令和元年 6 月 26 日

蘭国「RWS INFORMATIE –Werkwijzer Ontwerpen Waterkerende Kunstwerken – Ontwerpverificaties voor de hoogwatersituatie Groene versie 2018」(RWS INFORMATION – Werkwijzer (Work pointer) Design Waterkerende (water turning) Artworks - Design verifications for the high water situation Green version 2018)仮英訳

https://www.helpdeskwater.nl/publish/pages/158168/werkwijzer_ontwerpen_waterkerende_kunstwerken_-_groene_versie_november_2018.pdf (参照：2019/04/12)

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表紙

RWS INFORMATION –

Werkwijzer (Work pointer) Design Waterkerende (water turning) Artworks - Design verifications for the high water situation

Green version 2018

Date　November 1, 2018
Status　Definitive - green version

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Colophon

Issued by　RWS-WVL Water barriers

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Format
Date November 1, 2018
Status Final, green version
Version number D2

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1 Introduction

1.1 General

The vulnerability of the Netherlands to flooding is generally known and one the strong system of (primary) water defenses is therefore unanimously regarded as one basic condition for Dutch society. The Flood of 1953 and the near-floods in 1993 and 1995 show that water safety is continuous attention.

Most water defenses are of land. Artworks deviate in this sense that they are for a large part made up of other materials such as steel, concrete, wood, masonry and plastic. Artworks are used for other purposes water-retaining structures such as shipping, traffic and water management, but have to naturally comply with the legal standard for water defense.

The Guide for Art Works will be used for the design of water-retaining structures [Ref. 2.2] has been widely used to date. The Guideline Art Works dates from 2003 and is therefore designing flood defenses at the appearance of this Werkwijzer (work pointer) works of art about 15 years old. The most important reason for this work pointer to be changed is the legislation where an exceedance probability standard applies switched to the flood probability standard. In addition, the building regulations (Building Decree) has been adapted and for building constructions the work is currently being done with the so-called Eurocodes. Of course also give 15 years of development flood defense technique and 15 years of experience with the Guidelines for Art Works 2003 cause for adjustments and improvements.

The desired safety level of hydraulic engineering works with a flood defense function is anchored in the Water Act and the Building Decree. On both must be met and therefore the strictest is decisive. Because both laws based on different starting points are a direct mutual comparison is not always easy. This is particularly important constructive failure mechanisms. Chapter 7 provides practical handles given how to deal with this.

1.2 Goal and target group of the work pointer

This work pointer has as goal: offering methods with which the current knowledge and current tools can be verified in a practical way design of a hydraulic engineering work in a primary flood defense appropriate management and maintenance throughout the life of the law required water retaining capacity.

The work pointer is particularly focused on the designer / constructor with several years of work experience and also on the person who draws up the design specifications to which a work of art must comply.

Comments:
(1) The work pointer provides handles for the verification of a design based of the requirements of the Water Act / Building Decree, including the necessary for this schematics, but is not a manual for the integral design of a water-retaining artwork. Some general indications are given in 0 the course of a design process.

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(2) In the work pointer, therefore, there are no formulas that, based on the requirements, lead to the optimal dimensions or specifications of artwork parts. If to use the work pointer in this sense, one first has to use the estimate design quantities, calculate whether they meet and if necessary adjust estimates.

(3) The verification of a design has many similarities with the verification (assessment) of an existing construction. Yet there are important ones differences.

- Designs will continue to be looked into in the future. That's why it has to be with it design is taken into account (to a greater or lesser extent difficult quantifiable) uncertain aspects such as climate change, degradation, aging and settlements within the chosen design life.
- In design, other functions and aspects are also the only ones water safety (which, moreover, is outside the scope of this work pointer falls).
- Assessment is based on the existing characteristics of the artwork, while the artwork is designed for designs. As a result, uncertainties can be handled differently.

1.3 Scope

The scope of this work pointer is limited to water-retaining structures (including denominations, safety locks, drainage locks and barge locks, inlet and outlet locks, pumping stations). The work pointer does not deal with management (intersection) and, tunnels, longitudinal constructions or the large storm surge barriers, although the latter can fall within the validity range of some mechanism models.

The connections of these structures to the adjacent dike sections (Connection constructions) and Transition constructions (for example from the soil protection or the dike coverings on the work of art) are also up to it scope.

This work pointer is about designing and looking forward to the whole design lifespan (taking along, for example, sea level rise and soil subsidence). This means there are common ground with the Design Instrumentarium (including the Foundations for flood protection) but also with it Statutory Assessment Instruments (WBI). The management and maintenance of works of art are only discussed in general terms in this work pointer. For this, reference is made to the essays from the Guidelines for Art Works 2003 about this subject.

Obviously, an overall design verification must be broadened looked at the water-retaining function that is central to this guideline. In many it is up to the designer to make suitable preconditions and determine reliability requirements.

Because of the multidisciplinary character of the design of artworks too other laws, standards, guidelines and rules are important, such as for example the Machinery Directive for drive systems. These fall outside the scope of this Work pointer.

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1.4 Relationship with Guide to Works of Art 2003

In this work pointer the method is from appendices B1-B5 and B7-B9 of the Guideline Artworks 2003 [Ref. 2.2] adapted and brought into line with the probability of flooding and the current state of the art. The basic ideas underlying chapters 2 to 5 and Appendix B6 from the Guidelines for Art Works 2003 can in principle still be used. However, it must be borne in mind that the texts from these chapters are correct parts have been overtaken by time and no longer literally apply.

1.5 Status of this work pointer: green version

This document is a green version. The instruments that work with are relatively new. The proposed methods and methods have been tested at a limited number of cases. Therefore suggestions for improvement and / or optimization of the suggested methods of working. These can be provided via the Helpdesk Water.

1.6 Reading guide

In chapter 2, starting from the Water Act and the foundations for high water protection [Ref. 2.1], discussed the system in which the artwork functions as part of the primary flood defense and the potential flood situations which lead to the crossing of the basin power. On the basis of the main error tree for a flood defense, the probability of failure for the artworks. The assignment will be entered of this failure probability space to the different failure mechanisms (do not close, height, piping and structural failure).

Chapter 3 then discusses the hydraulic loads of apply when designing flood defense structures. Here the main instruments discussed and climate scenarios and other surcharges on the water level.

Chapters 4 to 7 then fail the failure mechanisms closing, height, piping and structural failure further elaborated. At the failure mechanism structural failure is also discussed the role of the Building Decree and the Eurocodes invoked therein.

Chapter 8 provides an initial impetus for the design of Connection Structures.

Chapters 9 and 10 give further information about cup storage and erosion resistance of soil protection as it is particularly important for the failure mechanisms do not close and height (chapters 4 and 5).

Finally, a case is discussed in chapter 11: a design verification for a lock.

Appendix A deals with general information about the design process. Appendix B is an addition to chapter 4. Appendices C, D and E belong to chapter 7 Constructive to fail. Appendix C deals with the verification of the construction in the few occurring situation that the own weight is dominant in the high water load situation. Appendix D deals with the Goda model for the determination of wave loads on constructions and Appendix E gives a concise step-by-step plan for a fully probabilistic verification.

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All references mentioned in this work pointer and not free or against payment are available, together with the work pointer uploaded on the Helspdesk Water.

1.7 References and background documents

[Ref. 2.1] Foundations for flood protection, Expertise network Water safety, Second revised edition, November 2017

[Ref. 2.2] Guidelines for Art Works 2003, TAW, May 2003

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2 Design verifications based on flood probability standards

2.1 Water Act and Building Decree

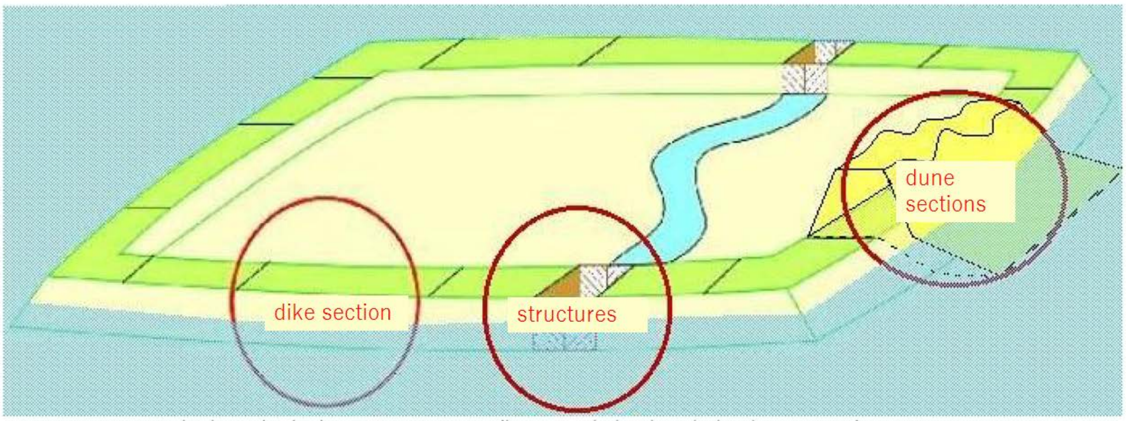
A design verification involves the assessment of whether a design meets the requirements. This concerns both usage requirements, such as requirements on the clearance height, as legal safety requirements. In this chapter the latter requirements centrally.

Works of art that are part of a primary flood defense must both meet the standards from the Water Act as well as the requirements of the Building Decree. The reliability requirements that become new construction under the Building Decree stipulated in the NEN-EN1990/NB Principles of the constructive design.

In this chapter (and also chapters 3-6) only the standards are discussed from the Water Act. The requirements from NEN-EN1990/NB are only relevant for verifications of structural safety. These requirements are included in chapter 7 Constructive failure addressed.

2.2 Flooding: exceeding the storage capacity

A water-retaining work of art does not stand alone. It is part of a dyke track that can consist of dyke bodies, quays, dunes and structures. The dike section forms the flood defenses for a high water discharge in its totality protect the underlying area. In this underlying area there is a certain degree of concomitant ability present.



※（Rijkswaterstaat, Central government　2018）15 頁より作成。

Figure 1 Artwork as part of a system (source: Safety Netherlands in Map 2)

The Water Act sets a flood probability per year[1] on a dyke project. The probability of flooding of a trajectory should be smaller than this flood probability. The concept of storage capacity can be derived directly of the definition of the probability of flooding as it is in the Water Act (Article 1.1) defined:

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[1] Opportunities per year are discussed in the context of the Water Act. This is actually a chance of failure in a continuous period (reference period) of 1 year. Where chances in this work pointer are concerned, this concerns chances of failure in a certain period of time, the reference period. That is why no dimensions are presented with regard to the opportunities.

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Flood probability is the chance of loss of water retaining power of one dyke route so that the area protected by the dyke section is so floods that fatalities or substantial economic damage originated.

Between the loss of water retaining power and the flooding of it protected area that are fatalities or substantial economic damage is the capacity for storage. The definition of the coming together power is:

> The capacity to store water is the loss of waterpower can still be stored in the underlying system without this leads to fatalities or substantial economic damage.

The water retaining capacity of the flood defense and the combining capacity of the protected area thus together determine the probability of flooding dyke route. It should be noted that loss of water retaining capacity must be made read as both failure due to failure of the flood defense and failure without failure of the flood defense (see explanation in section 2.3.1).

For substantial economic damage can be connected in the first instance with the practical guidance given above in the Foundations for high water protection [Ref. 2.1] is done:

> Substantial economic damage occurs when the average water depth at least one area or neighborhood with the same four-digit zip code (based on the district and

neighborhood map of the CBS) is greater than 0.2 m.

If desired or required, the above criterion can be substantiated deviated.

2.3 Failure mechanisms and resulting flood situations

A flood is the result of the loss of water retaining power should be read as failure due to collapse of the flood defense or failure without failure of the flood defense. The events that lead to the loss of water retaining capacity are described through failure mechanisms. That's how it is available storage capacity is exceeded by, for example, the collapsing of a work of art or dyke body, through transfer over a work of art or by inflowing into a closed and not collapsed work of art. In practice an important difference between artworks and dyke bodies that the chance of one flooding without the failure of the flood defense in dike bodies is very small is. This may be different for artworks.

2.3.1 Flooding on the site of works of art

Artworks always have a watercourse behind the artwork, except for denominations. This waterway usually forms part of a trial system which has secondary barriers. A work of art can be done in different ways failure, with a flood as a result:

A) Flooding without failure: A work of art can be so robust designed to ensure that large quantities of water are closed or not to be able to enter the bosom system without the artwork succumbs. When the maximum farm level is reached and a secondary barrier collapses, the chimney breast empties in the polder in a short time. **(17 頁)** Since outside water will continue to flow in via the artwork and the succumbed secondary capacity, the storage capacity will be exceeded. In this case, we speak of a flood without the collapse of it artwork as a result of a shortage of combining capacity as then there is fatalities and / or substantial economic damage in the underlying area[2].

B) Flooding due to failure with the occurrence of a progressive breach: when the artwork is unable to construct the inflowing flow constructively to resist it will collapse, leading to a breach. When then the flow rate of the inflowing water becomes so large that the adjacent floods also collapse is a progressive breach. In that case the combining capacity will almost certainly be exceeded and the precise one size no longer important. In that case, we speak of a flood due to failure of the artwork.

C) Flooding caused by failure with the occurrence of a limited breach: In case of constructive collapse where a breach arises with a limited scope, the volume of inflowing water remains limited (er). In that case the size of salvage capability important for the chance of failure after constructive succumb. This situation is less common than the situation in which one such a large gap is created that in any case there will be one flood. It is assumed that exceeding of the coming power almost certainly occurs after structural failure, conform flood situation B.

2.3.2 Flooding in dike bodies

Dike bodies generally have a toe ditch inside the dike that connects to a system of ditches lying in the ground level (so no quays around it).Depending on the quality of the turf, dike bodies can transfer through put of order 10-100 l/m/s resist without collapsing. Although such flows over large dike stretches result in a large inflow volume, the consequences often limited as long as the dike does not collapse. The same goes for the inflow due to seepage water. The system of inner-dike water ways will gradually fill and then water will come to the ground level. Leaving exceptions, wave overtopping and seepage water leads to flooding, but not resulting in flooding with substantial damage or casualties. Naturally, the determination of the storage capacity must be taken into account be kept with transshipment over dyke bodies and seepage water.

In practice one speaks of dike bodies of a flood in the case of the collapse or eroding of the defense, leading to a progressive breach. This is rarely made explicit in design guidelines and WBI documentation. It is assumed that the probability of exceeding the storage capacity after the occurrence of a breach is practically equal to the probability of a breach. This seems strongly on how to deal with structural failure in artworks.

2.4 Practical choices with regard to cup storage in designing artworks

The way in which the available storage capacity in the design process of a water-retaining artwork plays a role can vary per failure mechanism.

[2] An additional consequence is that after the drainage of the chimney breast, the decay over the artwork in question increases sharply and the artwork can still collapse with the possibility of a progressive breach in the primary flood defense.

(18頁)With some failure mechanisms, the size of the storage capacity is one direct design parameter and with other mechanisms is not recommended. Below are some points of attention with regard to dealing with cup storage in the design process.

2.4.1 Bowl storage; not closing, transfer and / or overflow

Particularly in the verification of the design with respect to non-closing and transfer and / or overflow, the combining capacity can be an important design parameter. In these failure mechanisms, it is generally advisable to design the artwork in such a way that flooding (transfer and / or overflow) or inflow (not closing) without collapse of the structure (flood situation A in section 2.3.1) is decisive for the failure probability. The consequences of unwanted inflow without the structural failure of a work of art are more manageable than the consequences of breaching and possibly breach growth. In addition, it is relatively easy to design a work of art so robust that overflow volumes in closed condition and through flowing water can be structurally resisted in the non-closed state. In general, the construction (concrete, steel) of the artwork is itself sufficiently robust from its primary function (s) (passing through shipping, drainage, etc.) in order to withstand the current flow load.

An exception to the above concerns the design of the soil protection. This is often heavily loaded by larger flow and flow rates due to the flow rates then occurring. For the vast majority of works of art, adjusting soil protection at such high flow rates does not lead to the most efficient design. This is because it is usually (much) cheaper to increase the deflecting height or to place an additional reticle than to strengthen the soil protection.

2.4.2 Bowl storage; piping and structural failure

In design verifications for piping and structural failure, it is less obvious to use the combining capacity as a direct design parameter. In both cases, the flood defense is closed at high tide and (part of) the construction will have to collapse before large volumes of water flow in and thus claim the storage space (flood situation B in section 2.3.1). Although the collapse of the artwork itself is not equivalent to a flood, it is in most cases not recommended in the design to look for the edges of the permissible and as a failure criterion to choose the overtaking capacity. The recommended approach is therefore to include the failure probability with respect to piping and structural failure to the chance of the structural failure of the artwork itself and the likelihood that the volume of inflowing water is subsequently greater than the existing storage space equals to 1. The failure criterion becomes identical to a failure criterion appropriate to the NEN-EN1990 / NB Principles of the structural design.

2.5 The probability of flooding in the Water Act

Two different standards for the primary have been included in the Water Act flood defenses:

1. Signaling value.
2. Lower limit or maximum permissible value.

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In the **statutory assessment** of the existing primary flood defense systems both the signaling values as the maximum permissible flooding probabilities of importance. The reference period for both chances is 1 year. In the legal assessment is the expected situation in the last year of the assessment round.

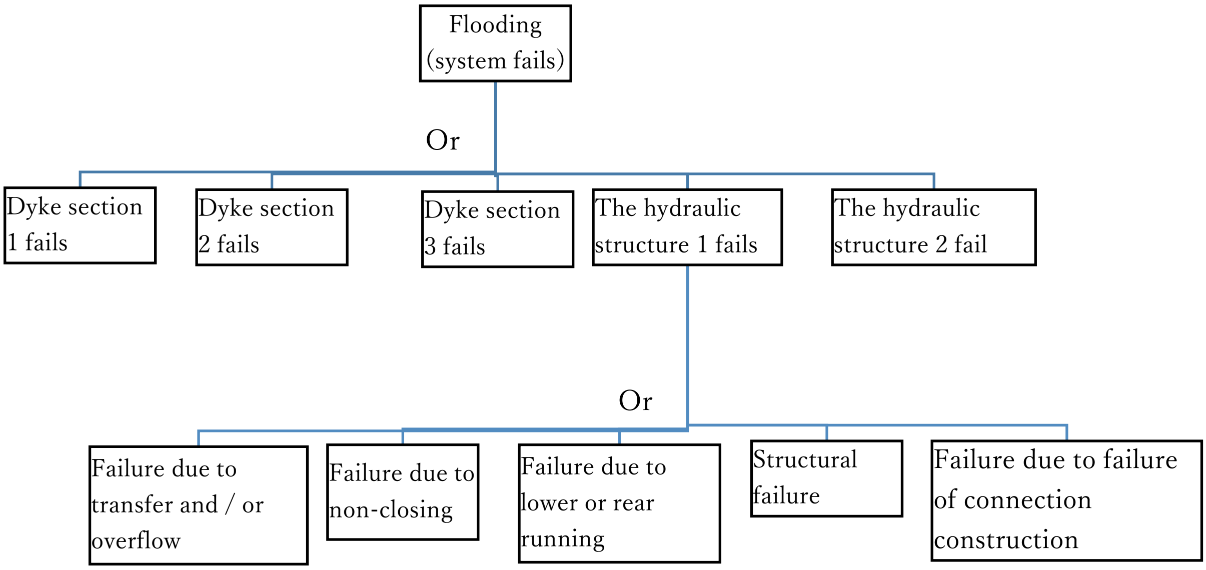
The **design** verification checks whether the requirement that the failure probability must be less than the maximum in each year of the intended life permissible failure probability. Usually this will be the last year because there is long-term effects such as sea level rise and material degradation. On this time aspect is discussed in more detail in section 2.6.4. The signaling values are not important for design verifications.

For further explanation on the different types of flooding or failure probabilities included in the Water Act reference is made to paragraph 4.4 of the Foundations for flood protection ([Ref.1.2]). The flood probability norms themselves can be found in various places such as https://waterveiligheidsportaal.nl/#/home or wetten.overheid.nl/BWBR0025458/2018-07-01#BijlageII respectively wetten.overheid.nl/BWBR0025458/2018-07-01#BijlageIII.

2.6 From flood probability standards per route to reliability requirements per failure mechanism for individual works of art

2.6.1 A route as series system

The maximum permissible probabilities of flooding from the Water Act do not relate to individual structures or dike sections, but to trajectories. A dyke project can be regarded as a series system. A series system is as strong as the weakest link: if one link fails, the system fails. The probability of flooding is thus equal to the probability that at least one of the components of the route will fail. This relationship is schematically shown in the error tree in Figure 2.



※（Rijkswaterstaat, Central government　2018）19 頁より作成。

Figure 2 Display of a trajectory as a series system in an error tree.

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The chance that a work of art fails will contribute to the probability of flooding of a project. A work of art fails if more water enters the area through or over the artwork than can be stored there without substantial damage or casualties. This can have several causes. These causes are also called failure mechanisms. Important failure mechanisms for artworks are:

- Transshipment and / or overflow;
- Non-closing;
- Piping (also called bottom and rear running);
- Structural failure;
- Failure of connection construction.

The probability that somewhere a work of art or construction component fails is in practice greater than the chance that one specific work of art or construction component fails. This phenomenon is also called the length effect. In order to be able to assess the reliability of an individual artwork or construction component, a requirement at the object or component level must be derived from the requirement at the level of the trajectory.

To arrive at a requirement for a particular failure mechanism for an individual artwork or construction component, the following steps must be taken in sequence:

1. Determine a reliability requirement for the relevant failure mechanism at route level.
2. Translate the reliability requirement for the relevant failure mechanism at trajectory level to a reliability requirement for the considered artwork or construction component.

2.6.2 Step 1: Determine failure probabilities per failure mechanism at route level

Failure probes at the trajectory level can be derived by dividing the flood probability standard over the different failure mechanisms. This breakdown is also called the failure probability budget. A default failure probability budget has been drawn up for WBI2017 (see [Ref.2.2]) and OI2014v4 ([Ref.2.3]). This is shown in Table 1. It is permissible to deviate from this default failure probability budget in order to avoid unnecessarily restricting requirements for certain failure mechanisms.

It is recommended to evaluate at an early stage of the design process if optimization of the failure probability budget is useful. It should also be realized that the failure probability budget applies to an entire dyke project and that only major shifts within the failure probability budget are of practical significance.

Table 1. Default failure probability budget from WBI2017 and OI2014.

Type of barrier	Failure mechanism	Sandy coast	Other (dikes)
Dike or artwork	Overflow or wave overtopping	0%	24%
Dyke	Cracking and piping	0%	24%
	Macro instability	0%	4%
	Damage to the	0%	10%

	covering and erosion of the dyke body		
Artwork	Not closing	0% *	4%
	Piping	0% *	2%
	Structural failure	0% *	2%
Dune	Dune exit	70%	0% (10%) **
Other		30%	30% (20%) **
Total		100%	100%

※（Rijkswaterstaat, Central government　2018）20 頁より作成。

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* Many sections along the sandy coast do not contain any works of art or dikes. If this is the case, then will have to deviate from the default failure probability budget.

** In the case of journeys consisting of only a part of dunes, the dune catch will often be a relatively small one contribute to failure probability. In the default failure probability budget in such cases a part of the post 'other' assigned to dune catch.

For more background to the failure probability budget, reference is made to Section 5.5.1 of the Accounting Policies [Ref. 2.1].

2.6.3 Step 2: Determine failure probabilities per failure mechanism for an individual artwork or construction component

The difference between a failure probability for a trajectory and the corresponding failure probability for an individual artwork or construction component is determined by the length effect. The length effect differs per failure mechanism. The length effect is small in the case of a failure mechanism such as transfer and / or overflow. In this mechanism the uncertainty with regard to the hydraulic load is dominant and it is spatially strongly correlated. The length effect is great for a failure mechanism such as piping. In this mechanism, the uncertainty with respect to the substrate properties is of relatively great importance and these properties are spatially very variable.

In principle, the failure probability per failure mechanism for a particular artwork or construction component can be determined on the basis of a probabilistic calculation for the trajectory. However, that is relatively laborious. For this reason, fixed length-effect factors have been included in the OI2014 ([Ref.2.3]) and in the WBI2017 ([Ref.2.2]) with which a failure probability for an individual artwork or construction component can be directly derived. In general, it can be written:

$$P_{eis,kw} = \frac{P_{eis}}{N} = \frac{P_{max} \cdot \omega}{N} \qquad 2.1$$

In which:

$P_{eis,\,kw}$	Failure probability for the failure mechanism considered for an individual artwork per year [-]
P_{eis}	Failure probability for the considered failure mechanism at route level per year [-]
P_{max}	Maximum permissible probability of flooding of the dike section (defined as a lower limit by law) per year [-]
ω	Failure probability factor for the relevant failure mechanism [-]
N	Long-term effect factor for the failure mechanism considered

Table 2 includes longitudinal effect factors from the OI2014v4 and the WBI2017 in order to arrive at reliability requirements for individual artworks. It is emphasized that these are first approximations. The values from Table 2 must be viewed in the light of the gradual transition to a probabilistic method in assessing the reliability of flood defenses. At one probabilistic method, failure probabilities per structure, dike and dune section are calculated and combined into failure probabilities at, for example, route level. This provides insight into which failure probability performance is desirable, so that the necessity disappears to work with previously estimated length-effect factors. It is important, however, that sufficient margin is retained in the design in order to be able to accommodate future changes within the process, such as the construction of more works of art in the process.

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It is advisable to work from coarse to fine in the design and only sharpen the length effects if that yields substantial savings. The values from the WBI2017 must be regarded as minimum requirements. This document connects to the approach taken in the OI2014v4.

Table 2 Length-effect factors in the OI2014v4 and WBI2017. This document can be regarded as a further elaboration of the OI2014v4 for works of art.

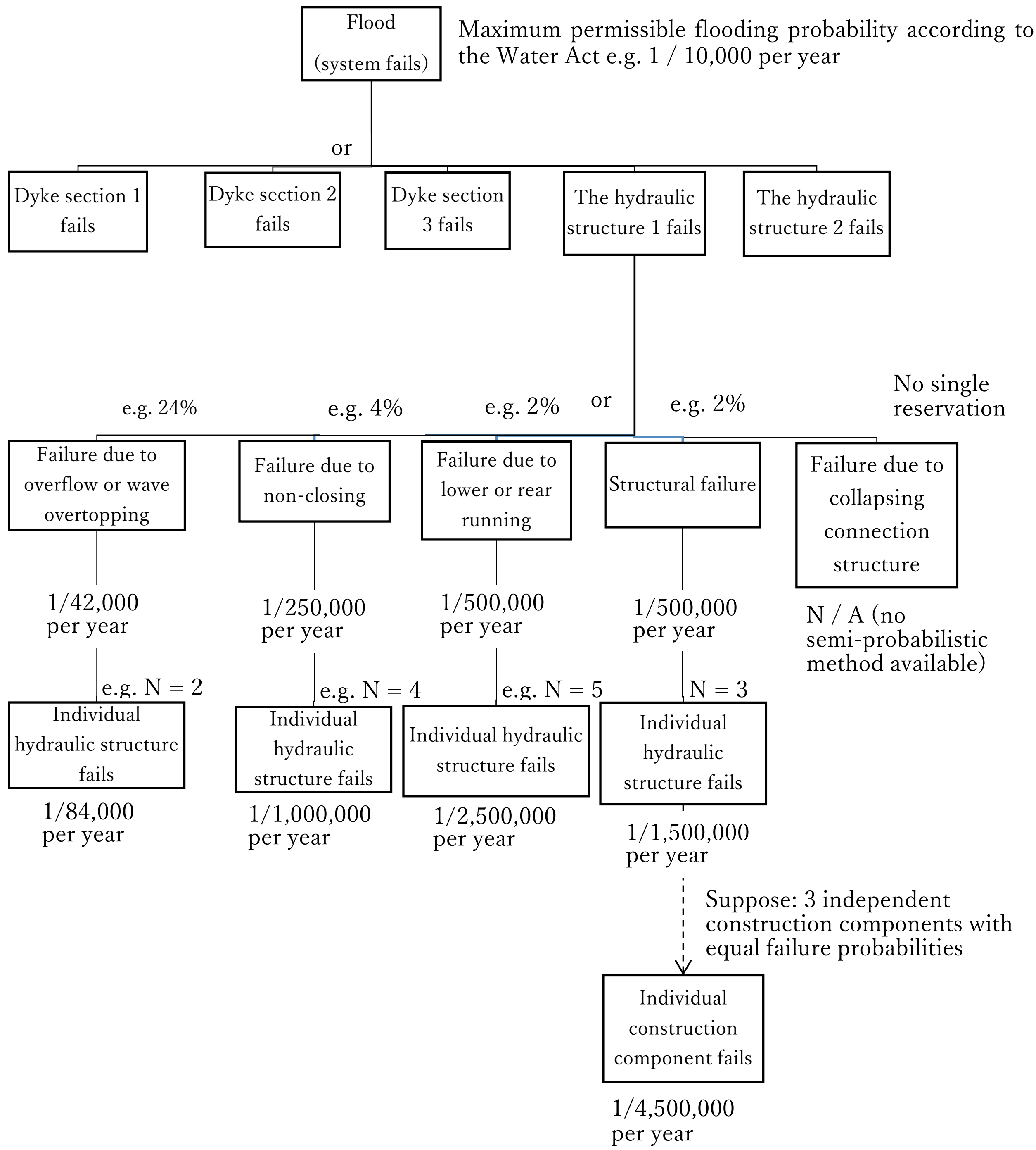
	OI2014v4(held here)	WBI2017(for information)
Overflow or wave overtopping	N = 1-3 (depending on the route, see Appendix A from OI2014v4)	N = 1-3 (depending on the route, see Scheme of Grass Coverings ([Ref 2.4]))
Do not close	N = min (n_{kw}; 10) With: n_{kw} Number of artworks in process where non-closing is relevant failure mechanism and closures minus or more independent from each other to fail (-). Possible to refine on the basis of probabilistic analyzes	N = max (1, 0.5 × $n_{kw,2a}$) With: $n_{kw,2a}$ Number of works of art whose probability of failure is not negligibly small according to the simple test (-). In other words: the number of artworks that could not be easily approved with regard to the reliability of the closure, so that a detailed assessment is required.
Structural failure	In the OI2014v4 there is no longitudinal effect factor recorded but is advised to start from CC3 from the NEN-EN1990. This Guide is recommended to start from N = 3	N=3
Piping (under and rear running)	N = min (n_{kw}; 10) With: n_{kw} Number of artworks in process where piping is relevant failure mechanism is (-). Possible to refine on the basis of probabilistic analyzes. It should be noted that this is for the time being only applies to the model from	There are no regulations in the WBI2017 recorded for under and backwardness with an explicit relationship with a goal reliability

	Sellmeijer. For the rest models are not prescriptions included an explicit relationship with a goal reliability (see chapter 6).	

