bottom of the asphalt layer is an important criterion in pavement design. Fatigue cracking occurs after a large number of strain repetitions which are caused by the traffic loads. Initiation and propagation of fatigue cracks depends on the magnitude of the applied strain and the fatigue resistance and resistance to crack propagation of the material (see Figure 8-21 (left)).

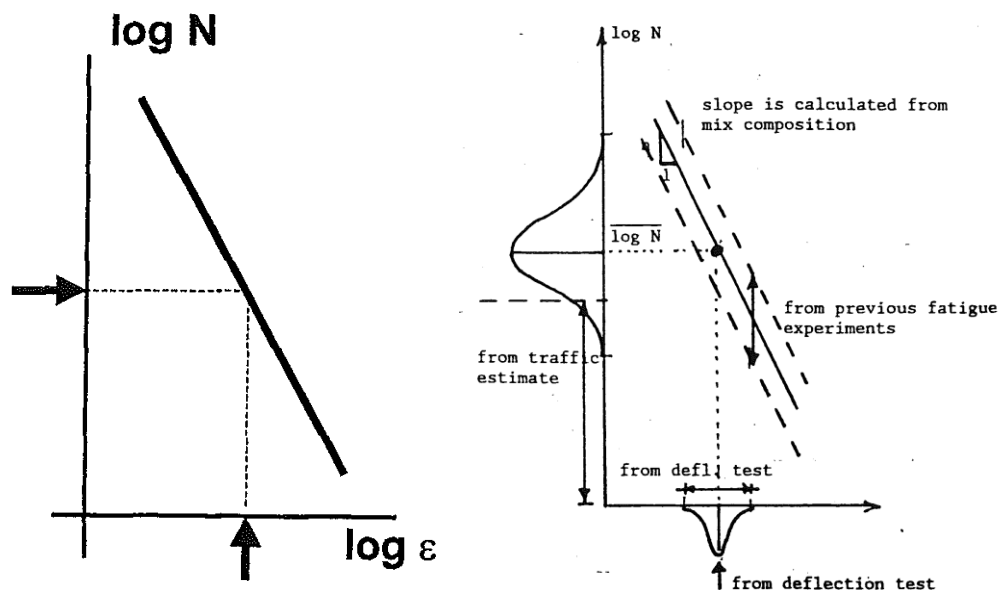


Figure 8-21: Principle of the assessment of fatigue life of an asphalt layer (left) and variation in pavement life as a result of the variation in occurring tensile strain and variation in fatigue resistance (right) (Molenaar, 1999).

On a particular stretch of pavement, the tensile strain at the bottom of the asphalt layer will not be the same at each location, because of variation in the layer thicknesses and layer moduli. Also the fatigue resistance of the asphalt mixture will show a certain amount of variation over the length of the road, because of the variation in mixture composition and degree of compaction. All in all this means that one also will observe a certain amount of

variation in fatigue life over the length of the asphalt pavement (see Figure 8-21 (right)). The function that describes the increase in amount of cracking with time takes a shape of the performance curve of:

$$F(t) = 1 - \frac{S_t}{S_T}$$

In which:

$F(t)$ = condition of the asphalt layer at t [-]

S_t = amount of cracks at t [-]

S_T = maximum achievable amount of cracks [-]

The maximum amount of damage is reached when the chance that the number of load repetitions is reached, equals 1 (or 100%).

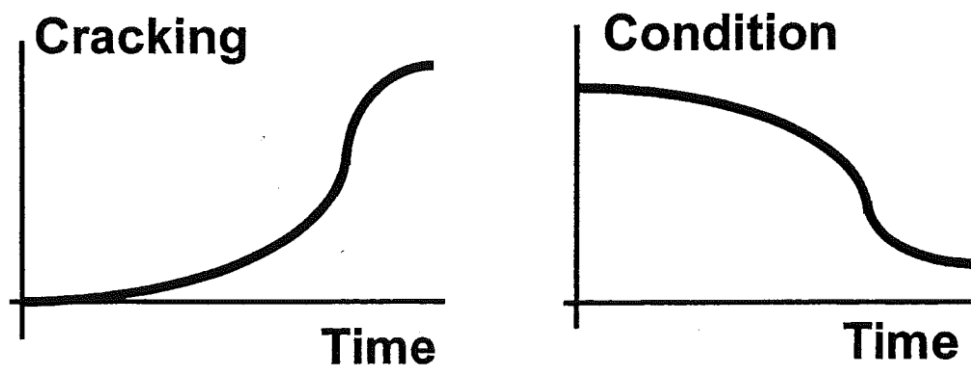


Figure 8-22: Damage development (left) and performance curve (right) (Molenaar, 1999).

In reality the development of cracking is a bit more complicated, because damage is not only initiating at various locations in the way as described above. Cracking is also propagating from locations where it has initiated. This means that the development of the amount of cracking is influenced by initiation and propagation.

The performance curve shows a Weibull distribution is theoretically the best function to describe failure of the asphalt pavement sections if the pavement is modeled as a very large number of small, discrete and independent elements which each fail at a certain moment. The shape of the Weibull distribution is:

$$F_W(t) = 1 - \exp \left[- \left(\frac{t}{\mu} \right)^\beta \right]$$

$$\log(\beta) = -0,341 + 0,295 \cdot \log(h)$$

In which:

$F_W(t)$ = annual probability of failure that an element has failed before year t [-];

t = time [yr]

β = curvature parameter [-]

μ = time parameter [yr]

h = thickness of the asphalt layer [mm]

It is clear that this distribution indeed allows to model the typical S shaped type progression of the damage as described earlier.

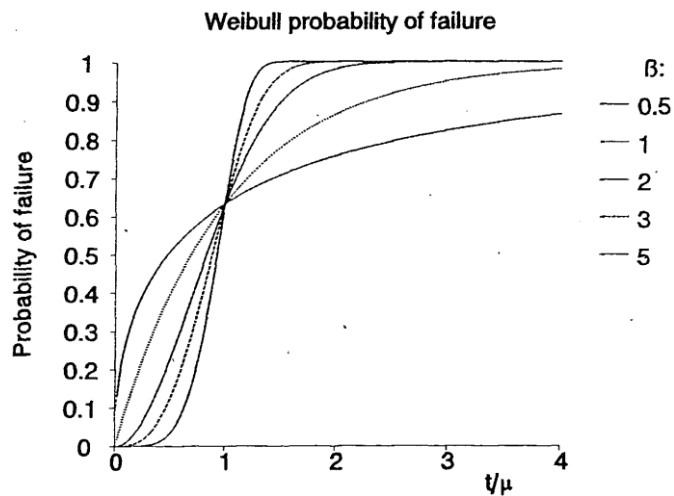


Figure 8-23: Shape of the Weibull distribution function (Molenaar, 1999).

Calculation

The percentage of cracking deals with the amount of wheel track cracking. The standard length of each section to be inspected is 100m. The asphalt pavement on the Afsluitdijk has been inspected and resulted in cracks over a length of 5m. The thickness of the asphalt layer is 200mm and therefore results in a curvature parameter:

$$\beta = 10^{-0,341+0,295 \cdot \log(200)} = 2,18$$

In 2007 the asphalt layer had been renewed and therefore the inspection was done 5 years after the asphalt pavement was constructed. The time parameter can be calculated now:

$$\frac{5 [m]}{100 [m]} = 1 - \exp \left[- \left(\frac{5}{\mu} \right)^{2,18} \right]$$

$$\text{Solve } \mu \rightarrow \mu = 19,53 \text{ yr}$$

The probability of failure of the asphalt road on the Afsluitdijk is:

$$F_W(t) = 1 - \exp \left[- \left(\frac{t}{19,5} \right)^{2,18} \right]$$

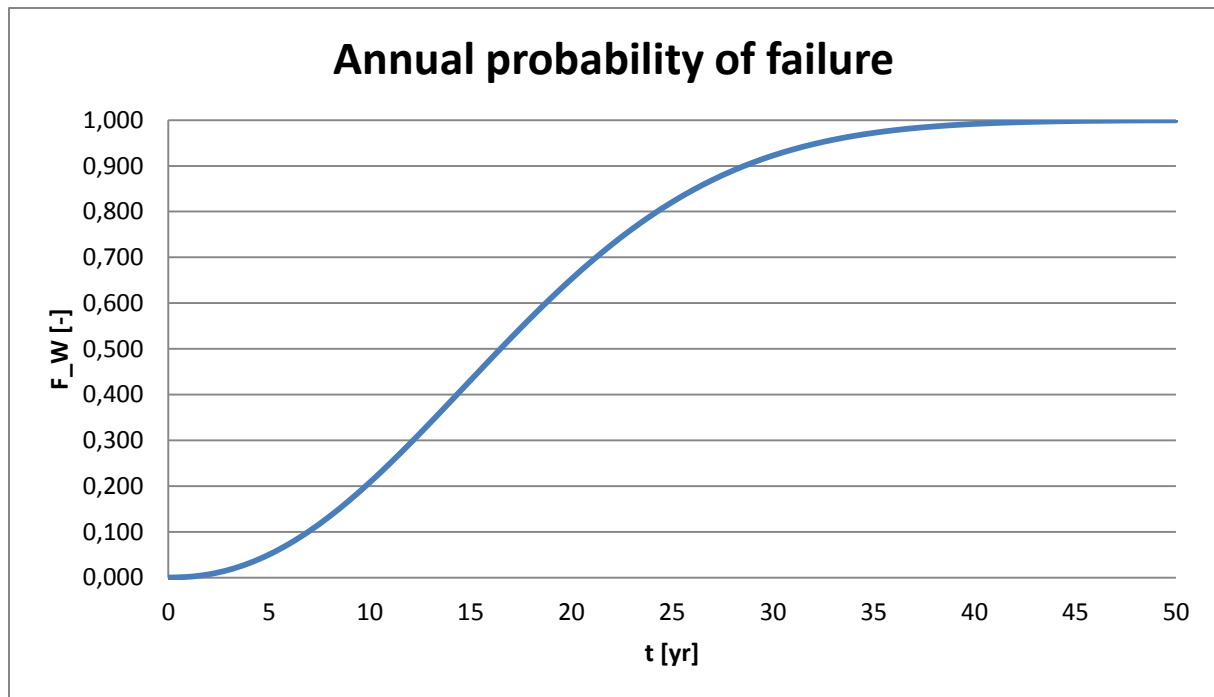


Figure 8-24: The curve of the probability of failure of the asphalt road on the Afsluitdijk

The costs will be divided between planned and unplanned maintenance. Maintenance activities which are planned will lead to congestion costs and the direct maintenance costs, but not to failure of the function and therefore has an availability of 100%. However, the unplanned maintenance due to failure of the function will, beside the direct maintenance (repair) costs, lead to unavailability of a certain amount of time and the corresponding damage. An estimation of the costs has been determined:

- Planned maintenance:
 - Congestion costs: € 460.000
 - Direct maintenance costs: € 8.000.000
- Unplanned maintenance:
 - Logistic and casualties damage: € 2.000.000
 - Repair costs: €16.000.000

The cost numbers have been determined very roughly based on expert judgment and rules of thumb.

The congestion costs are based on the rules of thumb numbers in paragraph 8.5.2 and the assumptions that renewing an asphalt layer (milling and asphalt) will cost 2 days per kilometre and the extra congestion time will be 1 hour. These set of number result in €460.000 congestion costs per maintenance activity (Expert Judgment).

The direct maintenance costs have been determined based on a prefix of €25 per metre square of renewing asphalt, including milling and other activities. The road is 10 metres wide and 32 kilometres long, which then will result in €8 million per maintenance activity. However, when the asphalt failed the costs to repair this will be significantly higher than for planned maintenance. Because not only the asphalt has to be repaired but also the layers

below should be maintained. Therefore, a prefix of €50 per metre square of repairing asphalt has been applied (Costs for asphalt layers, 2012).

The logistic and casualties damage have been determined by the logistic damage of making a detour (2 hours) and the casualties that may occur by direct failing of the road. These costs have been estimated as 4 times higher than the congestion costs in the planned maintenance (Inspection report IV-Infra, 2011).

The costs will be expressed by the expected net present value of the maintenance costs:

$$E(PV) = \sum_{n=1}^n (C_{PM} + D_{congestion}) \cdot \left(1 - \frac{1}{(1+r)^{n \cdot \Delta I}}\right) \cdot \frac{1}{r}$$

In which:

C_{PM} = costs of planned maintenance [€]

$D_{congestion}$ = costs of congestion due to maintenance [€]

r = annual discount rate [-]

ΔI = time interval of the maintenance activities [yr]

n = total amount of maintenance activities within the given time period [-]

The present value of the risk will be defined as follows:

$$PV_R = \sum_{t=1}^L F_W(t) \cdot (D_{logistic} + C_{UM}) \cdot \left(1 - \frac{1}{(1+r)^t}\right) \cdot \frac{1}{r}$$

In which:

$F_W(t)$ = annual probability of failure [-]

$D_{logistic}$ = logistic damage due to failure [€]

C_{UM} = repair costs [€]

r = annual discount rate [-]

t = moment of time when maintenance will be executed [yr]

L = total considered time frame [yr]

Maintenance interval results

Table 8-11: The present value of the total costs over a period of 100 years.

Year	Time [year]	F_W [-]	$E(CW_{tot})$ [€]
2012	0	0,050	3.961.544
2024	12		116.005.193
2042	30		195.808.031
2060	48	0,524	246.974.891
2078	66		279.781.337
2096	84		300.815.711

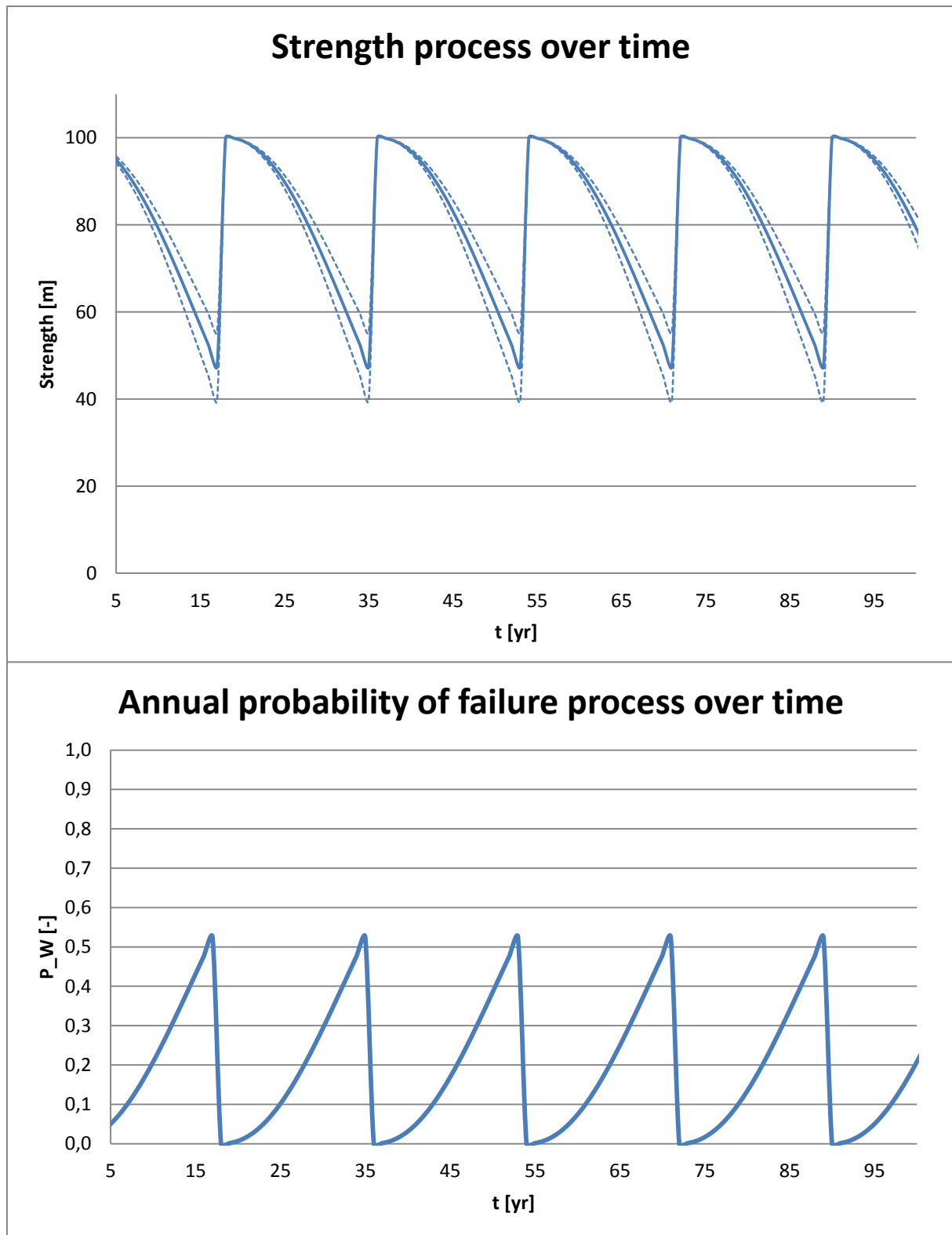


Figure 8-25: The process of the strength, indicated by the length of cracks in meters (above); and the annual probability of failure (below).