※（Rijkswaterstaat, Central government　2018）22 頁より作成。

The length-effect factors from Table 2 serve to derive failure probabilities for individual artworks. Sometimes a further breakdown of these failure probabilities over different physical or process components is required.（23 頁）For example, the probability per year of the failure of the alert must be less than the maximum probability per year of the failure of a work of art due to non-closure. After all, a closure can also fail due to other causes, see chapter 4. Also in the assessment of the structural safety of construction components a stricter requirement must be adhered to than the failure probability for structural failure at art level if there are independent construction components with more or less equal failure probabilities. This is discussed in more detail in Chapter 7.

Figure 3 shows a numerical example of the derivation of the reliability requirements of the standards from the Water Act for the purpose of design verifications of works of art.



※（Rijkswaterstaat, Central government　2018）23 頁より作成。

Figure 3 Numerical example for the derivation of reliability requirements of a maximum permissible flooding probability according to the Water Act for an individual artwork and / or construction component.

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2.6.4 Alternative verification methods

The failure probability budget and the length-effect factors are only tools for verifications based on the Water Act. They make it possible to carry out verifications per failure mechanism for individual artworks. In some cases, however, it may be more accurate or convenient to perform a verification not per failure mechanism and per artwork, but to consider different failure mechanisms and / or works of art collectively.

If there is a strong dependency between failure mechanisms, verification per failure mechanism can be unnecessarily conservative. The dependence between failure mechanisms can be included in a design verification by:

1. To add the failure probability for the failure mechanisms in question
2. The chance to determine that at least one of these failure mechanisms occurs.

The failure probability budget therefore remains the same, at most the percentages for the conscious failure mechanisms are no longer considered separately. The dependency between the failure mechanisms is taken into account on the side of the failure probability calculations: it ensures that the probability that at least one failure mechanism occurs is less than the sum of the probabilities per failure mechanism.

In addition to refinement by more accurately incorporating the dependencies between failure mechanisms, design verifications can sometimes also be refined by taking the interdependencies between artworks more accurately. This can be useful if:

1. A default length-effect factor is found to be excessively conservative and / or
2. Otherwise the interactions between artworks cannot be properly modeled.

If a default length effect factor is found to be too conservative, then the failure probabilities per artwork for the conscious failure mechanism can be combined into a failure probability at the trajectory level. This chance of failure can then be compared directly with the failure probability at the level of the trajectory. A breakdown of the failure probability at the trajectory level about the individual artworks by means of a length-effect factor is then no longer necessary. One could also use the result of a failure probability analysis for a series of existing artworks to further determine the requirement for a new artwork.

It is also possible that there are interactions between the (failure) behaviors of artworks that make it difficult to assess works of art separately. Consideration can be given to situations in which the consequences of transhipment over one artwork depend heavily on the quantity of transhipment that takes place over the other works of art. A possible solution is then to determine the inflow volume as a function of the outside water level for all the works of art together.[3] Then the probability can then be calculated that the storage capacity is exceeded. This opportunity must meet the requirement at the turning height at the level of the route. Such a method can be considerably more practical and accurate than the splitting of the storage capacity over the individual structures.

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[3] If the turning height of the structures is partly determined by whether or not closure means are used, then the chances of the various possible inflow flows can be given as a function of the outside water level. By combining this result with the probability distribution of the outside water level, the probability of exceeding the storage capacity can then be obtained. This is then the probability of failure due to either overflow or non-closure (assuming that failure due to scouring is not normative).

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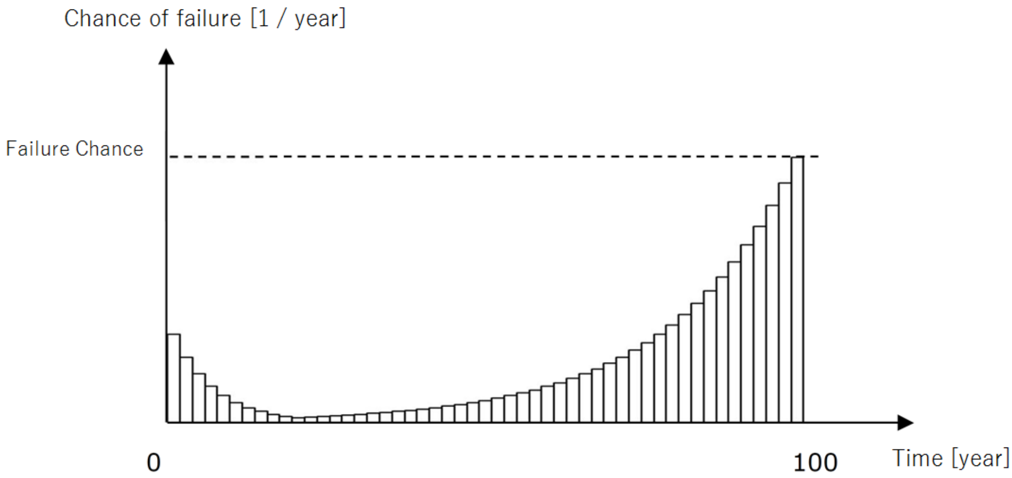
2.7 Development of strength and load over time

There are no specific reliability requirements for the design in the Water Act as in the NEN-EN1990 / NB *Foundations of the structural design*. Well is indicated which requirement a route must at least meet. Each trajectory is periodically assessed on the basis of this requirement. The parts of a construction are therefore designed in such a way that they are (expected or even with a high level) degree of certainty) in each year during the intended design life requirements from the Water Act.

Initially the failure probability of a new construction can be reduced by proven strength. Without interference, the chance of failure will always increase over time processes such as aging, degradation and relative mirror rise. Often the probability of failure in the last year is normative (see Figure 4)



※（Rijkswaterstaat, Central government　2018）25 頁より作成。
Figure 4 Illustration of a design based on the maximum permissible flooding probability from the Water law. The chance of failure must be smaller than the requirement in each year of the design life. The shown probability of failure corresponds to a construction that meets the failure probability for 100 years.

In practice it is uncertain how long a particular construction meets the requirements of the Water law will comply. That is because the development of the strength and the load cannot be predicted perfectly in time. In addition knowledge development and the introduction of new models lead to adjustment of the image of the reliability of a flood defense. Also these changes are surrounded with uncertainty. With these uncertainties has to be with the design taken into account.

The conservatism in the starting points with regard to the decrease in strength and increasing the load over time determine the probability that a flood defense will sooner than intended to exceed the requirements of the Water Act. Advised to the dealing with the uncertainties regarding the time to exceed the assess the standard on the basis of, for example, a scenario analysis in which the effects of alternative future developments (such as different ones) sea level rises) is checked.

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With relatively easy-to-adjust construction components, the optimal design durability primarily determined by the technical feasibility and the life-cycle cost (LCC). In that case, it is advisable to start from design verifications the expected decrease in the strength and the expected increase of the load time, roughly in line with the G or G + climate scenario. This matches an LCC analysis in which the time to reject as expected value or deterministic variable.

With hard-to-adjust construction components such as foundations and concrete constructions often determine the user functions the optimal design life. With such construction components, it is advisable to reduce the chance of the interim need to replace or strengthen. This can be done by to use conservative principles with regard to the decrease of the strength and increase of the load over time. An example is handling a W + climate scenario.

Management and maintenance affect the development of the probability of failure over time. These activities therefore affect the margins associated with the design must be adhered to future changes in the failure rate of the flood defense. This also applies to conditions of use that ensure that the chances of certain taxes are small or absent.

2.8 Authentication methods

To assess whether the reliability of a work of art is sufficient both probabilistic, semi-probabilistic and deterministic methods available. For more backgrounds in these methods, please refer to section 5.6 of the *Principles for flood protection* ([Ref.2.1]).

Chapters 4 through 7 are probabilistic and / or semi-probabilistic and / or deterministic verification methods presented for different failure mechanisms. In doing so, we always discuss:

1. The failure definition and the failure mechanism model (or: the limit state function),
2. The variables in the failure mechanism model and their uncertainties,
3. The reliability requirement for the considered failure mechanism at the level of a work of art or construction part and
4. The method for answering the question or to the reliability requirement is met: probabilistic, semi-probabilistic or deterministic. With every

semi-probabilistic rule, all are always representative values and partial safety factors specified.

2.9 References and background documents

[Ref. 2.1] Foundations for flood protection, Expertise network Water safety, Second revised edition, November 2017

[Ref. 2.2] Regulation for primary flood defenses 2017 - Appendix III Strength and safety, Ministry of Infrastructure and the Environment

[Ref. 2.3] Rijkswaterstaat WVL, Guide Designing with Flood chances - Safety factors and taxes for new ones Flood probability standards, version OI2014v4, February 2017

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[Ref. 2.4] WBI2017 - Scheduling manual for grass cover, Rijkswaterstaat WVL, version definitive 2.0, December 1, 2016

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3 Hydraulic loads

3.1 Introduction

This chapter discusses the determination of hydraulic loads necessary for performing a design verification. The method used in this chapter is usable for most objects in the Netherlands. Use is made here of the instruments as at the time of writing this work guide by the Ministry of Infrastructure and the Environment delivered. For situations where this 'standard set of instruments' is not suitable, as in harbor basins, analyzes and / or customized models are required. You can do this contact with the Helpdesk Water.

3.2 Explanation of available instruments

3.2.1 Introduction Hydra-NL

Below is an introduction to Hydra-NL specifically for Kunstwerken (works of art). In front of detailed information is available in the Hydra manual NL [Ref 3.6].

Hydra-NL was created by merging the previous Hydra-Sweet and Hydra Coast. Until recently the program was only suitable for dike profiles, but since 2017 it is also possible to schematize vertical walls[4]. With this Hydra-NL can be used as a design tool for determining the required height of artworks.

Hydra-NL is called a probabilistic model, in which the parameters (seawater levels, wind speed, lake levels, drainage, etc.) that lead to the hydraulic load on flood defenses (water level and waves) stochastic variables. For each tax system[5], databases are physics and statistics available. The physics databases provide the link between the local water level and the basic toasts (e.g. seawater level, drainage, wind speed). The statistic files describe the marginal statistic [6] of the basic randomists. Hydra-NL compiles these marginal statistics into the tax statistics location of the export locations, or the combined statistics on water level and waves. This takes into account the correlations between variables and the correlations over time. For a calculation on the site of a work of art, a bank location will be chosen in Hydra-NL, after which the model will be the combined generate tax statistics for the artwork.

The current version 2.4.1 of Hydra-NL has three modes:

- Assessment mode, where with the databases physics (these are the hydraulic preconditions databases) from the WBI 2017 is counted.
- Design mode, where for visual years 2050 and 2100 with adapted Statistics files and sometimes also adapted databases of physics are counted. In the statistics files, the effect of climate change on the statistics of discharge and / or seawater level / lake level is processed.

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[4] The transhipment and overflow formulas for vertical walls from the 2003 Artwork Guidelines [Ref 3.4] are programmed in Hydra-NL. For several reasons, the formulas from the EuroTop Manual [Ref 3.5] were not chosen. The formulas from the Guide to Works of Art are also somewhat more conservative, which is recommended in the case of a design. See also chapter 5 Height.

[5] Coast, lake area, upper river area, etc.

[6] This is the statistic of the individual stochastic variables

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(30 頁)For the upper rivers do the databases physics for designs differ from the databases for assess because of a different discharge distribution over the splitting points. In addition, it is conceivable that one wants to take account of spatial planning measures. Reference is made to the calculation prescriptions from OI 2014 ([Ref3.12]) for the correct (combination of) databases of physics and statistics.

- Test mode, in which you can create your own climate scenarios (by the user can be processed in custom statistics files).

The program determines the height of the failure mechanism Hydraulic Load Level (HBN), or the height of a dike body or artwork belonging to a specified transshipment / overflow flow rate and associated exceedance. The strength parameter - the critical transfer or overflow rate -is treated as a deterministic variable in the model. It is for design program is therefore suitable for the required crown height of the artwork determine; for a detailed explanation, see Chapter 5 Height. It can be the other way around program also calculate the failure probability if the crest height and transshipment / overflow flow rate are specified. This is useful for the statutory assessment.

In addition, for the failure mechanisms, Hydra-NL cannot close, piping and structural failure based on the marginal statistics calculate the exceedance frequency lines of outside water level and waves for each output location. Using Hydra-NL 2.4.1 it is not possible to determine the combined water level and wave statistics required for structural failure.

3.2.2 Introduction Water level course

Here is a brief introduction of the program, for detailed information see *User Manual Water level course* [Ref 3.2].

The Water Level Course tool generates one for six water systems [7] per outlet location course of the outside water level at a specified peak outside water level. In reality, rivers depend on many possible forms of the drainage wave of influences such as meltwater and quantity, period and location (s) of the precipitation. The variation of the high water level is smaller on the coast because of the astronomical tide, but the high water level can vary between one or more tides due to variation in storm duration and wind direction. To come to one water level course is used in the Water Level Course tool made of a combination of slow (drain) and fast (wind speed) stochastic. Because the Water Level Course tool is meant for geotechnical failure mechanisms focuses on the slow stochastic devices. Here is a combination of slow and fast stochasts chosen not the most likely is, but on the extreme side focused on the long term.

For art works, the Water Level Translation Tool is particularly useful for it sharpening the inflow volume over a closed work of art or by an opened / collapsed work of art for determining the chance of exceeding the maximum capacity for storage. It will be within the WBI water level course schematized as a block pattern with a certain storm duration.

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[7] Coast (including Wadden Sea and Western Scheldt, but not the Oosterschelde), Lakes (Ijsselmeer and Markermeer (Ijssel and Marker lake), Vecht and Ijssel delta, Benedenrivieren (Beneden River), Upper River Maas (also referred to as Upper Meuse) and Upper River Rhine (also referred to as Upper Rhine).

(31 頁) The default value for the storm duration is 6 hours, this value is adaptable within the WBI. For storm-dominated load systems with a relatively short-lived high-water wave are indeed 6 hours a conservative assumption. However, this is not the case for discharge-dominated systems in the case and use of the Water Level Reduction Tool inflow volume. This is discussed further in chapter 10 Cup storage.

Wind waves also play a role in determining the inflowing flow. When waves make a relatively large contribution to the inflowing flow [8], especially at transshipment situations, these must be combined with the water level course. This is discussed in more detail in section 3.6.

3.2.3 Introduction Risk

Risk (Ring Test) is a software application that assesses the WBI 2017 supports. With Riskeer hydraulic loads can be determined and for the failure mechanisms *height*, *do not close* and *constructional failure* can be a failure probability being calculated. In this case the user introduces a schematization of the strength of (parts of) the artwork, after which with the failure mechanism-model an analysis of the strength with respect to the taxes can be executed. The result is a failure probability for the failure mechanism in question. The Failure mechanism *piping* for artworks is not included in Riskeer. With the currently applicable Riskeer version 17.2.1, it is only possible to do so a given construction to determine a failure probability. It is not possible to have one target chance and thus the required value of one of the input parameters (e.g. the critical flow rate of the soil protection or the required strength of a structural component) ask. In addition, it is not possible in Risk version 17.2.1 with another year of vision than 2023 (the year of assessment). This makes that Risk currently still offers few options for performing one design verification. Extensive information about the operation of Riskeer can be found in the user manual [Ref 3.7].

3.2.4 Introduction Water data and Ten-year overviews

For some design verifications it is necessary to have water level statistics the high frequency range (probability of occurrence greater than 1/10 per year). For this Hydra-NL is only of limited use. On the Waterdata website (https://www.rijkswaterstaat.nl/water/waterdata-en-waterberichtgeving/waterdata) to find current water data such as water levels, drains and waves, but in addition, measurement series from the past can also be retrieved for a large number measuring stations. This allows local statistics to be generated in it high-frequency range.

A document can be downloaded for the coast on the page 'Characteristic values [9]' in which the characteristic values of the water level for all measuring stations along the coast. For all stations, the high water levels with frequencies are included 1 time per 10 years, 1 time per 5 years, 1 time per 2 years, 1 time per year, 2 times per year and 5 times per year.

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[8] Waves naturally play a role in the failure mechanism height, but reliability and closing and strength and stability in the vertical wall / high threshold inflow model also apply to the failure mechanisms

[9]https://www.rijkswaterstaat.nl/water/waterdata-en-waterberichtgeving/metingen/waternormalen/index.aspx

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(32 頁) The same document is currently being used for the rivers composed. As long as this is not yet available, use can be made of the ten-year overview 1981-1990 ([Ref 3.1]). Here are the high water levels with frequencies 1 time per 10 years, 1 time per 2 years and 1 time per year included.

3.3 Overview of hydraulic loads

3.3.1 Overview of hydraulic loads per failure mechanism

In short, the hydraulic loads exist on a water-retaining structure for the load situation high water from:

- An outdoor water level, sometimes also the course of the outside water level important in time.
- The associated waves.
- An inland water level.

In the chapters per failure mechanism is explained when outside and water levels and when water levels must be applied.

The way of determining and applying hydraulic loads can be different are for the various failure mechanisms of artworks. In the table below for each mechanism it is indicated which taxes are relevant and which instrument for the derivation of these taxes can be used. Follow up a short explanation is provided for each failure mechanism.

Table 3 Overview of load parameters and available tools for each failure mechanism

Mechanism	Parameters to be determined	Instrument to use * /sources
Height	Outside water level OR outside water level course Wave height Inland water level	Hydra-NL Hydra-NL + Water level expansion tool Hydra-NL Local data
Don't close	Outdoor water level OR Outdoor water level trend Wave height Inland water level	Hydra-NL / Ten-year overviews / Water data Hydra-NL + Water level progress tool Hydra-NL Local data
Piping	Outside water level Inland water level	Hydra-NL Local data
Constructively collapse	Outside water level Wave height Inland water level	Hydra-NL Hydra-NL Local data

※（Rijkswaterstaat, Central government　2018）32 頁より作成。

* The WBI toolkit Riskeer (see section 3.2.3) is not included in the table, although there are possibilities to use this. At the moment this is custom work, which requires a lot of extra effort and external assistance.

3.3.2　Further explanation of failure mechanism Height

Hydra-NL is used in a different way than for the failure mechanism for the other mechanisms. Hydra-NL can directly use this for height probabilistically determine the crest height by means of an HBN calculation at a specified critical flow rate and the failure probability for the mechanism. This counts Hydra-NL directly with the combined water level and wave statistics, so that none separate design water level and wave boundary conditions have to be determined to become.

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3.3.3　Further explanation failure mechanisms do not close and structural failure

Determination of high frequency water levels

Depending on the threshold height of the structure are lower or higher water levels required for significant inflow due to the non-closed structure. Hydra-NL and the Water Level Gradient tool have a limited water level range and are of limited use for water levels with exceedance frequencies higher than roughly once every 10 years. The precise range differs per water system / export location. In that case, use must be made of other sources such as Water data or Ten-year overviews ([Ref 3.1]).

When outside water levels with an exceedance probability greater than 1/10 per year already lead to problems with an open artwork can here in one design verification in two ways:

1. Extrapolation of the exceedance frequency line from Hydra-NL. This is mostly a conservative approach.
2. Crossing probabilities with local water level data to derive high frequency water levels. Applying local water level data does have a number of preconditions:
   - The measurement series must be representative of the situation to which looking at:
     - The water level statistics may not be significantly affected by changes in the water system in the past or must for this changes can be corrected.
     - No changes in the water system are to be expected within the planning period that can influence the statistics or for this purpose must be corrected. Think of the consequences of both spatial measures such as climate change. Notice that spatial measures in the rivers for the benefit of the flood risk management generally have a limited impact on the high frequency water levels (effort / effectiveness is meant for extreme conditions).
   - There must be enough data for a thorough statistical analysis. If this is not the case, the desired water level should be used fully based on the Hydra models.

For the series of measurements use can be made of the Ten-year overviews and www.rijkswaterstaat.nl/water/waterdata-en-waterberichtgeving/waterdata.

Decrease in incoming flow due to rising inland water level

Generally speaking, art works are either a small cup of storage directly rises with the outside water level or a large cup storage where the water level remains more or less constant. Sometimes, however, the situation is between these two extremes in and must take into account the decrease of the inflowing flow through the artwork because the inland water level increases. In that the case also needs to be counted with an inland water level course. This situation is not very common and is part of a customized analysis.

3.4 Applying climate scenarios

An important difference between a (legal) assessment and the design of a water-retaining work of art is the year of visibility of taxes. With designs it is visual year equal to the required life span of the artwork and in the case of an assessment is the visual year of the last year of the assessment round. This means that a design verification is actually an assessment at the end of its useful life with an estimate of the taxes and strength as assumed to be present. **(34 頁)**Many types of taxes have no or negligible differences in time, but hydraulic loads usually do. This can be caused by climate change (is processed in statistics files) but for example also through spatial measures (is processed in physics databases).

3.4.1 The basis: hydraulic preconditions for the Legal assessment

For every statutory assessment, the Ministry of Infrastructure and Water state databases physics[10] available for all load systems (coastal and coastal) lake area, rivers, etc.) that are valid for the period between two successive Legal Reviews. For this assessment round databases made available with year of view 2023. With these databases physics then the local hydraulic loads, with for example Hydra-NL, on the artwork or dyke body are derived for an assessment. In the meantime the physics databases can be used both in Risk and Hydra-NL, without that additional (uncertainty) allowances are required. However, small differences can occur due to the different computational techniques used in both programs, but they are irrelevant to this manual.

3.4.2 Translation to hydraulic boundary conditions for designs

For designs, an extrapolation must be made according to the year of vision that belongs to it at the end of the chosen design life. As a result of climate change (leading to modified statistics files compared of the assessment) and planned spatial measures (leading to adapted databases physics for the upper rivers area) are there for hydraulic taxes (large) differences to be expected between the present and the distant future. Reference is made to the calculation prescriptions from OI 2014 ([Ref. 3.12]) the right (combination of) databases of physics and statistics.

The IPCC (Intergovernmental Panel on Climate Change) proposes global climate scenarios, after which the KNMI translates these into national scenarios for the Netherlands. The climate scenarios currently used are in accordance with the KNMI'06 climate scenarios and the DGWB policy choices. For the determination of the hydraulic boundary conditions for a visual year in the future with the help of Hydra-NL a climate scenario should be chosen.

From January 2018 the KNMI-2006 climate scenarios (2 units: G and W +, for further explanation see www.klimaatscenarios.nl/knmi06),which are based on IPCC-2005/2006. These scenarios may become available Hydra-NL eventually replaced by the KNMI-2014 scenarios (at least 2 units), which are based on IPCC-2014. For the choice of the climate scenario affiliated with the OI 2014 v4 which states that for each design lifetime the W + climate scenario is assumed. It is also stated that a design on "end-of-life" should indeed meet the W + requirement, but adaptive can be applied to a middle scenario (G or G +) provided it design is expandable. The latter will especially for the foundation and hard construction parts for artworks do not apply as quickly.

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[10] These can be found on the WBI2017 ftp server. Access to this can be requested via the Water Help Desk.

(35 頁) In the design mode of Hydra-NL three vision years can be chosen: 2023, 2050 and 2100. Although the climate effects are exponential, it is a first conservative approach to interpolate linearly when the required visual year between these three view years. No policy has yet been developed for view years after 2100. Although climate effects may be underestimated, is currently being advised to linearly extrapolate for visual years after 2100. In the event that one more accurate distraction is desired can be assistance through Helpdesk Water asked. Incidentally, it is always possible to own Hydra-NL test mode climate scenarios and other visual years. You can make your own estimates for sea level rise, lake level rise and drain statistics. It is recommended to use specialist help in this.

3.5 Other surcharges on the water level

When determining the hydraulic loads in the year of visibility, take into account be kept with the following surcharges on the water level:

• A surcharge for outdoor oscillations and gusts .
• A surcharge for seiches.
• A surcharge for (local) noise if not already in the water level statistics has been processed.

In the *Guidelines for Art Works 2003* ([Ref.5.5]), account was also taken of an extra safety in the form of a minimum guard height (during wave overtopping) than however, safety surcharge (at overflow) of 0.30 m in connection with possible uncertainties in the water level. Since these uncertainties are currently explicit included in the hydraulic loads comes this minimum guard height / safety surcharge.

3.5.1 Temper oscillations and pulses

Punches and blown oscillations are short-term changes in the water level due to heavy showers and major changes or fluctuations in the wind. Right now such fees will only be charged in the databases for it Europoort area. For other tax areas where outside oscillations and gusts possibly relevant (coast, lake area and deltas) it is up to the designer to estimate whether and, if so, which surcharge should be charged here. Reference is made to Section B2.2.3 of the *Guide for Sea and* Lake Dykes [Ref 3.3].

3.5.2 Seiches

Seiches are standing waves in a (semi) closed basin, which are short-lived increase the water level. Presumably the Europoort area and IJmuiden the most seiche-sensitive locations. Only in the Europoort area seiches were considered relevant enough to take them into the hydraulic tax databases of the WBI as a surcharge on the water level; a separate surcharge so not necessary here.

For other port basins where seiches may play a role is an exploratory research done ([Ref 3.10] 11). The seiche effect is for these port basins usually not relevant or unambiguous from the measurement results. If the designer considers a seiche surcharge necessary, then this can be done with specialist assistance to the hydraulic load database.

[11] The results from this report cannot simply be used; an explanation can be obtained via the Water Helpdesk

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3.5.3 Local wind

Local windings are included in the hydraulic loads at the site export locations, except in the upper river area. In the upper rivers area local noise is also not relevant because the water level discharge-dominated and the wind does not play a major role in situations with a high level drain. Therefore, there is no separate surcharge for local calls to become.

3.6 Wave growth during the flood wave

Wind waves also play a role in determining the inflowing flow. In this paragraph deals with how these wind waves interact with it water level course can be combined. A distinction is made here made to the tax systems such as these in the Water Level Course tool to be distinguished: Coast (including Wadden Sea and Western Scheldt, but not the Oosterschelde (Eastern Scheldt)), Lakes (IJssel and Markermeer), Vecht and Ijssel delta, Lower rivers, Upper rivers Meuse and Upper rivers Rhine.

The basic idea is that the wind set-up is combined in such a way with the water level course that the peak of the wind set-up (and thus wave height) coincides with the peak of the water level. This is a conservative approach. In In the paragraphs below, this is further explained for the different tax systems worked out.

The peak wave height is read from the illustration point of the Hydra-NL-calculation of the required crown height. In this way is as good as possibly taken into account the correlation between water level and wave height before a specific location. See also chapter 11 Case.

3.6.1 Coast

The water level course at a random location along the coast consists of the time course of the storm set-up and the time series of the average astronomical tide. The top of the set-up and the top of the astronomical tide do not coincide. The phase shift depends on the location. The storm design has one trapezoidal course for all locations along the coast. The parameters of the trapezium depend on the water system, see also the table below from *Background Report Hydraulic Loads* [Ref

3.8]:

Table 4 Parameters trapezia time course storm set-up and phase shift coastal areas (source: [Ref 3.8])

Area / location	Time course storm set-up	Phase difference design and tide
Zeeland and Holland coast	Trapezium, base duration = 44 hours and peak duration = 2 hours	2.5 hours
Hook of Holland	Trapezium, duration = 30 hours at half meter level and flanks from 12 hours to zero meter level	-4.5 hours
Wadden Sea	Trapezium, base duration = 45 hours and peak duration = 2 hours	5.5 hours

※（Rijkswaterstaat, Central government　2018）36 頁より作成。

In general, the following procedure can be used:

• Make an export of the water level course from the tool Water level course. Figure 5 is a typical illustration of the water level course along the coast.

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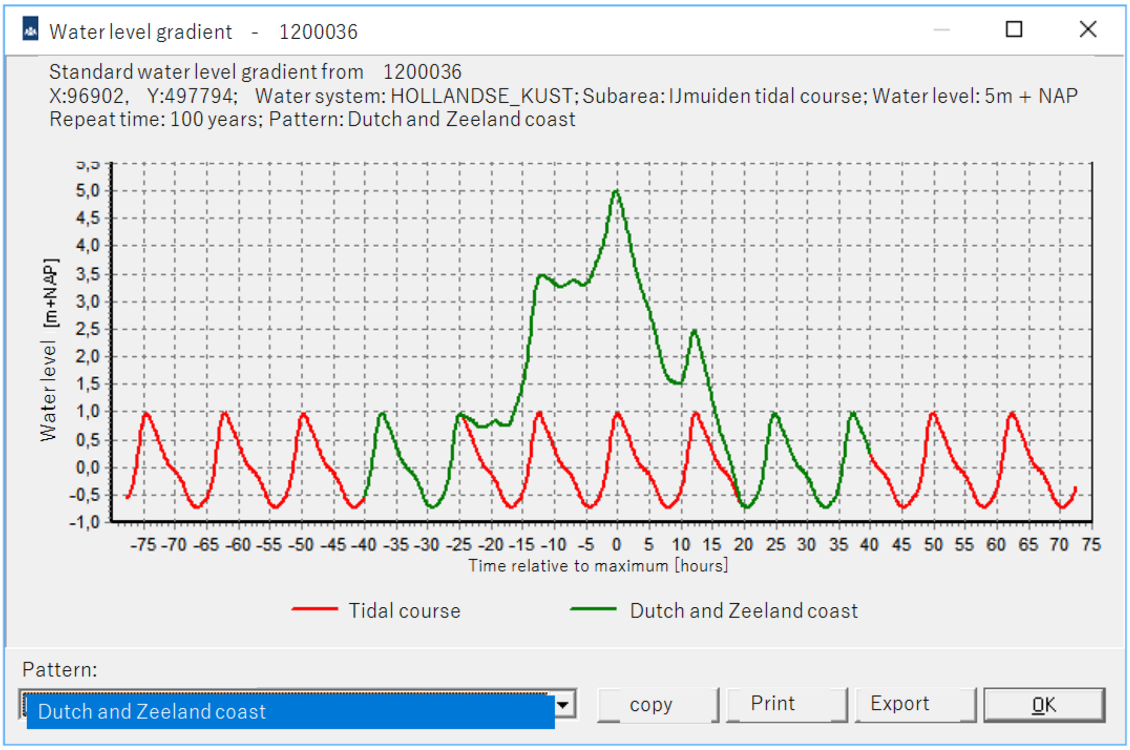

※（Rijkswaterstaat, Central government　2018）37 頁より作成。
Figure 5 Export tool Water level course Coast

- In the Excel file obtained in this way there are two tabs included: 'Name of sub-area' (e.g. Dutch and Zeeland) coast) and ' Tidal walk '. By finishing the tide course of the water level the design can be reconstructed. Because of the phase shift, the top of the water level set-up does not lie at T = 0 but at the Dutch and Zeeland coast for example at T = -2.5 hours.
- The wave growth follows the design of the water level and grows from 0 on the basis of the trapezoid shape of the setup to the maximum value (1.50 m in this example) at the top of the storm (in this example at T = -2.5 hours). In Figure 6 this is visualized for one example location along the coast.