The calculation shows that the economical most beneficial maintenance point is every 18 years for the current asphalt properties (asphalt mixture, thickness, and etcetera), climate condition and traffic intensity. The corresponding probability of failure is more than 50% and the expected present value will be more than €300 million.

The reliability of the asphalt system can be calculated by the following:

$$R = 1 - F$$

In which:

R = reliability [-]

F = annual probability of failure over the maintenance interval [-]

Filling in the numbers:

$$R = 1 - 0,524 = 0,476$$

The availability can be calculated by the following:

$$A = 1 - U = 1 - (U_{unpl} + U_{pl})$$

$$U = U_{unpl} + U_{pl} = F_W \cdot \frac{MTTR}{I} + (1 - F_W) \cdot \frac{M}{I}$$

In which:

A = availability [-]

U = unavailability [-]

U_{unpl} = unavailability due to unplanned maintenance (repair) [-]

U_{pl} = unavailability due to planned maintenance [-]

F_W = annual probability of failure [-]

MTTR = mean time to repair [d]

M = mean time to execute the planned maintenance activities [d]

I = maintenance interval (time duration between the starting point (t = 0) and the moment of starting maintenance activities) [d]

The annual probability of failure (F_W) has been determined in the analysis above and be given by 0,524. The maintenance interval (I) which has been determined on the most economical beneficial point has been calculated on 18 years (or 6570 days). The reparation and maintenance durations has been determined on expert judgment and respectively be given by 182 days (/half a year) and 128 days (/18 weeks). Together this will result in the availability of the road connection.

Filling in the numbers:

$$U = 0,524 \cdot \frac{182}{6570} + (1 - 0,524) \cdot \frac{128}{6570} = 0,0145 + 0,0093 = 0,0238$$

$$A = 1 - 0,0238 = 0,9762$$

From an economical point of view the reliability should be 47,6% and the availability should be 97,6% per year on average.

8.8.2 Navigation lock

The deterioration model that will be used for the navigation lock will focus on the stability of the bed protection in front of the navigation lock on the Wadden Sea side. Inspection from RWS indicated that this is the largest risk which endangers functioning of the lock complex. Other risks will not be considered like the strength of the structure (i.e. lock gates) itself.

NB. This deterioration model only describes the shear and stability of the bed protection and not the degradation of the bed protection itself. So the bed protection wants to keep the scour hole as far as possible away from the navigation lock to prevent collapse.

The bed protection behind the navigation lock is needed in order to avoid instability of the lock itself due to scouring. In these situations the scour protection is not intended to prevent scour altogether, but only to ensure that the scour hole occurs far enough away from the lock in order to avoid instabilities. The required length of protection is therefore a function of the expected depth of the scour hole at the end of the protection and the expected upstream slope angle of the hole.

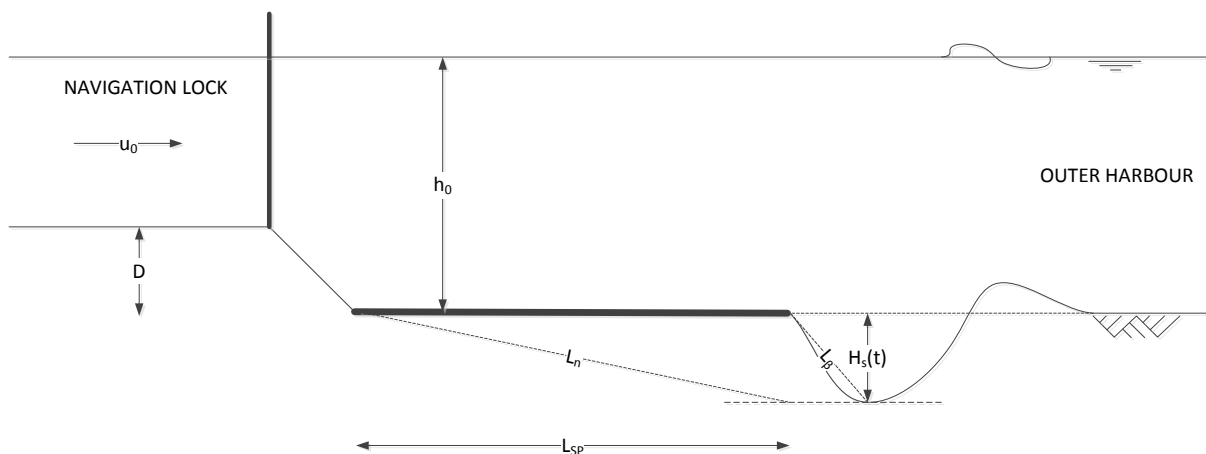


Figure 8-26: Schematisation of the bed protection problem at the navigation lock (not to scale).

To start with the calculations of the depth of the scour hole over time it's necessary to have some measured parameters. These parameters are as follows:

Table 8-12: Input parameter values.

Deterministic parameter	Symbol	Value	Unit
The undisturbed water depth of the lock	h_0	9,0	[m]
The height of the sill of the lock	D	4,0	[m]
The ratio D/h_0	D/h_0	0,44	[-]
The flow velocity on top of the sill	u_0	2,0	[m/s]
The exposure time	t	7,6	[d]
The grain size of the sand	d_{n50}	0,2	[mm]
The length of the bed protection	L_{SP}	150	[m]
The packing density of the sand	The soil is loosely packed		

To calculate with this model two assumptions have been made:

- The flow velocity is relatively high at the beginning of the locking process and decrease to zero. Here, the assumption has been made of an average flow velocity of 2 m/s over the total lock process.
- The exposure time forms a summation of the time that it takes to lock a ship. Opening the gates will take approximately 10 minutes and this happens 3 times a day which together lead to a fictive continuous process of 7,6 days in one year.

The length of the scour protection has to be calculated based on failure of the dam due to the scour hole at the end of the bed protection. When the scour hole will erode too much, the slope will be unstable and the soil will collapse. One should prevent this kind of situations from failing. The allowed slope of the scour protection should not be steeper than 1:15²⁵. There is also a maximum slope angle at the scour hole given by the parameter β , which cannot be steeper than 1:2 based on model experiments²⁶.

To calculate the required length of the scour protection one should first determine/calculate some parameters. The following formulae are used based on turbulent currents ($\psi = 0,03$):

- 'Smoothness' coefficient according to Chézy²⁷:

$$\text{Hydraulic radius: } R \cong h_0 = 9 \text{ m}$$

$$\text{Equivalent roughness: } k_r = 2d_{n50} = 0,4 \text{ mm}^{28}$$

$$C = 18 \log \left(\frac{12R}{k_r} \right) = 18 \log \left(\frac{12 \cdot 9}{2 \cdot 0,2 \cdot 10^{-3}} \right) = 97,76 \text{ m}^{1/2}/\text{s}$$

- Shields relation with correction parameters is the basis for all stability relations in critical flow situations²⁹:

$$\text{Relative density: } \Delta = \frac{\rho_s - \rho_w}{\rho_w} = \frac{2650 - 1000}{1000} = 1,65$$

$$\bar{u}_c = \sqrt{d_{n50} \psi_c \Delta C^2} = \sqrt{0,2 \cdot 10^{-3} \cdot 0,03 \cdot 1,65 \cdot 97,76^2} = 0,308 \text{ m/s}$$

- The average flow velocity at the end of the scour protection (which decreases linear to the velocity on the top of the sill)³⁰:

$$\bar{u}_s = \frac{h_0 - D}{h_0} \bar{u}_0 = \frac{9,0 - 4,0}{9,0} \cdot 2,0 = 1,11 \text{ m/s}$$

- The slope angle of the scour hole³¹:

$$\text{Smooth protection: } f_c = \frac{C}{40} = \frac{97,76}{40} = 2,444^{32}$$

²⁵ (Schierck, 4.3.6 Stability and slides, 2004)

²⁶ (Schierck, 4.3.6 The slope angle B, 2004)

²⁷ (Schierck, 2.3.1 Uniform flow, 2004)

²⁸ (Schierck, 3.2.5 Practical application, 2004), assumed that $\psi = 0,03$, which is a practical choice.

²⁹ (Schierck, 3.2.5 Practical application, 2004), assumed that the density of sediment $\rho_s = 2650 \text{ kg/m}^3$

³⁰ Assume that the amount of water, which flows over the sill, will flow over the end of the protection. It will not turn too much due to the contraction.

³¹ (Schierck, 4.3.6 The slope angle B, 2004)

$$\text{Turbulence: } r_0 = 1,2 \frac{\sqrt{g}}{C} = 1,2 \cdot \frac{\sqrt{9,81}}{97,76} = 0,0384^{33}$$

$$\beta = \arcsin \left[3 \cdot 10^{-4} \frac{\bar{u}_s^2}{\Delta g d_{50}} + (0,11 + 0,75 r_0) f_c \right]$$

$$= \arcsin \left[3 \cdot 10^{-4} \frac{1,11^2}{1,65 \cdot 9,81 \cdot 0,2 \cdot 10^{-3}} + (0,11 + 0,75 \cdot 0,0384) \frac{97,76}{40} \right] = 26,96^\circ$$

- The maximum depth in the scour hole developed for clear-water scour behind a bed protection³⁴:

$$\text{Exposure time: } t = 7,6 \text{ d} = 182,4 \text{ h}$$

$$\text{Dustbin parameter: } \alpha_L = 2,5 \text{ based on } \frac{D}{h_0} = 0,44^{35}$$

$$h_s(t) = \frac{(\alpha \bar{u}_s - \bar{u}_c)^{1,7} h_0^{0,2}}{10 \Delta^{0,7}} t^{0,4} = \frac{(2,5 \cdot 1,11 - 0,308)^{1,7} \cdot 9^{0,2}}{10 \cdot 1,65^{0,7}} \cdot t^{0,4} = 0,507 \cdot t^{0,4} \text{ m}$$

Failure mechanism

The major failure mechanism for the navigation lock is the instability of the structure itself due to scouring. By simplification, this failure mechanism will be represented by the length of the bed protection. This case study has not been executed to indicate the exact probability of failure, but to describe a method by doing that.

The most realistic failure mechanism will be introduced based on the failure mechanism of instability. One should however realize that this is a simplification of the reality to indicate a method to determine the most economical maintenance interval. The limit state function will therefore be formulated as:

$$Z = R - S = L_{SP} - (L_n - L_\beta) \cdot h_s$$

In which:

Z = limit state function [m]

L_{SP} = length of the scour protection [m]

L_n = slope of the scour protection [-]

L_β = slope of the scour hole [-]

h_s = maximum depth in the scour hole developed for clear-water scour behind a bed protection [m]

The system failure will be defined:

- When the depth of the scour hole over time becomes too deep that the length of the bed protection cannot guarantee the stability of the lock (Z < 0).

³² (Schierck, 4.3.3 Protection length and roughness, 2004)

³³ (Schierck, 2.3.1 Uniform flow, 2004)

³⁴ (Schierck, 4.3.1 Scour development in time, 2004)

³⁵ (Schierck, 4.3.2 Factor a, 2004), see also figure 4-14.

The costs repair and failure will be estimated by:

- $C_i = \text{€ } 2.500.000$ per maintenance activity;
- $c = \text{€ } 200.000$ per meter bed protection;
- $D_f = \text{€ } 300.000.000$ per failure event.

The costs of maintenance activities have been estimated by the amount of materials. Over the time the scour hole will increase and the maintenance activity therefore will exist of sand and bed protection materials (stones). Over the complete width of the lock the maintenance costs per meter will be €200.000 (Schiereck, 2004).

The damage costs due to failure of the lock have been estimated on a new navigation lock. When stability of the subsoil fails, the complete lock will be broken what lead to the construction of a new lock. A new lock with the same properties as the current one will approximately cost €300 million (Costs of an average sea lock, 2012).

The period for maintenance will be set on 100 years.

The costs will be expressed by the expected net present value of the maintenance costs:

$$E(PV) = \sum_{n=1}^n \left(C_i + c \cdot E(L_{SP} - L(t)) \right) \cdot \left(1 - \frac{1}{(1+r)^{n \cdot \Delta I}} \right) \cdot \frac{1}{r}$$

In which:

C_i = costs per maintenance activity [€]

c = initial costs of repairing the bed protection [€/m]

$E(L_{SP} - L(t))$ = expected value of the need to repair the bed protection [m]

r = annual discount rate [-]

ΔI = Time interval of the maintenance activities [yr]

n = total amount of maintenance activities within the given time period [-]

The present value of the risk will be defined as follows:

$$PV_R = \sum_{t=1}^L P_f(t) \cdot D_f \cdot \left(1 - \frac{1}{(1+r)^t} \right) \cdot \frac{1}{r}$$

In which:

$P_f(t)$ = annual probability of failure at moment t [-]

D_f = damage of instability due to failure [€]

r = annual discount rate [-]

t = moment of time when maintenance will be executed [yr]

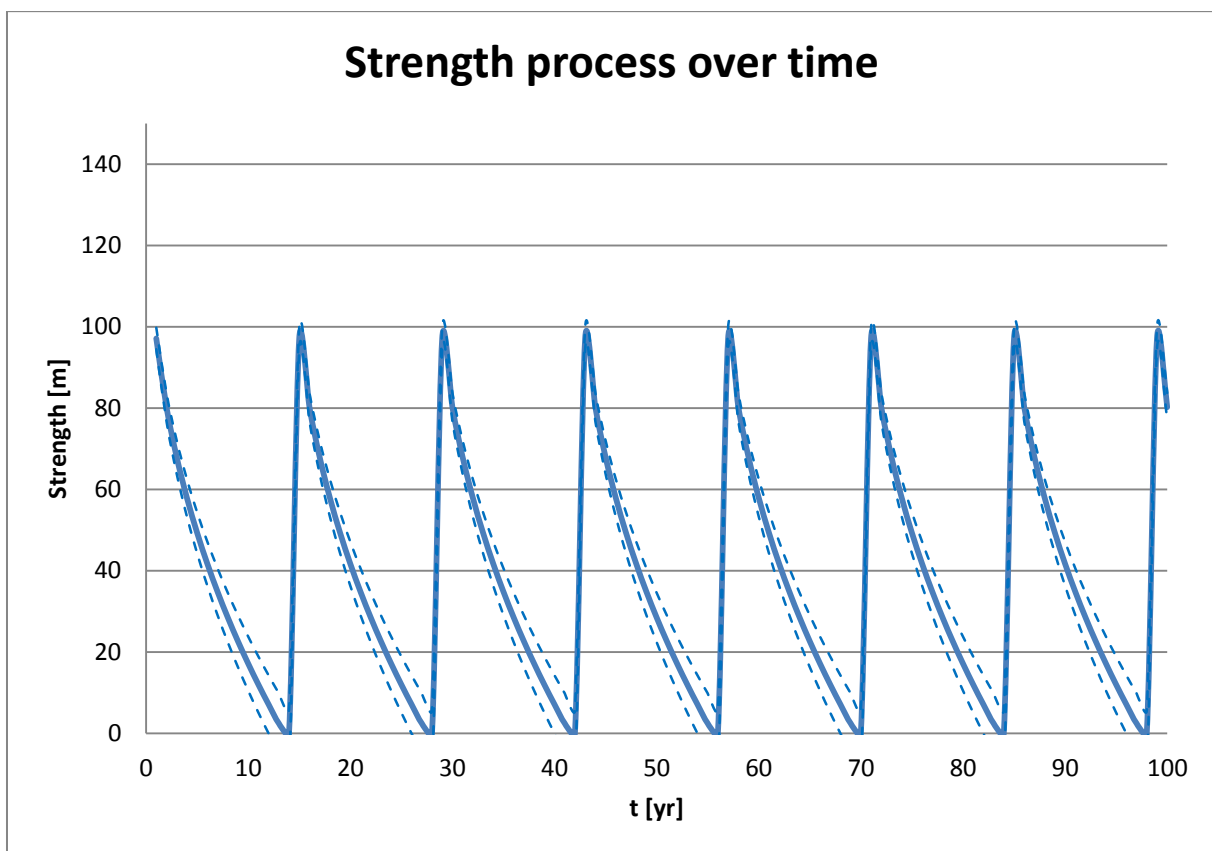
n = total amount of maintenance activities within the given time period [-]

Maintenance interval results

The results of this maintenance optimization have been elaborated in Appendix 12.A.

Table 8-13: The present value of the total costs over a period of 100 years.

Year	Time [yr]	F_w [-]	$E(CW_{tot})$ [€]
2012	0	0	0
2026	14	0,0893	384.605.537
2040	28		656.801.336
2054	42		849.441.705
2068	56		985.778.533
2082	70		1.082.267.814
2096	84		1.150.555.903
2110	98		1.198.885.240



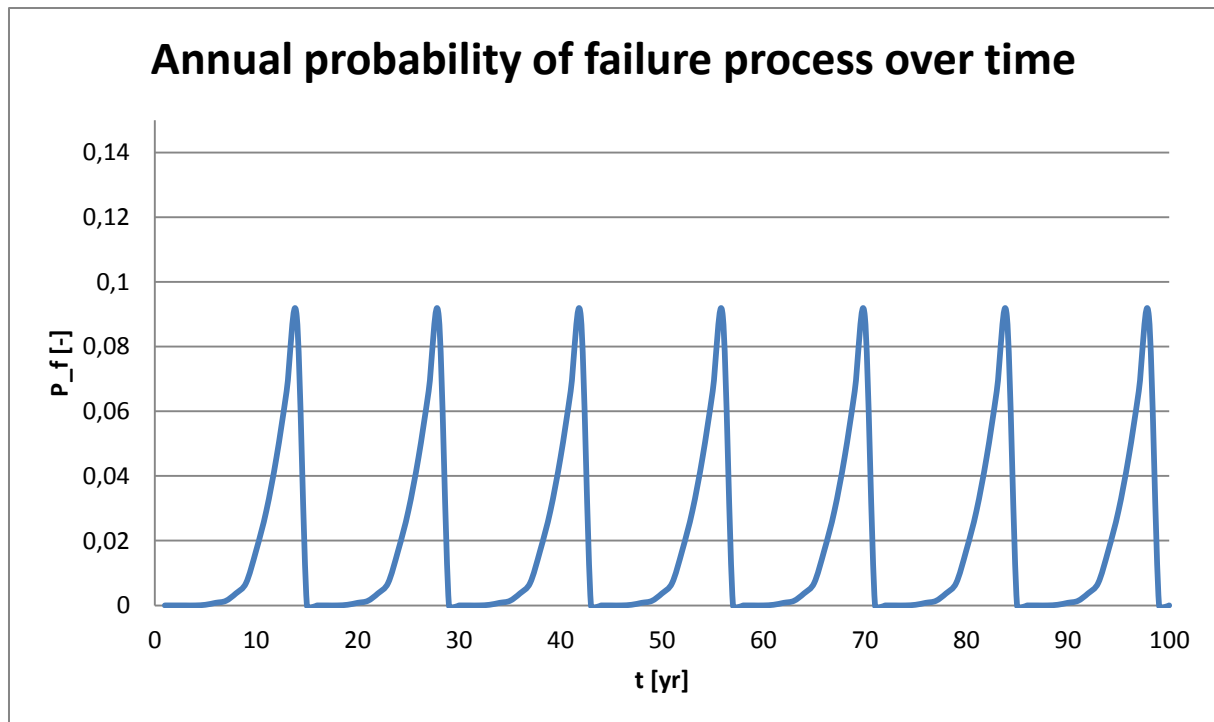


Figure 8-27: Process of strength, indicated by the length of cracks in meters (previous page); and the probability of failure (above).

The calculation shows that the economical most beneficial maintenance point is every 14 years for the current bed protection, climate condition and navigation intensity. The corresponding probability of failure is 8,93% and the expected net present value will be almost €12.000 million.

The reliability of the bed protection system can be calculated by the following:

$$R = 1 - F$$

In which:

R = reliability [-]

F = annual probability of failure [-]

Filling in the numbers:

$$R = 1 - 0,0893 = 0,9107$$

The availability can be calculated by the following:

$$A = 1 - U = 1 - (U_{unpl} + U_{pl})$$

$$U = U_{unpl} + U_{pl} = F_W \cdot \frac{MTTR}{I} + (1 - F_W) \cdot \frac{M}{I}$$

In which:

A = availability [-]

U = unavailability [-]

U_{unpl} = unavailability due to unplanned maintenance (repair) [-]

U_{pl} = unavailability due to planned maintenance [-]

F = annual probability of failure [-]

MTTR = mean time to repair [d]

M = mean time to execute the planned maintenance activities [d]

I = maintenance interval (time duration between the starting point ($t = 0$) and the moment of starting maintenance activities) [d]

The annual probability of failure (F) has been determined in the analysis above and be given by 0,0893. The maintenance interval (I) which has been determined on the most economical beneficial point has been calculated on 14 years (or 5110 days). The reparation and maintenance durations has been determined on expert judgment and respectively be given by 365 days (/one year) and 30 days (/4 weeks). Together this will result in the availability of the road connection.

Filling in the numbers:

$$U = 0,0893 \cdot \frac{365}{5110} + (1 - 0,0893) \cdot \frac{30}{5110} = 0,0095 + 0,0051 = 0,0146$$

$$A = 1 - 0,0146 = 0,985$$

From an economical point of view the reliability should be 91,1% and the availability should be 98,5% per year on average.

NB. In this example, the definition of maintenance has been given by restoring the system to its initial condition, but this leads to regular maintenance activities over time. By taking other measures it is also possible to reduce or even eliminate this kind of maintenance. This can be done by i.e. filling the scour hole with grits or underwater concrete (see for example the Dutch 'Oosterschelde kering') instead of just filling the hole with sand (initial situation). However, these possible measurements have not taken into account in this case study.

8.8.3 Discharge sluice

The model that will be used for the drainage sluice will focus on the piping mechanism in front of the drainage sluice on the Wadden Sea side to the IJsselmeer. Inspection from RWS indicated that this is the largest risk which endangers functioning of the sluice complex. Other risks will not be considered like the strength of the structure (i.e. sluice doors) itself.

A dike or structure fails due to piping in case the soil particles below the dike/structure are washed out due to excessive seepage. An example is shown in Figure 8-28 for an earthen dike with a clay blanket on top of a sand layer. Failures due to piping not only occurred in the Netherlands in the past, but also in Germany (Kolb, 1964). Four stages are defined: In the first stage, water pressures develop below the inside clay blanket. In the second stage, the clay blanket is cracked due to excessive pore pressures and sand boils start to develop. In the third phase, a canal develops below the dike. In the fourth stage, this canal progressively increases until an open connection between the outside water and the inside is formed. The open connection can finally cause the dike to collapse due to subsidence and cracking of the dike's body.

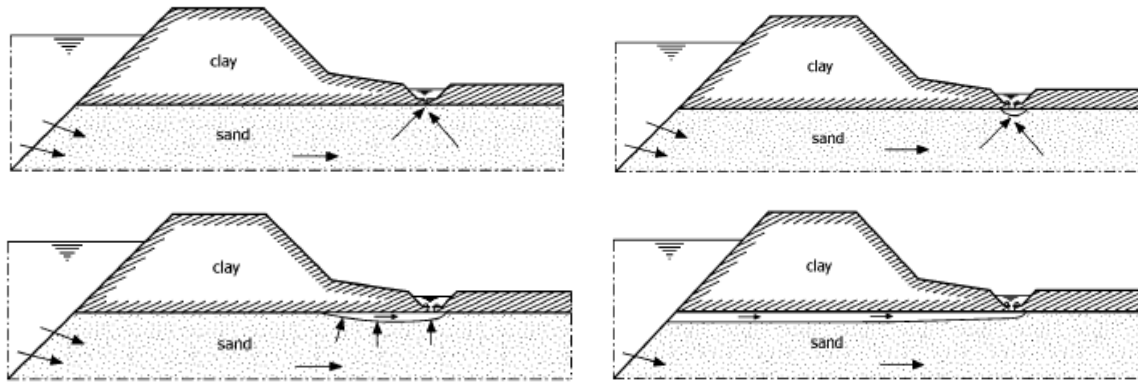


Figure 8-28: The principle of piping in case of an earthen dike (TAW, 2002).

Studies of the development of a pipe below a glass plate show that the pipe is not a single pipe, but a series of meandering pipes, creating new branches and closing some old ones, but progressively growing in length until it has reached the other side in case of high loading.

The reliability of the dike with respect to piping can be assessed with the methods of Bligh and Lane or with the more advanced method of Sellmeijer (Sellmeijer, 1988; TAW, 2002). Sellmeijer takes into account most influences and is used for the assessment of dikes in the Netherlands. According to Sellmeijer, a critical water level h_p is defined:

$$h_p = \alpha c L \left(\frac{\gamma_p}{\gamma_w} - 1 \right) (0,68 - 0,1 \ln(c)) \tan(\theta) > 0$$

Where α includes limited thickness of sand layer, c incorporates the erosion resistance of the sand layer, L is the leakage length, γ_p is the density of the grain particle, γ_w is the density of the water and θ is the rolling friction angle. Piping occurs in a corrected load H is higher than h_p . H is the water level minus $0,3 \times D$ (D is the layer thickness). For more information is referred to (TAW, 2002)

Bligh

The method of Bligh is a simple test to detect piping, the seepage length is tested against the critical seepage length according to Bligh. The seepage length present has been obtained through study. Monitoring for more seepage lines is carried out, each with accompanying head, if the exit point cannot be determined beyond doubt.

Bligh's creep factor is determined on the basis of the estimated grain diameter of the sand in the water bearing sand layer. If there is no data available on the sand, then a value of 18 is used for the creep factor. This is the maximum seepage line factor. This value can be used to quickly determine piping, if the entry and exit point are only roughly known.

The method of Bligh and the values indicated for C creep do not have an extensively probabilistic basis. It is assumed that it is a safe approach. The minimum factor must be used for the seepage length, or the different parts and for the ground surface inside the dike. If the seepage length present is less than the required seepage length according to Bligh, then there is a danger of piping. The evaluation can continue with the more advanced calculation rule according to Sellmeijer. This generally results in a more favorable (shorter) required

seepage length, although this is not necessarily the case. This is likely if the D/L ratio (thickness of the water-bearing sand layer and the seepage length) is high.

Bligh formula:

$$\Delta H \leq \Delta H_c = \frac{L}{C_{creep}}$$

In which:

ΔH = hydraulic head over the flood defence [m]

ΔH_c = maximum permissible gradient [m]

L = minimum seepage length [m]

C_{creep} = 'creep' factor [-]

Lane

It is recommended to only use this method to check for piping if the Bligh or Sellmeijer methods cannot be applied; for example when cut-off walls are used on the upstream side or in the middle under the flood defence. This method can also be applied to check for heave, both for monitoring and designing.

The seepage length, to be calculated, is composed of vertical and horizontal components. These can consist of cut-off walls, a vertical section at the outflow and the seepage line under the foreland and under the dike. One-third of the horizontal section is used to calculate the seepage length present. The weighted seepage line factor of Lane must be known in order to determine the critical seepage length. This depends on the type of material in the water-bearing layer, an estimate of the coarseness of the sand is sufficient. The method of Lane, just like the method of Bligh, is based on empiricism. The (best estimate of the) minimum seepage length and the reduction at normative outside water level must be applied for monitoring.

Lane formula:

$$\Delta H \leq \Delta H_c = \frac{\frac{1}{3}L_h + L_v}{C_{w,creep}}$$

In which:

ΔH = hydraulic head over the flood defence [m]

ΔH_c = maximum permissible gradient [m]

L_h = horizontal seepage length [m]

L_v = vertical seepage length [m]

$C_{w,creep}$ = 'creep' factor [-]

Sellmeijer

The critical seepage length is calculated more accurately with the Sellmeijer method. The Sellmeijer method almost always results in a lower critical seepage length, if the thickness of

the water-bearing sand layer is relatively limited. In general it is useful to apply the Sellmeijer method if the thickness of the sand layer is less than the seepage length. Use of the Sellmeijer method is always recommended if the necessary information is available.

The following additional information is required:

- grain distribution; and
- permeability of the sand layer.