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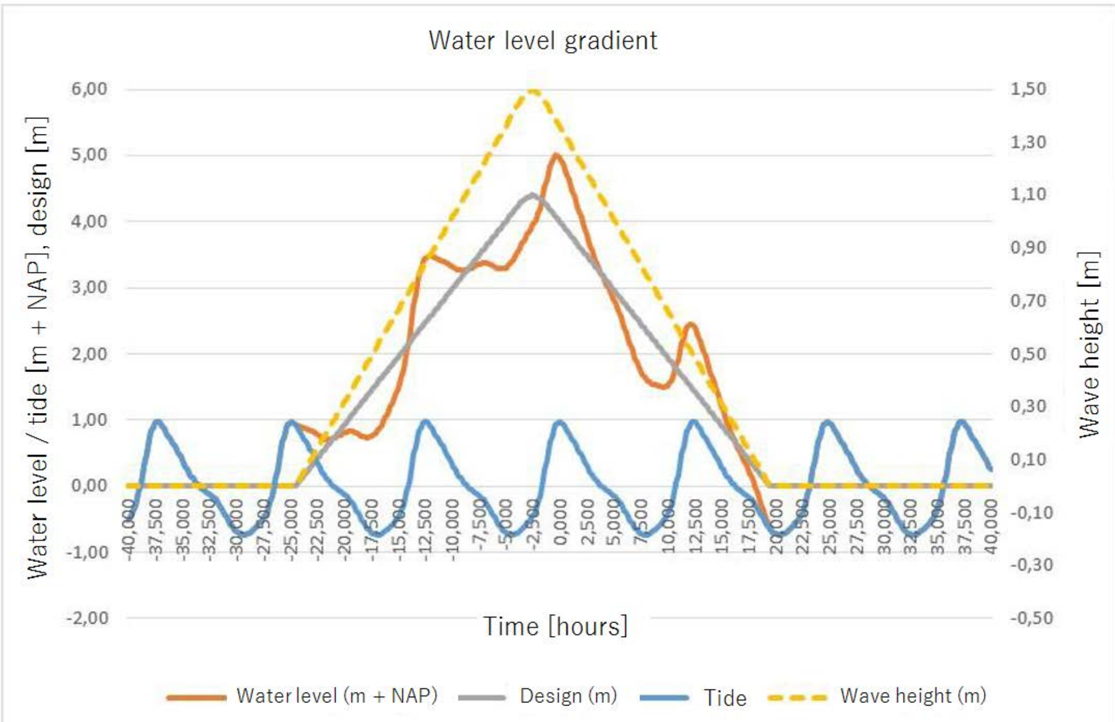
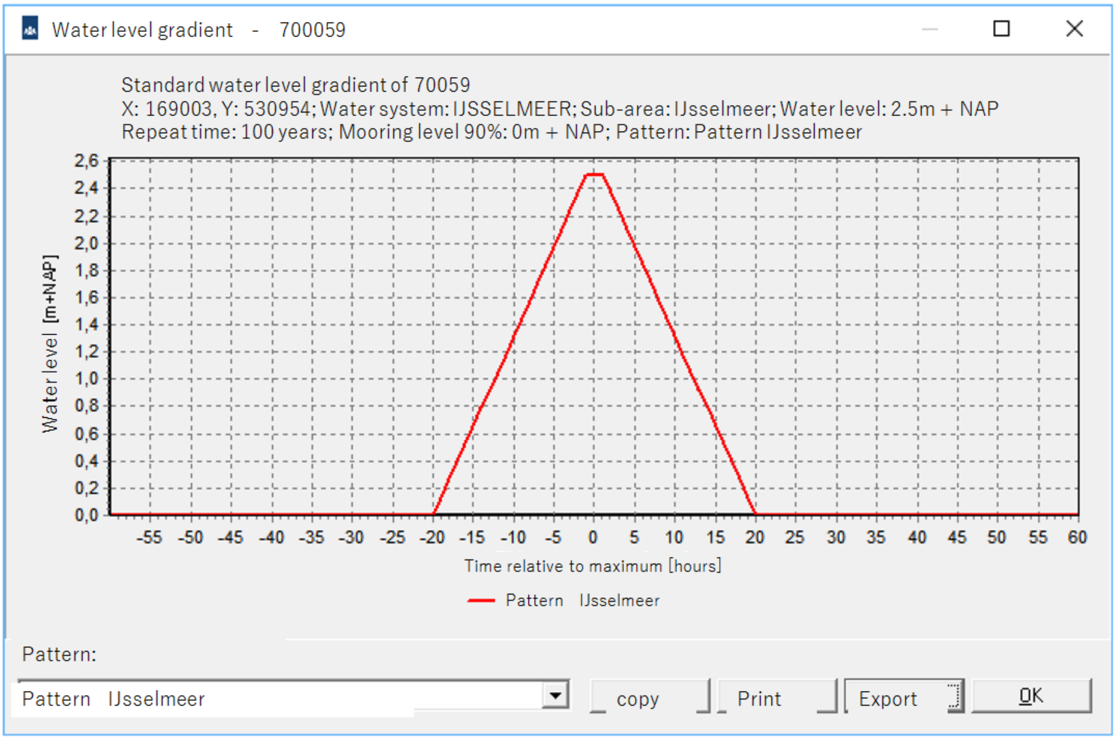


※（Rijkswaterstaat, Central government　2018）38 頁より作成。

Figure 6 Course of water level (left vertical axis) and wave growth (right vertical axis) up example location IJmuiden

3.6.2　　Lakes

The water levels for the IJsselmeer and the Markermeer (IJssel and Marker laker) are described by a trapezoidal shape of the wind set superimposed on a stationary course of the lake level at the level of the 90% percentile. The wind set has a trapezoidal shape with a base duration of 40 hours and a peak duration of 2 hours. Figure 11 shows a characteristic image of the water level course a random location in the lake area.



※（Rijkswaterstaat, Central government　2018）38 頁より作成。

Figure 7 Export tool Water level course Lakes

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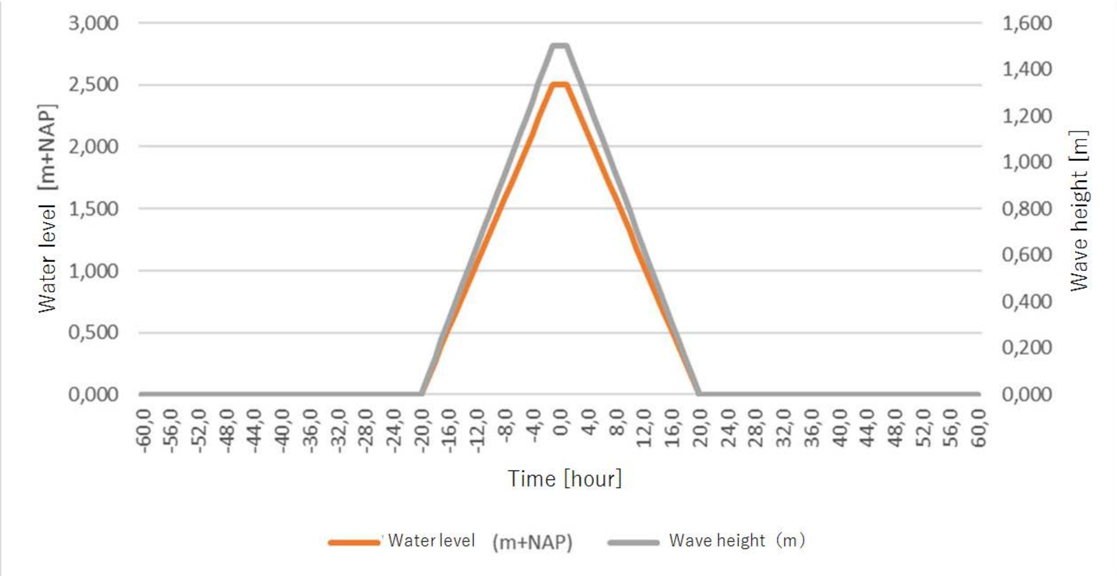
It can be seen that the course of the water level exactly follows the wind set up and also has a trapezoidal shape with a base duration of 40 hours and a top duration of 2 hours. The wave growth follows the design of the wind / water level and thus grows from 0 on the base from the trapezium shape (T = -20 hours) of the set-up to the maximum value on the top of the storm at T = -1 hour. The wave height then remains at a maximum until T = + 1 hour and walk back to 0 at T = + 20 hours. In Figure 8 this is visualized for a random location where the wave height in the illustration point of the Hydra- NL calculation is 1.50 m.

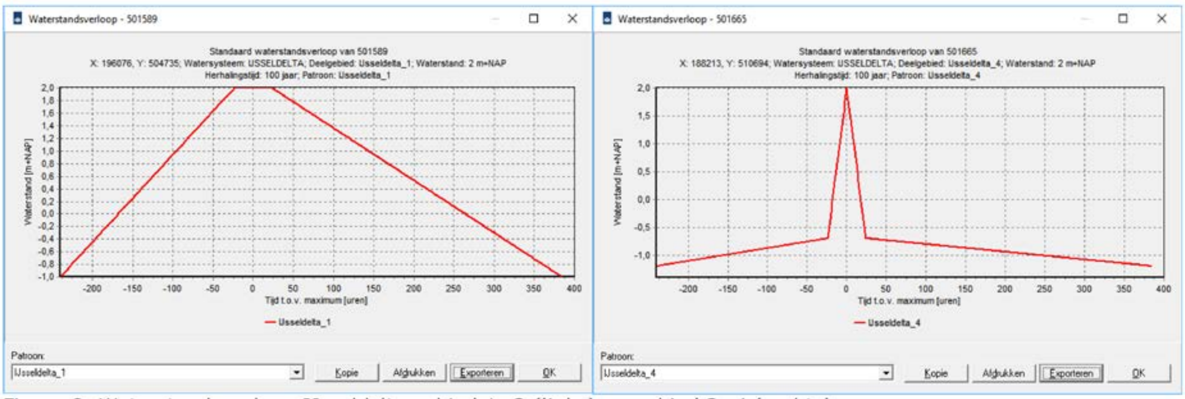


※（Rijkswaterstaat, Central government　2018）39 頁より作成。

Figure 8 Course of water level (left vertical axis) and wave height (right vertical axis) on example location Ijssel Lake

3.6.3 Fighting and ice delta

Both the Vecht delta and the Ijssel delta are divided into four sub-areas [12]. For the Ijssel delta applies that there are two principle forms for it water level course, for the Vecht delta each sub-area has one water level gradient line with its own shape. In Figure 9 and Figure 10 this is displayed.



※（Rijkswaterstaat, Central government　2018）39 頁より作成。

Figure 9 Water level course Ijssel delta area 1+2 (left) and area 3+4 (right)

[12] The boundaries can be found in paragraphs 10.3.3 and 10.3.4 of [Ref 3.8]

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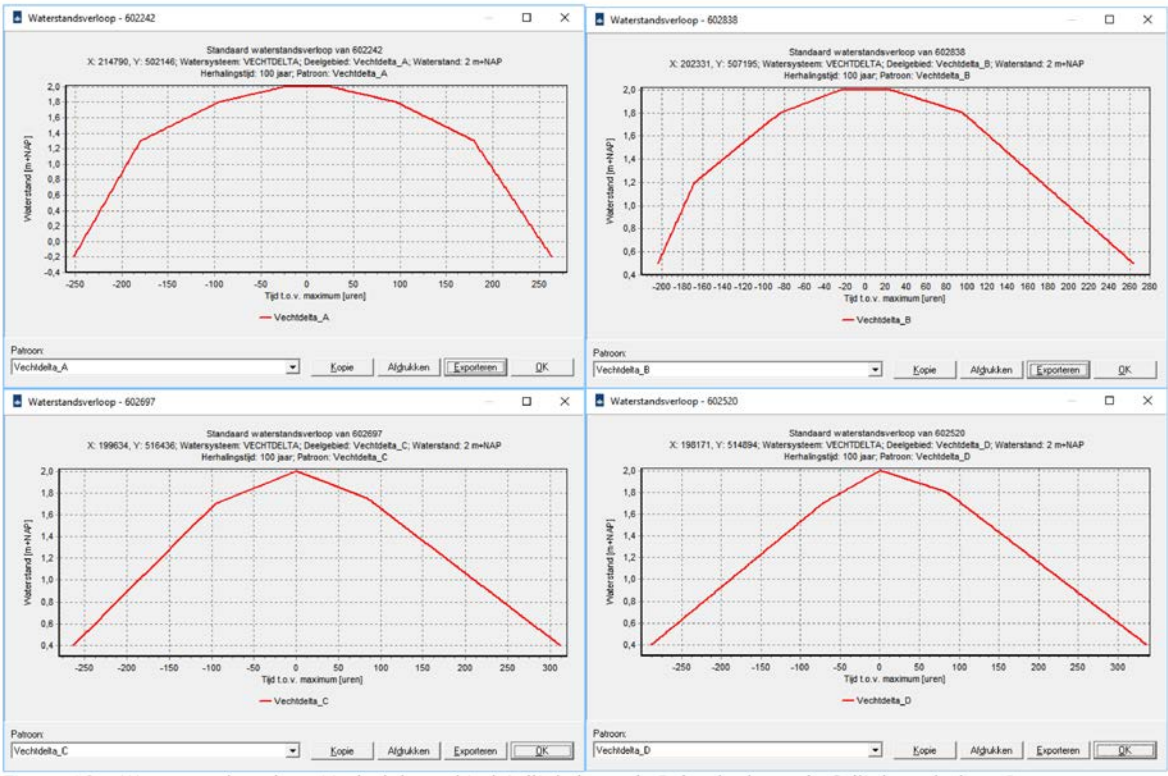

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Figure 10 Water level course Vecht delta area A (upper left), B (top right), C (lower left) and D (bottom right)

It can be seen that each sub-area has its own breakpoints:

Table 5 Overview of the articulation points of the water level course in the Vecht and IJssel delta areas

IJssel Delta 1-4	Vecht delta A	Vecht delta B	Vecht delta C	Vecht delta D
T = -264 hours	T = -252 hours	T = -204 hours	T = -240 hours	T = -288 hours
T = -24 hours	T = -180 hours	T = -168 hours	T = -96 hours	T = -72 hours
T = + 24 hours	T = -96 hours	T = -84 hours	T = + 84 hours	T = + 84 hours
T = + 384 hours	T = -24 hours	T = -24 hours	T = + 312 hours	T = + 336 hours

	T = + 24 hours	T = + 24 hours		
	T = + 180 hours	T = + 96 hours		
	T = + 384 hours	T = + 264 hours		

※（Rijkswaterstaat, Central government　2018）40 頁より作成。

From *Hydraulic Loads Vecht and IJssel Delta* [Ref. 3.9] shows that it is for sub-areas Vecht delta C and D would be too conservative the wave growth directly on the water level design because of the long duration that this would entail bring. That is why for the Vecht and IJssel Delta the wave growth is linked to the wind set up.

In the calculation of the hydraulic loads in the Vecht and Ijssel delta is account is taken of a trapezoidal course of the wind set-up. In The figure below from *Hydraulic Loads is Vecht- and Ijssel delta* [Ref 3.9] the course of the wind design (see the black line), which is characterized by a basic duration of 48 hours and a top duration of 2 hours.

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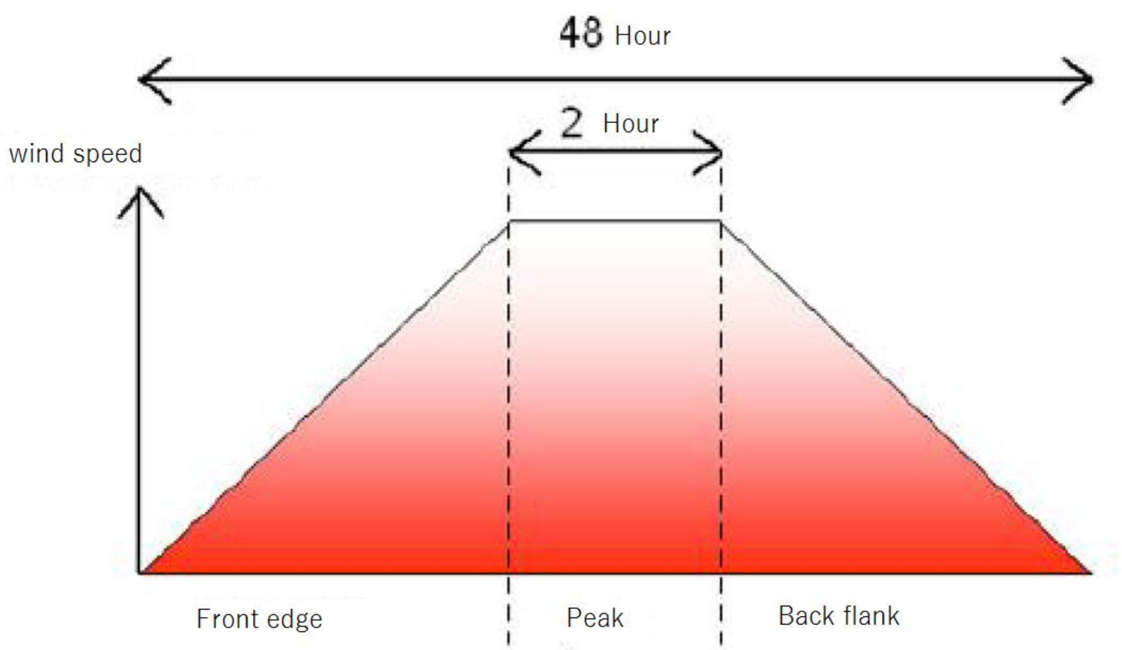
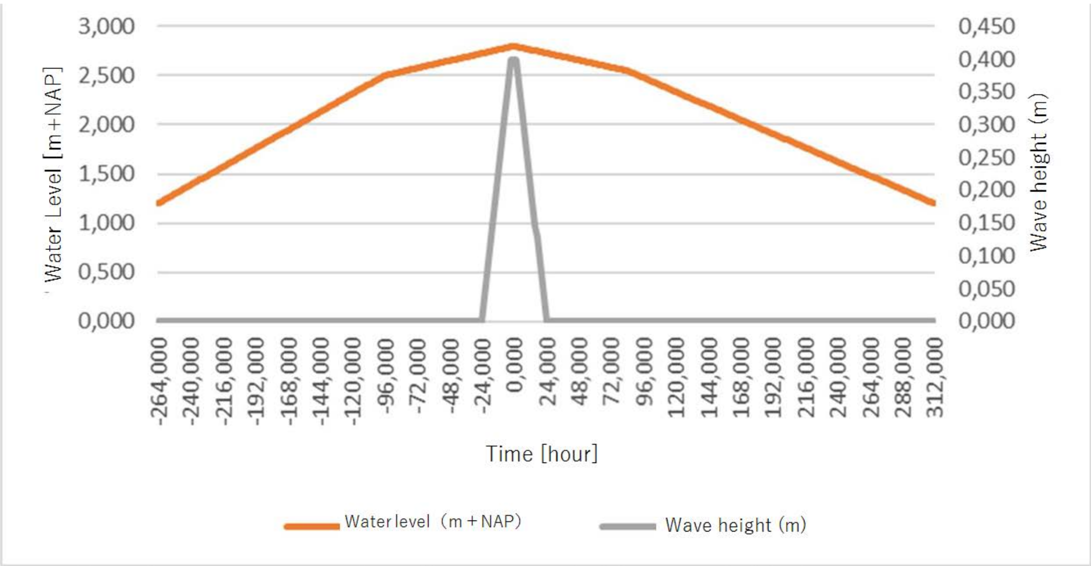


※（Rijkswaterstaat, Central government　2018）41 頁より作成。

Figure 11 Schematic representation of the time course of the storm (wind speed) (source: [Ref 3.9])

The wave growth follows the design of the wind / water level and thus grows from 0 on the base from the trapezium shape (T = -24 hours) of the setup to the maximum value on the top of the storm at T = -1 hour. The wave height then remains at a maximum until T = + 1 hour and runs back to 0 at T = + 24 hours. In Figure 12 this is visualized for a random location in sub-area C where the wave height in the illustration point of the Hydra-NL calculation is 0.43 m.



※（Rijkswaterstaat, Central government　2018）41 頁より作成。

Figure 12 Course of the set-up of water level and wave growth at sample location Vecht delta C

3.6.4 Lower rivers

There is a large number of sub-areas in the sub-river area (> 10). It goes too far to give an example of all of these areas water level course. Some typical examples are in shown below.

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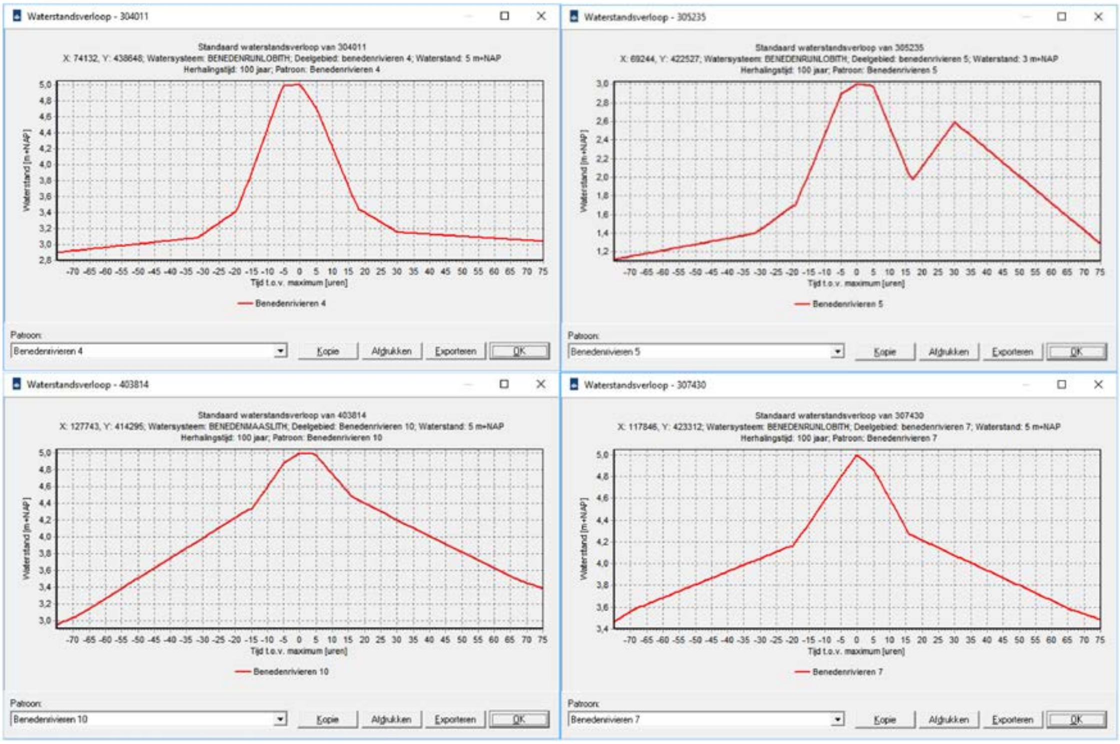
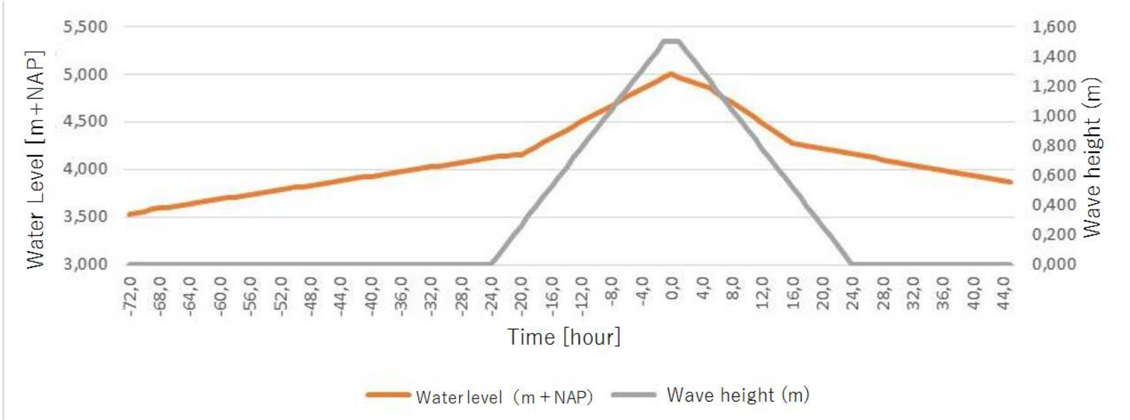


©Rijkswaterstaat, Central government, Netherlands (2018)
※（Rijkswaterstaat, Central government　2018）42 頁より作成。
Figure 13 Water level course for sub-rivers sub-area 4 (upper left), 5 (top right), 7 (bottom right) and 10 (bottom left)

In the calculation of the hydraulic loads in the sub-river area consider the same trapezoidal course of the wind set-up as in the Lake District (*Background Report Hydraulic Loads* ([Ref 3.8]) page 60). This is shown in Figure 11 from [Ref 3.9] which is included in the previous one section. The course of the wind set-up is also in the lower river area characterized by a basic duration of 48 hours and a top duration of 2 hours.

The wave growth follows the design of the wind / water level and thus grows from 0 on the base from the trapezium shape (T = -24 hours) of the setup to the maximum value on the top of the storm at T = -1 hour. The wave height then remains at a maximum until T = + 1 hour and runs back to 0 at T = + 24 hours. In Figure 14 this is visualized for an arbitrary location in the Lower River sub-area 7 where the wave height at the illustration point of the Hydra-NL calculation is 1.50 m.



※（Rijkswaterstaat, Central government　2018）42 頁より作成。

Figure 14 Course of the design of the water level and wave growth at the example location Lower Rivers 7

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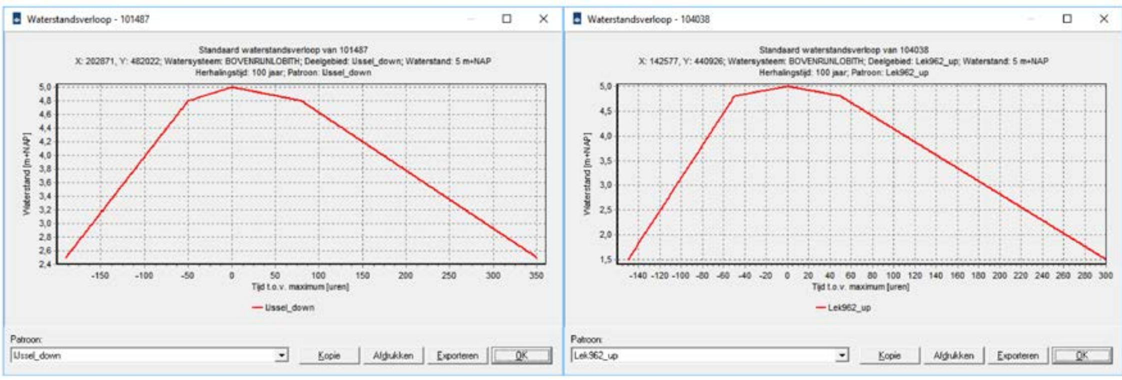
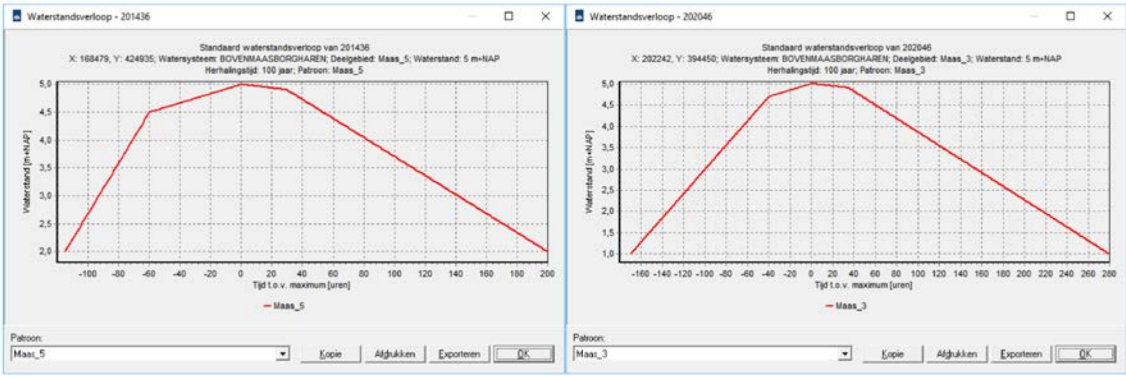
3.6.5 Upper rivers Maas and Rhine

There is also a large number of sub-areas in the upper rivers area (> 10). It goes too far to give an example of all of these areas water level course. Some typical examples are in shown below figures. It can be seen that the drainage waves are all up main lines have the same shape.



※（Rijkswaterstaat, Central government　2018）43 頁より作成。

Figure 15 Water level course over rivers Rhine part area IJssel_down (left) and Lek962_up (right)



※（Rijkswaterstaat, Central government　2018）43 頁より作成。

Figure 16 Water level course over rivers Meuse area Maas_5 (left) and Maas_3 (right)

In the calculation of the hydraulic loads in the upper rivers area no account is taken of the duration of the wind. For the calculation of the wave growth is therefore assumed that the wind set-up is the same trapezoidal course as in the lower river area (see previous section). The course of the wave growth over time can then be done in an analogous manner be determined.

3.7 Artworks behind harbor dams

It often happens that works of art are located behind harbor dams that the reduce waves on the outside water. The output points of the hydraulic however, tax databases are usually outside these port dams. In first authority can be chosen to design with the tax combinations outside the port. No physical processes are taken into account such as wave transmission, reflection, refraction, diffraction and local wave growth may affect wave conditions in the port. Incidentally, the wave conditions can for a location in the port are also less favorable than the wave conditions at the location before the port entrance (due to specific properties of a port in combination with said physical processes). As a design with wave loads outside the port may produce an inefficient design in more detail to the Hydraulic loads are looked at in the port. Any feeling for this can are obtained by varying the orientation of the artwork. **(44 頁)** In that case, the hydraulic boundary condition must be at the location of the export location can be translated to the precondition at the location of the artwork. The Hydraulic Charges for Harbors (HB Havens) tool is available for this. With using this tool, a simple and advanced analysis can be made the hydraulic loads in the harbor basin. More information about this tool can be found in [Ref 3.11]. In particular the advanced analysis requires one comprehensive and multidisciplinary approach. Specialist is recommended for this turn on help.