Sellmeijer formula:

$$\Delta H \leq \Delta H_c = \alpha c L \left(\frac{\gamma_p}{\gamma_w} - 1 \right) (0,68 - 0,1 \ln(c)) \tan(\theta)$$

$$\alpha = \left(\frac{D}{L} \right)^{\frac{0,28}{\left(\frac{D}{L} \right)^{2,8} - 1}}$$

$$c = \eta d_{70} \left(\frac{1}{\kappa L} \right)^{\frac{1}{3}}$$

$$\kappa = \frac{\nu}{g} k = 1,35 \cdot 10^{-7} k$$

In which:

ΔH_c = critical hydraulic head over the flood defence [m]

γ_w = volume weight of water [kN/m³]

γ_p = (apparent) volume weight of sand grains under water [17 kN/m³]

θ = rolling resistance angle of the sand grains [°]

η = drag force factor (coefficient of White) [-]

κ = intrinsic permeability of the sand layer [m²]

d_{70} = 70 per cent value of the grain distribution [m]

D = thickness of the sand layer [m]

L = length of the seepage line (measured horizontally) [m]

Results

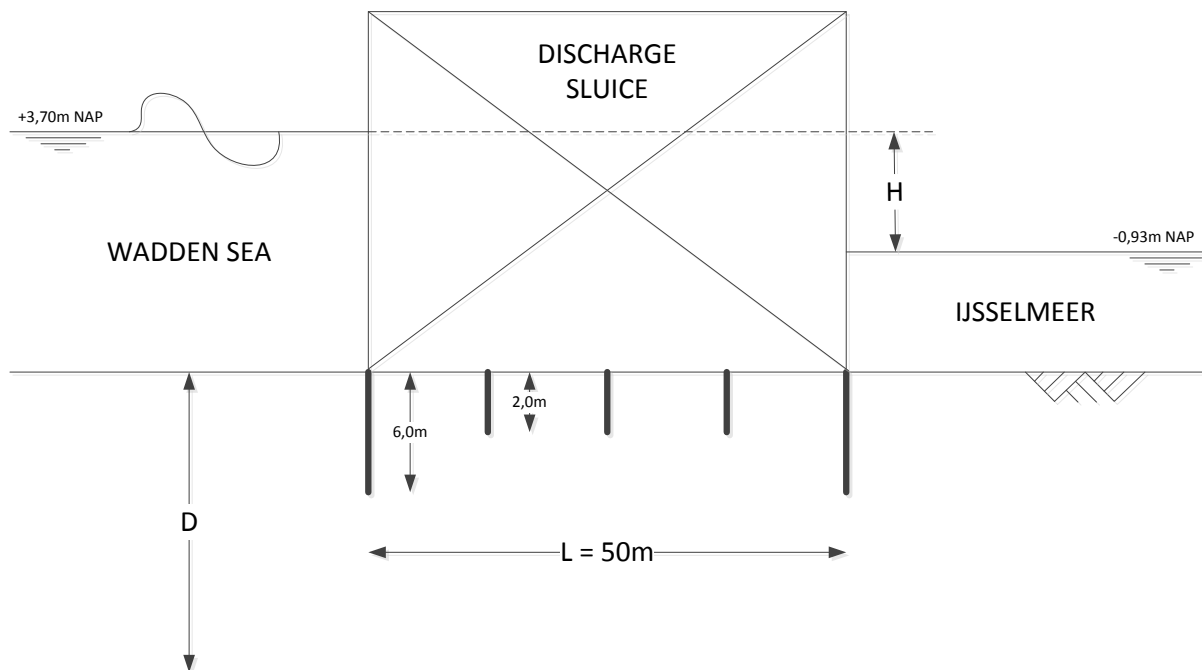


Figure 8-29: Schematisation of the piping problem at the discharge sluice (not to scale).

Different methods to assess the vulnerability for piping are used in this study. Since not all parameters are known, the results should be regarded as rough estimates. The methods of Bligh, Lane and Sellmeijer all indicate the structure is sensitive to piping, see Table 8-15. In this table, Δh_{\max} is the maximum occurred water level and h_{critical} is the critical water level for piping. The leakage length is estimated to 23 m; the coefficient of Bligh is 15 m and the coefficient of Lane 7 m. For Sellmeijer, the rolling friction angle is 43 degrees, d_{50} is 0,23 mm and the permeability is $4 \cdot 10^{-4}$ m/s. For other coefficients is referred to TAW (1999).

Table 8-14: Values of the parameters.

Assessment method	Parameter	Value	Unit
Bligh	C_{creep}	15	[-]
	L	68	[m]
Lane	$C_{w,\text{creep}}$	7	[-]
	L_h	50	[m]
	L_v	18	[m]
Sellmeijer	D	10	[m]
	L	68	[m]
	θ	43	[°]
	η	0,25	[-]
	γ_p	18	[kN/m ³]
	γ_w	10	[kN/m ³]
	K	$2,5 \cdot 10^{-4}$	[m/s]
	d_{70}	0,23	[mm]

Table 8-15: Piping sensitivity of discharge sluice breach.

Assessment method	Δh_{\max} [m]	h_{critical} [m]	Sensitive
Bligh	4,63	4,53	Yes
Lane	4,63	4,95	No
Sellmeijer	4,63	3,99	Yes

It may be obvious that the structure does not fulfill the required piping length according to all the three assessment methods (Bligh, Lane, and Sellmeijer). Because of the fact that this is not a deterioration model over time, the maintenance activity will only be executed when this activity does not exceeds the present value of the monetary risk of the total life cycle (here: €800.000).

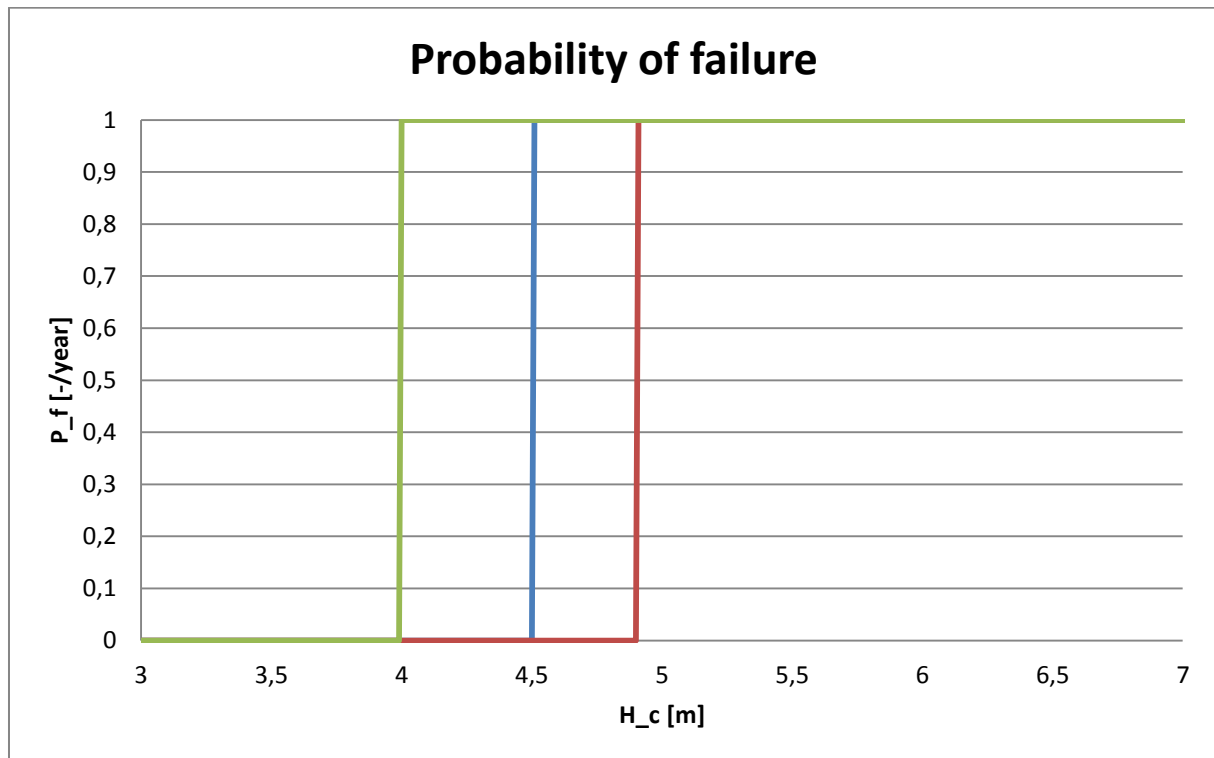


Figure 8-30: A step function³⁶ of the criterion of the piping mechanism according to the assessment methods of Bligh, Lane, and Sellmeijer.

The assessment methods of Bligh and Sellmeijer do not make any difference between the horizontal and vertical length in contrary to the method of Lane. Therefore an assumption has been made of taking measurements of only vertical lengths which has the most effects according to Lane.

The costs of increasing the vertical length will be estimated by using the [TAW, 2001], see Table 8-16.

³⁶ The piping step-function (or staircase function) can be classified as a Heaviside function $H(x)$. It is the mathematical concept behind some test signals, such as those used in determine the step response of a dynamical system.

Table 8-16: Extra needed vertical length to prevent piping and maintenance costs according to the three assessment methods.

Assessment method	Construction costs [€/m]	Extra vertical length needed [m]	Maintenance costs [€]
Bligh	37.000	1,5	53.650
Lane		0	0
Sellmeijer		11	407.000

Advantages of vertical cut-off wall outside of the discharge sluice are mainly based on easiness of maintenance in the coming future. Because this wall is below water level, there is no necessity of maintenance to this construction. Besides, this wall also does not take any space of the structure and does not lead to any limitations. So, this is a solution with a high maintainability.

For more specific research one can also consider other maintenance activities, like a piping berm, vertical cut-off wall inside the dike, filter structure, (artificial) increasing water level of the IJsselmeer.

The maintenance activity to prevent piping can be realized by construction a vertical cut-off wall of 11 m (or 2 walls of each 5,5 m) where the costs are estimated on €407.000 including preparations of the soil, materials, labor men, and etcetera.

This failure mechanism piping formed the largest risk of the discharge sluice. The measurements against this failure mechanism can be solved by constructing an extra vertical cut-off wall. The maintenance costs of €407.000 are less than the total present value over the life time of the discharge sluice (€800.000). Therefore, this maintenance activity is feasible with respect to the monetary risk of the structure. Of course, there are other risks that should be analyzed, but these risks are often applied on one of the discharge sluice and do not involve all the three sluices (like the case of piping, which result in instability of the total discharge complex).

Influence of time-dependence on outside water level on piping

The tides component in the hydraulic head over the flood defence on sea is generally considerable. Depending on the situation, tidal fluctuations in the outside water level in an inward direction will be absorbed by the sand layer. Indications of this can be obtained using 13-hour measurements of the water pressure response.

Although theoretically well-founded calculation models are not available to estimate the influence of fluctuations on the erosion process, in the case of strong damping near to the exit point it may be worthwhile to include only part of the tidal amplitude in the calculation of the hydraulic head present over the flood defence. It is recommended that experts are consulted in relation to this. For the heave mechanism the current maximum gradient at the site of the cut-off wall is normative. There is no simple methodology to calculate the time-dependent gradient due to tidal fluctuations in a substrate configuration with cut-off walls. Modeling with a multipurpose program for groundwater flow and consolidation will have to be set up case by case, and preferably calibrated to the results of 13 hour water pressure response measurements. Also here it is recommended that experts be consulted.

NB. This maintenance strategy has not been based on a deterioration model which does not have any consequence on the reliability and availability of the structure. Although, constructing a vertical cut-off wall off course lead to a higher reliability and availability: R = 99,9% and A = 99,9%.

The deterioration model that will be used for the dike will focus on the consolidation of the dike body on the compressible subsoil. Here, the assumption has been made that the subsoil completely exists of (impermeable and compressible) clay. The schematisation of the situation for heightening the dike crest has been illustrated in Figure 8-31. The length of the consolidation period of the subsoil can be estimated by using the formula:

$$c_v = \text{consolidation coefficient [m}^2/\text{s]}$$



Table 8-17: Data from the subsoil.

Parameter	Mean value	Standard deviation
γ_{clay} [kN/m ³]	18	0,1
D [m]	15	1
c_v [m ² /s]	$1,0 \cdot 10^{-6}$	$1,0 \cdot 10^{-7}$

The calculation of the consolidation will be made by using the formulas of Terzaghi and Koppejan. Although this simple schematization of the situation, it will not change anything about the optimization of the maintenance period.

The formula of Koppejan estimates the final consolidation at t :

$$\underline{\varepsilon_e} = \int_0^D \left(\frac{1}{C_p} + \frac{1}{C_s} \cdot \log \left(\frac{t}{t_1} \right) \right) \cdot \ln \left(\frac{\sigma'}{\sigma'_1} \right) dz = \left(\frac{1}{C_p} + \frac{1}{C_s} \cdot \log \left(\frac{t}{t_1} \right) \right) \cdot \underline{l_p}$$

$$\sigma_{zz}' = \sigma - p = (\gamma_{\text{clay}} - \gamma_{\text{water}}) \cdot z$$

$$\sigma'_{zz,1} = \sigma' + Q \cdot e^{\left(-\frac{2z}{\pi B} \right)}$$

In which:

- ε_e = consolidation at t_e [m]
- C_p = primary consolidation coefficient [m⁻¹]
- C_s = secondary consolidation coefficient [m⁻¹]
- t_0 = unit of time (often 1 day) [d]
- t = end of the consolidation activities [d]
- σ' = original soil stress of the subsoil at depth z [Pa]
- γ_{clay} = volumetric mass of clay [kN/m²]
- γ_{water} = volumetric mass of water [kN/m²]
- σ'_1 = new soil stress of the subsoil at depth z [Pa]
- B = width of the dike body [m]
- l_p = solution of the integral [-]

Hereby the consolidation coefficients will be chosen based on the chosen unit of time t_0 and the assumption these coefficients are independent of the depth z . The primary and secondary consolidation coefficients can be determined by the 'Constant Rate of Strain' test (CRS-test). These tests result in an average value and a standard deviation which means the coefficients are stochastic variables. The moment of time t_e depends on the consolidation coefficient c_v and the thickness of the compressible subsoil. It may be obvious that the compressible subsoil as well as the c_v will not be equally distributed and therefore t_e should be schematized as a stochastic variable. However, t_e equals often to a large number (and the partial derivative of S_e will therefore be very small), and will therefore be assumed as a deterministic variable.

Parameter	Mean value	Standard deviation
C_p [kN/m]	15	2
C_s [kN/m]	85	10
t [d]	1302	42
t_0 [d]	1	-
B [m]	66	-
Q [kN/m]	16,2	-

The Mean Value approach will result in a normal distribution for the final consolidation. The process of the consolidation over time has been given by the consolidation theory:

$$\underline{\varepsilon}(t) = \underline{U}(t) \cdot \underline{\varepsilon}_e$$

$$\underline{U}(t) = 1 - \frac{8}{\pi^2} \cdot \sum_{j=1}^{\infty} \frac{\exp\left(- (2j-1)^2 \frac{\pi^2 c_v \cdot t}{D^2}\right)}{(2j-1)^2}$$

In which:

$\varepsilon(t)$ = consolidation on t [m]

ε_e = final consolidation [m]

$U(t)$ = degree of consolidation [-]

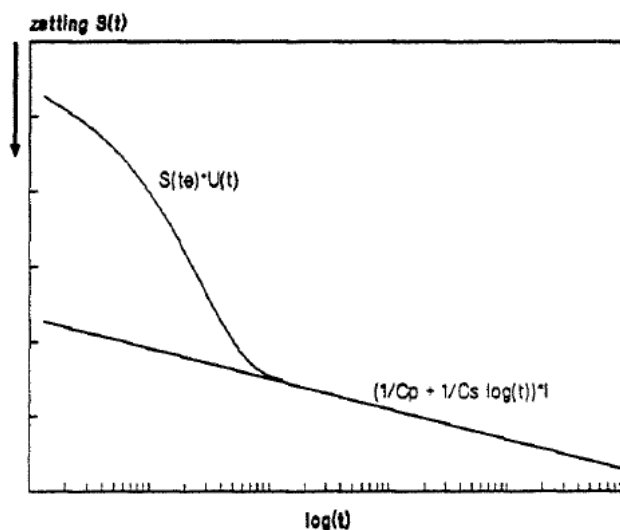


Figure 8-32: Consolidation process over time (Kuijper, 1991).

The strength of the dike will be expressed in the height over the dike and can be given by the formula:

$$\underline{H}(t) = H_0 - \underline{\varepsilon}(t) = H_0 - \underline{U}(t) \cdot \underline{\varepsilon}_e$$

In which:

H_0 = original strength [m]

$H(t)$ = strength on t [m]

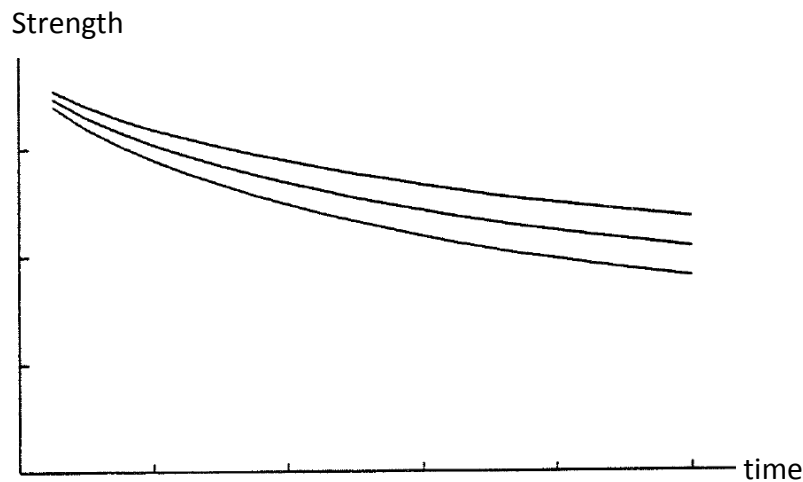


Figure 8-33: Strength process over time (Kuijper, 1991).