3.8 References and background documents

[Ref 3.1] Ten-year overview 1981-1990, National Institute for Coast and Sea / RIKZ, 1994 [Ref 3.2] User manual Water level course - version 3.0.1, HKV LIJN IN WATER in the name of Rijkswaterstaat WVL, March 2017

[Ref 3.3] Guidelines for Sea and Lake Dikes - Basic report, Technical Advisory Committee on Flood Defense (TAW), December 1999

[Ref 3.4] Guideline for Art Works 2003, TAW, May 2003

[Ref 3.5] EurOtop - Manual on wave overtopping or sea defenses and related structures, Second edition, Pre-release October 2016

[Ref 3.6] Hydra-NL User Manual version 2.4, HKV LINE IN WATER in order Rijkswaterstaat WVL, May 2018

[Ref 3.7] Ring Test User Manual, version 17.2.1, Deltares, November 27, 2017

[Ref 3.8] Background report Hydraulic Loads, Legal Assessment instrumentation 2017, Deltares, feature 1230087-008- HYE-0001, September 2017

[Ref 3.9] Hydraulic Loads Vecht- en IJsseldelta, Legal Assessment instrumentation 2017, Deltares, feature 1230087-005, April 2017

[Ref 3.10] Seiches - Analysis of water level measurements and influence generating mechanisms, Deltares, feature 11200537-009-ZWS- 0001, final, October 2017

[Ref 3.11] Hydraulic Loads Ports - User manual and method, Aktis Hydraulics and HKV LIJN IN WATER by Rijkswaterstaat WVL, December 2017

[Ref.3.12] Operating instructions for the hydraulic design prerequisites - Supplement OI2014, version 5 (Hydra-NL 2.4.1), Deltares, characteristic 11202226-009-GEO-0002, final, May 2018

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4 Do not close

4.1 preface

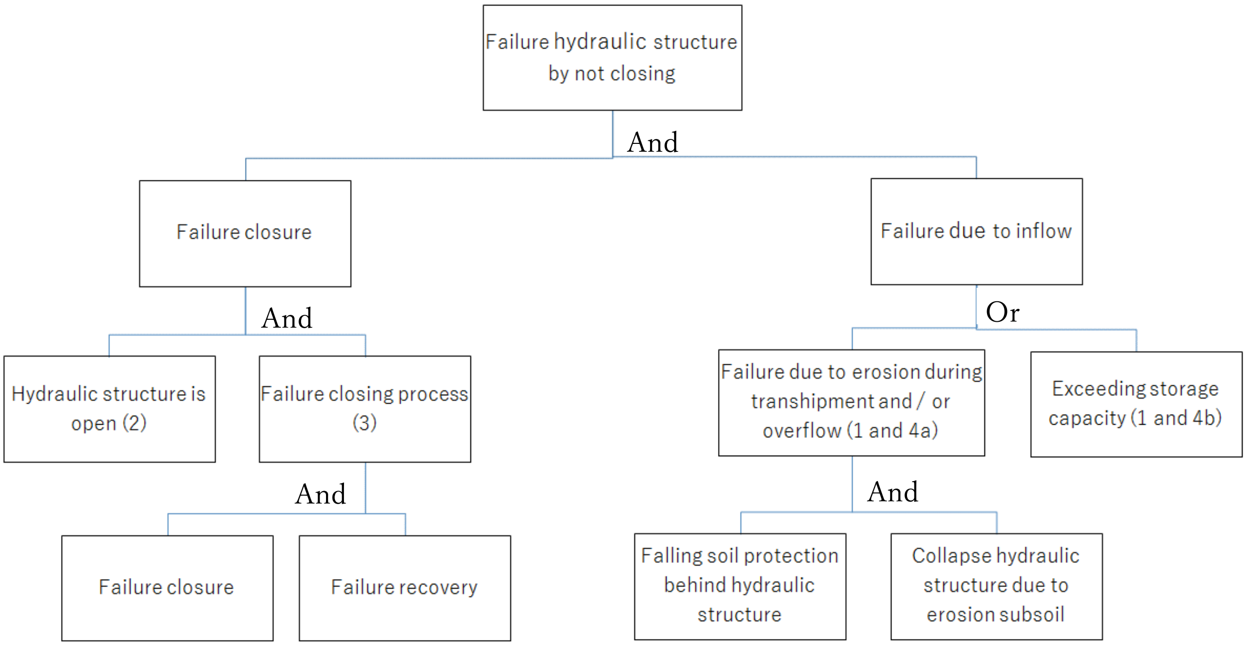
4.1.1 Introduction failure mechanism

The presence of a water-retaining artwork in a flood defense is first authority for a function other than water retaining. The primary function relates to the desire to goods, people, vehicles or liquids or gases pass through the flood defense. Without this primary after all, the artwork was not needed. As a result of this primary function should offer an artwork an opening in the flood defense. This in particular, makes a water-resistant work of art stand out from a flood defense consisting of dikes / quays and / or dunes. The failure mechanism is not closing is therefore a unique failure mechanism that only plays with artworks. To be able to turn a high water will be the opening and thus the artwork (must be closed at high tide).

The choice of reversing means and thus also the reliability of closing is strongly dependent on the primary function. Of course, the water defense function also serves to be covered, but both functions can lead to contradictory requirements direction. As an example, an inlet lock, where the primary function requires water to flow from outside to inside (matching reticle: slider) but the function watering is most efficiently filled in if one reversing device that closes (flow) on its own when influx a check valve or waking door.

4.1.2 Phenomenological description

A detailed description of the failure mechanism not close can be recovered in the test report of the WBI [Ref. 4.1]. The failure tree is shown in Figure 17 below.



※（Rijkswaterstaat, Central government　2018）45 頁より作成。

Figure 17　Do not close failure tree failure mechanism

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Failure to close failure due to the failure mechanism occurs when it inflowing volume of water through an opened work of art is so large this leads to substantial damage and / or casualties (flood consequences). To fail of the flood defense does not occur if not closed:

(1)　Showing a high water,

AND (2)　The artwork prior to this high water is open,

AND (3)　The closure of the artwork fails, through which unwanted outside water can flow in,

AND (4a)　The artwork itself succumbs as a result of failure of soil protection with substantial damage and / or victims (flooding) as a result [13] .To do this, the soil protection must first be left behind collapse the artwork. Then arise pitfalls in the (no longer protected) substrate, after which the stability of the artwork is lost with a result (advancing) breach in the flood defense. It is assumed that this situation always leads to the exceeding of the storage capacity.

OR (4b)　The artwork itself remains standing but it flows in volume is not possible due to the closed artwork are stored in the underlying (water) system without causing substantial damage and / or victims (flooding).

The final phase (phase 4) concerns the actual failure of the flood defense.

4.1.3 Step-by-step plan and design strategy

The starting point for the step-by-step plan for the design verification for non-closing is that in addition to the dimensions and the strength of the soil protection, the number and type reversing means required from the primary function (s) is known. In the following the step-by-step plan (see Figure 18) is then considered the situation where the artwork is not closed high water and a high water applies. This implies that the requirements with respect to the functioning of the reversals from the primary process have a direct impact on this failure mechanism. Below is a step-by-step plan that is a possible one method to come to the design with respect to not closing. Every step is briefly explained below the figure. The order given in the figure of steps is not required, because the present design assignment may not fit exactly on the basis of the step-by-step plan. In that case, all steps are needed to go through, but perhaps a different order than optimal.

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[13] In a practical sense it is possible that a work of art collapses as a result of the inflowing water, without this leading to major consequences (damage and / or casualties). This is because the storage facility is very large and the growth in the breach remains limited. From the primary function of the work of art, however, such an approach will not be desirable in the design. In the case of an assessment based on water safety, it can be used to demonstrate that the requirements are met.

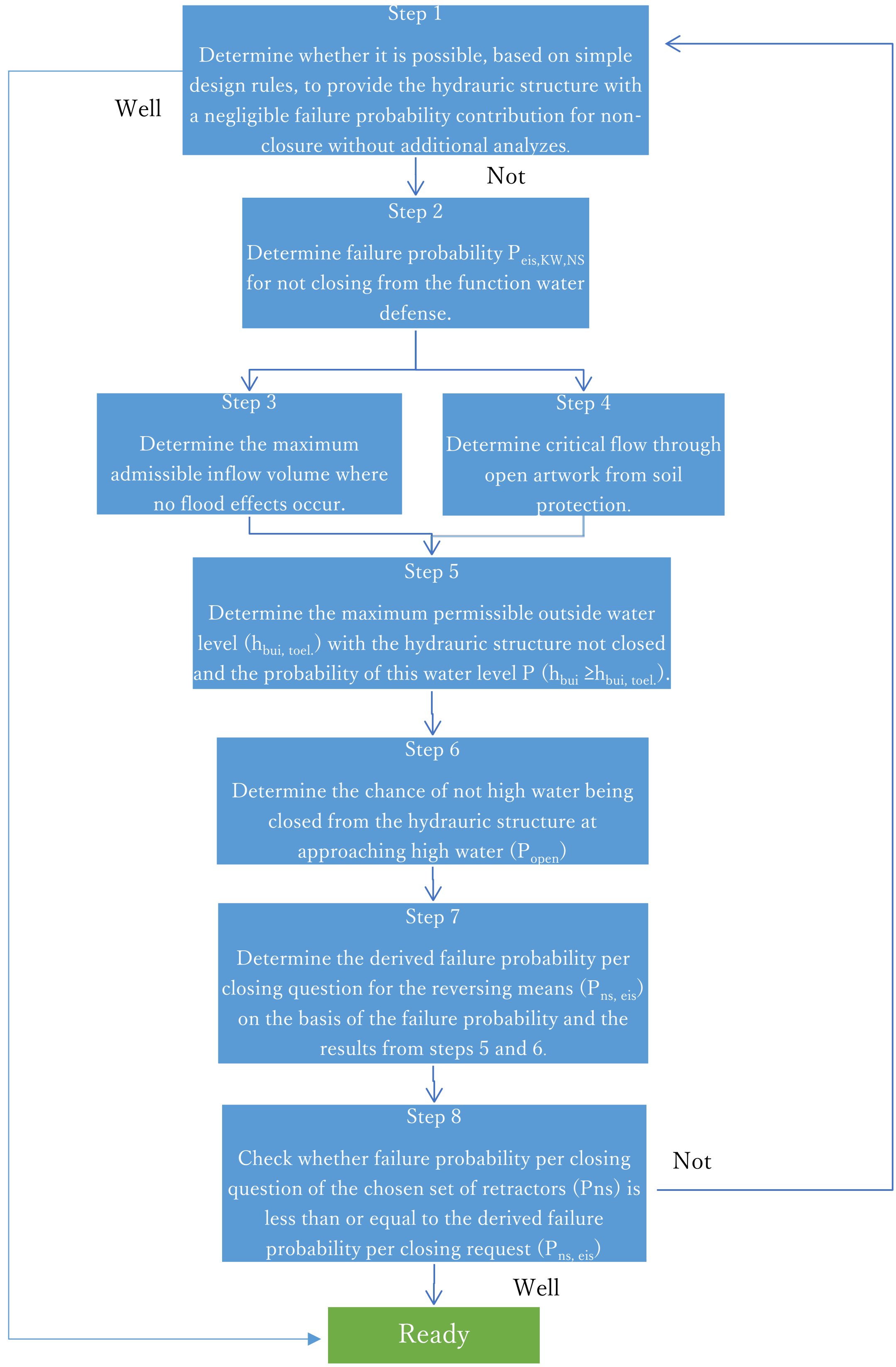
Step 1
Determine whether it is possible, based on simple design rules, to provide the hydraulic structure with a negligible failure probability contribution for non-closure without additional analyzes.
Well
Not
Step 2
Determine failure probability $P_{eis,KW,NS}$ for not closing from the function water defense.
Step 3
Determine the maximum admissible inflow volume where no flood effects occur.
Step 4
Determine critical flow through open artwork from soil protection.
Step 5
Determine the maximum permissible outside water level ($h_{bui,\ toel.}$) with the hydraulic structure not closed and the probability of this water level P ($h_{bui} \geq h_{bui,\ toel.}$).
Step 6
Determine the chance of not high water being closed from the hydraulic structure at approaching high water (P_{open})
Step 7
Determine the derived failure probability per closing question for the reversing means ($P_{ns,\ eis}$) on the basis of the failure probability and the results from steps 5 and 6.
Step 8
Check whether failure probability per closing question of the chosen set of retractors (Pns) is less than or equal to the derived failure probability per closing request ($P_{ns,\ eis}$)
Not
Well
Ready

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Figure 18 Do not close step-by-step plan for design failure mechanism

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Below is a brief explanation of the steps:

Step 1. In this step it is checked whether it is from the primary function, but also costs and other preconditions are possible for the artwork and designing his headlights in such a way that the artwork is evidently safe with respect to not closing. The basis for this is simple design rules (see section 4.3.1). When this is possible, there is no need further analysis before not closing.

Step 2. The failure probability for failure to close (probability of failure per year) can be determined on the basis of the (standard) failure probability budget and the failure probability contributions of the other objects in the dike section concerned (see paragraph4.1.4.1).

Step 3. As explained in section 2.2, a flood occurs at the exceeding the coming capacity due to the inflow of outside water. In this step, the maximum admissible inflow volume of water (that is, the storage capacity). The maximum permissible inland water level with no flooding effects yet occurrence (OKH) is of importance here, together with the surface and the characteristics of the cup storage. This becomes chapter 10 further elaborated.

Step 4. If the artwork is not closed in a high-water way, water flows through the object. The soil protection is then taxed by this flow. The strength of this soil protection can be expressed in one maximum allowable flow rate and then on the basis of the Inland water level is calculated back to a maximum permissible inflowing flow (see also chapter 9 Soil protection). The decay and the outdoor water level also determines the occurring flow velocities.

Step 5. With the help of the results from step 3 and 4 the maximum outside water levels are determined when the water is not high water closed artwork is not just a too large inflowing volume water (step 3, cup storage) or a too large inflowing flow (step 4, soil protection). The inland water level and the inflowing volume cannot be seen separately, because the Inland water level during a flood water is influenced by the inflowing volume. When the bowl is relatively large, the decay will be over increase the artwork during a high water wave because the Inland water level does not rise or only partially rises with the outside water level. This higher decay then leads to higher flow velocities. The previous means that when determining the maximum permissible outside water level the basis of the analysis is formed by a consideration of the cup storage. The course of the outside water level (high water wave) and thus the course of the inflowing flow, the total inflow volume and ultimately the course of the Inland water level in time are the components of the required consideration.

The frequency of exceedance determined in the above manner maximum permissible outside water level actually indicates how often per year or with what chance there is such a high water that there is one flooding would occur if the object never turned upside down would be closed. This exceedance frequency is equal to the number closing questions per year from the high-water function. In this step you can the Hydra-NL and Water level course.

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Step 6. The use of the artwork from its primary function determines how often and how long a work of art is not closed high water. This can be determined with the use of the artwork how great is the likelihood that the artwork will still be present at an approaching high tide must be closed. This opportunity is indicated with P_{open}[14].

Step 7. With the help of the data from step 5 and 6, starting from the failure probability for not closing (step 2), determine the maximum failure probability per closing question of the reversing means.

Step 8. In this last step, it is checked whether the chosen reversing means (from the primary function and perhaps also from the water retaining function) a failure probability of closure per question (possibly including a recovery option) that is less than or equal to the derived failure probability for the closure per question (step 7). If this is not the case, the various steps are again followed by tightening or other choices are made. One can even go back to step 1 and a whole consider new barrier concept. In step 2 you can look again whether the derived failure probability for non-closing can be tightened by reduce the number of artworks that are of interest. This is possible for example by a further analysis of the probabilities of failure to close to perform the rest of the works of art or perhaps even to adjust it of other works of art by applying other inversions or optimizing the closing protocol.

4.1.4 Safety format

The starting point of the safety format is the requirement that the failure probability for non-closure of $P_{f,\ KW,\ NS}$ is less than or equal to the failure probability for not closing $P_{eis,\ KW,\ NS,}$ or:

$$P_{f,KW,NS} \leq P_{eis,KW,NS} \qquad 4.1$$

This forms the basis for the verification of the design with respect to non-closure. Below is a detailed discussion of the determination of the failure probability (section 4.1.4.1) and the calculated failure probability (section 4.1.4.2).

4.1.4.1. Determination of failure probability is *not closed*

This requirement is derived from the legal requirement for the standard route and can be determined using the following formula:

$$P_{eis,KW,NS} = \frac{P_{max} \cdot \omega_{NS}}{N_{NS}} \qquad 4.2$$

In which:

$P_{eis,\ KW,\ NS}$	Failure Chance for not closing an individual artwork for a reference period of 1 year [-]
P_{max}	Failure Chance for the entire dike section (standard route) based on the maximum permissible probability of flooding from the Water Act15 [-]
ω_{NS}	Failure probability factor for not closing [-]
N_{NS}	Length effect factor for non-closing [-]

---

[14] There are situations where P_{open} does not concern a chance but a frequency. This can be the case in particular with frequently deployed structures (for example drainage locks).

[15] In the Water Act this is referred to as the lower limit

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In the WBI2017 [Ref. 4.3] and OI2014v4 [Ref. 4.2] is a standard failure probability budget for a standard route. In this standard failure probability budget for the failure probability space factor for *non-closing* (ω_{NS}) a value of 0.04 arrested.

Deviating from this is possible, see chapter 2 for this design verifications.

The length-effect factor N_{NS} is *not* equal to the number of works of art for *not closing* the dike section considered, where the failure mechanism *does not close* a non- negligible contribution to the probability of flooding and whose closures more or less independent of each other. That means that at it design of a work of art the number of existing works of art in it dyke track must be known as well as their contribution to the failure probability of it dyke section for *not closing* . If the contribution of the other works of art does not known, it can be assumed that these other works of art all contribute. In addition, sufficient margin must be established when determining the N value are held to record future changes within the process able to capture, such as the construction of more works of art in the process. Thereby being a maximum of 10 independent, equivalent failure probability contributions are advised to prevent too much conservatism.

4.1.4.2. Determination of failure probability not close

The failure probability for failure to close $P_{f,\,KW,\,ns}$ follows from the failure tree in Figure 17. The following events can be distinguished:

1. The artwork is open at an approaching high water, or there is a need for closure (P_{open}).
2. Failure of the high-water closure of the work of art (P_{ns}) and failure of repair of the failed regular closure ($P_{f,herstel}$).
3. 3. Failure of the soil protection behind the structure (limit state function Z_{NS1}).
4. Failure of the artwork as a whole after failure of the soil protection has taken place (limit state function Z_{NS2}).
5. Exceeding the storage capacity (limit state function Z_{NS3}).

It can be seen from the failure tree in Figure 17 that:

$$P_{f,KW,NS} = P_{open} \cdot P_{ns} \cdot P_{f,herstel} \cdot P(\{Z_{NS1} < 0 \; EN \; Z_{NS2} < 0\} \; OF \; Z_{NS3} < 0) \tag{4.3}$$

Here is:

$P_{f,\,KW,\,NS}$	Risk of flooding after non-closure [per year]
P_{open}	Opportunity on open flood barrier with a closing question [-]
P_{NS}	Probability of failure closure with a closing question [-]
$P_{f;herstel}$	Probability of failure to recover from failed closure [-]

P (Z_{NS1} <0)	Risk of failure of soil protection [-]

P (Z_{NS2} <0)	Chance of collapsing artwork in the event of a collapsed soil protection, also called $P_{f,\ KW\ |erosie\ bodem}$ called [-]

P (Z_{NS3} <0)	Probability of exceeding the storage capacity [-]

The first term in formula 4.3 concerns the probability that the artwork is open when a high water is present. The second and third term indicate what the chances are that the artwork is not closed at the moment the outside water level it becomes so high that there are flood effects on the inside occur. This concerns a failure probability per closing question from the function high water times.

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The last term indicates how many closing questions per year arise from the function of high water times. If it is assumed that the chance of failure of a work of art after erosion of the soil protection is equal to 1, then this last term reduces to:

$$P(\{Z_{NS1} < 0\ EN\ Z_{NS2} < 0\}\ OF\ Z_{NS3} < 0) = P(Z_{NS1} < 0\ OF\ Z_{NS3} < 0)$$

Then the following applies:

$$P_{f,KW,NS} = P_{open} \cdot P_{ns} \cdot P_{f,herstel} \cdot P(Z_{NS1} < 0\ OF\ Z_{NS3} < 0) \qquad 4.4$$

Either:

$$P_{f,KW,NS} = P_{open} \cdot P_{ns} \cdot P_{f,herstel} \cdot \mathrm{P}\{\min(Z_{NS1}; Z_{NS3}) < 0\} \qquad 4.5$$

The limit state functions for failure of the soil protection ZNS1 (formula 4.6) and for exceeding the composting power Z_{NS3} (formula 4.7) are shown below:

$$Z_{NS1} = Q_c - Q_{in|open} = q_c \cdot B_{sv} - Q_{in|open} = u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv} - Q_{in|open} \qquad 4.6$$

$$Z_{NS3} = V_c - V_{in|open} = A_{kom} \cdot \Delta h_{kom} - t_s \cdot Q_{in|open} \qquad 4.7$$

Here is:

Q_c	Critical flow rate at which soil protection collapses [m³ / s]

$Q_{in|open}$	Acting inflowing flow through the artwork at a certain water level given that the artwork is not closed high water [m³ / s]

q_c	Critical inflowing flow with respect to the soil protection [m³ / s / m]

Bsv	Power-carrying width soil protection [m]

u_c Critical flow speed soil protection [m / s]

h_{bi} Inland water level in relation to NAP [m]

h_{bb} Height top soil protection [m + NAP]

V_c Maximum available volume of storage capacity in the hinterland, where no significant consequences (flooding effects) occur [m³]

$V_{in|open}$ Inflowing volume due to the closed artwork for a high water flow a high water period [m³]

A_{kom} Coming away surface [m²]

Dh_{kom} Maximum permitted level increase in the hinterland [m]

t_s Duration of the high tide wave [s]

Under the simplifying assumption that all strength terms in formulas 4.6 and 4.7 are (almost) deterministic, the last term in formula 4.5 can finally be rewritten to:

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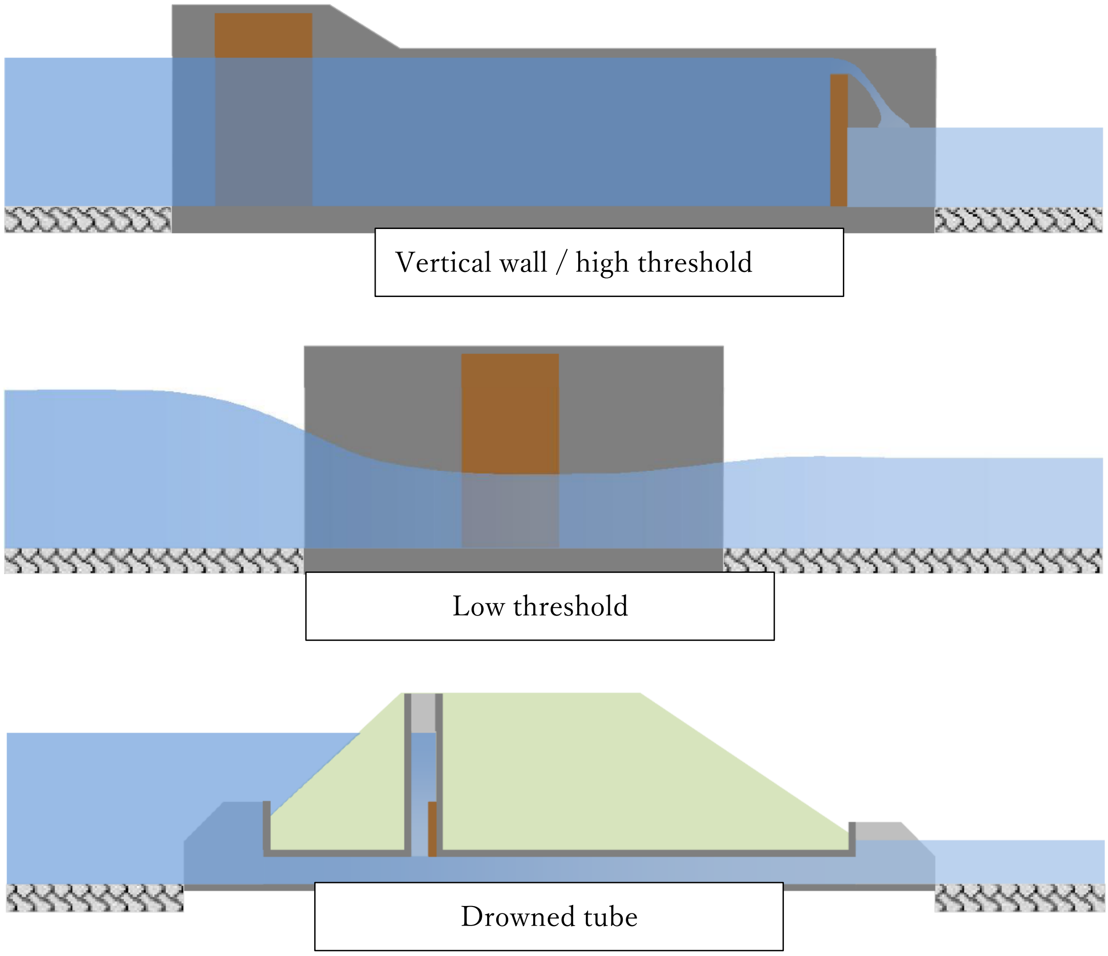
$$P\{\min(Z_{NS1}; Z_{NS3}) < 0\} = P\left(Q_{in|open} > \min\left\{u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv} \; ; \; \frac{A_{kom} \cdot \Delta h_{kom}}{t_s}\right\}\right) \quad 4.8$$

An important load variable in both comparisons is the total inflowing flow $Q_{in|open}$ by the non-flooded closed artwork. Depending on the geometry of the artwork when it is not high-water-tight, different models are applicable to determine this inflowing flow. Roughly, the following models exist to determine the inflow rate:

- Vertical wall / high threshold. There is no direct contact between inside and outside water. This situation occurs, for example, in the case of a lock in which the flood-tight closing means are not closed and water flows over the lock doors.
- Low threshold (spillway). There is direct contact between inside and outside water and the inflow surface is not limited at the top. This situation occurs, for example, in the case of a flood barrier, when the flood defense is not closed.
- Drowned tube. There is direct contact between inside and outside water and the inflow surface is limited on the upstream side. The waterway in the artwork through which water flows in is completely under water. This situation occurs, for example, in the event that a discharge channel consisting of a tube is not closed by the flood defense.



※（Rijkswaterstaat, Central government　2018）52 頁より作成。

Figure 19 Do not close diagrammatic representation of impulse models

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The strength variables V_c and q_c/u_c (depending on the soil protection, see chapter 9 Soil protection) are the available storage and the critical inflow flow/the critical flow rate for soil protection. These are further elaborated in section 4.2.3.

For the strength variables are also calculation values. For the collapse of a work of art given the collapse of the soil protection is initially maintained that this always happens as soon as the soil protection has collapsed. Tightening of this is possible requires a specialist consideration outside the scope of this work guide falls. The use of this additional strength, if any, fits incidentally better when assessing than with designs. Recommended for designs to be cautious about this.

4.2 Taxes and incoming flow

The hydraulic loads determine in combination with the inland water level and the geometry of the non-high-watered closed artwork the inflowing flow $Q_{in|open}$. For the hydraulic loads, this depends on the geometry and the associated in-flow model for a combination of water levels and waves (for model vertical wall / high threshold) or sec water levels (for models low threshold and drowned tube). Besides the absolute values of water levels and waves also play a role in the water level. This allows the outdoor water level to be determined with the available storage room (V_c) is still sufficient to reduce the inflow volume during one high water ($V_{in|open}$).

4.2.1 Water levels and waves

The different in-flow models as included in this manual will be fed with data on water levels (all models) and waves (only model vertical wall / high threshold, see table 6)

Table 6 Need for input of various influx models

Inset model	Outside water level	Waves
Vertical wall / high threshold	Yes	Yes
Low threshold	Yes	No
Drowned tube	Yes	No

※（Rijkswaterstaat, Central government　2018）53 頁より作成。

This applies to the 'vertical wall / high threshold' inflow model should be made of combined statistics of water levels and waves. This combined statistic can be derived from Hydra-NL (see method at *height* in chapter 5). This allows the chance of occurrence of a combination of outside water level and waves are determined in the case of a known defensive height in a closed state that is not high water leads to exceeding the critical flow or critical inflow volume.

This applies to both the 'low threshold' inflow model and 'drowned tube' No role 16 games with the incoming flow. Here you can use Hydra-NL exceedance frequency / probability of that outside water level shall be determined at these in-flow models lead to the critical flow or the critical inflow volume.

[16] With the low threshold model in particular, situations are conceivable in which

waves do indeed contribute to the inflow flow. Consider a situation with a relatively small water depth above the threshold in combination with large waves. In that case, better use can be made of the vertical wall / high threshold model.

(54 頁) To this end, an exceedance frequency line should be used with Hydra-NL made. If the failure mechanism *does not close*, the inflow can occur at the not being closed is already leading to a relatively high frequency at outside water levels to flooding consequences. In Hydra-NL frequencies are higher than once in the 10 year (exceedance probability is 1/10 per year) not included in the databases. When outside water levels with an exceedance probability greater than 1/10 per year already lead to problems with an open work of art then there are two methods possible. These are included in section 3.3.3.

4.2.2 Water level course outside water

The course of the flood wave in time determines how much water there is during a high tide wave due to the artwork not closed to high water can flow in. This volume of incoming water is important to be able to assess whether the combating capacity is exceeded. In the chapter cup storage (chapter 10) will be discussed further here.

4.2.3 Emerging incoming flow when his work is not closed

This section deals with the formulas with which the inflowing flow can be determined at the time the artwork is not high-watering is closed. This inflowing flow depends on the situation (geometry) of the artwork at the moment that it is not closed (see section 4.1.4.2).

In the literature (including *Applied fluid mechanics, hydraulics for hydraulic engineers* ([Ref.4.4])) can provide more information about the models and the coefficients are found.　Especially the drain coefficients can be subject are of extensive studies. For the usual works of art it is recommended to these coefficients in the first instance with simple approximations to determine then to accept. It should be noted that when designing the primary function the drainage coefficients can already be looked at. There can be when designing the water-retaining function may already be used.

4.2.3.1. Inlet vertical wall / high threshold

For the 'vertical wall / high threshold' inflow model, please refer to section 5.2.3 of the Height section. This model has been extended described.

4.2.3.2. Inflow model low threshold.

For this situation, there are two models that both have their own formula know for the inflowing flow: the complete and the imperfect spillway. The formulas for the spillway are presented in the literature in various ways displayed. Here is chosen for a way in which work is done with levels compared to NAP. It is also assumed that the outside water is so large that the water flow speed towards the artwork is very small, causing the energy level outside the dike is equal to the outside water level.

For the imperfect spillway, the inland water level has an influence on the size of the inflowing flow. The model has the following formula and the associated precondition with regard to the outside water level in relation to the inland water level and the threshold height.