The consolidation process of the compressible subsoil after heightening the crest of the dike will be different from the process after the construction of the dike itself. This can be explained by the fact that the increasing stress in the subsoil after heightening the crest level way smaller is than compared with the construction phase. Besides, the layer will be stiffer when heightening the crest level than during the construction of the Afsluitdijk.

Failure mechanism

The major failure mechanisms for the Afsluitdijk have been determined in paragraph 8.6.4: overflow and wave overtopping. By simplification, these failure mechanisms will be represented by the height of the dike. This case study has not been executed to indicate the exact probability of failure, but to describe a method by doing that.

A fictive failure mechanism will be introduced based on the failure mechanisms overflow and overtopping. One should realize that will not represent the reality, but must be seen as a simplification to indicate a method of how to determine the most economical maintenance strategy. The (fictive) limit state function will therefore be formulated as:

$$Z = R - S = \underline{H_R} - \underline{H_S}$$

In which:

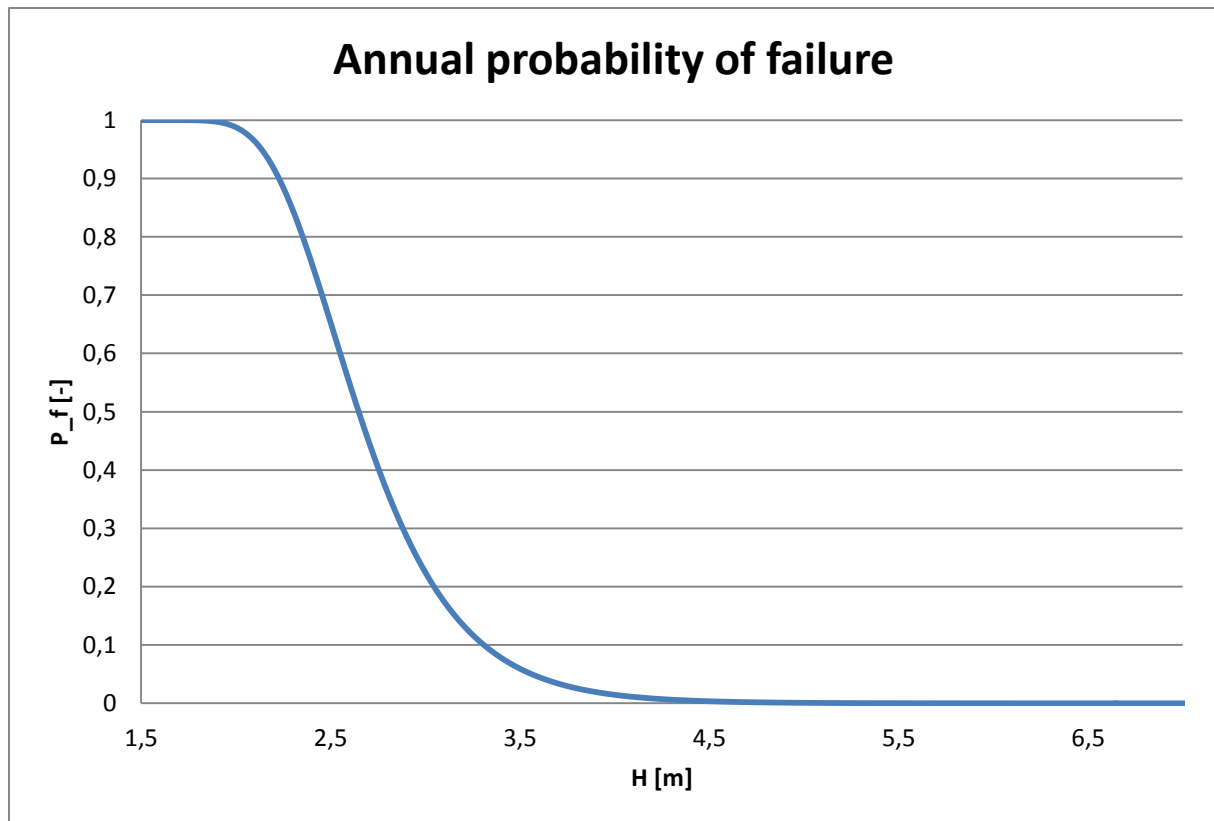
Z = limit state function [m]

H_R = strength of the dike [m]

H_S = load on the dike [m]

The annual probability of failure has been determined in paragraph 8.6.4:

$$P_f(H < HWS) = 1 - \exp(-\exp(-2,85(H - 2,52)))$$



The system failure will be defined:

- The flood defence system should always fulfil the Dutch water law (Waterwet) and therefore when the probability of failure exceeds 1/10.000 per year the system fails;

The costs repair and failure will be estimated by:

- C_i = € 40.000.000 per maintenance activity;
- c = € 2.000.000 per meter heightening crest level (see paragraph 8.6.4);
- D_f = € 11.000.000.000 per failure event (see paragraph 8.5.5);

The period for maintenance will be set on 10 years.

The costs will be expressed by the expected net present value of the maintenance costs:

$$E(PV) = \sum_{n=1}^n \left(C_i + c \cdot E(H_0 - H(t)) \right) \cdot \left(1 - \frac{1}{(1+r)^{n \cdot \Delta t}} \right) \cdot \frac{1}{r}$$

In which:

C_i = costs per maintenance activity [€]

c = initial costs of heightening the dike [€/m]

$E(H_0 - H(t))$ = expected value of the need to increase the dike [m]

r = annual discount rate [-]

Δt = time interval of the maintenance activities [yr]

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n = total amount of maintenance activities within the given time period [-]

The present value of the risk will be defined as follows:

$$PV_R = \sum_{t=1}^L P_f(t) \cdot D_f \cdot \left(1 - \frac{1}{(1+r)^t}\right) \cdot \frac{1}{r}$$

In which:

$P_f(t)$ = probability of failure at moment t [-]

D_f = damage of flooding due to failure [€]

r = annual discount rate [-]

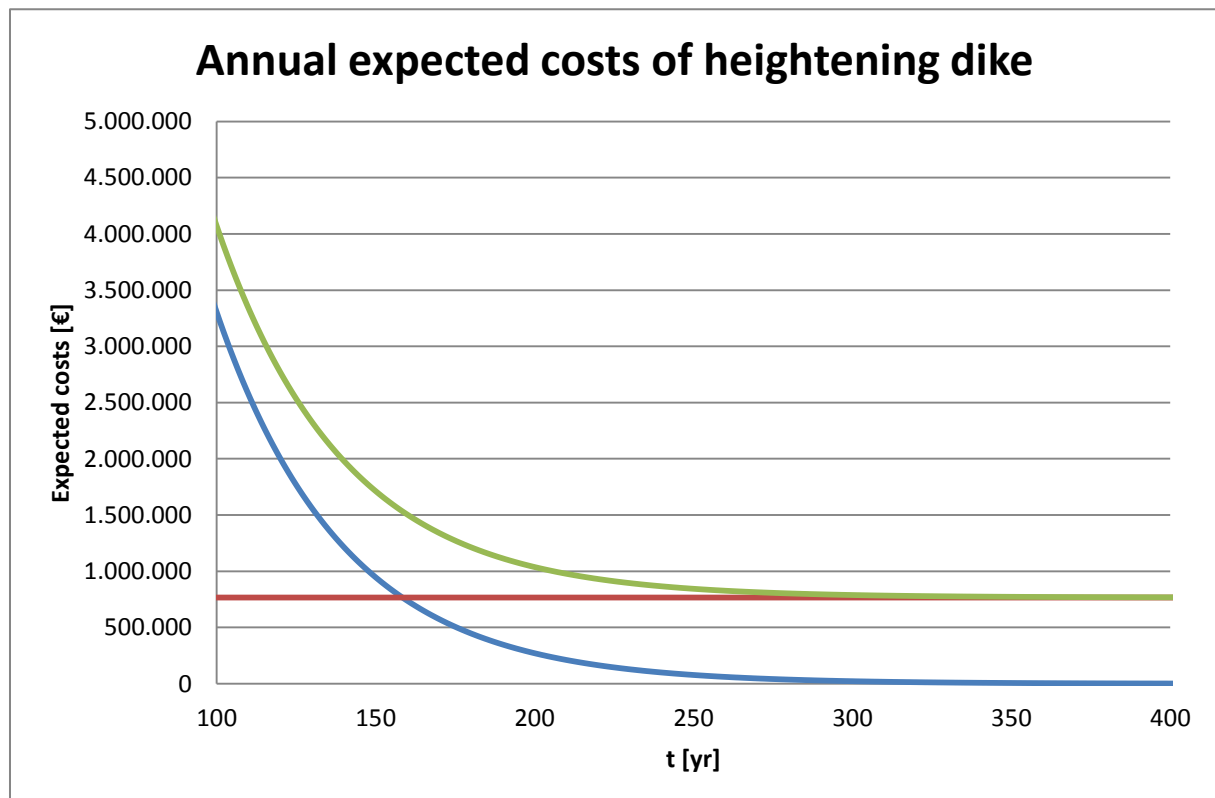
t = moment of time when maintenance will be executed [yr]

L = total considered time frame [yr]

Result

Table 8-18: Result of the consolidation process of the dike expressed in present value.

Time [years]	Thickness clay [m]	Consolidation [m]	Present value maintenance [€]	Present value Risk [€]	Present value Total [€]
0	15	0	0	527.232	527.232
10	14,8685	0,1315	0	6.643.823	6.643.823
100	14,8685	0,1315	0	28.061.453	28.061.453
1000	14,8685	0,1315	0	30.656.440	30.656.440



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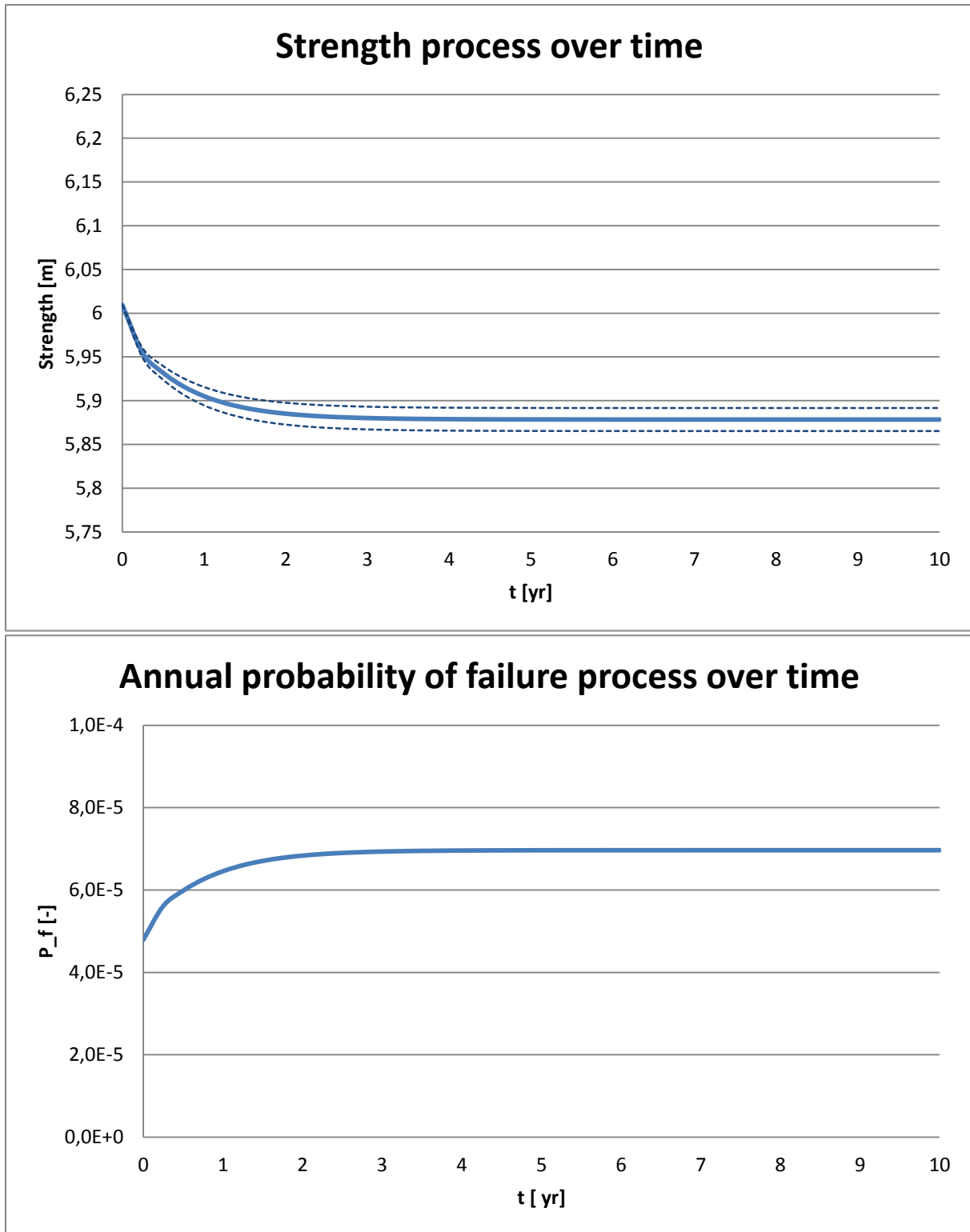


Figure 8-34: The process of the strength, indicated by the crest height in meters (above); and the probability of failure (below).

According to the two fail criteria there will be no maintenance in the form of increasing the consolidated dike. The present value of the total costs has been presented in the right column.

Because of the fact that the dike had been constructed about 80 years ago, the consolidation process is going on for a while. The relatively small increasing of the dike height (0,81m) will lead to a final consolidation activity of 0,132 m. The dike height therefore will not get below 5,88 m which corresponds with an annual probability of failure of $6,97 \cdot 10^{-5}$. This means that the first criterion of the probability of failure must stay below 10^{-4} will always be met over time. The second criterion described that the maintenance must be applied on the economical most beneficial moment in time and due to the small final consolidation, it is more beneficial to do nothing (no maintenance) than to increase the dike again to its original level.

The reliability of the asphalt system can be calculated by the following:

$$R = 1 - P_{f,end}$$

In which:

R = reliability [-]

$P_{f,end}$ = final probability of failure over the maintenance interval [-]

Filling in the numbers:

$$R = 1 - 0,0000697 = 0,9999303$$

The availability can be calculated by the following:

$$A = 1 - U = 1 - (U_{unpl} + U_{pl})$$

$$U = U_{unpl} + U_{pl} = F_W \cdot \frac{MTTR}{I} + (1 - F_W) \cdot \frac{M}{I}$$

In which:

A = availability [-]

U = unavailability [-]

U_{unpl} = unavailability due to unplanned maintenance (repair) [-]

U_{pl} = unavailability due to planned maintenance [-]

F = annual probability of failure [-]

MTTR = mean time to repair [d]

M = mean time to execute the planned maintenance activities [d]

I = maintenance interval (time duration between the starting point ($t = 0$) and the moment of starting maintenance activities) [d]

The annual probability of failure (F) has been determined in the analysis above and be given by $6,97 \cdot 10^{-5}$. The maintenance interval (I) which has not been determined, because it is not profitable to plan any maintenance activities and therefore the maintenance interval will be set on one year. The reparation and maintenance durations has been determined on expert judgment and respectively be given by 182 days (/half a year) and 0 days (because there is no planned maintenance). Together this will result in the availability of the road connection.