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$$Q_{in,onvolk.} = B \cdot m_{onv} \cdot (h_{bi} - h_{dr}) \cdot \sqrt{2 \cdot g \cdot (h_{bu} - h_{bi})} \qquad h_{bu} < {}^{3}\!/_{2} \cdot h_{bi} - {}^{3}\!/_{2} \cdot h_{dr} \qquad 4.9$$

For the complete spillway, the inland water level no longer has any influence on the inflowing flow:

$$Q_{in,volk.} = B \cdot m_{vol} \cdot \sqrt{g \cdot (h_{bu} - h_{dr})^3} \qquad h_{bu} \geq {}^{3}\!/_{2} \cdot h_{bi} - {}^{3}\!/_{2} \cdot h_{dr} \qquad 4.10$$

Here is:

$Q_{in,onvolk.}$ Emerging incoming flow through the artwork involving one imperfect flow [m^3 / s]

$Q_{in,volk.}$ Emerging incoming flow through the artwork involving one complete flow [m^3 / s]

B Width of the flow opening [m]

m_{onv} Drainage coefficient for the imperfect spillway [-]

m_{vol} Discharge coefficient for the complete spillway [-]

h_{bi} Inland water level compared to NAP [m]

h_{dr} Top threshold compared to NAP, which in practice is the top of the bottom of the artwork is [m]

h_{bu} Outside water level compared to NAP [m]

4.2.3.3. Immersion immersed tube

In this situation the water-carrying element is completely under water. In front of a drowned tube the following intake formula applies:

$$Q_{in,koker} = \mu \cdot A \cdot \sqrt{2 \cdot g \cdot (h_{bu} - h_{bi})} \qquad 4.11$$

Here is:

$Q_{in,koker}$ Acting incoming flow through the artwork involving one drowned tube [m^3 / s]

A Minimum area of the through-flow openings [m^2]

μ Discharge coefficient for drowned tube [-]

h_{bi} Inland water level in relation to NAP [m]

h_{bu} Outdoor water level compared to NAP [m]

The formula for the drowned tube (formula 4.11) has many similarities with the formula for the imperfect spillway (formula 4.9). The drain coefficients are explicitly different.

4.3 Strength

The strength of a work of art with regard to non-closing consists of a number of parts (see also formula 4.3c):

1. The reliability of the closure (P_{ns}), which also includes the failure probability of the recovery ($P_{f,herstel}$).
2. The likelihood that a work of art will not be closed at a high water level when such a high water occurs that in the event of a non-closed work of art this leads to flooding consequences in the hinterland (P_{open}).

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3. The geometry of the work of art in a non-high water closed condition, consisting of the then present retaining height and the dimensions of the through-flow opening (s). The geometry, in combination with the inland water level, determines the load on the soil protection (the inflowing flow $Q_{in|open}$) and the cup storage (the inflowing volume $V_{in|open}$).
4. The characteristics and scope of the available storage capacity of the underlying water system (V_c). (See chapter 10 Bowl storage)
5. The structure and dimensions of the soil protection behind the artwork.

The first two parts determine the chance of the artwork not standing high-water closed at the moment that this is required. Parts 3 to 5 relate to the result when they are not closed in a high-water condition.

4.3.1 Simple design rules reliability closure

A work of art can be designed in such a way (design, functioning and number and type of reversing means) that an analysis with respect to non-closing can be omitted immediately, because it is clear in advance that the probability of flooding is negligible. To this end, a number of simple design rules have been drawn up that apply to specific types of works of art. A few simple test rules are also included in Appendix III Strength and safety of the WBI2017 ([Ref 4.3]). The design rules are partly derived from this. The design rules are more specific and stricter. The latter in particular leads to fewer simple design rules than simple test rules.

The simple design rules mentioned in the following subsections do not cover all conceivable situations. This means that in practice works of art can occur of which it can already be indicated in advance that the contribution to the flood risk is negligible, but that do not fall under the simple design rules. If a detailed analysis is not carried out in that case, it must be demonstrated by means of a qualitative analysis, supported by global quantitative considerations, that the contribution to the flood risk is negligible. Arguments for this can be found in the (limited) dimensions of the water-carrying elements, the threshold height in the non-closed state, the use of the artwork or the (very) limited consequences when the artwork remains open.

Regardless of how it is designed, it is desirable to include at least one reversal element in the current-carrying elements of a work of art at all times[17].

4.3.1.1. Locks

If a lock is provided with at least two high-watering heads and also the lock chamber is capable of turning the same high water, a further analysis of non-closing is not necessary as long as the following condition is met: the lock doors are never all open at the same time (for example, to grant a free passage to the ship in the event of no decay, or to allow water to be discharged via an opened lock).

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[17] The simple testing rules of the WBI2017 or further analyzes may show that no

turning device is required to meet the reliability requirement under the Water Act. For a design, however, it is not considered desirable to include any turning agent with a work of art in the primary flood defense. In the event of a disaster or maintenance, it is important that a work of art can still be closed off, even if it is only to prevent damage.

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If the above simple design rule is met, this means that the lock is always closed high water. Only in case of a calamity (collision) it can occur that this is not the case. Acceptance is considered under strength and stability and therefore does not affect the failure mechanism to close.

4.3.1.2. Grinded

When a pumping station is provided with two independent high-level reversing devices which are connected to the pumping operation, no further analysis of non-closing is required. 'Switching the pump operation' means in this case that they open when the pumping station starts to rotate and that they automatically close when the pumps are stopped. The independence implies that one of the reversing means consists of a non-return valve (or possibly a guard door) and that the other reversing device is, for example, an automatically closing butterfly valve. These reversing devices are independent because they do not have the same drive and energy source and are far enough apart so that they cannot both be blocked simultaneously by the same obstacle.

When a pumping station is provided with several milling passes (pressure cookers) and all of these are provided with two reversing means which satisfy the description above, a further analysis of the pumping station is not required.

When a pumping station is equipped with reversing means which comply with the above, the chance of undesired opening and the inflow of such amounts of water that this leads to flooding consequences is negligible.

4.3.1.3. Load sluice / sewer overflow

Drainage locks can be risky objects with respect to not closing. However, there are cases where this risk is very small. In the following case, a further analysis of non-closure is not required.

A drainage sluice with an internal diameter of less than or equal to 1 m, provided with at least one high-water reversal that closes automatically and of which the water-carrying element (conduit) enters the outside of the impact zone of the flood defense in an (inspection) pit, the access of which at least at ground level.

In this case the risk is negligibly small, because the chance of a closed reversing agent is large. In addition, the consequences of failure of the closure will be limited, because the pipe diameter is limited and the risk of breaching is negligible.

4.3.2 The failure probability of closure

- The probability that the closure of a work of art fails is one of the most important parameters for the failure mechanism. The size of this opportunity depends on a number of aspects:
- The use of the artwork in relation to its primary function, being the passing of the flood defense by water, goods, people, vehicles or vessels.
- The number and type of reversals.
- The number of flow openings.

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- The emergency organization of the manager concerned.
- Possibilities for possibly closing the artwork in an alternative manner with a failing regular closure ($P_{f,herstel}$).

When designing a work of art, it is recommended to not initially take into account the positive contribution of the chance of recovery. This implies a failure probability of recovery ($P_{f,herstel}$) of 1. This does not detract from the fact that the practical design can take into account possible possibilities for an alternative to closing a work of art. This could include, for example, accessibility and accessibility of the power-carrying elements of the artwork by people and equipment.

4.3.2.1. Failure of regular high-water closure (P_{ns})

The probability of regular closure is indicated by the parameter P_{ns} and knows the unit per question [1 / question]. The size of this opportunity is influenced by four sub-processes: alarm, mobilization, operation and technical failure. The emergency organization, elaborated in, among other things, the flood scenario, has a major influence on these sub-processes.

The value of P_{ns} can roughly be determined by three methods.

1. With the help of score tables (see Determine the probability of not closing per closing question with score tables ([Ref 4.5] and Guide to assurance of reliability of closure in scenarios [Ref 4.6]) This method applies to reversing devices that are not closed at exercise of the primary function (s) of the artwork but only when approaching a high water.
2. Using standard failure probabilities for reversals that are closed regularly from their primary function. In Appendix B, this has been done for a number of frequent reversals.
3. Using an advanced error tree analysis. Such an analysis is particularly important for larger, more complex works of art.

For works of art, a combination of methods 1 and 2 can also be useful if both specific high-water retainers are present and reversals that are also used in the primary process.

Depending on the chosen method, the operation of the artwork and the interdependence regarding the failure of the closure, the number of flow openings in the analyzes can be included. In the first method, a failure probability of the closing of the specific high-water defenses is determined per flow opening. If several flow openings are present, the use of these flow channels determines how they can be taken into account when determining the total chance of the closure failure.

Example: A drainage lock has two tubes that are always used simultaneously. The reversing means consist per tube of a non-return valve and a manually operated slide. It is known that the failure probability of non-closing is dominated by technical failure. The total failure probability of non-closing P_{ns} is determined using method 1 (score tables) for the slide and method 2 (default failure probabilities) for the non-return valve. The failure of the closure of one tube is virtually independent of the closure of the other sleeve. **(59 頁)** The table below shows the various events with their chance of occurrence. It should be noted that the probability of failure per box (P_{ns}) is in any case less than 10^{-2} per question.

Scenario	Tube 1	Tube 2	Opportunity
Both closed	Close	Close	$(1\text{-}Pns)\cdot(1\text{-}Pns) \approx 1$
Sleeve 1 not closed	Do not close	Close	$Pns\cdot(1\text{-}Pns) \approx Pns$
Tube 2 not closed	Close	Do not close	$(1\text{-}Pns)\cdot Pns \approx Pns$
Both are not closed	Do not close	Do not close	$Pns\cdot Pns = Pns^2$

※（Rijkswaterstaat, Central government　2018）59 頁より作成。

The probability of all scenarios added together 1. When failure occurs if at least one of the tubes remains open (the probability of this event equals $P_{ns} + P_{ns} - P_{ns}{}^2 \approx 2P_{ns}$) serves in the further analysis (inflowing flow) to take into account a water-carrying cross section equal to the flow area of one tube. The other event where failure of the flood defense occurs is when both tubes do not close. The probability of this is equal to $P_{ns}{}^2$ per closing question. In that case, the flow-through surface of both tubes must be applied together in the further analysis. In practice it is almost always the case that the probability of failure per tube is small, so that the probability that one tube is not closed ($2 \cdot P_{ns}$) is substantially greater than the chance that both tubes are not closed ($P_{ns}{}^2$). The last chance is often negligibly small.

4.3.2.2. Failure of recovery from a regular closure

In determining the probability of failure of a closure, the chance of recovery from a failed regular closure ($P_{f,herstel}$) can also be included. This means the possibility that a failure in closing can possibly be closed in an alternative manner.

Example: An inlet lock consists of a pipe through the barrier with a pipe diameter of 1.0 m and a sliding shaft in the crown. Such an inlet lock can possibly still be closed if the retention means are not closed by pouring clay or sand bags into the pipe via the slide shaft.

In the score tables (see method 1 in section 4.3.2.1) recovery of the regular closing process is already included, as well as the influence of a possible second turning means. An additional (alternative) closing method can, however, still be charged separately.

For the second method to determine P_{ns} (standard failure probabilities), the probability of recovery can be explicitly included. This is not yet processed in the standard failure probabilities.

In an advanced error tree analysis, recovery possibilities in case of a failed closure are included as standard.

The probability that the recovery of a failed regular closure will depend to a large extent on the available time between the moment of the regular closure and the moment the water is so high that inflow can no longer be stopped. **(60 頁)** In addition, issues such as the accessibility and dimensions of the artwork play an important role.

4.3.3 The chance that a work of art is open

When a high water approaches and the artwork is already closed, the failure mechanism does not necessarily close. It is therefore important to consider how often and / or during which part of the time an artwork is open. This obviously has a direct relationship with the primary function (s) of the artwork and more specifically with that of the high-water retainers. When designing a work of art, it is therefore important to make an estimate of the required input from this function (s).

The probability that a work of art is not closed (P_{open}) consists of two components (see also WBI2017 [Ref. 4.1]).

1. The first component concerns the period over which the artwork actually exercises its primary function (s) and the high-water retarding means are therefore not closed. In case of an approaching high water, a closure is then required to make the barrier high-water-bearing.
2. The second component concerns the conditional probability of the artwork standing open after a closure of high-water retaliators has failed due to their primary function. This component therefore represents the period over which the flood defense is not closed in a high-water way because the reversing means are under repair and a closure is therefore also not possible.

The above components must be viewed separately in an analysis of a work of art and be included in the considerations. Because every situation and every work of art is unique again, it is not possible to give a generally applicable wording for P_{open}, which means that you can always work.

Formula 4.12 gives a global general formulation for P_{open}. This is also included in WBI2017 ([Ref 4.1]). This formula is based on a number of assumptions. The most important are:

- Closures occur throughout the year, regardless of the chance of high water;
- The probability of non-closure (Pns) in a regular closure is the same as the probability of non-closure if this occurs from the point of view of high water;
- Repairs that prevent the barrier from closing can only be caused by the failure of the closure. Other causes such as maintenance are not considered. The implicit assumption is therefore that maintenance will be carried out in a period in which a high water level cannot be expected (monitoring water levels) or where measures have been taken to guarantee the high-level function during maintenance (alternative flood defense).
- In the case of repairs as a result of a failed closure, the barrier remains in the open state during the entire repair period.

$$P_{open} = N_{open} \cdot \left(T_{open} + T_{rep}\right) \qquad 4.12$$

In which:

P_{open} Total chance that a work of art is open at the moment of an approaching high water for a period of one year [-].

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N_{open} The number of closing questions from the primary function (s) of a work of art [1/year].

T_{open} The average duration per cycle of opening and closing in which the artwork comes from his primary function (s) is open [year].

T_{rep} The average amount of time needed to make a failed closure of the high-water mark reversing / repairing means from the primary function (s) [year].

The repair duration can be estimated with a design based on possible failure causes and the expected speed with which these failure causes can be remedied. It should be borne in mind that this is the time required to restore the high-water function, possibly with temporary facilities.

Formula 4.12 is somewhat special because it does not result in a pure chance. This becomes clear when the product of the number of closing questions (N_{open}) and the repair time of a failed closure (T_{rep}) is large. As a result, it is possible that in the above formulation P_{open} gets a value that is greater than 1. This thus indicates that there is a somewhat forced formulation and that for each situation a separate consideration of the possibility of the artwork standing open in the event of an approaching high water to be made. In a number of cases the given formula will be directly applicable.

Example

A lock with lock doors and a separate high-water door that is closed as standard at the end of each day (after 12 hours). If the recovery time of a failed closure of the high-water door is 8 hours, the following may apply:

$$P_{open} = N_{open} \cdot \left(T_{open} + T_{rep}\right) = 365 \cdot \left(\frac{12 + 8}{365 \cdot 24}\right) = \frac{5}{6} \qquad 4.13$$

In the above result it is visible that the formulation of P_{open} as given by formula 4.12 initially allows the repair time to participate directly in the probability of being open. Under the condition that the probability of non-closing at a regular closure (at the end of the service) is equal to the probability of non-closing in the situation that the lock is in operation and high water is present, this will eventually lead to interpretation of the basic formula 4.3 to the correct approach to the failure probability for non-closure

4.3.4 Geometry of the closed artwork that is not high water

The geometry of the artwork determines how much water flows in and how this happens, given that the artwork is not closed high water. This geometry is included in the various intake models (see section 4.2.3).

4.3.5 Bowl storage

For the purpose of determining the reliability of the closure, the storage capacity must be converted into a flow per running meter by the non-high-water-tight closed artwork. From recasting of formula 4.7 it follows that there is / is no question of exceeding the capacity of the storage as follows:

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$$Q_{in|open} = \frac{A_{kom} \cdot \Delta h_{kom}}{t_s} \quad 4.14$$

Here is:

$Q_{in|open}$ Average inflow through the artwork [m³ / s]

A_{kom} Compressing surface [m²]

Δh_{kom} Permitted level increase for storage [m]

t_s Duration of high-water wave [s]

It should be noted that the above formula is based on a schematization of the high water wave as a block with a constant time duration and a constant inland water level (for example when the combing capacity is very high). For systems with a 'short' load duration (coast, lakes) this is usable, for systems with a long-term load (rivers, deltas) not. For these systems, the output of the Water Level Gradient tool must be used to correctly determine the inflow volume. For more information, see chapter 10 Cup storage.

If inflow can be schematized with the model vertical wall / high threshold (see section 4.2.3.1), the above formula is included in a schematic high water wave as a block with a constant duration in formula 5.15 (see chapter 5). It should be noted that this may be applicable in the case of a lock, in which the deflecting height of the inner door (s) that is not flooded is to be derived or verified with the requirement for not closing.

The method of determining the parameters A_{kom}, Δh_{kom} and t_s is discussed in the chapter on Reservoir Storage (chapter 10).

4.3.6 Soil protection

The strength of the soil protection must be converted to a flow per running meter by the closed artwork that is not turned upside down. From recasting of formula 4.6 follows that:

$$Q_{in|open} = u_c \cdot (h_{bi} - h_{bb)} \cdot B_{sv} \quad 4.15$$

Here is:

$Q_{in|open}$ Average incoming flow through the artwork [m³ / s]

u_c Critical depth average flow speed soil protection [m / s]

h_{bi} Inland water level in relation to NAP [m]

h_{bb} Height top soil protection compared to NAP [m]

B_{sv} Power-carrying width soil protection [m]

Because the inflowing flow rate in the in-flow models 'drowned tube' and 'low threshold' depends on the inland water level and can rise during undesired inflows, this inflowing flow is not constant. However, the critical flow from soil protection is also not constant because it also depends on the inland water level. For these in-flow models it is therefore recommended to first look at storage, so that the course of the inland water level during a high water wave is clear, and then determine the critical flow rate over time.

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For the 'vertical wall / high threshold' model, the above formula goes over in the formula given in the chapter on height:

$$q_{os/ol} = \frac{u_c \cdot (h_{bi} - h_{bb)} \cdot B_{sv}}{B} \quad 4.16$$

Here is:

$q_{os/ol}$ Average throughput / overflow flow over the artwork per linear meter [m^3 / s / m]

B Width of the crown of the artwork [m]

The critical flow rate of soil protection is often known from the design of the other function (s) of the artwork. If this is not the case, then this critical flow rate can be determined on the basis of Chapter 9 Soil Protection. The parameters h_{bi} and h_{bb} speak for themselves and are also discussed in chapter 9.

4.4 Concrete design recommendations

The design strategy for non-closure aims to design the artwork in such a way that it meets the requirements for not closing from flood risk management. Because the design and the use of the artwork are mainly determined by the primary function of the artwork, the design strategy for non-closure is largely based on the determination of which direction means can meet the requirement for not closing. It is recommended to initially strive for a robust solution. This means that when the required safety can be achieved relatively easily by applying conventional reversing means, this is preferable to the optimal use of the residual strength in the form of cup storage and strength of the soil protection.

4.4.1 Simple design rules

A design based on simple design rules is preferred. After all, this ensures that not closing is not a normative mechanism and can be easily tested in the statutory assessment. However, the number of situations in which simple design rules can be used is limited.

In addition to the limits to the practical application of the simple design rules, it also applies that from an economic point of view it must lead to a desired solution. For example, the cost of turning a lock into a double high water can be considerably higher than for a lock with a cavity and the inner head substantially lower than the high water turning outer head.

4.4.2 Detailed design

If the simple design rules cannot be met, further analysis is required. This can be done on the basis of the step-by-step plan presented in this chapter. Depending on the specific situation, a limited number of steps may suffice. An example of this is a very large cup storage, in which it is clear in advance that this will never be decisive.

When designing via the step-by-step plan, it is recommended to initially design robust as well. The extra costs, for example in the form of an additional retraction, are often relatively limited at construction, while relatively much certainty can be gained. **(64 頁)**With large objects this is usually not the case and extensive analysis can hardly be prevented.

4.5 References and background documents

[Ref. 4.1] Delhez, R, WTI 2017 Test rules for art works - Test report Reliability Closure, Deltares, 1220087-002 GEO-0009, Version D1, December 2015

[Ref. 4.2] Rijkswaterstaat WVL, Guide Design with Flood Opportunities – Safety Factors and Taxes for new Flood probability standards, version OI2014v4, February 2017

[Ref. 4.3] Ministry of Infrastructure and the Environment, Regulations for the

Safety of Primary Welds 2017 - Appendix III Strength and safety,
[Ref. 4.4] Nortier, Applied fluid mechanics, hydraulics for hydraulic engineers, ISBN 90-401-0318-6, 1996
[Ref. 4.5] Rijkswaterstaat WVL, Working method to determine the probability of not closing per closing question with scoring tables, 1 November 2017, Definitive
[Ref. 4.6] Rijkswaterstaat WVL, Guide to assurance of reliability of closure in scenarios, Background report for the use of the score tables for the failure mechanism not to be closed, November 2017, Definitive
[Ref. 4.7] WBI2017 - Schematization manual reliability closure of the work of art, Rijkswaterstaat WVL, final version, January 2, 2017

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5 Height

5.1 Introduction

5.1.1 Introduction failure mechanism

The deflecting height of high-water structures must be large enough to keep the amount of wave overtopping or overflow within acceptable limits. A design verification of the requirements of the Water Act involves a flood with significant consequences. In this chapter, handles are given to determine the required height of a water-retaining work of art. This is relevant for works of art that independently take care of the vertical height, such as lock locks, flood barriers, loading and unloading locks and denominations.

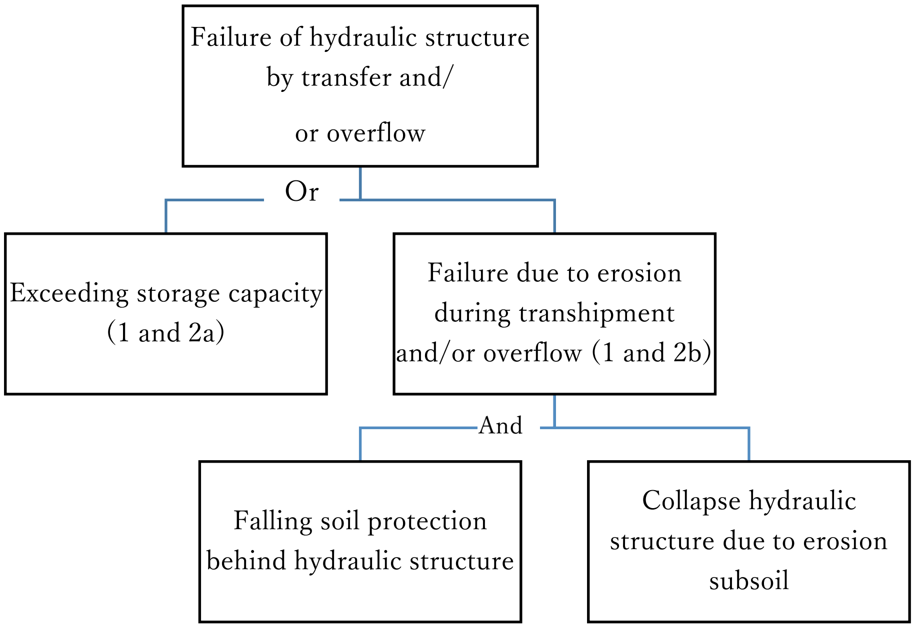
For works of art in which the vertical height is provided by the dike body (such as divers and pressure pipes), a verification of the artwork on transhipment and/or overflow is not relevant.

By far most of the constructions where transfer and/or overflow is relevant, it concerns a vertical rigid wall on relatively deep water. For this type of construction the method described in this chapter is therefore arranged. Different design types are discussed briefly in section 5.5.

The method explained in this chapter is aimed at an assessment using Hydra-NL. This was chosen because, at the time of writing this Riskeer, Riskeer is still structured as an assessment tool and not as a design tool. For example, Riskeer is currently not equipped to take climate developments into account. It is expected that the functionality of Riskeer will eventually be made suitable for designs.

5.1.2 Phenomenological description

A detailed description of the failure mechanism handling and/or overflow can be found in chapter 4 of the WBI test track report Height [Ref. 5.1]. The failure tree is shown in Figure 20 below.



※（Rijkswaterstaat, Central government　2018）65 頁より作成。

Figure 20 Failure tree failure mechanism transfer and/or overflow

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Failure as a result of a shortage of altitude occurs when there is so much water flowing over the high-water-tight closed artwork or connecting structure as a result of wave overtopping and/or overflow, which leads to substantial damage and/or casualties (flooding consequences). Failure of the flood defense occurs when:

(1) presenting a high water

AND (2a) The work of art itself will remain, but the inflowing volume due to wave overtopping and/or overflow cannot be recovered in the underlying (water) system without this leading to substantial damage and/or casualties (flooding).

OR (2b) The artwork itself collapses as a result of failure of soil protection with substantial damage and/or casualties (flooding) as a result. For this, the soil protection must first collapse behind the artwork. Subsequently, excavation pits form in the (not more protected) subsoil, after which the stability of the artwork is lost, resulting in a (progressive) breach in the flood defense. It is assumed that this situation always leads to the exceeding of the storage capacity.

5.1.3 Step-by-step plan and design strategy

5.1.3.1. Roadmap

For transhipment and/or overflow the situation is considered where the artwork is closed high-water. Figure 21 shows a possible method that can be used in many situations. It is assumed that the soil protection has already been dimensioned from other functions of the artwork and that there is a considerable storage space behind the artwork. Figure 21 shows the steps that are set in the design process in this method. Each step is briefly explained below the figure.

There are also situations conceivable in which a different method is optimal, for example when storage is clearly decisive. In that case, an assessment can be made of the critical flow in step 2 on the basis of storage and verification will take place in step 5 on the basis of the strength of the soil protection and the structural strength. It is important that in the design verification all components (soil protection, structural strength and cup storage) are discussed.

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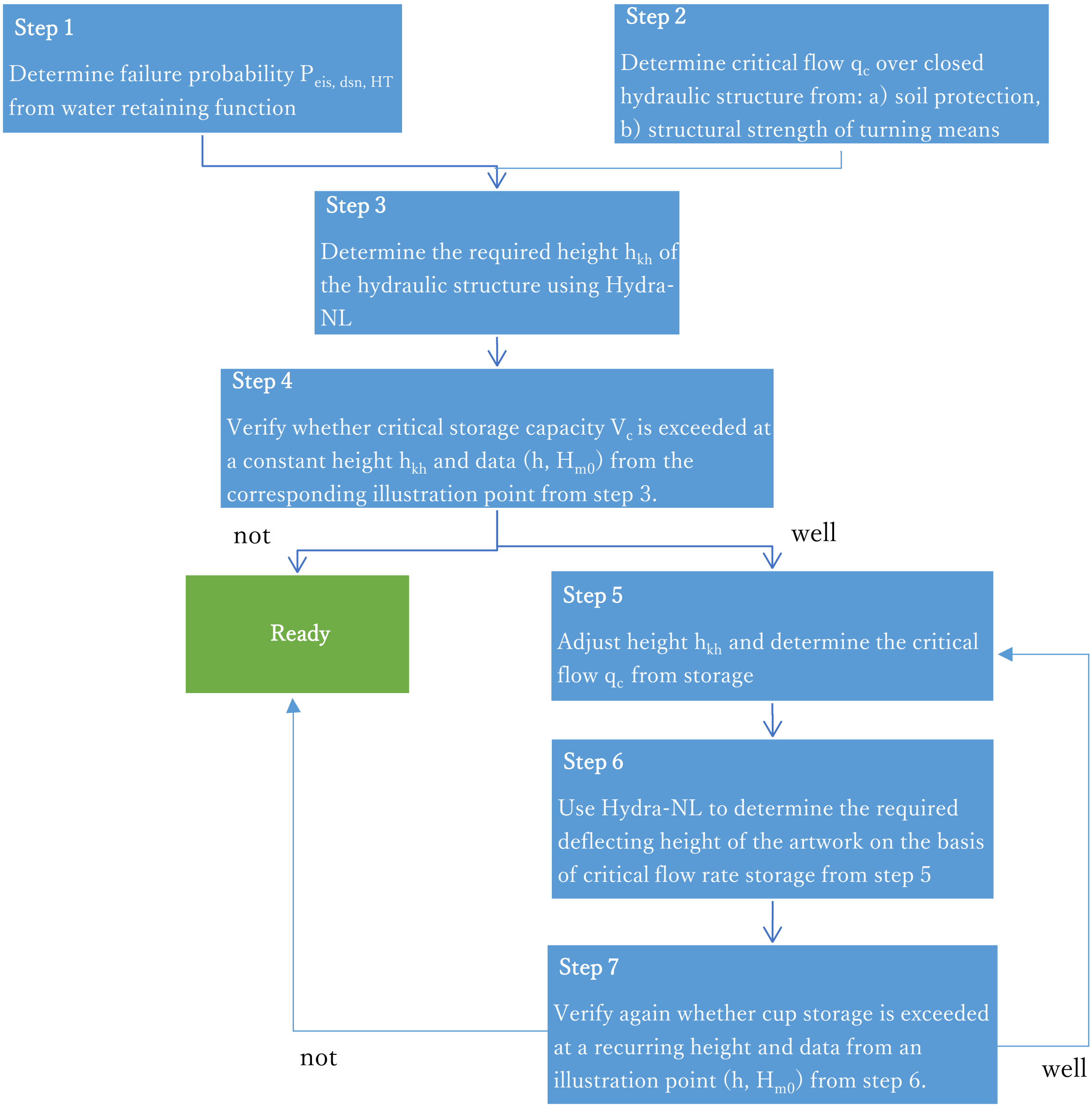

※（Rijkswaterstaat, Central government　2018）67 頁より作成。

Figure 21 Step-by-step plan for design failure mechanism handling and/or overflow

Below is a brief explanation of the steps indicated:

Step 1. The failure probability for height can be determined on the basis of the failure probability space distribution and the length effect factor N for height in the relevant standard path. This step is explained in more detail in section 5.1.4.

Step 2. In this step, it is determined which flow rate due to wave overtopping and/or overflow over the closed art work is permissible without this leading to (a) collapse of the soil protection behind the artwork or (b) structural collapse of the windings as a result of dynamic effects due to the overflowing beam. The smallest flow rate of both failure modes is used as input for step 3. This is further elaborated in section 5.3.3 (soil protection) and section 5.3.4 (constructive failure of retarding materials).

Step 3. As soon as the failure probability and the critical transfer/overflow flow rate are known, the required turning height of the artwork can be determined. Use can be made here of Hydra-NL. In Hydra-NL, the hydraulic load level HBN is determined with the failure probability and the critical flow rate as input parameters. Section 5.4 discusses the determination of the required deflection height in more detail.

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Step 4. It must be verified whether the combining capacity is not exceeded at the calculated turning height from step 3. The water level course can be used to determine the course of the water level, the water level and wave height from the illustration point of the Hydra-NL-calculation from step 3 as input. This step is further elaborated in paragraph 5.3.2. If the combining capacity is not exceeded, the calculated turning height is calculated in step 3 and the calculation process is completed. If the storage capacity is exceeded, then continue with step 5.