Filling in the numbers:

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$$U = 6,97 \cdot 10^{-5} \cdot \frac{182}{365} + (1 - 6,97 \cdot 10^{-5}) \cdot \frac{0}{6570} = 0,0000349 + 0 = 0,0000349$$

$$A = 1 - 0,0000349 = 0,999965$$

From an economical point of view the reliability should be 99,9% and the availability should be 99,9% per year on average.

8.8.5 Recapitulate economical maintenance optimization

On beforehand one expects that the road connection and navigation lock can be described by an invariable deterioration process and the dike by a variable deterioration process. Although, this result in different probability of failure values. Therefore also the reliability and availability became different. This has been summarized in the table below, see Table 8-19.

Table 8-19: An overview of the reliability and availability numbers and the corresponding maintenance activity.

Function	Reliability [%]	Availability [%]	Maintenance interval [yr]	Maintenance
Road connection	47,6	97,6	18	Renewing of asphalt pavement
Navigation lock	91,1	98,5	14	Repairing bed protection/scour hole
Discharge sluice	99,9	99,9	-	Vertical cut-off wall
Dike	99,9	99,9	-	Heightening crest level

It is not that remarkable that the reliability and availability of the dike are higher than the other functions (road connection and navigation lock), because of the fact that the damage due to failure of the function is way higher.

The fact that the dike does not need any regular maintenance in this case depends on the situation, but in general the maintenance intervals increase because the subsoil becomes stiffer over time. This, of course, can be explained by the variable deterioration process of the dike. Instead of the dike, the other functions describe an invariable deterioration process and therefore show a constant maintenance interval over the life cycle.

9. RAMSSHEEP requirements

In the introduction of the case study (paragraph 8.2) an illustration has been shown of the two possible approaches to solve the case study of the Afsluitdijk. The top-down approach describes a solution from the basic aspects of the RAMSSHEEP definitions and will work from the predetermined aspects from the acronym. The bottom-up approach determines where the solution can be calculated in the economical most beneficial situation. This classical approach also forms the basis for the acronym RAMSSHEEP. It determines whether what is feasible with respect to the engineering possibilities and also the feasibility of the engineering solution in the real world, or in other words the working space from a social point of view. All these working fields together form the solution space in which a feasible solution can be found.

Because of the fact that the acronym RAMSSHEEP came from a different part of engineering and because this acronym has been formulated in a general way, the case study has been applied to assess the correctness of the acronym. This assessment will be done by the bottom-up approach towards a solution of the problem which can finally be translated to the original aspects of the acronym. Finally, this translation can be compared with the original acronym solution (or in other words the top-down approach). This can be done because the acronym RAMSSHEEP should be build up from the underlying (, original) methods.

9.1 Bottom-up approach: Case study

The bottom-up approach is the most classical way of determining whether a solution is more beneficial (economical) than the original situation. Therefore this also forms the basis for many models that had been created over the last years.

The case study has been approached by this bottom-up method and now these results will be translated to the acronym RAMSSHEEP. By doing this, one is able to assess whether the acronym (new model) gives the same result or even is correctly defined. First of all, the result has been translated to the RAMSSHEEP aspects, see below.

Table 9-1: RAMSSHEEP requirements based on the results of the case study.

RAMSSHEEP aspect	Requirement description
Reliability	The road connection may fail a maximum amount of 19,4% per year.
	The navigation lock may fail a maximum amount of 13,3% per year.
	The drainage sluice may fail a maximum amount of 0,01% per year.
Availability	The dike may fail a maximum amount of 0,01% per year.
	The road connection should be 97,4% per year available for road traffic between North-Holland and Friesland.
	The navigation lock should be 98,5% per year available for its navigation

RAMSSHEEP aspect	Requirement description
Maintenance	function.
	The drainage sluice should be 99,9% per year available for its discharge function.
	The dike should be 99,9% per year available to retain water from the Wadden Sea.
	The road connection must be maintained (renewing asphalt layer) every 18 years.
	The navigation lock must be maintained (repairing bed protection/scour hole) every 14 years.
Economics	The discharge sluice must be maintained (constructing a vertical cut-off wall of 11,0m) immediately and does not need any regular maintenance.
	The dike must be heightened by 0,81 metre and does not need any regular maintenance afterwards.
Politics	The system must be maintained in the economic most beneficial way in which the expected costs of safety, security, health and environment has been implemented in the damage numbers.
	Making decisions based on the economical results within the system (lowest decision level).

NB. Here can be seen that not all the aspects of the acronym has been filled in.

9.2 Top-down approach: Acronym

The top-down approach has been applied according to the given definitions of the acronym. The RAMSSHEEP aspects has been formulated by a contractor of RWS and myself. This will be an example of how RWS approaches maintenance problems on the market.

This approach has been executed from the perspective of the contractor. This perspective has been chosen because of the fact that the contractor has to execute the maintenance activities and need therefore RAMSSHEEP aspects and requirements which can be used to steer on. In other words, the contractor needs concrete information about the requirements (RAMSSHEEP aspects) that need to be fulfilled, to steer on several aspects to increase the efficiency and therefore also the contentment of RWS. The RAMSSHEEP aspects have been formulated on the requirements which has been given by RWS and replenished by requirements by the law. This replenishment is used to give the contractor more grips on the aspects to steer on, so it has been formulated by concrete numbers or referred to guide books with concrete information.

All the numbers cannot be verified by RWS and one does not know where these numbers come from. RWS wishes the highest possible reliability and availability against the lowest possible costs. The result of the acronym RAMSSHEEP for the maintenance solution will be presented in the table below.

Table 9-2: RAMSSHEEP requirements based on the maintenance Afsluitdijk tender to the contractor.

RAMSSHEEP aspect	Requirement description
Reliability ³⁷	The primary flood defence system (The Afsluitdijk) ought to retain water levels with a probability value larger or equal of one in the ten thousand years (source: Dutch water law (Waterwet)).
	The dike, which is managed by RWS, is able to retain water and suffices to the hydraulic requirements from 2006 (source: Beheer- en Ontwikkelingsplan Rijkswateren)
	The road bridge may fail (unplanned unavailability; no road or water traffic possible) a maximum amount of two times per year.
	The lock gates may fail ³⁸ a maximum amount of two times per year.
	All three discharge compartments together may fail a maximum amount of one time a year.
	To preserve the reliability of the elements in the system it is obligatory use these elements only by its original purpose.
	The reliability never may be harmed by damages or defects on the elements with respect to the current regulations.
Availability	The water level in the IJsselmeer area should, under normal (climatological) conditions, is equal or lower than the governing high water level (NAP + 4,90m).
	The navigation lock should 99,9% ³⁹ per year be available for its navigation function.
	At least one of the discharge sluices should 99,9% per demand be available for its drainage function.
	The road connection should 99,9% ⁴⁰ per year be available for road traffic between North-Holland and Friesland.
	The road bridge should 99,9% per year be available for road traffic or navigation (bridge is operational (open or closed)).
Maintainability	Crucial elements should be in stock nearby the system for emergency repairs, like lock door, scupper gates (elements), etcetera.
	The lock chamber should be maintained to fulfil to the required intervention level.

³⁷ Failure means in this context not able to fulfil its original function.

³⁸ Technical failure, external or natural causes, deviation of water level upper reach or constructional failure.

³⁹ The number of 99,9% per year availability is based on the operational hours per year. For example, a navigation lock is only for use from 5:00 am. until 0:00 pm., so that means that the availability requirement is only applicable for this time period. Moreover, this 99,9% availability per year is excluded of planned maintenance.

⁴⁰ Unavailability of 0,01% per year = 9 hours per years.

RAMSSHEEP aspect	Requirement description
Safety	The access route (roads, waterways, etcetera) should be big enough for large materials and equipment and the extra weight (impulse force) should not harm the safety of the dike.
	When critical elements ⁴¹ do not fulfil to its given intervention level (or standard) should be replaced by new ones with a better price/maintainability ratio.
	All the critical elements within the system should be checked on visible damage every year and after a severe storm.
	Planned maintenance on the system should never lead to obstruction of the road traffic or navigation (the users).
	It is not acceptable when humans die or have serious injuries due to not functioning of the system over a period of 100 years.
	All installations within the system should suffice to the safety, health and welfare policy ⁴² and the NEN6787 (and other regulations).
	The road bridge, drainage scuppers and navigation lock gates should be used in a safe and smooth way with due observance of extreme situations ⁴³ and the reliability and availability requirements (RA-aspects).
	The professional lock master (with direct sight on the lock chamber or by CCTV) should control the navigation activities (professional and recreational) in a safe way without harming anyone. ⁴⁴
	At the lock site it is obligatory to have presented some life-buoys and life-safe hooks. Beside this a ladder has to be present within the lock chamber.
	An annually check-up of the fire alarm and lightning protection system and the absence of redundant inflammable substances in the system.
	The road A7 should not have any obstructions (due to mutations in the system) in the sight lines of the users and the road marking should be in good condition.
	Inspection to all mechanical and electrical components should be conforming the regulation in the NEN3140.
	The accessibility of rescue service (ambulance, fire department, police) should be conform the regulations (time of arrival < 30 minutes) and

⁴¹ Critical elements are elements which have a high potential failure frequency with large damage, or in other words risk (risk = probability of failure * damage). These elements can be determined by a FMECA.

⁴² The safety, health and welfare policy is not something one can use to steer on during the performance agreement. This policy has been documented by the Dutch law system, so it is a bit trivial to mention (because everyone should keep up to the law).

⁴³ Extreme situations depend on a couple of parameters, like wind direction, wind speed, water level/tide, precipitation, etcetera. One should act according the protocols which has been documented for this extreme situation. Besides, measures against extreme weather conditions may not lead to dangerous situations, like spreading salt during a frost period, puddles on the dike which endangers the strength, etcetera.

⁴⁴ Although, one is not able to steer on this aspect, this may even well lead to damage to the lock complex by using the system not correctly. So, it has indirect connections to the maintenance.

RAMSSHEEP aspect	Requirement description
Security	twice a year drills should be organized.
	With respect to the control buildings of the navigation lock, discharge sluice and bridge should not be accessible from unauthorized people.
	The control buildings should be under surveillance by cameras.
Health	Crucial elements, which are in stock, should be fenced off from unauthorized people.
	In general a good quality of life (physical health) due to the direct and indirect usage of the system ⁴⁵ .
	The direct and indirect life in the system (users and all areas which will be influenced by the Afsluitdijk) should form a solid and safe dike (psychologically) and not lead to an increase of the stress level of the users due to high water levels. ⁴⁶
Environment ⁴⁷	The water quality should at least suffice to required values which have been documented in Kaderrichtlijn Water (KrW).
Economics	The costs to maintain the system to its requirements should be within the budget against the highest benefits (optimization).
Politics	The contentment of the users should be as high as possible (maximisation). ⁴⁸

9.3 Evaluation

In this evaluation part the two approaches will be discussed based on their results. First of all, a comment has to be made on both approaches: the bottom-up approach is the most commonly accepted way of solving a problem in the engineering world. The cost-benefit analysis forms therefore always the main driver by solving the problem by finding the minimum of the expected cost function. The top-down approach, which can be described by the acronym RAMSSHEEP, is an extended version of the cost-benefit analysis. One should always approach a problem by balancing the costs and benefits and the origin of the RAMSSHEEP aspects have been defined based on this point of view. Or in other words, the acronym should result in the same (or more or less the same) as the case study did.

⁴⁵ With the physical health term one should think of special and dangerous material use (i.e. asbestos) or dangerous natural damage (i.e. processionary caterpillar, sick seals, and etcetera).

⁴⁶ This physical health can be caused by news from kinds of media. Messages in the media which say that the Afsluitdijk is below its safety level will lead to an increase of the stress level.

⁴⁷ The environment aspects should at least fulfill the requirements in the following laws: Waterwet, Wet milieubeheer (WM), wet bodembescherming (WBB), wet belasting op milieugrondslag (WBM), besluit bodemkwaliteit (BBK), wrakkenwet, ontgrondingswet, natuurbeschermingswet 2005, flora en faunawet (FFW) and Kaderrichtlijn water (KrW).

⁴⁸ Contentment of the users can be translated to concrete terms like no flooding of the Netherlands, no endless congestion of motorists and captains, no victims of certain measurements/calamities (by the contractor), and etcetera.

By using the bottom-up approach (BUA) as the standard, it is surprisingly that the result from the BUA is way smaller than the top-down approach (TDA). By taking a closer look at the results it is obvious that the requirements of the TDA are determined on many different operational levels. The reliability, availability and economics have given clear requirements on top level where no specific conditions have been implemented. When looking at the maintainability, safety, security, environment and health, these aspects are more specific concentrated on the operational phase over time and not based on the measurements to fulfil to the systems requirements. By looking at the BUA, these formulated aspects operate on the same level and are concrete to the focus of the problem: maintaining the system in order to retain the Afsluitdijk technical state or reverting it back to this state.

As a second point of discussion, the source of the numbers of the reliability and availability has been unknown. By analysing the numbers the best guess that can be made is that RWS aims on the highest contentment of the users of the system, but without looking at the consequence for the costs. It would be a coincidence when the reliability and availability of all the functions are 99,9% based on the economical most beneficial situation. Therefore one may conclude that the cost of keeping the system working at a high level is way more important than the corresponding costs to do this. Although, it is strange the budget is always too short. Moreover, in the economics part of the acronym requirements describes: *the costs to maintain the system to its requirements should be within the budget against the highest benefits (optimization)*. This is obviously contrary to each other, so probable RWS wishes to have it all (low costs, high reliability and availability) which is not feasible for these kinds of problems.

As third and last point of discussion the list of requirements of BUA is smaller than TDA. This can mainly be explained by the fact that most of the RAMSSHEEP aspects have been implemented with the economics. This means that there are no special terms determined and have been expressed in a possible damage number. So it is all coming back to the economics, which is not the case at the TDA. Here every single aspect has been elaborated to something concrete, but sometimes a certain aspect is not relevant or can be implemented in another aspect.

10. Conclusion and Recommendation

10.1 Conclusion

In this chapter of conclusions the results of the theory and case study will be discussed. The objective of this research is to assess whether the acronym RAMSSHEEP is applicable as a risk-driven maintenance model. Comparing the Probabilistic Approach (PA) and RAMSSHEEP resulted in some similarities and many differences what has been presented by advantages and disadvantage of these approaches.

The two approaches, PA and RAMSSHEEP, have more or less the same purpose; a basis document to optimize a maintenance plan for a certain system or structure. Both approaches result in requirements in which the system should be sufficient to fulfill its functions. These requirements can be based on two approaches: (1) economical optimization and (2) maximum contentment of its users. Below an overview has been given of the advantages and disadvantages of PA and RAMSSHEEP.

Similarities

- A basis for a maintenance plan;
- More or less the same aspects will be used in the analyses.

Differences

- PA formulates the solution by starting at the problem. The solution space has been defined by the options in the real world, society, and technical (engineering) solutions.
- RAMSSHEEP takes a certain statement and bases system requirements on that statement. In this case the statement has been formulated as the highest possible contentment of its users, what automatically leads to high requirements.
- PA approaches the problem on a relatively high level. Or in other words, the aspects that have been analyzed and calculated are active on the same operation level.
- RAMSSHEEP is analyzing on more than one level on the same time. The different aspects of the acronym are active in the top level (system), but also on element level (specific objects).