Step 5. In this step the height is determined whereby the available cup storage is not exceeded. This includes a certain value of the maximum transfer/overflow flow rate.

Step 6. With the maximum occurring transfer/overflow rate from step 5 and the failure probability from step 1, a new turning height and corresponding illustration point are found in Hydra-NL by means of an HBN calculation.

Step 7. On the basis of the height from step 6 and the water level and wave height from the corresponding illustration point, it can be determined again with the help of the Water Level Gradient tool whether the cup storage is exceeded. If this is not the case then the calculation is finished.

If this is the case, then steps 4 through 7 are repeated again.

Steps 4 to 7 are further elaborated in section 5.3.2 and chapter 11 Case.

5.1.3.2. Preferred strategy

In the above-mentioned step-by-step plan, the required deflecting height for a certain failure probability is determined on the basis of the available strength from storage and soil protection[18]. In other words: the turning height is the design variable. In principle, the strength of the soil protection or the size of the cup storage can also be adjusted. For the vast majority of works of art, however, this does not lead to the most efficient design. This is because it is usually (much) cheaper to increase the turning height than to strengthen the soil protection or to increase the storage space. The storage capacity and the strength of the soil protection are therefore taken as starting points in this method.

It is good to realize that the strength of the soil protection and (less obvious) the size of the cup storage are parameters that can be influenced by the designer in situations where the further raising of the artwork is not possible or no longer efficient.

5.1.4 Safety format

The starting point of the safety format is the requirement that the failure probability for transshipment and/or overflows $P_{f,\,KW,\,HT}$ is less than or equal to the failure probability for transhipment and/or overflow $P_{eis,\,KW,\,HT}$ or:

$$P_{f,KW,HT} \leq P_{eis,KW,HT} \tag{5.1}$$

[18] The critical flow rate of soil protection is often known from the design of the other function (s) of the work of art.

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This forms the basis for the verification of the design with regard to height. Below is a detailed discussion of the determination of the failure probability (section 5.1.4.1) and the probability of failure present (section 5.1.4.2).

5.1.4.1. Determination of failure probability

The failure probability is derived from the legal requirement for the standard route and can be determined with the aid of the following formula:

$$P_{eis,KW,HT} = \frac{P_{max} \cdot \omega_{HT}}{N_{HT}} \qquad 5.2$$

In which:

$P_{eis,\,KW,\,HT}$	Failure Chance for height of an individual work of art for a reference period of 1 year [-]
P_{max}	Failure Chance for the entire dike section (standard route) based on the maximum permissible probability of flooding from the water act[19] [-]
ω_{HT}	Failure-space factor for altitude [-]
N_{HT}	Length-effect factor for height [-]

A standard failure probability distribution for a standard route is included in Appendix III Strength and safety of the WBI2017 ([Ref.5.5]) and OI2014v4 ([Ref 5.2]). In this standard failure probability distribution a value of 0.24 is used for the failure probability factor for height (ω_{HT}). This value applies to both the height requirement of the dikes (failure mechanism for grass cover erosion crown and inner slope) and of the structures in a standard route. Deviating from this is possible but rarely takes place in practice. Note: it is not possible to adjust the failure probability room factor for height for (individual) artworks. Adjustment of the failure probability distribution can only be carried out at standard path level. See Chapter 2 for this.

The length-effect factor N_{HT} is equal to 1, 2 or 3 and is independent of the number of works of art in the standard trajectory. The value of N_{HT} can be found for each route in Appendix A of OI2014v4 ([Ref 5.2]).

5.1.4.2. Determination of failure probability of transfer and/or overflow

The failure probability for transshipment and/or overflows $P_{f,\,KW,\,HT}$ follows from the failure tree in Figure 20. The following three partial failure mechanisms can be distinguished:

1. Failure of the soil protection behind the artwork (limit state function Z_{HT1}).
2. Probability that the artwork as a whole will collapse after failure of soil

protection has taken place (limit state function Z_{HT2}).

3. Exceeding the storage capacity (limit state function Z_{HT3}).

It can be seen from the failure tree in figure 20 that:

$$P_{f,KW,HT} = P(\{Z_{HT1} < 0\ EN\ Z_{HT2} < 0\}\ OF\ Z_{HT3} < 0) \quad 5.3$$

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[19] In the Water Act this is referred to as the lower limit

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Here is:

P (Z_{HT1} <0)	Risk of failure of soil protection [-]
P (Z_{HT2} <0)	Chance of collapsing artwork in case of collapsed soil protection, also called $P_{f,KW|erosie\ bodem}$ called [-]
P (Z_{HT3} <0)	Likelihood of exceeding the storage capacity [-]

If it is assumed that the chance of failure of a work of art after erosion of soil protection is equal to 1, then formula 5.3 reduces to:

$$P_{f,KW,HT} = P(Z_{HT1} < 0\ OF\ Z_{HT3} < 0) = \quad \mathrm{P}\{\min(Z_{HT1}; Z_{HT3}) < 0\} \quad 5.4$$

The limit state functions (Z-functions) associated with the part-feeding mechanisms 1 (failure of soil protection) and 3 (exceed capacity of the comb) are defined as follows:

$$Z_{HT1} = Q_c - Q_{os/ol} = \ q_c \cdot B_{sv} - q_{os/ol} \cdot B \ = u_c \cdot (h_{bi} - h_{bb)} \cdot B_{sv} - q_{os/ol} \cdot B \quad 5.5$$

$$Z_{HT3} = V_c - V_{os/ol} = \ A_{kom} \cdot \Delta h_{kom} - t_s \cdot q_{os/ol} \cdot B \quad 5.6$$

Here is:

Q_c	Critical flow rate at which soil protection collapses [m³/s]
$Q_{os/ol}$	Acting transhipment/overflow flow over the artwork for a specific one water level [m³/s]

q_c Critical transfer/overflow flow with respect to soil protection [$m^3/s/m$]
B_{sv} Power-carrying width soil protection [m]
B Width of the artwork [m]
$q_{os/ol}$ Average throughput/overflow flow rate [$m^3/s/m$]
u_c Critical flow speed soil protection [m/s]
h_{bi} Inland water level in relation to NAP [m]
h_{bb} Height top soil protection [m NAP]
V_c Maximum volume of storage capacity in the hinterland, with no significant consequences [m^3]
$V_{os/ol}$ Incoming volume as a result of transhipment/overflow over the closed work of art during a high water period [m^3]
Δh_{kom} Permitted level increase for storage [m]
A_{kom} Compressing surface [m^2]
t_s Storm duration [s]

Under the simplifying assumption that all strength terms in formulas 5.5 and 5.6 are (almost) deterministic, formula 5.4 can finally be rewritten to:

$$P_{f,KW,HT} = P(q_{os/ol} > \min\left\{\frac{u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv}}{B} ; \frac{A_{kom} \cdot \Delta h_{kom}}{t_s \cdot B}\right\}) \qquad 5.7$$

An important load variable in both comparisons is the flow rate $q_{os/ol}$. Hydra-NL can be used to determine this. In Hydra-NL the relevant load parameters are considered as stochastic variables.

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This is discussed further in section 5.2.

The strength variables V_c and q_c/u_c are respectively the available storage behind the barrier and the critical transfer and/or overflow flow rate/the critical flow rate for soil protection. These are further elaborated in section 5.2.2.

For limit state function 2 (probability of collapse artwork given failure of soil protection), it is initially assumed that this mechanism always occurs as soon as the soil protection has collapsed. Tightening of this is possible but requires a specialist consideration that falls outside the framework of this Work Guide. The utilization of this additional strength, if any, fits in better with assessment than with designs. It is advisable to deal with this with caution.

5.2 Taxes and occurring transfer and/or overflow flow

The hydraulic loads, in combination with the geometry of the artwork, determine the flow rate ($q_{os/ol}$) that flows through wave overtopping and/or overflow over the flooded closed artwork. For the taxes this concerns a combination of water levels and wave heights on the outside. These can be determined with Hydra-NL for the failure mechanism. The water level course also plays a role in being able to calculate the available storage space back to a critical transfer/overflow flow rate that can be used in Hydra-NL to determine the required height or failure probability at a given height.

5.2.1 Combination of water level and waves

The occurring transfer/overflow flow rate is always determined by a combination of water level and waves. A high water level with small waves can give the same overtopping/overflow as a lower water level with very large waves. Data on this combined statistics of water levels and waves can be derived from Hydra-NL.

5.2.2 Water level course outside water

The course of the flood wave in time also determines how much water can flow in over a closed artwork during a flood. This volume of incoming water is important to be able to assess whether the storage capacity is exceeded. This will be discussed in more detail in the chapter on the storage of coffers (chapter 10).

5.2.3 Occurring transfer and/or overflow flow

This section gives the formulas that are included in the Hydra-models to determine the occurring transfer/overflow flow. These formulas are valid for vertical walls on relatively deep water (wave height just before the construction does not exceed about 1/3 of the water depth) and no heavy wave breaking occurs just before the construction. For vertical walls with a foreshore on which the waves adapt, reference is made to Section 5.5.

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When calculating the transshipment/overflow flow, three situations are distinguished:

1. Outdoor water level <turning height artwork: only wave overtopping
2. Outdoor water level> deflecting height artwork AND no or off-shore waves: overflow
3. Outdoor water level> deflecting height artwork AND waves: combination formula for wave overtopping and overflow

5.2.3.1. Golf conditions

To make the transition from onshore to offshore waves gradual, an influence factor γ_s is used in Hydra-NL (and Riskeer):

$$H_{m0} = \gamma_s \cdot H_{m0;input} \tag{5.8}$$

with:

$$\begin{array}{lll} 0 < \beta \leq 80 & \Rightarrow & \gamma_s = 1{,}0 \\ 80 < \beta \leq 110 & \Rightarrow & \gamma_s = (110 - \beta)/30 \\ 110 < \beta \leq 180 & \Rightarrow & \gamma_s = 0{,}0 \end{array} \tag{5.9}$$

and:

$$\begin{array}{lll} |\theta - \Psi_{KW}| \leq 180 & \Rightarrow & \beta = |\theta - \Psi_{KW}| \\ |\theta - \Psi_{KW}| > 180 & \Rightarrow & \beta = |(|\theta - \Psi_{KW}|) - 360| \end{array} \tag{5.10}$$

Here is:

H_{m0}	Significant wave height after correction with influence factor [m]; this is the wave height used to calculate the overtopping flow
γ_s	Influence factor offshore waves [-]
$H_{m0,\ input}$	Significant wave height in illustration point Hydra-NL calculation [m]
β	Angle of wave attack [°]
θ	Wave direction [°]
ψ	Normal of the artwork [°]

5.2.3.2. Situation with wave overtopping only

The formula for the inflow rate as a result of wave overtopping is (see formula B2.4 from the Guidance for Art Works 2003 ([Ref.5.5])):

$$q_{os} = m_{os} \cdot \sqrt{gH_{m0}^3} \cdot e^{\left(-3{,}0 \cdot \frac{h_{kh}-h}{H_{m0}} \cdot \frac{1}{\gamma_\beta \cdot \gamma_n}\right)} \tag{5.11}$$

with:

$$0 < \beta \leq 20 \quad \Rightarrow \quad \gamma_\beta = 1{,}0$$ 5.12[20]
$$20 < \beta \leq 180 \quad \Rightarrow \quad \gamma_\beta = max\{cos(\beta - 20)\,;0{,}7\}$$

Here is:

q_{os}	Average throughput over a vertical wall [m³/s/m]
m_{os}	Model factor for transsshipment flow = 0.13 [-]
g	Gravitational acceleration (9,81) [m/s²]

[20] This formulation differs slightly from [Ref. 5.5] and [Ref. 5.6], where it is stated that $\gamma_\beta = 0$ if $\beta > 90°$. This was done to allow the transition from onshore to off-shore waves to proceed gradually

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H_{m0}	Significant wave height after reduction with influence factor γ_s [m]
h	Local outdoor water level compared to NAP [m]
h_{kh}	Turning height artwork compared to NAP [m]
β	Angle of wave attack [°]. This is the angle between the wave direction and the normal of the artwork.
γ_n	Influence factor nose construction [-]
γ_β	Influence factor skew wave attack [-]

A nose construction can be fitted to limit the occurring transfer/overflow flow. This is hardly the case with artworks. The influence factor γ_n for a nose construction is not included in Hydra-NL (and not in Riskeer). The formulas for calculating the influence of a nose construction on the transshipment flow rate can be found in section B2.4.1 of the Guidelines for Art Works 2003 ([Ref.5.5]). For a method to subsequently translate this influence to a required crown height, reference is made to Section 7.3.6 of the Scheme for height of work of art ([Ref. 5.4]).

In the Guidelines for Art Works 2003 ([Ref.5.5]) an additional wind factor γ_w is applied if small transhipment rates occur ($q_{os} \leq 10$ l/s/m). Because such small transhipment rates are rarely used in the design of works of art, the backgrounds will suffice with a reference to the Guidelines for Art Works 2003 [Ref. 5.5] and the Background Report keyboard track Height I - Modeling overflow/overflow flow rate ([Ref 5.6]).

5.2.3.3. Situation with overflow only

If the artwork is closed high water, there is no direct contact between inside and outside water. The inland water therefore has no influence on the size of the inflowing flow. The overflow formula is therefore also derived from a situation with a complete spillway:

$$q_{ol} = m_{ol} \cdot 0{,}55 \cdot \sqrt{g \cdot (h - h_{kh})^3} \qquad 5.13$$

Here is:

q_{ol} Overflow flow over a vertical wall [m³/s/m]

m_{ol} Model factor for overflow flow rate [-]

The factor m varies depending on the radius R of the crown and the overflow height H:

a. the maximum value of m_{ol} is 1.3 at R/H = 0.6 to 2; for larger R/H m decreases; at R/H = 6 approaches m_{ol} to 1 (one long spillway);

b. for R/H <0.6 the flow is released and m_{ol} also decreases;

c. for a sharp, aerated spillway (R/H = 0) m also approaches 1, provided that the overflow height is much smaller than the upstream water depth or the width of the jet is much smaller than the upstream width. Usually a value of 1.1 is used for m_{ol}.

g Gravitational acceleration (9,81) [m/s²]

h Local outdoor water level compared to NAP [m]

h_{kh} Turning height artwork compared to NAP [m]

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5.2.3.4. Situation with both wave overtopping and overflow

The formula for the inflowing flow if there is both wave overtopping and overflow (see formula B2.17 from the Guidance for Art Works 2003 ([Ref.5.5])):

$$q_{os+ol} = m_{ol} \cdot 0{,}55 \cdot \sqrt{g \cdot (h - h_{kh})^3} + m_{os} \cdot \sqrt{g \cdot H_{m0}^3} \qquad 5.14$$

Here is:

q_{os+ol} Average transfer and overflow rate over a vertical wall at the same time occurrence of transhipment and overflow [m³/s/m]

m_{ol} Model factor for overflow flow rate [-]. Usually a value of mole is given for moles 1.1, see explanation in section 5.2.3.3.

g Gravitational acceleration (9,81) [m/s²]

h	Local outdoor water level compared to NAP [m]
h_{kh}	Turning height artwork compared to NAP [m]
m_{os}	Model factor for transshipment flow = 0.13 [-]
H_{m0}	Significant wave height after reduction with factor γ_s [m]

5.2.3.5. Overarching formula

The above can be summarized in the following formula:

$$q_{os/ol} = ALS(h \leq h_{kr}; q_{os}; ALS(H_{m0} = 0; q_{ol}; q_{os+ol})) \quad 5.15$$

Here is:

$q_{os/ol}$ Combined average throughput and overflow flow rate over a vertical wall [m³/s/m]

5.2.4 Wave overtopping as a non-stationary phenomenon

The critical transfer rate is a time average. In reality there is no question of a constant flow, but per incoming wave a quantity of water goes over the structure and then again for some time nothing. These transshipment amounts per wave depend on the wave height and the distance between the water level (stagnant water line) and the deflecting height. The greater the wave height, the greater the quantities per overtopping wave at the same average overtopping rate. This aspect must be taken into account when determining the critical transhipment rate in connection with the strength of the soil protection. For determining the inflow volume for cup storage, only the time-averaged transfer/overflow flow rate is important and this aspect does not play a role.

A relationship was established between the wave height and the average flow rate in EurOtop 2016 ([Ref 5.3]). It must be remembered that:

- This is particularly important for works of art where the transhipment rate directly taxes the soil protection (especially denominations). For works of art where the transhipment flow in a body of water ends up behind the artwork, the volume per wave is much less important.
- The verification for the high water situation involves a process of ongoing erosion of soil protection. Incidental/local damage as a result of a single outlier in the transshipment volume does not necessarily lead to failure (but can of course be of importance for other functions).

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- All available formulas to determine the critical strength of the soil protection are based on a time-averaged flow rate at the location of the soil protection.

All in all, there are hardly any handles for the designer to determine the maximum permissible volume per individual wave. As a rule of thumb, it can be maintained that at a large wave height (for example, greater than 3 m), the lesser average critical wave overtopping flow must be maintained for the erosion mechanism. Some guidance can then be found in section 3.3 of EurOtop 2016 ([Ref 5.3]).

With overflow the phenomenon is much more stationary and large constant flows can occur. In case of mainly overflows in combination with wave overtopping, incidental outliers in the transshipment flow are also of secondary importance.

5.3 Strength

The strength of a work of art with regard to height consists of four parts:

1. The geometry of the work of art: the deflecting height and width of the high-water retainers and the adjacent structural parts determine the load on the soil protection (the inflowing flow rate $Q_{os/ol}$) and the cup storage (the inflowing volume $V_{os/ol}$).
2. The characteristics and scope of the available storage capacity of the underlying water system (V_c) (see chapter 10 Bowl storage).
3. The structure and dimensions of the soil protection behind the artwork.
4. The structural strength of the turning means.

The above components are reflected in formulas 5.5 and 5.6 for the determination of the failure probability for transhipment and/or overflow and are explained in more detail in the paragraphs below.

5.3.1 Turning height and width of the artwork

The deflecting height of the high-water retainers and the adjoining water-retaining structures determine the transfer/overflow flow that flows in per linear meter over the work of art. Their width then determines the total flow that flows in over the closed work of art. Adjacent construction parts where the spill/overflow water also ends up behind the work of art and the soil protection or the cup storage must also be taken into consideration.

The width of the artwork is actually always a given from the other function (s) of the artwork. The required deflection height can be determined with Hydra-NL (see section 5.4). The schematization of the width plays a role here. Usually the width to be maintained is equal to the width of the reversing means. If the adjacent structural parts have more or less the same height as the reversing means then this must be taken into account. Instructions for this can be found in section 7.3.6 of the Schematisation guide for tall works of art [Ref. 5.4].

5.3.2 Bowl storage

For the purpose of determining the required deflecting height, the composting capacity must be converted to a flow rate per linear meter over the reversing means and adjacent structural parts. **(76 頁)**From conversion of formula 5.6 follows that there is just no question of failure if:

$$q_{os/ol} = \frac{A_{kom} \cdot \Delta h_{kom}}{t_s \cdot B} \qquad 5.16$$

Here is:

$q_{os/ol}$	Average transhipment/overflow flow over the artwork per linear meter [m^3/s/m]
A_{kom}	Compressing surface [m^2]
Δh_{kom}	Permitted level increase for storage [m]
t_s	Duration of high-water wave [s]
B	Width of the artwork [m]

The method of determining the parameters A_{kom}, Δh_{kom} and t_s is discussed in the chapter on Reservoir Storage (chapter 10). The parameter B is in the above section 5.3.1.

It should be noted that the above formula is based on a schematisation of the high water wave as a block with a constant duration. For systems with a 'short' load duration (coast, lakes) this is usable, for systems with a long-term load (rivers, deltas) not. For these systems, the output of the Water Level Gradient tool must be used to correctly determine the inflow volume. For more information, please refer to the chapter 'Reservoir storage' (chapter 10), for practical application to the chapter Case study (chapter 11).

5.3.3 Soil protection

The strength of the soil protection can be converted to a critical flow rate per linear meter over the reversing means and adjacent structural parts for the determination of the required deflecting height. After all, conversion of formula 5.4 follows that there is just no question of failure as:

$$q_{os/ol} = \frac{u_c \cdot (h_{bi} - h_{bb)} \cdot B_{sv}}{B} \qquad 5.17$$

Here is:

$q_{os/ol}$	Average transhipment/overflow flow over the artwork per linear meter [$m^3/s/m$]
u_c	Critical flow speed soil protection [m/s]
h_{bi}	Inland water level in relation to NAP [m]
h_{bb}	Height top soil protection [m NAP]
B_{sv}	Power-carrying width soil protection [m]
B	Width of the crown of the artwork [m]

The critical flow rate of soil protection is often known from the design of the other function (s) of the artwork. If this is not the case, then this critical flow rate can be determined on the basis of the Soil Protection chapter (chapter 9). The parameters h_{bi} and h_{bb} speak for themselves and are also discussed in the Soil Protection section. The parameter B is discussed in section 5.3.1.

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5.3.4 Maximum transhipment flow from a constructive point of view

It is recommended that the critical flow rate from both soil protection and storage should not exceed 0.1 to 1.0 $m^3/s/m$. This is a rule of thumb; at a flow rate of 1.0 $m^3/s/m$, a water disc of about 0.6 m over the deflecting parts of the structure enters the overflow situation. With such inflowing flow rates, dynamic aspects as a result of air inclusions under the overflowing jet may play a role. Larger flow rates are permissible provided that the stability of the retaining components is then demonstrated.

5.4 Determine required turning height

The required turning height of the artwork can be determined with Hydra-NL (and in the future also with Riskeer). Both methods are briefly explained below.

5.4.1 Determine the required height with Hydra-NL

In Hydra-NL, the required crest height can be calculated directly for view years 2023, 2050 and 2100 (for other view years, it must be interpolated or extrapolated). Here, the smallest critical flow rate $q_{os/ol}$ resulting from the partial-track storage (see formula 5.16) and soil protection (see formula 5.17) is entered in the program. Note: this concerns the flow per linear meter barrier! The failure probability in section 5.1.4.1 is also entered. Hydra-NL then calculates the hydraulic load level HNB, or the required deflecting height, where the sum of the probability of occurrence of all combinations of water levels and wave heights leading to exceeding the imposed overtopping flow is equal to the failure probability. To this end, a vertical wall module is included in Hydra-NL with the formulas as included in section 5.2.3. In the case (chapter 11) this working process is explained in more detail and the relevant input screens are provided with a brief explanation.

The critical transhipment flow is introduced in Hydra-NL as a deterministic parameter. With this, Hydra-NL deviates from Riskeer. Also the hydraulic loads are derived in Hydra-NL in a slightly different way than in Riskeer. This means that the calculated crest heights with Hydra-NL for visual year 2023 can deviate only slightly from Riskeer (order 0.1 m).

5.4.2 Determine the required height with the help of Riskeer

In Riskeer a fully probabilistic calculation is performed. The method of calculation is (obviously) aimed at the assessment of existing works of art whose geometry is fixed. With the deflecting height as one of the input parameters, a failure probability is determined. Through an iterative process, the turning height can be determined in Riskeer, where the artwork meets the failure probability. For the way in which the various parameters must be schematized and introduced in Riskeer, reference is made to the WBI toolkit.

It should be noted that Riskeer is still set up as an assessment instrument at the time of writing this work guide; it is not equipped as standard to work with statistic files that take account of climate developments. It is expected that the functionality of Riskeer will eventually be made suitable for designs.

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5.4.3 Sharper determination of throughput

The occurring transshipment/overflow flow rate is determined both in Riskeer and in Hydra-NL on the basis of the formulas from the Guidelines for Art Works 2003 ([Ref.5.5]). Since 2003 knowledge development has taken place in this area, making it possible to determine the transhipment rate more sharply[21]. The most recent knowledge on this subject is laid down in EurOtop 2016 ([Ref 5.3]). Section 5.5 shows how EurOtop 2016 ([Ref 5.3]) can be used to determine the required crown height more sharply.

5.4.4 Construction height calculation

When determining the installation height of the crown, the following surcharges must be taken into account at the calculated crown height:

- A surcharge for outdoor oscillations, pipe fittings and seiches.
- A surcharge for (local) noise if not already included in the water level statistics.
- The locally expected subsidence over the plan period.
- The expected curvature due to settlement of the subsurface over the planning period, after completion.

The high water rise over the planning period is already included in the Hydra-NL calculation.

For the determination of the surcharges for external oscillations, pipe fittings, seiches and local windings, see chapter 3 Hydraulic loads.

5.5 Relationship with the EurOtop manual

5.5.1 When to use EurOtop?

EurOtop 2016 ([Ref 5.3]) has a broader scope than the formulas for determining the transshipment/overflow flow rate in section 5.2, which originate from the Guidelines for Art Works 2003 ([Ref.5.5]). The formulas in section 5.2 are in principle intended for a vertical wall on relatively deep water (no heavy wave breaking for the construction). EurOtop 2016 ([Ref 5.3]) is more widely applicable and can also be used to determine the required crown height if there is no vertical wall on relatively deep water.

With non-breaking waves for the construction, it is not necessary to use EurOtop2016. As can be seen from Figure 22, the formulas from the Guidelines for

Art Works 2003 - and which are also used in Riskeer and Hydra-NL - are somewhat conservative with respect to the formulas from EurOtop 2016 ([Ref 5.3]). EurOtop2016 can be used in this situation to achieve a sharper design. The 'gain' that can be achieved with this is order 0.1 to 0.2 times the wave height in the part of the graph where most constructions lie (freeboard between 0.5 and 1.5 times the wave height).

If there are waves of refraction for the artwork, then the formulas in the 2003 Guideline Art Works (see section 5.2), especially for larger freeboards[22], are not always conservative. It is recommended in that case to explicitly verify whether it is necessary to use EurOtop 2016 to determine the required crown height.

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[21] In [Ref. 5.6] the background is included why the decision has been made to continue to use the formulas from the Guide to Works of Art 2003

[22] Height of the crown above the local still water level

(79 頁) Figure 22 can hereby be used as an aid.

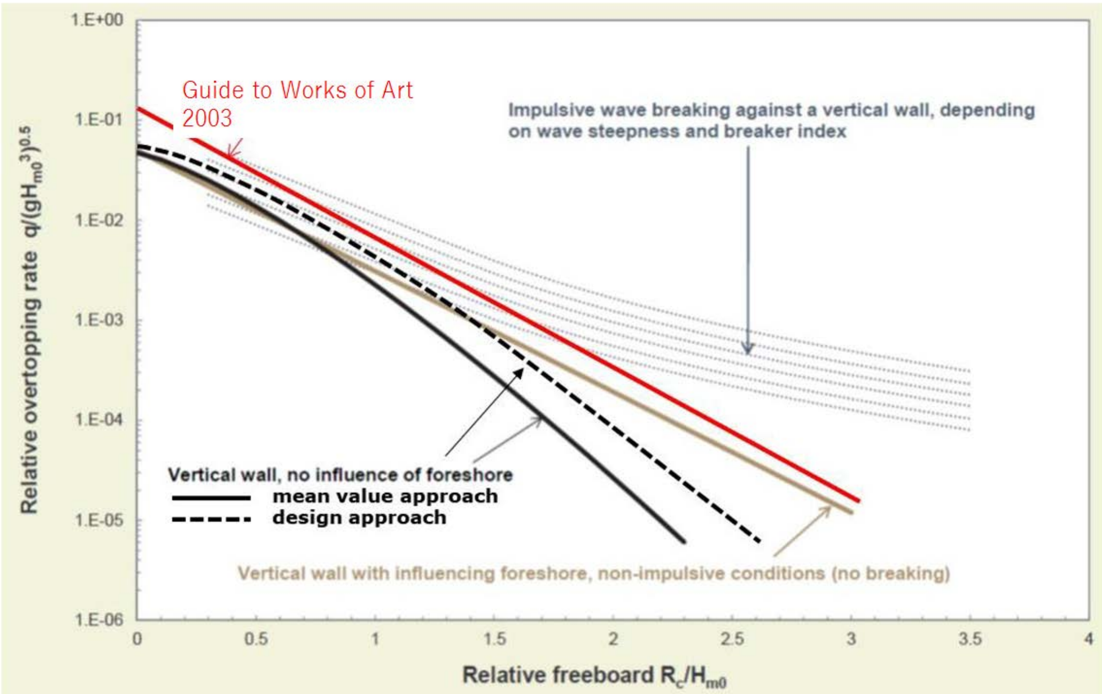

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Figure 22 Overview formulas in graphic form for vertical constructions from EurOtop 2016 and Guide to Works of Art 2003. The freeboard (R_c) is the height of the crown above the local standstill level. Base: figure 7.5 from EurOtop 2016.

5.5.2 Dealing with different construction forms in EurOtop 2016

With the help of EurOtop 2016, the transshipment flow rate for a range of composite constructions can be calculated. For the design of water-retaining structures, the spill overflow in vertical constructions (Chapter 7 from EurOtop 2016) is particularly important. For vertical walls, EurOtop 2016 distinguishes between three situations:

1. Vertical walls on relatively deep water. The waves hardly change due to depth limitation. This is by far the most common among flood defense structures.

2. Vertical walls with a front bank on which the waves adapt. This situation rarely occurs with works of art. Depending on the water depth for the construction and the steepness of the waves, breaking or non-breaking waves occur against the vertical wall.
   a) Non-breaking waves can reach the top of the structure and then transfer, similar to vertical walls on deep water. The transhipment formula also resembles that for vertical walls on deep water, but the transhipment flow is slightly higher.
   b) Breaking waves against the construction go high into the air after which a part passes over the construction. This situation always gives wave overtopping regardless of the height of the barrier.

The occurring transhipment rates per meter width for the cases 1, 2a and 2b outlined above are given in Figure 22. The corresponding formulas are given in paragraph 7.3.2 of EurOtop 2016. (80 頁)Please note: for the design, the formulas associated with the so-called 'design approach' must be applied in EurOtop 2016. This concerns formula 7.2 for situation 1 and formula 7.6 for situation 2a. Formulas 7.9 and 7.10 apply for situation 2b. In addition, various formulas for composite constructions have been included in section 7.3.3 and further.

5.5.3 Tax statistics when using EurOtop-manual

When using the formulas from EurOtop 2016 to determine the required height of the artwork, the required height is always a function of the wave height and the overtopping flow. The transhipment flow follows from the considerations regarding storage and soil protection. However, a wave height must also be entered.

For this, the illustration points from Hydra-NL can be used. Although the formulas from the Guide to Art Works 2003 have slightly different coefficients, they are exactly the same as the formulas from EurOtop 2016. Therefore it can be expected that the illustration point of a calculation with Hydra-NL would have been the same as the formulas from EurOtop 2016 would have been included. Therefore, a first calculation can be made with Hydra-NL. In case of any tightening on the basis of EurOtop 2016, the required wave height can be obtained from the illustration point of the Hydra-NL calculation (take the wind direction with the largest probability contribution). This is worked out in more detail in chapter 11 Case.