- PA translates all aspects within the system in a certain amount of costs (damage, investment). This has been done for maintenance activities (including safety), security, health and environment.
- RAMSSHEEP does not clarify which elements have been expressed in costs if this is even done.
- PA obviously acts on economical optimization of the system. Therefore all aspects within the system will be approached by costs what eventually leads to an estimation of the costs in the complete lifecycle.
- RAMSSHEEP aims at economical optimization, but after the requirements have been determined (without any background analysis). This means that there is not much steering space available to create the most beneficial maintenance plan.
- The input of the PA forms an important part of the analysis. The determined numbers have been based on many sources and therefore an uncertainty has to be introduced. The results of the PA are sensitive to a different input what also may lead to a different decision.
- RAMSSHEEP has less input than PA, because the numbers have been determined on contentment of its users and not on economic analysis. Therefore, this leads to less sensitivity in the results.
- PA forms a basis for the maintenance plan, and it has been explicitly added to the analysis parts. So, maintenance over the years has to be optimized by another analysis and be based on the PA results.
- RAMSSHEEP has implemented the maintenance in the same analysis. However, there has not been given any specific information on the optimization of the maintenance activities, but it does form a part of the analysis.
- PA gives a straight-forward working method which clearly describes the steps that should be taken to calculate the results. The descriptions are SMART⁴⁹ formulated which gives a robust and valuable approach.
- RAMSSHEEP describes the requirements broad and vague. Only the reliability and availability are requirements on which a contractor can steer on. The other requirements are often too specific or too vague.

⁴⁹ Specific goals, Measureable goals, Action-Oriented goals, Realistic goals, Time-based goals.

Table 10-1: Overview of the similarities and differences between PA and RAMSSHEEP approach.

Aspects	PA	RAMSSHEEP
<u>Reliability</u>	Amount of time functional [%]	Amount of time functional [%]
<u>Availability</u>	Amount of time available for its users beside planned and unplanned maintenance [%]	Amount of time available for its users beside planned and unplanned maintenance [%]
<u>Maintainability</u>	Mean Time To Repair (MTTR)	Measures to ease the maintenance on the system
<u>Safety</u>	Costs of unsafe/danger situation of the system	Using and maintaining the system according to the Safety manual
<u>Security</u>	-	Safe system with respect to vandalism, terrorism and human errors (including all kinds of sabotage of the system)
<u>Health</u>	Casualties have been translated into possible damage to the system	Minimization of the casualties due to function failure.
<u>Environment</u>	Pollution, contamination, and etcetera have been translated into possible damage to the system	To meet certain requirements which have been secured in Environmental Acts one suffices the rules of a good and clean environment.
<u>Economics</u>	Decision will be made by this main driver. The cost-benefit balance aims at the most optimal situation.	A serious reflection in terms of a Cost-Benefit Analysis must be made to provide more insight for an economical choice.
<u>Politics</u>	Politics gives the level of strategy on which the cost-benefit analysis will be executed: micro-, macro economy or political science.	A rational decision has to be made.

The PA started in 2001 by calculating the probability of dike breaches for the most important dike rings in the Netherlands. It also implemented the possible damage to a certain dike breach which eventually resulted in the monetary risk of the system. This information can be used by determining the most economic beneficial solution to improve the safety of the system.

The original intention of the PA project was to present a method that analyses the economical beneficial solution for RWS. This could be used in future tender projects for RWS. However, nowadays most tenders are based on RAMSSHEEP with given requirements.

The current approach of RAMSSHEEP is not optimal, because it aims to the most economical beneficial situation but also with highest contentment of its users. There has to be made a compromise between these two requirements: high contentment of its users leads to large amount of costs and economical optimization leads possibly to lower contentment of its users. The available budget of RWS to maintain the primary flood defence system is always short and it would therefore be interesting to create an economical optimal solution.

10.2 Recommendation

The recommendations of this research are based on two parts; (1) proposal for improvements on RAMSSHEEP and (2) recommendations for further research.

10.2.1 Improvements of RAMSSHEEP

The improvements of RAMSSHEEP come from the conclusions above. It may be clear that the approach of RAMSSHEEP should be adjusted. The following improvements are recommended:

- (1) Choose just one political strategy for the cost-benefit analysis: economic optimization or contentment of its users. Hereby the economic optimization will be recommended.
- (2) Using PA for making decisions of measurement/investments to increase the safety of the system bases on the economic most beneficial solution.
- (3) An optimization of the maintenance can be based on physical deterioration models (verified on its current situation) which leads to a maintenance plan.
- (4) Results of PA can be translated to an optimal reliability and availability of the system which can be used as a level of intervention.

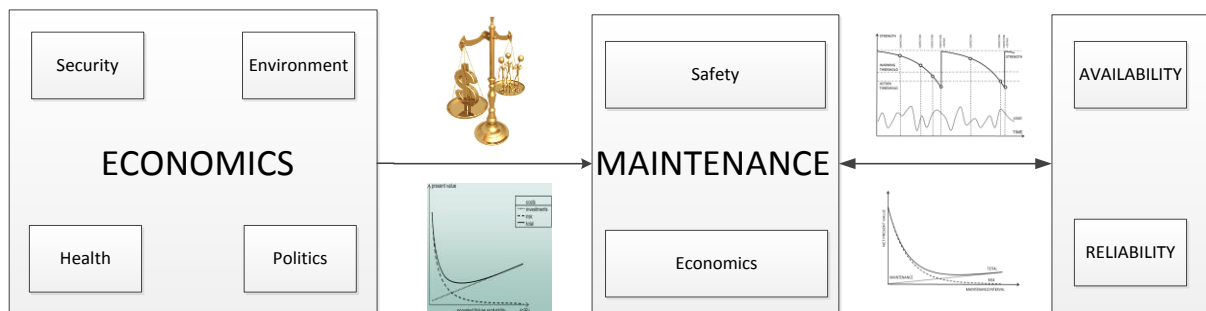


Figure 10-1: Recommended approach of economic most beneficial maintenance intervals.

Gathering all the recommendations (see above) will lead to

Figure 10-1. This new 'model' will be called EMAR. This is nothing more than a gathering of existing models which all have been based on economic optimization.

According to EMAR a certain technical problem will be solved by considering the engineering possibilities and societal feasibility. As illustrated in

Figure 10-1 the first step is to make a cost-benefit analysis (CBA) according to the PA. A political cost-benefit level has to be determined up front, because this is essential to determine the values of the benefits. The CBA contains costs of the current risk and the investment costs to reduce the risk what should balance to an optimal situation: lowest possible costs or highest possible benefits.

The next step is the optimization of maintenance over the life cycle of the structure and system. Hereby the highest risks will be translated from a physical model to a scientifically deterioration model which can be used to determine the probability of failure. The optimization of maintenance activities acts on the same principle as cost-benefit analysis in the first step, during the optimization one is searching for the most economical maintenance interval. This maintenance interval should be determined on the summation of the expected costs of the risk (unplanned failure) and the investment costs to repair the system to its original performance level. Eventually, this will lead to reliability and availability of the system, but based on the future situation instead of the current situation. Results of probabilistic analysis can be translated to an optimal reliability and availability of the system which can be used as a level of intervention.

The name of the 'new' model EMAR has been chosen, because the individual letters represent the four main processes in the solution. The optimization (economical) starts by the Economics (E) which determines whether or not adaptations to the system should be made. The results will lead to a basis for the optimization of the Maintenance (M) which can be translated to the optimal intervention levels of Reliability and Availability (RA). This last process can be checked by inspection whether or not the deterioration models are still applicable for that system.

10.2.2 Further research

The result of this research leads to more options to investigate other parts of engineering. The main recommendation for further research therefore will be:

The model EMAR has been used on a primary flood defence system (Afsluitdijk) and resulted in a basis for planned maintenance by economical optimization of the maintenance intervals. Is this model also applicable on other parts of the technical engineering (like maintenance on high/rail ways, hydraulic structures, electrical elements, and etcetera)? So can this model be used in a more general way or should this model be used on more specific parts (scaling)?

Beside this main recommendation, also further research should be done by the specific verification of this research:

- The investments in the probabilistic calculation have been concentrated mainly on reducing of the probability of failure and not on reducing the expected costs of the damage. Is it profitable (or cost-effective) to divide a polder in different parts by a compartment? Or is it profitable to build new house estates on higher levels within the polder?
- Many limit state functions are based on rather simple models. More research could lead to more sophisticated (numerical) models for reliability methods and a better representation of reality.
- A sensitivity analysis of the numbers in the case study can be applied to get insight in the reliability of the results. The largest sensitive numbers can be prioritized in a list and also result in a research in which more reliable numbers can be determined.

- Nowadays many innovations have been discovered and many will come. More research could lead to changes in maintenance intervals or expected costs for the several functions (see also next page).
 This further research in innovations can be an important subject, because maintenance itself is not the main part of the costs per year:

Depreciation	5% per year	~45%	
Interest	5% per year	~45%	
Maintenance	1% per year	~10%	+
Total	~11% per year	~100%	

Although, maintenance does have a direct connection with the depreciation of the systems elements. So innovations in i.e. materials can reduce the annual costs and is therefore an interesting subject to do some research on.

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12. Appendices

A. Water level data

	Maximum annual water level		Maximum annual water level		Maximum annual water level
Year	[m]		[m]		[m]
1932	2,38	1967	2,90	2002	2,85
1933	1,89	1968	2,30	2003	2,64
1934	2,14	1969	3,01	2004	2,85
1935	2,52	1970	2,97	2005	2,45
1936	3,00	1971	2,84	2006	3,21
1937	1,83	1972	2,23	2007	3,61
1938	2,74	1973	3,19	2008	3,15
1939	2,57	1974	2,75	2009	2,48
1940	3,00	1975	2,12	2010	2,43
1941	2,57	1976	3,70	2011	2,69
1942	2,69	1977	2,98	2012	2,55
1943	2,83	1978	2,28		
1944	2,89	1979	2,78	Parameter	
1945	2,93	1980	2,22	X _m	2,71 [m]
1946	2,45	1981	2,95	s	0,42 [m]
1947	2,23	1982	2,84		
1948	2,20	1983	3,43	Coefficients	
1949	2,55	1984	2,78	a	2,52 [-]
1950	2,01	1985	2,55	b	2,85 [m]
1951	2,61	1986	2,75		
1952	2,19	1987	2,44		
1953	3,43	1988	2,77		
1954	3,62	1989	2,98		
1955	2,53	1990	3,56		
1956	2,40	1991	2,92		
1957	2,12	1992	2,23		
1958	2,62	1993	3,37		
1959	2,23	1994	3,39		
1960	3,08	1995	3,34		
1961	2,34	1996	2,48		
1962	3,14	1997	2,17		
1963	2,33	1998	2,73		
1964	2,48	1999	2,82		
1965	2,72	2000	2,88		
1966	2,91	2001	2,88		

B. Probability of occurrence of data points

i	Year	Maximum annual water level [m]	Probability of exceedance: p [-]	Return period: T [years]
1	1976	370	0,0122	82
2	1954	362	0,0244	41
3	2007	361	0,0366	27
4	1990	356	0,0488	21
5	1983	343	0,0610	16
6	1953	343	0,0732	14
7	1994	339	0,0854	12
8	1993	337	0,0976	10
9	1995	334	0,1098	9
10	2006	321	0,1220	8
11	1973	319	0,1341	7
12	2008	315	0,1463	7
13	1962	314	0,1585	6
14	1960	308	0,1707	6
15	1969	301	0,1829	5
16	1940	300	0,1951	5
17	1936	300	0,2073	5
18	1989	298	0,2195	5
19	1977	298	0,2317	4
20	1970	297	0,2439	4
21	1981	295	0,2561	4
22	1945	293	0,2683	4
23	1991	292	0,2805	4
24	1966	291	0,2927	3
25	1967	290	0,3049	3
26	1944	289	0,3171	3
27	2001	288	0,3293	3
28	2000	288	0,3415	3
29	2004	285	0,3537	3
30	2002	285	0,3659	3

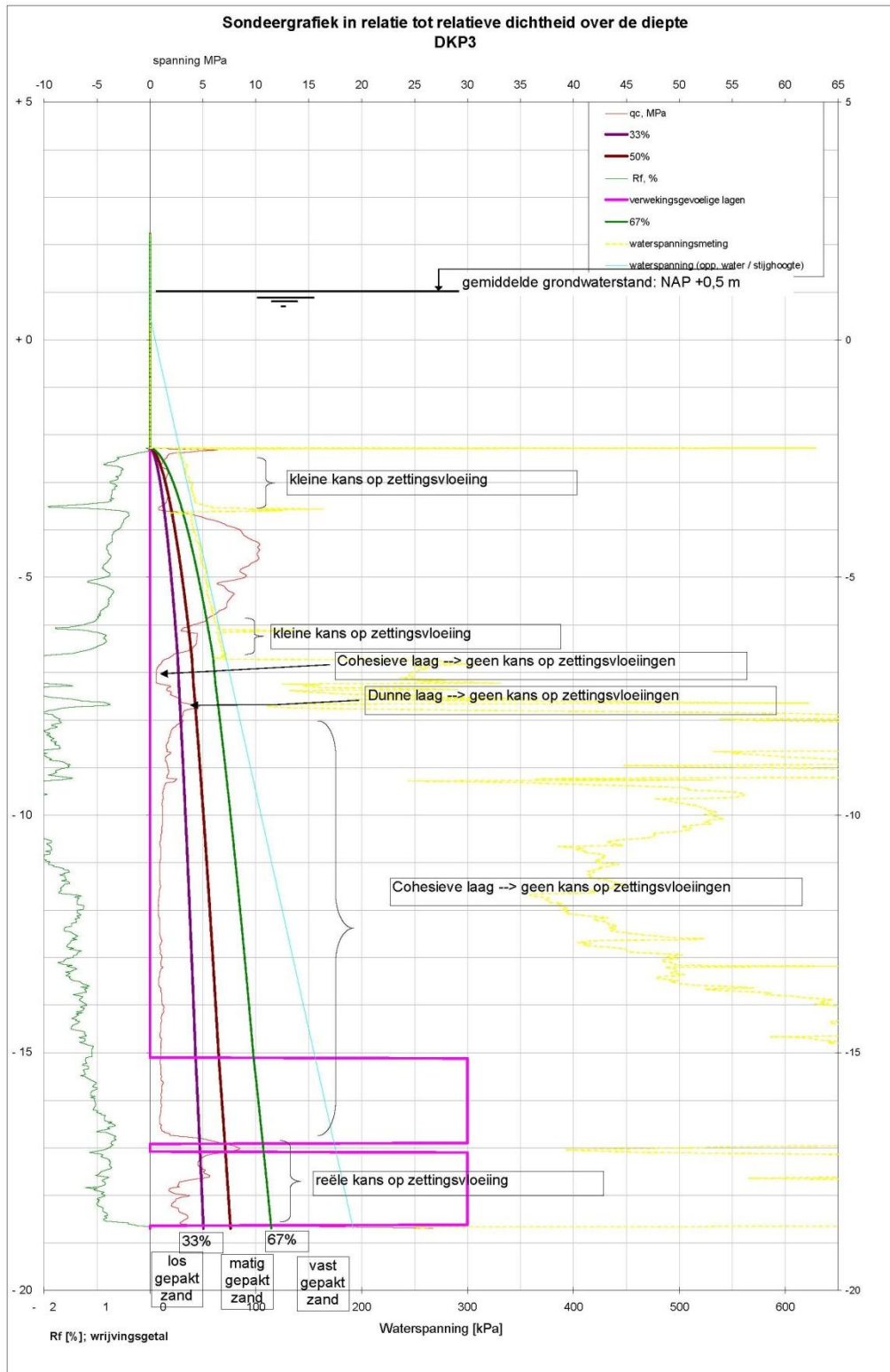
i	Year	Maximum annual water level [m]	Probability of exceedance: p [-]	Return period: T [years]
31	1982	284	0,3780	3
32	1971	284	0,3902	3
33	1943	283	0,4024	2
34	1999	282	0,4146	2
35	1984	278	0,4268	2
36	1979	278	0,4390	2
37	1988	277	0,4512	2
38	1986	275	0,4634	2
39	1974	275	0,4756	2
40	1938	274	0,4878	2
41	1998	273	0,5000	2
42	1965	272	0,5122	1,95
43	2011	269	0,5244	1,91
44	1942	269	0,5366	1,86
45	2003	264	0,5488	1,82
46	1958	262	0,5610	1,78
47	1951	261	0,5732	1,74
48	1941	257	0,5854	1,71
49	1939	257	0,5976	1,67
50	2012	255	0,6098	1,64
51	1985	255	0,6220	1,61
52	1949	255	0,6341	1,58
53	1955	253	0,6463	1,55
54	1935	252	0,6585	1,52
55	2009	248	0,6707	1,49
56	1996	248	0,6829	1,46
57	1964	248	0,6951	1,44
58	2005	245	0,7073	1,41
59	1946	245	0,7195	1,39
60	1987	244	0,7317	1,37

i	Year	Maximum annual water level [m]	Probability of exceedance: p [-]	Return period: T [years]
61	2010	243	0,7439	1,34
62	1956	240	0,7561	1,32
63	1932	238	0,7683	1,30
64	1961	234	0,7805	1,28
65	1963	233	0,7927	1,26
66	1968	230	0,8049	1,24
67	1978	228	0,8171	1,22
68	1992	223	0,8293	1,21
69	1972	223	0,8415	1,19
70	1959	223	0,8537	1,17
71	1947	223	0,8659	1,15
72	1980	222	0,8780	1,14
74	1952	219	0,9024	1,11
75	1997	217	0,9146	1,09
76	1934	214	0,9268	1,08
77	1975	212	0,9390	1,06
78	1957	212	0,9512	1,05
79	1950	201	0,9634	1,04
80	1933	189	0,9756	1,03
81	1937	183	0,9878	1,01

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C. Properties of subsoil



D. Probability of failure of navigation lock – MatLab (Monte Carlo)

The navigation lock at Den Oever can be schematized by water flowing over the lock sill to the Wadden Sea. This situation is present for approximately 10 minutes per lock process whereby water is flowing from the lock to the sea. Because of this flow a bed protection is needed behind the lock in order to avoid instability of the lock due to scouring.

In these situations the scour protection is not intended to prevent scour altogether but only to ensure that the scour hole occurs far enough away from the lock in order to avoid instabilities. That required length of the protection is therefore a function of the expected depth of the scour hole at the end of the protection and the expected upstream slope angle of the hole.

The bed protection length in front of the lock has a length of 150 m which can be seen as the strength. The load depends on the depth of the scour hole (h_s) and increase over time. Together this forms the Z-function:

$$Z = R - S = L_{SP} - (L_n - L_\beta) \cdot h_s$$

The variables will be given by means and standard deviations. Also these variables will be distributed by several distribution types (normal, lognormal, triangular, and etcetera). Putting all these aspects in a MatLab-script and this will determine the probability of failure using the Monte Carlo calculations. Eventually this indicates what the best maintenance interval is economically.

To start with the calculations of the depth of the scour hole over time it's necessary to have some measured parameters. The input parameters which are used for the execution of the MatLab-script are as follows:

Table 12-1: Input parameters for the MatLab-script.

Nr.	Variable	MatLab Name	Unit	Distribution type	Mean μ	Standard Deviation σ
1	ρ_s	Rhos	[kg/m ³]	Normal	2.650	10
2	ρ_w	Rhow	[kg/m ³]	Normal	1.000	3,0
3	Ψ_c	Phic	[-]	Normal	0,03	0,005
4	d_{n50}	dn50	[m]	Lognormal	0,0002	0,00001
5	u_0	udot	[m/s]	Normal	2,0	0,1
6	α	Alpha	[-]	Normal	2,5	0,1
7	h_0	hdot	[m]	Normal	9,0	0,2
8	t	T	[hours]	Triangular	182	180; 184
9	D	D	[m]	Normal	4,0	0,1
10	L	L	[m]	Triangular	150	147; 153
11	L_n	Ln	[-]	Normal	15	1,0
12	L_β	LB	[-]	Triangular	1,56	1,0; 2,0
13	k_r -factor	krfactor	[-]	Triangular	2,0	2,0; 5,0

To calculate with this model two assumptions have been made:

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- The flow velocity is relatively high at the beginning of the locking process and decrease to zero. Here, the assumption has been made of an average flow velocity of 2 m/s.
- The exposure time forms a summation of the time that it takes to lock a ship. Opening the gates will take approximately 10 minutes and this happens 3 times a day which together lead to a fictive continuous process of 7,6 days in one year.

It is obvious that the parameters do have different distribution types and means and standard deviation values. To make these decisions more clear, you will find below an explanation to the choices that are made and to understand why.

Density of sediment: ρ_s

The density of the sediment particles has a normal distribution, because the deviations to the mean value can both be lower and higher without the chance being negative. The mean has been set on the assumed value and this is also the most used and common number. The standard deviation can be determined and is very low due to the fact that the weight can be measured accurately.

Density of water: ρ_w

The density of the water has a normal distribution, because the deviations to the mean value can both be a bit lower or higher (depends on the air pressure, salinity value, etc.) without the chance being negative. Due to the fact that the situation takes place in a river mouth means that one may assume that there is only fresh water. The mean had been set on the most common density of fresh water. The standard deviation is relatively small because the water in the river is nearly fresh water without many deviations.

Shields (stability) parameter: ψ_c

A practical approach gives a value of 0,03, which can be shown in (Schiereck, 2004). The figure 3-6 in this chapter gives the ψ_c against the ratio of k_r/d_{n50} . It can be seen that the value of ψ will vary from 0,02 to 0,04 and therefore one can set the distribution type to normal. The mean value is based in the practical choice. The standard deviation can be calculated by using information that the two sigma boundaries rule gives a probability of 95%. This gives a standard deviation of 0,005.

Median nominal diameter: d_{n50}

The median nominal diameter should always have a positive number (larger than zero) and due to the fact the d_{n50} has a very low value it is preferable to give this parameter the lognormal distribution. This distribution is always larger than zero and does not have a long tail. The mean has been set on the given value which is probably measured by some sediment samples. The standard deviation can therefore also have a small value, because from these sediment samples it may be assumed that the sediments are measured accurately.

Flow velocity on top of the sill: u_0

The flow velocity over the sill depends on many parameters and the sum of several distributions will eventually give a normal distribution. The mean has been set on the given

value which is probably measured in the field by using a boat with a GPS. The standard deviation is relatively very large because of 2 reasons. First that the velocity at the water surface is larger than at the bed, so there is a distribution over depth and the second reason is that the velocity varies over the exposure time of 10 days, what makes it unpredictable.

Dustbin parameter: α

The Dustbin parameter depends on the height of the sill and the water height in equilibrium state. Both parameters are normally distributed so the value of alpha will also have a normal distribution. The mean value has been set on the average number of the ratio of D/h_0 . The standard deviation can be determined by using information that the two sigma boundaries rule gives a probability of 95%. The boundaries of the alpha value are set on 2,3 to 2,7. This gives a standard deviation of 0,1.

The undisturbed water depth of the river: h_0

The water depth depends on some parameters and is therefore normally distributed. The mean has been set on the given value which is measured over several points. It is not very easy the measure the depth very accurately, but the numbers will be sufficient. Therefore the standard deviation is not extreme large but also not that small.

The exposure time: t

The exposure time depends on the construction time and it is more likely that the project will take more time than determined on beforehand. So the exposure time can be seen as a triangular distribution with boundaries from 228 hours to 264 hours. The mean can be set to the given value which is based on the time chart determined by some engineers.

The height of the sill: D

The height of the sill has a normal distribution, because the height can both be higher and lower than the number which has been measured. The mean has been set on the given value which is determined on several places so this will represent a good value. The standard deviation is relatively large due to the fact that it is very hard to construct a sill in a river mouth, where extreme values can occur, with a constant height.

The required length of the scour protection: L

The length of the scour protection is a man-made parameter which can be controlled by the contractor. The contractor prefers an exact length of the scour protection, because to keep the costs as low as possible. On the other hand his company wants maximum safety and therefore it is necessary to construct the scour protection a bit larger than determined on beforehand. The lower boundary is therefore set on $\mu-3$ [m] and the upper boundary is set on $\mu+6$ [m].

The slope of the scour protection: L_n^{50}

The slope of the scour protection is a natural process. The collapsed soil must have an equal or larger slope than 1:25 (flat slope boundary) and it cannot be steeper than 1:6 (steep slope

⁵⁰ (Schierck, 4.3.6 Stability and slides, 2004)

boundary). A reasonable average value is often used like 1:15. The standard deviation does not reach to the upper and lower boundaries and is therefore set on 1:1.

The slope of the scour hole: L_β ⁵¹

The slope of the scour hole can be described by a triangular distribution. Some experiments show that the maximum value for the slope is 2 (upper limit). There is no minimum value determined and therefore it is reasonable to choose a lower boundary of 1. The mean has been set on the calculated value in the hand-calculation.

Equivalent roughness of bottom: k_r ⁵²

The equivalent roughness of the bottom is based on a practical choice. However Van Rijn (1986) also found values like 4 á 5. Therefore it may seem logical to choose a triangular distribution with a lower boundary value of 2 and an upper boundary of 5. The mean will still be based on the practical value which also depends on the Shields (stability) parameter.

Results

Time [years]	Z-function [m]	Probability of failure (MC) [-/year]	Maintenance cost [€]	Risk cost [€]
0	0	0	13.077.700	0
1	52,9	0	16.457.359	0
2	69,8	0	18.914.957	0
3	82,1	0	20.916.845	0
4	92,1	0	22.636.280	0
5	100,7	0,0002	24.159.665	60.000
6	108,3	0,0008	25.537.241	240.000
7	115,2	0,0014	26.801.173	420.000
8	121,5	0,0037	27.973.478	1.110.000
9	127,4	0,0069	29.069.981	2.070.000
10	132,8	0,0170	30.102.493	5.100.000
11	138,0	0,0292	31.080.100	8.760.000
12	142,9	0,0457	32.009.957	13.710.000
13	147,5	0,0662	32.897.821	19.860.000
14	152,0	0,0893	33.077.700	26.790.000
15	156,2	0,1202	34.565.590	36.060.000
16	160,3	0,1539	35.352.680	46.170.000
17	164,3	0,1841	36.112.456	55.230.000
18	168,1	0,2338	36.847.306	70.140.000
19	171,7	0,2619	37.559.300	78.570.000
20	175,3	0,2979	38.077.700	89.370.000

⁵¹ (Schierck, 4.3.6 The slope angle B, 2004)

⁵² (Schierck, 3.2.5 Practical application, 2004)

MatLab-script

```
function [resultMC] = prob vdMeer example
stochast = struct(...
    'Name', {
        'Rhos'...           % [kg/m3]   Density of sediments
        'Rhow'...           % [kg/m3]   Density of water
        'Phic'...           % [-]      Shields (stability) parameter
        'dn50'...           % [m]       Median nominal diameter
        'udot'...           % [m/s]    Flow velocity on top of the sill
        'Alpha'...          % [-]      Dustbin parameter
        'hdot'...           % [m]       Undisturbed water depth of the river
        't'...              % [hours]  Exposure time
        'D'...              % [m]       Height of the sill
        'L'...              % [m]       Required length of the scour protection
        'Ln'...             % [-]      Slope of the scour protection - 1:Ln
        'LB'...             % [-]      Slope of the scour hole - 1:LB
        'krfactor'...       % [-]      Equivalent roughness of bottom
    },...
    'Distr', {
        @norm_inv...        % [kg/m3]   Density of sediments
        @norm_inv...        % [kg/m3]   Density of water
        @norm_inv...        % [-]      Shields (stability) parameter
        @logn_inv...        % [m]       Median nominal diameter
        @norm_inv...        % [m/s]    Flow velocity on top of the sill
        @norm_inv...        % [-]      Dustbin parameter
        @norm_inv...        % [m]       Undisturbed water depth of the river
        @trian_inv...       % [hours]  Exposure time
        @norm_inv...        % [m]       Height of the sill
        @trian_inv...       % [m]       Required length of the scour protection
        @norm_inv...        % [-]      Slope of the scour protection - 1:Ln
        @trian_inv...       % [-]      Slope of the scour hole - 1:LB
        @trian_inv...       % [-]      Equivalent roughness of bottom
    },...
    'Params', {
        {2650 10}...        % [kg/m3]   Density of sediments
        {1000 3}...         % [kg/m3]   Density of water
        {0.03 0.005}...     % [-]      Shields (stability) parameter
        {@logn_moments2lambda 0.0002 0.00001} {@logn_moments2zeta 0.0002 0.00001}}...% [m]
        Median nominal diameter
        {2.0 0.1}...        % [m/s]    Flow velocity on top of the sill
        {2.5 0.1}...        % [-]      Dustbin parameter
        {9 0.2}...          % [m]       Undisturbed water depth of the river
        {180 182 184}...    % [hours]  Exposure time
        {4 0.1}...          % [m]       Height of the sill
        {147 153 150}...    % [m]       Required length of the scour protection
        {15 1}...           % [-]      Slope of the scour protection - 1:Ln
        {1 2 1.56}...       % [-]      Slope of the scour hole - 1:LB
        {2 5 2}...          % [-]      Equivalent roughness of bottom
    },...
    'propertyName', { % specify here to call the z-function with propertyname-propertyvalue
pairs
    true...                % [kg/m3]   Density of sediments
    true...                % [kg/m3]   Density of water
    true...                % [-]      Shields (stability) parameter
    true...                % [m]       Median nominal diameter
    true...                % [m/s]    Flow velocity on top of the sill
    true...                % [-]      Dustbin parameter
    true...                % [m]       Undisturbed water depth of the river
    true...                % [hours]  Exposure time
    true...                % [m]       Height of the sill
    true...                % [m]       Required length of the scour protection
    true...                % [-]      Slope of the scour protection - 1:Ln
    true...                % [-]      Slope of the scour hole - 1:LB
    true...                % [-]      Equivalent roughness of bottom
    } ...
);

%% main matter: running the calculation
% run the calculation using Monte Carlo
resultMC = MC(...
    'stochast', stochast,...
```

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```

    'NrSamples', 1e5,...
    'x2zFunction', @prob_vdMeer_example_x2z);

%% Z-function
function z = prob_vdMeer_example_x2z(varargin)

%% create samples-structure based on input arguments
samples = struct(varargin{:});

%% calculate z-values
% pre-allocate z
z = nan(size(samples.Rhos)); % loop through all samples and derive z-values
for i = 1:length(samples.Rhos)
    Delta = (samples.Rhos(i) - samples.Rhow(i)) / samples.Rhow(i); % [-] Relative density
    R = samples.hdot(i); % [m] Hydraulic radius
    C = 18*log((12*R)/(samples.krfactor(i)*samples.dn50(i))); % [m^(1/2)/s] 'Smoothness'
    coefficient according to Chezy
    Uscour = samples.udot(i)*(samples.hdot(i)-samples.D(i))/samples.hdot(i); % [m/s] Velocity
    at scour hole
    Ucrit = sqrt(samples.Phic(i)*Delta*samples.dn50(i)*C^2); % [m/s] Critical velocity
    hs = (((samples.Alpha(i)*Uscour-Ucrit)^1.7*samples.hdot(i)^0.2)/(10*Delta^0.7))*samples.t(I
)^0.4; % [m] Scour depth
    z(i,:) = samples.L(i)-(samples.Ln(i)-samples.LB(i))*hs; % [m] Z-function
end

```