5.6 Concrete design recommendations height artwork

When designing a work of art, a number of choices can be made that can significantly reduce the chance of failure. A number of choices are mentioned below.

First of all, it is good to realize that designing the required deflecting height often does not have to take place at the sharpest point of the cut. The application of a little extra height is in fact relatively inexpensive.

In the design of a new work of art, it may also be wise not to fully utilize the claim to the available storage space. In any case, the developments that occur in the water system over time must be taken into account. Because artworks often have a long design horizon (100 years is common) it is impossible to predict the developments in the water system for this period. It is therefore advisable not to load the cup storage as much as possible so that some margin remains in the underlying system.

Particularly for denominations where the foreland is the same as that of the adjacent dike body, it makes little sense to make the height of the artwork higher than the height of the adjacent dike body. However, it is important to ensure an expandable construction and to take account of degradation and higher future loads when dimensioning structural elements. After all, the dikes are usually designed on a shorter planning period.

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5.7 References and background documents

[Ref. 5.1] Bree, B. van, WTI 2017 Key rules for works of art - Test track report Height, Deltares, 1220087-001-GEO-0010, Version D1, December 2015

[Ref. 5.2] Guide to Design with Flood Opportunities - Safety Factors and Loads for new Flood Risk Standards, version OI2014v4, Rijkswaterstaat WVL, February 2017

[Ref. 5.3] EurOtop - Manual on wave overtopping or sea defenses and related structures, Second edition, Pre-release October 2016

[Ref. 5.4] WBI2017 - Schematisation manual for height of artwork, Rijkswaterstaat WVL, version final, January 2, 2017

[Ref. 5.5] Guide to Works of Art 2003, TAW, May 2003

[Ref. 5.6] Bree, B. van, WTI 2017 Works of art Background report of the exam track Height I - Modeling overflow/overflow flow rate, Deltares, 1220087-001-GEO-0004, Version D1, December 2015

[Ref. 5.7] Regulations for primary flood defenses 2017 - Appendix III Strength and safety, Ministry of Infrastructure and the Environment

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6 Piping

6.1 Introduction

6.1.1 Introduction failure mechanism

Underneath a structure, concentrated seepage streams can be formed, resulting in leaching of material from the subsurface. In this chapter handles are given to control this problem.

6.1.2 Phenomenological description

A detailed description of the failure mechanism for piping can be found in Chapter 4 of the WBI Test Track Report Piping [Ref. 6.1].

In the event of failure by piping, hollow pipe-like spaces under (underflow) or around (backward running) create a work of art through the rinsing of soil particles as a result of a concentrated seepage stream. If this erosion process does not stop in time, the artwork can collapse.

> By "backwardness" is meant the formation of channels or hollow spaces on the side of a work of art as a result of the rinsing of soil. The normative seepage route is usually purely horizontal (a seepage flow along the structure at the interface of a cohesive layer), but can also contain vertical components (think of an entry or exit point under a wing wall). In practice, however, backwardness is often linked 1 to 1 to situations with a purely horizontal seepage path.
>
> Substance concerns the creation of cavities under a work of art as a result of a concentrated seepage stream in which soil particles are entrained. Here, the seepage flow passes underneath the work of art on the interface between construction and sand. Quarry screens are usually present underneath a work of art, as a result of which the seepage flow also includes vertical components. However, this does not always have to be the case. Think of long divers and pipes where no seepage screens are present. In practice, however, underflow is often linked 1 to 1 to situations with a (partially) vertical seepage line.

In line with the WBI2017, the Lane and heave models can be used in situations where the seepage line contains one or more vertical components. For situations with a purely horizontal seepage path, the models of Bligh[23] and Sellmeijer are available (for works of art). In Figure 2-3 of the schematization manual Piping works of art [Ref. 6.3] a more detailed overview is included in which situation the different models apply. A detailed description of these models can be found in chapters 5 and 6 of the WBI test track report Piping [Ref. 6.1]. The models are concisely summarized in section 6.1.3.

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[23] This differs from the Sand-bearing Wells Research Report [DLT-ozw 2012], in which it is recommended that Bligh's model should no longer be used. However, this recommendation is entirely based on research carried out for dikes. Because works of art due to the presence of seepage screens have a different groundwater flow image than dikes, it has been decided to maintain Bligh's model for the time being when testing and designing works of art. This is partly due to the lack of suitable calculation models that can replace the Bligh model.

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In verifications of under- or back-running, the strength is often expressed as a critical decay. The critical decay is defined as the decay where no continuous pipe is created below or next to the artwork. Before the artwork as a whole succumbs after exceeding the critical decay, a number of follow-up processes have to be completed. After the creation of a continuous pipe beneath or next to the work of art, such an erosion of soil material must occur that the overall stability of the artwork is insufficient. This can be done by tilting or shifting the entire work of art, or by a process in which successive parts of the work of art collapse structurally with the collapse of the total work of art as the final result. It is also possible that the artwork remains standing, but the adjacent dike collapses as a result of the erosion process and eventually collapses. A considerable residual strength is often still present after exceeding the critical decay. This is charged through the right branch of the failure tree in figure 23.

However, concrete models are lacking to quantify the residual strength and to express it in a chance of failure of the artwork as a whole after the critical decay has been exceeded. Therefore, residual strength is rarely taken into account in practice and the chance of exceeding the critical decay determines the probability of failure of the artwork as a whole. Incidentally, it is also recommended for designs not to take any residual strength into account anyway. For assessment, taking along residual strength may be relevant.

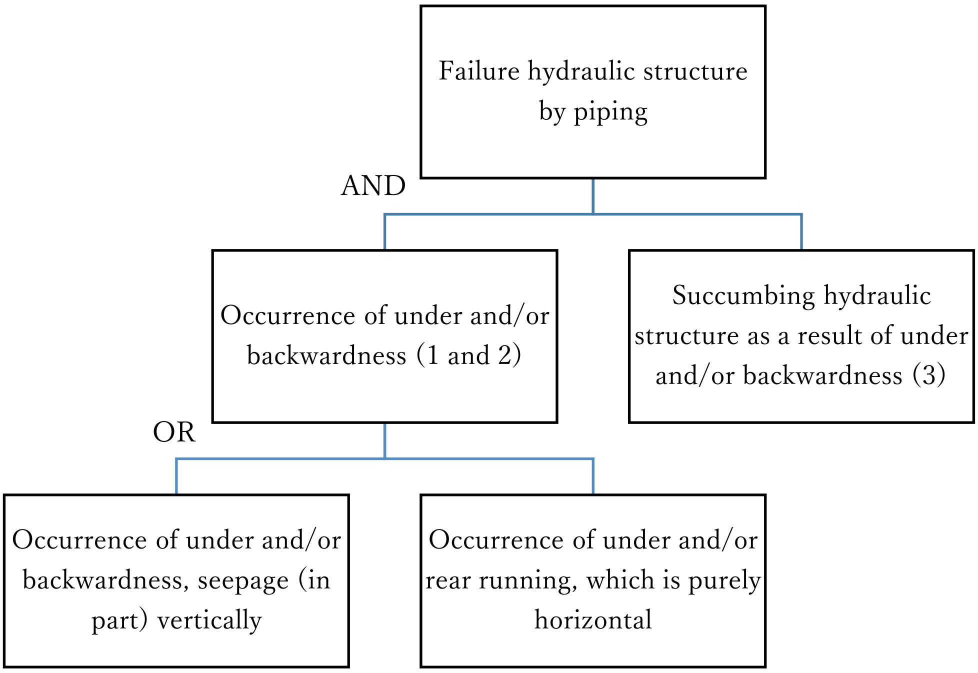

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Figure 23 Failure tree failure mechanism piping

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Figure 23 shows that failure of the flood defense occurs when:

(1) presenting a high water

AND (2) As a result of a concentrated seepage flow such

(a) Leaching of soil particles under (underflow) or

(b) Around (backwardness) the artwork is created that creates a continuous erosion channel (pipe).

AND (3) The artwork collapses due to this piping with substantial damage and/or casualties (flooding) as a result.

In section 4.2 of the WBI test track report Piping [Ref. 6.1] a more detailed description of the above failure process is included.

6.1.3 Safety format and concise model descriptions

The starting point of the safety format is the requirement that the failure probability for piping $P_{f,\,KW,\,PI}$ is less than or equal to the failure probability for piping $P_{eis,\,KW,\,PI}$ or:

$$P_{f,KW,\mathrm{PI}} \leq P_{eis,KW,\mathrm{PI}} \qquad 6.1$$

6.1.3.1. Determination of failure probability

The failure probability is derived from the legal requirement for the standard route and can be determined with the aid of the following formula:

$$P_{eis,KW,\mathrm{PI}} = \frac{P_{max} \cdot \omega_{PI}}{N_{PI}} \tag{6.2}$$

In which:

$P_{\mathrm{eis,\,KW,\,PI}}$	Failure Chance for piping an individual artwork for a reference period of 1 year [-]
P_{max}	Failure Chance for the entire dike section (standard route) based on the maximum permissible probability of flooding from the water act[24] [-]
ω_{PI}	Failure probability factor for piping [-]
N_{PI}	Length effect factor for piping [-]

In the standard failure probability distribution for a standard trajectory as included in the WBI2017 and the OI2014v4, a value of 0.02 is used for the probability of failure of the piping (ω_{PI}). Deviation from this is possible but rarely occurs in practice. See the chapter on Design Verifications (chapter 2).

An upper limit of the length-effect factor N_{PI} can be obtained by making it equal to the number of structures in the dyke section where piping is a relevant aspect. Because the probability of failure per artwork is rarely identical, a maximum of $N_{\mathrm{PI}} = 10$ is recommended in the OI2014v4. For a more accurate estimate of the length effect, the failure probabilities for piping of the other structures can be considered in the range. Among other things VNK2 results can be used for this.

[24] This is referred to as the lower limit in the Water Act

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In addition, sufficient margin must be maintained when determining the N value in order to be able to absorb future changes within the process, such as the construction of more works of art in the process. If the available chance of failure for the new artwork to be designed is too small, then the options are as follows:

1. Adapt the failure probability budget, so that the failure probability factor for piping in artworks becomes larger.

2. Create extra chance of failure through improvement measures for the other structures in the dyke section.
   The latter is not very obvious because in many engineering works improvement measures will be necessary to adjust the failure probability substantially (e.g. by a factor of 5-10).

Lack of relationship between safety factor and probability of failure for Bligh, Lane and heave models
For Bligh, Lane and heave models, there is as yet no explicit relationship between the calculated safety factor and the probability of failure. The failure probability is presumed to be met if the barrier conforms to these models at an outside water level that is equal to the water level at the standard. Data for a semi-probabilistic design verification (WBI) is only available for the Sellmeijer model. In the following sections the safety format is discussed for the individual models.

6.1.3.2. Safety format and model descriptions Bligh and Lane
For Bligh and Lane models, it is unclear how reliable a flood defense is when it is approved or designed with one of these rules. Within the VNK2 project, a probabilistic approach to both models was introduced at the time. The properties of the stochastic variables in both models are estimated on the basis of expert judgment. Based on this, semi-probabilistic calculation rules for both Bligh and Lane were derived in the WBI. In this Work Guide, however, it was decided to connect with the choice made in the WBI not to count (semi-) probabilistic with these models.

In both models it is verified whether the calculation value of the occurring decay on the artwork ΔH (the load) is smaller than the calculation value of the critical decay ΔH_c on the artwork (the strength):

$$\Delta H < \Delta H_c \qquad 6.3$$

The calculation value of the occurring decay is the decay with an exceedance probability that is numerically equal to the maximum permissible flooding probability.

The calculation value of the critical decay ΔH_c over the artwork is calculated according to the Lane model as follows:

$$\Delta H_c = \frac{L_v + {L_h}/{3}}{C_{w\ creep}} \qquad 6.4$$

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According to Bligh's model, the critical decay ΔHc over the artwork is calculated using formula 6.5:

$$\Delta H_c = \frac{L_h}{C_{creep}} \qquad 6.5$$

If a crack channel is present through which sand must be discharged outside, the fluidized sand in the channel will provide extra resistance. On the basis of tests by Sellmeijer (1981), the Technical Report Sand Meering Wells [Ref. 6.10] proposes the following correction to the Bligh model if a burst channel is present:

$$\Delta H - 0{,}3\ d < \Delta H_c = \frac{L_h}{C_{creep}} \qquad 6.6$$

Here is:

ΔH_c Critical decline over the artwork [m]

ΔH Present decay on the artwork [m]

L_v Total length of the vertical parts of the seepage line [m]

L_h Total length of the horizontal parts of the seepage line [m]

C_{creep} Creep factor from Bligh (material constant of the substrate) [-]

$C_{w,\ creep}$ Weighted creep-factor of Lane (material constant of the subsurface) [-] In Table 7 the values given by Bligh and Lane are for different types of material in the soil layer. These values can be understood as calculation values.

d Length of crack channel [m]

Table 7 Creep factors (nominal values) for the Lane and Bligh rules

Soil type	Median grain diameter [μm] [1]	$C_{w,\ creep}$ (Lane)	C_{creep} (Bligh)
Extremely fine sand, silt	< 105	8,5	
Very fine sand	105 – 150		18
Very fine sand (mica)		7	18
Moderately fine sand	150 – 210	7	15

(quartz)			
Moderately coarse sand	210 – 300	6	
Very/extremely coarse sand	300 – 2000	5	12
Fine gravel	2000 – 5600	4	9
Moderately coarse gravel	5600 – 16000	3,5	
Very coarse gravel	> 16000	3	4

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[1] Indications in accordance with NEN 5104 (September 1989)

6.1.3.3. Safety format and model description heave model

For engineering works - in contrast to dikes - only a deterministic verification is available. Here it is verified whether the calculation value of the occurring displacement over the downstream seepage screen i (the load) is smaller than (a calculation value of) the critical displacement i_c over the downstream seepage screen (the strength):

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$$i = \frac{\varphi_o - h_p}{d} < i_c \tag{6.7}$$

In the deterministic calculation, the calculation value $i_{c;d} = 0.5$ is maintained[25]. When checking heave, it is thus checked whether the calculation value of the occurring displacement over the downstream seepage screen is smaller than the critical vertical displacement $i_{c,d}$ of 0.5. It should be noted that this value is generally regarded as sufficiently safe, but that further substantiation is still lacking for the time being.

6.1.3.4. Safety format and model description model of Sellmeijer

For the Sellmeijer model, a semi-probabilistic method is available. The determination of the failure probability is discussed in section 6.1.3.1. Below we will discuss the determination of the probability of failure with this model.

Determination of failure probability

The failure probability for piping $P_{f, KW, PI}$ follows from the failure tree in Figure 23. Herein are the distinguish the following three partial failure mechanisms:

1. Occurrence of under and/or backwardness, seepage (partly) vertical (limit

state function Z_{PIP1})

2. Occurrence of under and/or backwardness, seepage path purely horizontal (limit state function Z_{PIP2})
3. The total collapse of the artwork after under and/or rear running has taken place (limit state function Z_{PIP3})

The Sellmeijer model may only be used for purely horizontal seepage roads in one direction. This means that limit state function Z_{PIP1} is not relevant here.

For limit state function Z_{PIP3} (collapsing artwork in the event of occurrence of under/or rearward running), it is initially assumed that collapse of the artwork always occurs as soon as there is under and/or backward running (P (Z_{PIP3} <0) = 1). Tightening of this is possible but requires a specialist consideration that falls outside the framework of this Work Guide.

For limit state function Z_{PIP2}:

$$Z_{PIP2} = \Delta H_c - (\Delta H - 0{,}3d) \tag{6.8}$$

Here is:

ΔH_c	Critical decay as calculated using the Sellmeijer model [m]
ΔH	Emerging decay on the artwork [m]
d	Thickness of the covering layer [m]

---

[25] This calculation value for the critical relocation is higher than for dikes. The difference between heave behind a seepage screen and heave over a covering layer is that with a seepage screen there must always be a vertical path due to the sand present. This requires fluidization of the sand package downstream of the seepage screen, for which the required gradient is approximately 1. A safety factor of approximately 2 has been applied to this, resulting in a critical gradient of 0.5. With a cover layer, the resulting crack is filled with a liquid sand-water mixture: in principle, this crack is almost never completely filled with packed sand. As a result, the water velocity through the cover layer can become so large that the hole is more or less flushed, which results in a smaller critical ratio of 0.3.

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The strength variable ΔH_c is determined using the Sellmeijer model. The model of

Sellmeijer is described in paragraph 7.3 of the Research Report Sand Meandering Wells ([Ref 6.5]). The updated version included in the WBI2017 is as follows:

$$\Delta H_c = L \cdot F_{resistance} \cdot F_{scale} \cdot F_{geometry} \quad 6.9$$

$$F_{resistance} = \frac{\gamma_p'}{\gamma_w}\{\eta \tan(\theta)\} \quad 6.10$$

$$F_{scale} = \frac{d_{70m}}{\sqrt[3]{\kappa L}}\left(\frac{d_{70}}{d_{70m}}\right)^{0,4} \quad 6.11$$

$$F_{geometry} = F(G) = 0,91\left(\frac{D}{L}\right)^{\frac{0,28}{\left(\frac{D}{L}\right)^{2,8}-1}+0,04} \quad 6.12$$

Here is:

L	Pathway (measured horizontally) [m]
$F_{resistance}$	Resistance factor, describes the boundary balance of grains of sand on the soil from the pipe [-]
F_{scale}	scale factor, reflects the ratio between the process scale of the mechanism for pellet transport and the process scale of the groundwater flow that drives this transport mechanism [-]
$F_{geometry}$	Geometry factor, describes the influence of the shape of the geometry of the subsurface on the groundwater flow [-]. The formula presented is valid for a standard configuration with one homogeneous sand layer below it impermeable work of art. With a deviating geometry, the factor must be $F_{geometry}$ are determined with the piping module from the groundwater flow model MSeep.
Υ'_p	(Apparent) volume weight of the sand grains under water [kN/m^3] = Υ_p - Υ_w with Υ_p = 26 kN/m^3 and Υ_w = volume weight of water [kN/m^3]
θ	Roll resistance angle of the sand grains (θ = 37) [°]
η	Coefficient of White (η = 0.25) [-]
κ	Intrinsic permeability of the piping-sensitive/upper sand layer [m^2] = $\kappa = \nu \cdot k/g$ k = specific permeability of the piping-sensitive/upper sand layer [m/s] v = kinematic viscosity of water at 10 ° C (v = 1.33 · 10^{-6} m^2/s} g = acceleration of gravity (g = 9.81 m/s^2)
d_{70}	70 percentile value of the particle size distribution [m]

d_{70m} Average d_{70} of the sand types used in the small scale tests, on which this formula is fitted ($2.08 \cdot 10^{-4}$) [m]

D Thickness of the sand package [m]

Remarks relating to the application for works of art are included in Chapter 6 of the Test Track Report piping for works of art [Ref. 6.1].

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When determining the critical decay, use should be made of characteristic values (5% and 95% fractiles) for the input parameters where applicable.

Safety factors

In the semi-probabilistic design verification, the following safety factors must be applied to the critical decay:

γ_{pip} Safety factor for the failure mechanism piping. This depends on the reliability requirement.

γ_b Partial factor for the uncertainty about the subsurface and the water (over) voltages (schematization factor)

With this, the verification requirement based on the Sellmeijer model is as follows:

$$\frac{\Delta H_c}{\gamma_{pip} \cdot \gamma_b} > (\Delta H - 0{,}3d) \qquad 6.13$$

The necessary safety factors are determined as follows:

- Safety factor piping

  The safety factor for piping γ_{pip} that is used in the Sellmeijer model is determined using the following formula 6.14:

H

$$\gamma_{pip} = 1{,}04 \cdot e^{(0{,}37\beta_{eis,KW,PI} - 0{,}43\beta_{max})} \qquad 6.14$$

e

rin is:

$\beta_{eis, KW, PI}$ Reliability index associated with the failure probability $P_{eis, KW, PI}$ (-): $\beta_{eis,KW,PI} = -\Phi^{-1}(P_{eis,KW,PI})$

β_{max} Reliability index associated with the maximum permissible flood probability P_{max} (-): $\beta_{max} = -\Phi^{-1}(P_{max})$

Φ^{-1} Inverse of the standard normal distribution

- Schematization factor γ_b
  For the determination of the schematization factor γ_b, the usual method from [Ref. 6.8].

6.1.3.5. Connection to Eurocode 7

Section 10.5 of the Eurocode 7 briefly discusses the design of measures against piping. Reference is made here to the old Guideline for the design of river dikes; part 1-upper rivers area. In addition, a safety factor is specifically introduced for the Lane model, which varies between 1.5 and 2 depending on the risk class used. A substantiation of this factor-dependent safety factor is not given in the Eurocode 7. The use of this factor has therefore not been included in this Guideline.

6.1.4 Step-by-step plan for design

Figure 24 shows the steps that are set in the design process. Each step is briefly explained below in the figure.

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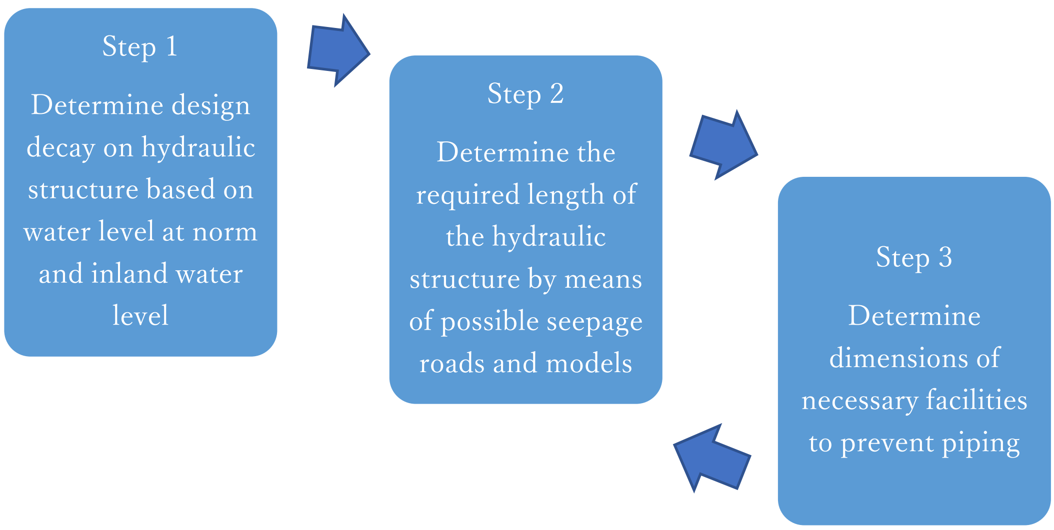

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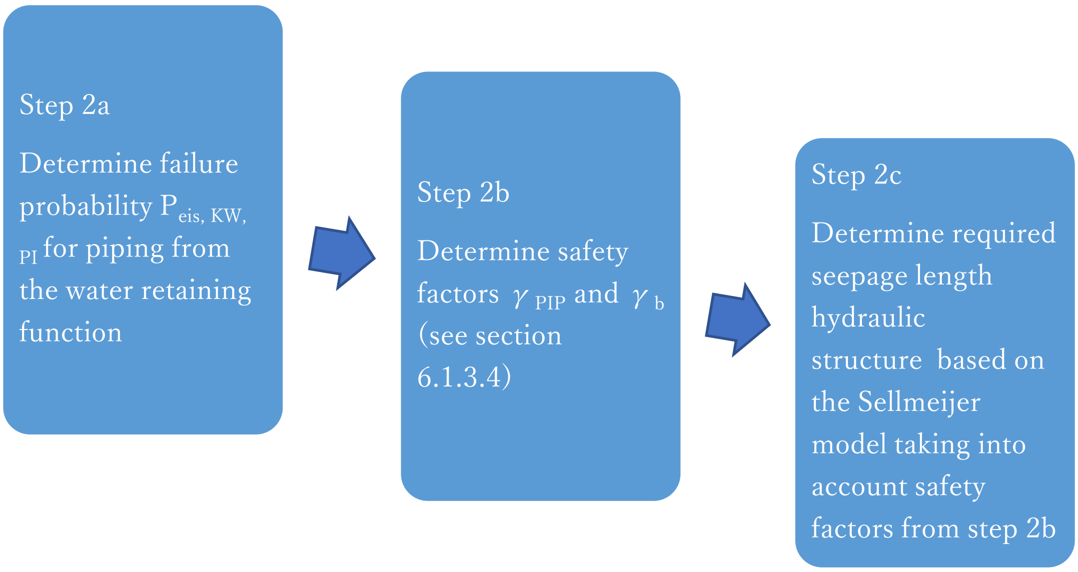
Figure 24 Step-by-step plan for design failure mechanism piping

Below is a brief explanation of the steps indicated:

Step 1. Based on the standard of the dyke stretch, the outside water level is determined using Hydra-NL. For the outside water level, the water level must be adhered to the standard. Furthermore, the inland water level is determined which is considered normative for piping. Together, these two parameters determine the decay of the artwork where the necessary measures to prevent piping are explained. This step is explained in more detail in section 6.2.

Step 2. In this step it is determined which seepage path length is required to prevent piping with sufficient certainty. The model with which this is done is determined by the route of the seepage roads that are possible under and/or next to the work of art. This is further elaborated in section 6.3. The models themselves are given in section 6.1.3.2 to 6.1.3.4. Please note: with the Sellmeijer model (see section 6.1.3.4), a safety factor must be charged that is dependent on the failure probability as determined in section 6.1.3.1. Specifically for the Sellmeijer model, step 2 is as follows:



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Step 3. As soon as the required seepage length is known, the required dimensions of the provisions against piping (for example filters and seepage barriers) can be further determined. Because the location and dimensions of seepage barriers also affect the possible seepage roads below and/or around the work of art, this is an interaction with step 2. This is discussed in section 6.5.

6.2 Taxes

Piping occurs at high outside water levels, where the gradient over the flood defense is relatively large. The duration of the high water tax also plays a role. Both are elaborated below. Waves and current play no role.

6.2.1 Decline on the artwork

The decay is determined by the combination of the outside water level and the inland water level. In the context of this Work Guide, only a load combination is considered that is related to high water conditions. Note that the decay of the artwork from other functions (e.g. management and maintenance) may be decisive.

The design value of the outside water level must be determined on the basis of the water level with an (annual) exceedance probability that is numerically equal to the maximum permissible flooding probability. This water level can be determined with the help of Hydra-NL (water level).

For the inland water level, a characteristic low value that fits the high water situation must then be taken into account. It should be borne in mind that this is influenced by the water level management just before and during high tide and the orientation of the inland waterway. In designs, any reductions in water level within the design lifetime must be taken into account. At the same time, the influence of noise and dust on the design level in the various design situations must also be taken into account.

In addition to hydraulic loads, no other loads (such as traffic loads, for example) play a role in the failure mechanism of piping.

6.2.2 Suspension over downstream seepage screen

In a design verification with the heave model, it concerns the transfer over the downstream seepage screen. A groundwater flow analysis is required for the determination of the occurring precipitation. In principle this can be done with any suitable calculation model. Most common are:

- Calculation using a (multi-purpose) computer program for numerical groundwater flow analysis, based on a finite element or finite difference method (EEM or EDM).
- Calculation with a semi-analytical calculation model (fragment method). This method has been developed in TAW (now ENW) framework, specifically for heave inspections at dikes or flood defense structures with vertical seepage barriers.

Reference is made to paragraph 5.4 of the Test Track Report piping for works of art [Ref. 6.1] for a more detailed description of the heave model and the fragment method.

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6.2.3 Time dependency of the load

For a flooding caused by the piping phenomenon, the driving force that triggers the erosion process must first build up and then remain present for a longer period of time so that the piping process can take place completely. The duration of the load influences the development of the water stresses in the water-bearing and piping-sensitive layers below and next to the artwork:

- In the case of 'long' duration water levels, there is stationary groundwater flow.
- With water levels of 'limited' duration, there is non-stationary groundwater flow, or time-dependent groundwater flow.

In the case of non-stationary groundwater flow due to short-term high water, the water tension in the aquifers below and behind the dike is less high at the same outside water level than in the case of a stationary groundwater flow due to prolonged high water. This depends on the storage capacity of the soil layers, the permeability and thickness of the soil layers and the duration of the load. This means that with non-stationary flow the force on the granules and with this the chance of piping is smaller than with stationary groundwater flow. In the estuaries, the lake district and along the coast the load is storm dominated and thus short-term, which means that there is almost always a non-stationary groundwater flow under high water conditions. In the upper rivers area the load is depleted and long-term, so that time dependency hardly plays a role and should be counted as a stationary one. In the sub-river area there is a combination of storm-dominated (thus short-term) and discharge-dominated (so long-term) tax. The following applies in general: the further downstream, the shorter the tax duration.

In order to be able to take into account time dependency, the response of the outside water level in the water-carrying package must be estimated. Monitoring well measurements can be an important aid here. Because these monitoring well measurements are carried out at lower external water levels, the measurement results must be extrapolated. For the analysis of these measurements and a prediction of the response, several methods are available, see for example Comparison methods determination time dependence rise height [Ref. 6.9] and Technical Report Water tensions at Dikes [Ref. 6.11].

The response of the outside water level in the water-carrying package can also be calculated using analytical or numerical models. To this end, the Technical Report Water pressures at Dikes [Ref. 6.11] instructions given. For this, use can be made of the Water Level Course tool developed in the framework of the WBI2017 for the course of the external water level.

6.3 Strength

The strength of the artwork with respect to piping consists of three components:

1. The length of the seepage roads below and/or next to the work of art.
2. The characteristics of the soil in which the artwork is founded.
3. The presence of filter constructions at the artwork.

The first two contribute to the resistance that the groundwater flow under and next to the artwork experiences. A filter construction does not so much contribute to this resistance, but prevents soil particles from being entrained as a result of groundwater flow. This is explained in more detail in the paragraphs below.

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6.3.1 Pathway length

The seepage length is directly related to the geometry of the artwork and is determined by dimensions of elements along which a seepage flow can occur (see for example Figure 25). The foundation method of the artwork plays a role in this.

Geometry

The main dimensions of the artwork are usually determined by the primary function (s) of the artwork. The seepage line can then be extended with the aid of seepage barriers below and next to the work of art. The required length of the seepage line can be calculated using the Bligh and Sellmeijer models if the seepage path is purely horizontal, and with the Lane and heave models if the seepage line contains vertical parts.

With regard to the choice between Bligh and Sellmeijer models, the following applies:

- Sellmeijer's model is intended for horizontal groundwater flow in one direction. For situations in which there is horizontal groundwater flow in one direction (i.e., bottom and rear run screens are lacking, for example in pipes and some box constructions), the situation in structures is completely analogous to the situation at dikes as long as the construction is properly aligned with the subsoil. In this situation, the calculation rule of Bligh may not be applied.
- With artworks, however, there are almost always (sub) and rear walk-through screens. As a result, the seepage flow is forced to change direction, as a result of which the seepage path becomes longer and the resistance to piping increases. This effect cannot be quantified and included in the Sellmeijer model. The calculation rule of Bligh can be applied for backward running. Incidentally, the Sellmeijer model can also be used, provided that the additional seepage length around the rear running screens is left out of consideration. However, that is conservative.

If the seepage line contains vertical elements, both the Lane model and the heave model can be used. In exceptional cases, Lane's model is not safe enough and the heave model must be applied. These exception cases are described in paragraph 5.5 of the Tracking Report on piping for works of art [Ref. 6.1].

Fund method

The foundation method determines whether the part of the seepage line underneath the structure also contributes to the resistance that the groundwater flow under and next to the work of art experiences. This is the case for works of art based on steel. For works of art that are founded on poles or sheet piles, it is possible that the subsoil will drop, but the artwork will not. This creates a gap between artwork and subsoil, which greatly reduces the resistance over this part of the seepage line. This part of the seepage path then does not contribute to the resistance that the groundwater flow under the artwork experiences. The horizontal parts of the seepage line under a pile foundation are therefore not included in considerations with the Bligh, Lane and Sellmeijer models.

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6.3.2 Basis

The resistance that the groundwater flow under and next to the work of art is strongly dependent on the material of which the substrate consists. In cohesive, poorly permeable soil layers the groundwater flow is very slow. In addition, the mutual cohesion of such soil layers ensures that no soil particles leach out. Cohesive, poorly permeable soil layers are therefore not susceptible to piping. In non-cohesive, well-drained soil layers (sand) the groundwater flow is much larger. In addition, there is no mutual connection between the particles, so that soil particles can easily be transported through the groundwater flow.

The extent to which non-cohesive soil layers are sensitive to piping depends on the grain diameter. Two opposite effects play a role in this. The resistance of the individual grain against leaching increases as the grain size increases. The permeability of the subsurface, and with it the groundwater flow, also increases with increasing grain diameter. The first (resistive) effect is, however, stronger, so that fine-grained materials are more sensitive to piping than coarse-grained. In Bligh and Lane formulas this is immediately visible because the so-called creep factor, which is a measure of the ratio between decay and required seepage length, is greater for fine-grained materials (see Table 7).

6.3.3 Soil protection and filter constructions

On both the in and outflow side of works of art, a bottom defense is almost always present from the primary function (s) of the artwork. Important are the type of soil protection (waterproof or water permeable) and the location of the soil protection (on the inside or outside of the artwork). Stamped concrete, colloidal concrete or asphalt mastic are the most common waterproof soil protection systems. Soil protection constructions that can be regarded as water-permeable are block mats, stone asphalt mats, bricks and granular filters.

Soil protection on the outside of the artwork

If a waterproof bottom protection is present on the outside, then the length of the soil protection can be included as a horizontal seepage so long as the connection with the work of art (floor, wing walls, impermeable slope) is good. In the design, it must be ensured that a good, watertight connection is realized.

A water-permeable soil protection on the outside is of course never included in the seepage length.

Soil protection on the inside of the artwork
A watertight soil protection on the inside of a work of art can only be included in the seepage length if erosion of this soil protection can be excluded. To this end, the occurring water pressure under the soil protection must be calculated, after which it must be checked whether erupting occurs at the calculation value of the decay. Naturally, the soil protection can be designed in such a way that cracking does not occur. If the length of the soil protection is taken into account as a seepage length, the connection with the artwork (floor, wing walls, impermeable slope) must of course be good.

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Filter constructions
The bottom protection on the inside can be designed in such a way that it is sand-tight and water-permeable. The soil protection can then act as a filter, so that piping cannot occur. It must be demonstrated here that during the design life span the filter remains water-permeable and sand-tight and therefore continues to function as a filter.

This is difficult for soil protection constructions on a geotextile. For this reason, it is recommended that this type of soil protection be considered as clogged and therefore watertight, unless management measures show that this is not the case and the soil protection can function as a filter during its life. The length of the soil protection can be included in a closed-down geotextile as horizontal seepage if it is shown that the soil protection does not erupt.

For filter constructions that are completely granular according to the 'filter rules' (see section 6.5.1), it is likely that the filter effect can be guaranteed during the design life of the filter construction. However, it may be necessary to include specific maintenance measures in the management maintenance plan of the artwork.

Incidentally, other filter systems can also be included in the design. If these have a role in the prevention of piping, the permeability and sand-tightness of the design durability must also be guaranteed with sufficient reliability.

6.3.4 Degradation

Various aging mechanisms can influence the resistance of the artwork to piping. Examples of this are cavity forming under the artwork through settlements, drawing sheet piles from the construction by negative adhesion, corrosion of steel screens/rotting of wooden screens, loss of entry resistance due to erosion of the contiguous soil, etc. The design must be taken into account here by the designer. This can be done by adjusting the modeling or, where necessary, devising a suitable design solution.

6.4 Schematisation

The search for relevant seepage roads under and/or next to the work of art is the basis for a good analysis of the probability of the occurrence of piping. The possible seepage roads are determined by the interplay of geometry of the artwork (dimensions of artwork, seepage barriers, foundation method) and the existing soil structure.

6.4.1 Geometry artwork

The identification of seepage roads starts with a 3D visualization of the artwork. With a simple work of art, this analysis can still take place with 2-dimensional tools (longitudinal and cross-sections on the artwork). For more complex works of art (structures with multiple seepage screens and different soil levels), 3D tools should be regarded as standard.

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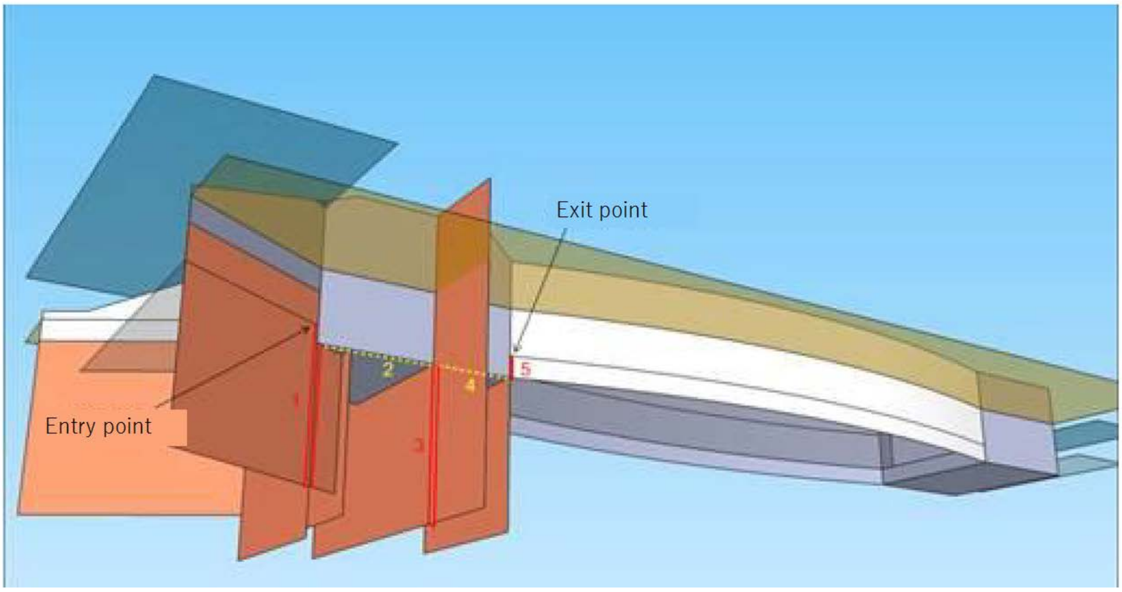

※（Rijkswaterstaat, Central government　2018）97 頁より作成。

Figure 25 A three-dimensional elaboration of a complex work of art in Google SketchUp for the purpose of determining normative seepage roads for under- and back-running (source: Tauw). The figure shows a sluice at an angle from below seen with a seepage line with horizontal (yellow dotted lines) and vertical (red line) components. It can be seen that the seepage path continues under the most upstream screen (1), then continues horizontally to the middle screen (2), this middle screen passes vertically (3) and again after a bit of horizontal seepage (4) just next to the downstream screen vertically in the lock chamber (5).

In inventorying potentially relevant seepage roads, both purely horizontal seepage roads and seepage roads with a vertical component must be considered.

6.4.2 Soil composition under and next to the artwork

In a design verification, the soil structure underneath and next to the artwork is often known to a large extent. For example, it is known from the design where and with what material is or will be supplemented next to the artwork. The existing soil structure can be derived from the available soil survey, supplemented if necessary with information from the WBI-SOS [Ref. 6.12]. Any missing information can often be obtained with limited effort.

If several scenarios concerning the soil structure around the work of art are possible, then each scenario concerning the soil structure has its own chance of occurrence. Each scenario then has its own probability of piping. The probability of a scenario must be estimated on the basis of the available information. The uncertainty about which scenario is actually present can be covered by the application of a schematization factor if the Sellmeijer model is applied (see also section 6.1.3.4). For the derivation of this schematization factor, reference is made to Section 3.4 of [Ref. 6.8]. The schematization theory from the Technical Report on Ground Mechanical Schematisation at Dikes [Ref. 6.8] can serve as an example in the reasoning of the choice of schematics for an analysis with Bligh and Lane models. For a detailed example of its application, reference is made to Section 12.3.7 of Research Report Sand-Liferous Wellen [Ref. 6.5]. Of course, a design can also be drawn up on the basis of an evidently safe choice for the subsurface. In that case no schematization factor needs to be applied.

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In view of the relatively limited costs of soil research when constructing structures, further research is always recommended if there are scenarios where the probability of piping (or another geotechnical failure mechanism) is substantially greater.

6.5 Measures to prevent piping

Piping is usually prevented by installing filter constructions or placing seepage barriers. Both are briefly discussed below.

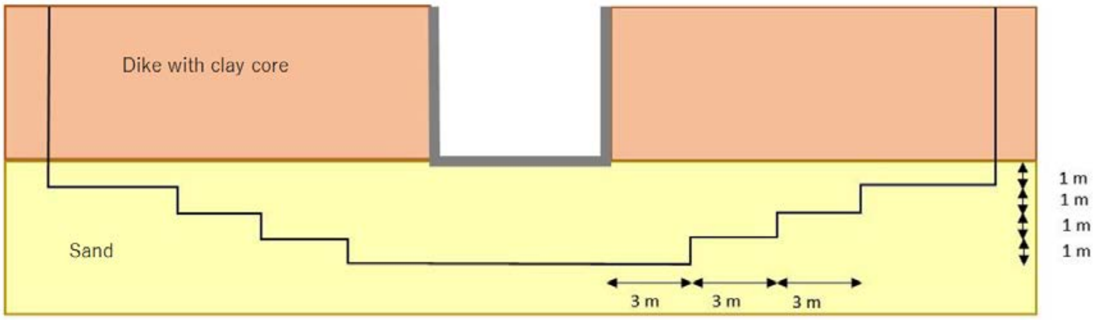
6.5.1 Installing filter constructions

The failure mechanism of piping can practically be ruled out by providing a filter at the outlet side of the seepage stream in which outflow takes place. A properly functioning filter ensures that the water exits in the filter, and that no sand particles are entrained. To this end, possible exit points of seepage water must have been well visualized. The filter must be designed according to the 'filter rules'. For an overview of these 'filter rules', reference is made to Section 5.4.3 and Section 8.6 of the Research Report on Sand-raging Wells [Ref. 6.5].

6.5.2 Application of seepage barriers

The preferential solution of many flood defense managers to allow the probability of piping to meet the failure probability is the fitting of seepage barriers. The total required seepage length can be calculated using the models from section 6.1.3. It is then up to the designer to determine where the seepage barriers can best be placed. The following aspects can be taken into account:

- To prevent leaching of sand particles at the site of construction transitions often (short) seepage screens are used. These can of course be included in the determination of the total length of the seepage required.
- Seepage must be sufficiently high, at least up to the design water level.
- For works of art on a pile foundation where piping is excluded because the work of art is enclosed by an impermeable soil package, short seepage screens must be installed. With this a possible opening between artwork and surface is sealed. It is recommended that a minimum size of 2 meters be maintained for the vertical length of such seepage barriers.
- The following applies to seepage barriers that are placed next to the work of art to prevent backward running (see figure 26):
  - from the deepest point below the work of art the depth per 3 m from the construction can be reduced by 1 meter
  - the screen must be at least 1 meter get stuck in the sand



※（Rijkswaterstaat, Central government　2018）98 頁より作成。

Figure 26 Front view of seepage screens under a work of art at clay dyke

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- In principle, backwater running does not play a role in artworks in sand dikes; in this situation, a test for microstability of the adjacent soil body must be done. At the same time, the rear running screens must have been kept at a certain minimum size outside the artwork; as a practical measure, take the length of the seepage screen below the structure (see figure 27).

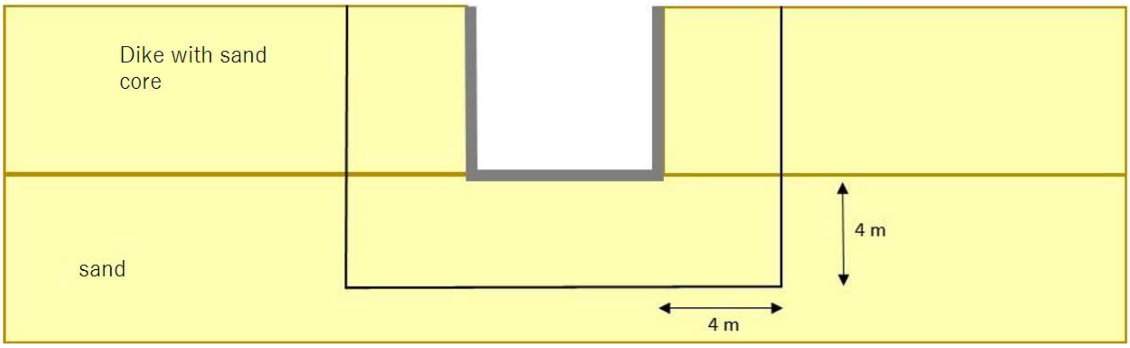

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Figure 27 Front view of sawn-screens underneath a work of art at sand dike

Finally, in the simple test rules (section 2.5 of [Reference 5.1]), a pipe with a pipe diameter smaller than 0.50 m does not need to be evaluated. In the design of such a pipeline a provision against piping must be included if the models in section 6.1.3 show that this is necessary.

6.6 Tightening necessary provisions against piping

Tightening of the required dimensions of seepage screens as determined with the models in this chapter may possibly take place on the basis of:

- Perform numerical groundwater flow calculations, taking into account, among other things, the time dependence of the load.
- Probabilistic piping or heave analysis.
- Analysis of monitoring well measurements.
- 'Proven strength' analysis.

Handholds for this can be found in paragraph 8.3 of the Test Track Report piping for works of art [Ref. 6.1]. It should be noted that the last two aspects can only be used in the improvement of an existing artwork.

6.7 Other draft recommendations

When designing a work of art, there are a number of choices that can be made to significantly reduce the chance of failure or with which piping can be practically excluded. A number of choices are mentioned below.

First of all, it is good to realize that designing the necessary provisions against piping often does not have to take place at the cutting edge. Applying some additional seepage length costs comparatively not much extra. This must be weighed against the possible costs of further research.

It also applies that in a design situation uncertainties can often be eliminated by choosing a solution where piping can be excluded on phenomenological grounds, such as by applying filters.

Furthermore, it is recommended that direct monitoring wells be installed when constructing works of art. In this way, the operation of the seepage barriers can be monitored over the lifetime and used calculation models can be calibrated for the specific situation. **(100 頁)**Moreover, gathering data about the response of the hydraulic head in the water-bearing sand layers around the artwork on the outside water level can provide more insight into the phenomenon of piping and eventually lead to better calculation models.

6.8 Example

In chapter 11 of this Work Guide an example is included in which the required dimensions of the seepage screens to prevent piping are determined. Furthermore, integral examples are included in paragraph 12.3 of the Research Report on Sand-Mealing Wells [Ref. 6.5] and in section 7.3 of Technical Report Sand-bearing Wells [Ref. 6.10]. Various examples related to the determination of specific input parameters for the Bligh, Sellmeijer, Lane and heave models can be found in chapter 7 of the Schematisation guide piping artwork [Ref. 6.3].

6.9 References and background documents

[Ref. 6.1] Bree, B. van, WTI 2017 Key rules for works of art - Test report Piping, Deltares, 1220087-003-GEO-0004, Version D2, December 2015

[Ref. 6.2] Guide to Design with Flood Opportunities - Safety Factors and Loads for new Flood Risk Standards, version OI2014v4, Rijkswaterstaat WVL, February 2017

[Ref. 6.3] WBI2017 - Schematisation guide piping artwork, Rijkswaterstaat WVL, version final 1.0, January 2, 2017

[Ref. 6.4] Guidelines for Art Works 2003, TAW, May 2003

[Ref. 6.5] Förster U., Van den Ham G., Calle E., Kruse G., Research Report Sand-Luring Welsh, Deltares, feature 1202123-003-GEO-0002, 2012

[Ref. 6.6] Jongejan R.B., Van Balen W., Calibration of semi-probabilistic test prescriptions for under- and lagging, Deltares, reference 12006006-

006-ZWS-0002, 2012

[Ref. 6.7] Förster, U. et al., WTI2017 Schematisation guide piping at dikes, Deltares, feature 1220084-006-GEO-0001 version 03, March 2016

[Ref. 6.8] Technical Report Soil Mechanic Schematising at Dikes, Expertise Network Flood Defense, 2011

[Ref. 6.9] Lambert, J. (2015). Comparison methods determination time dependency head, Deltares note 1220088-003-VEB 0007-m

[Ref. 6.10] Technical report Sand-bearing wells, Technical Advisory Committee on Flood Defense, March 1999

[Ref. 6.11] Technical Report Water tensions at Dikes, Technical Advisory Committee on Flood Defense, 2004

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[Ref. 6.12] Hijma, M.P. and Lam, K.S., Global stochastic subsurface schematisation (WTI-SOS) for the primary flood defense systems, Deltares report 1209432-003-GEO-006, 2015

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7 Structural failure

7.1 Introduction

This chapter deals with verifications for the failure mechanism constructive failure of flood defense structures in the design phase. The chapter serves as a support for constructors who take care of the actual dimensioning of the construction. In order to fully understand the Work Guide with regard to this failure mechanism, knowledge of structural design, the probability of flooding and the applicable building regulations (Building Decree) with the safety philosophy used in it is required.

Given the scope of the present Work Guide, the emphasis in this chapter is on the verification of the design with regard to the high water load. The flood tax is relevant both for the Water Act and the Buildings Decree, with the Eurocodes called by the Decree (NEN-EN 1990 to 1999). Since the Eurocodes pay little attention to hydraulic constructions, this manual also focuses briefly on taxes related to the other functions of hydraulic engineering works.

7.2 Scope and reading guide

This chapter describes the way in which the design of the structure must be verified in accordance with the requirements of the Water Act and the Building Decree for loads that the manufacturer considers relevant.

The first part of the chapter (sections 7.3 to 7.8) deals with the backgrounds of the structural failure. The second part of the chapter (sections 7.9 to 7.12) is a practical description of the design verification. In most cases, certainly for the works of art for which this Work Guide is written, the verification will be semi-probabilistic. The question when only a Building Decree or both a Building Decree and Water Act Verification must be made is answered in section 7.8.1. A step-by-step plan has been provided in section 7.9 for both verifications. In short, this elaboration comes down to the way in which reliability requirements have to be translated into calculation values. The calculation value of the tax effect service must always be smaller than the calculation value of the strength.

The way in which tax effects and the strengths of construction components are calculated is beyond the scope of this Work Guide. For this, reference is made to literature on construction mechanics.

7.3 Phenomenological description structural failure

Structural failure can be caused by the failure of water-retaining construction components and by instability of the structure and the adjoining ground body. The failure tree is shown in Figure 28 below. This failure tree differs from the tree in the WBI, which also has a branch for failure due to collision. Collision is considered in this Work Guide for the design verification as part of event 2 (failure of water-retaining construction elements), see also section 7.10.6.2.

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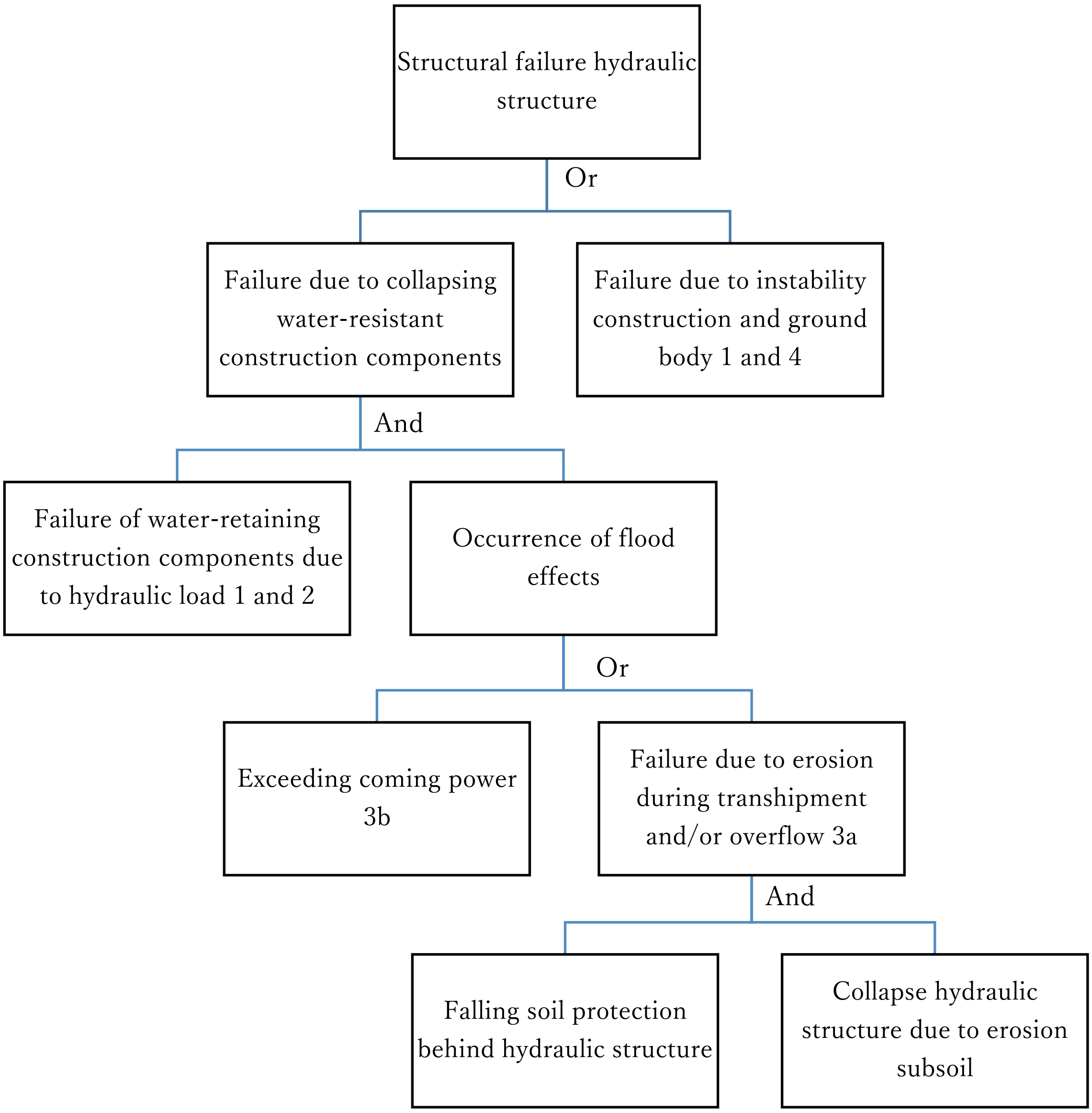

※（Rijkswaterstaat, Central government　2018）104 頁より作成。
Figure 28 Failure tree failure mechanism structural failure

7.3.1 Failure as a result of failure of water-retaining construction components

Failure occurs when:

(1) presenting a high water

AND　(2) As a result, such large loads occur that the strength of the water retaining components more adequate and these are collapsing. A large volume of water now flows in.

AND (3a) The soil protection behind the artwork collapses, after which excavation pits in the (not protected) subsoil are created. The artwork loses its stability with the result of a (progressive) breach in the flood defense and substantial damage and/or casualties (flooding) .[26]

OR (3b) The artwork maintains its stability and remains standing, but the inflowing volume due to the construction as a result of the failure of the water retaining components (2) cannot be recovered in the underlying (water) system without this leading to substantial damage. and/or victims (flooding).

[26] In some cases, after the emergence of pit pits and loss of standing certainty, it may happen that no progressive breach occurs, because, for example, the artwork is completely embedded in hard quay constructions. In that case the situation corresponds to 3b. From the primary function of the work of art, however, such an approach will not be desirable in the design. In the case of an assessment / assessment based on water safety, it can be used to demonstrate that the requirements are met.

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In most cases it is plausible that event 3a occurs instead of 3b, because there is often a large decline over the artwork at the moment that construction components collapse. The flow velocities that then occur after collapse are so great that the soil protection usually cannot cope with them.

7.3.2 Failure due to instability of construction and ground body

Failure occurs if:

(1) Presenting a high water

AND (4) As a result, there are such loads on the artwork and the adjacent ground body that instability occurs in the form of vertical or horizontal movement or tilting. This loss of stability leads to the occurrence of a (progressive) breach in the flood defense and substantial damage and/or casualties (flooding)

The difference with the partial failure mechanism as a result of failure of water-retaining construction components lies in the fact that failure due to instability automatically causes the loss of stability of the total structure. This does not require a failure of the soil protection. Exceeding the casing capacity is not considered here because the inflow volume is very large after total stability loss.

7.4 Failure and failure

The Foundations for Flood Protection [Ref. 7.2] states:

The exceeding of an ultimate limit state is also referred to as failure. Failure and failure are not the same. With collapse, loss of coherence or large geometry change is indicated. A flood defense can fail without collapsing. For example, the water can flow over the flood defense and cause a flood, without the barrier collapsing. Conversely, a flood defense can collapse without failing. For example, a superficial shearing of the inner slope of a dike does not have to lead directly to flooding. Of course in that case a repair is necessary, because the flood defense function is affected for the future.

From the perspective of the Water Act, failure is therefore equivalent to the occurrence of a flood. This means that failure encompasses all successive events that ultimately lead to a flood. For the structural failure mechanism, therefore, the chance of the top event from Figure 28, a combination of different events, should be formally verified. In practice, however, with most of the flood defense structures after the occurrence of an initial collapse mechanism, event 2 or 4 in Figure 28, there will also be an overshooting of the storage capacity or the collapse of the artwork after the failure of the soil protection. For this reason, design is pragmatically recommended for initially looking at the initiation of failure, or event 2 or 4, instead of the top event.

In the Eurocodes reliability requirements have been set for the failure of a construction. In the case of flood defense structures, this concerns events 2 or 4. **(106 頁)**All events after the structural failure that may possibly lead to a flood are not included in the verification. Of course the Building Decree does look at the consequences of collapse, but these are discounted in the reliability requirements imposed on structural failure. The greater the consequences of collapse, the stricter the reliability requirement for collapse. Rijkswaterstaat (Directorate-General for Public Works and Water Management) and other managers generally set the highest reliability requirement for flood defense structures, namely the requirement associated with Consequence 3 (CC3) 27. This is motivated by the idea that flood consequences also occur in most circumstances after structural failure. When the costs are highly dependent on the chosen consequence class and after failure no flooding is expected or the probability of flooding from the Water Act is relatively smooth (e.g. + 1/100 per year), a lower class could also be chosen.

7.5 Limit position function collapse constructively

As discussed in the previous section, the Buildings Decree focuses on constructive failure and, from the Water Act, it is also wise to look at constructive failure only when designing.

The occurrence of structural failure can be described by a limit state function. This function indicates for each possible combination of loads and strength properties whether the barrier will fail or not. A limit state function is often called a Z function. This function has a negative value if the load is greater than the strength and the barrier fails. In a limit state function all dimensions, variables and parameters occur that describe the strength of a structure (component) and the load on a structure (component).

A construction can fail due to multiple failure mechanisms. Thus, a structural component can collapse due to a shortage of shear force, moment or normal force capacity in a particular cut. But chicken or kink instability or fatigue can also be a problem. All these failure mechanisms have their own border state function.

The ultimate limit state is exceeded if the available strength of the structure is lower than the load effects. This is expressed in the following generic limit state function:

$$Z = R - E \quad \text{[28]} \qquad 7.1a$$

In accordance with NEN-EN 1990:

$$R = \sum f(X_i, \theta_i, a_i)$$
$$E = \sum f\left(F_j, \theta_j, a_j\right)$$

Σ means "combination of"

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[27] In Dutch design practice, the English expression Consequence Class 3 (CC3) is also often used. The English term is also used in this Work Guide.

[28] In the literature, this generic limit state function often uses S instead of E. However, S stands for load and E stands for load effect. With structural failure the load effect is used in the limit state function, in accordance with NEN-EN1990. NB: in NEN-EN1990, F is used instead of S.

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Here is:

R	Strength of construction (part)
X_i	Material strength i
θ_i	Model uncertainty i
a_i	Geometric data of construction (component) i
E	Tax effect
F_j	Tax j
θ_j	Model uncertainty j
a_j	Geometric data of construction (component) j

The total tax effect (E) is the result of (a combination of) considered taxes (Fi) in combination with geometrical data of the construction and model uncertainties. For the high water tax, this concerns the decay tax, wave tax and to a lesser extent the own weight.

The total strength R with respect to the strength magnitude in question is determined by the relevant structural components with their material strengths (xi) and the strength models used in the structural mechanics with their uncertainties.

The limit state is exceeded when:

$$R < E \qquad 7.1b$$

Either:

$$Z < 0 \qquad 7.1c$$

In the Eurocode NEN-EN 1990 Foundations of the structural design, a distinction is made between serviceability limit states and ultimate limit states. Constructive collapse[29] is a ultimate limit state, in which NEN-EN 1990 distinguishes between [30]:

- STR: Internal collapse or excessive deformation of the structure or structural elements, including foundations on steel, piles, basement walls, etc., where the strength of construction materials of the construction is decisive;

- GEO: Settling or excessive deformation of the ground where the strengths of soil are decisive for the resistance to be delivered;
- FAT: failure of the construction or structural elements due to fatigue.

For the high water tax, only the STR and GEO limit states are relevant from the point of view of the Water Act, which corresponds to the following assessment traces in the WBI2017: Strength Construction components (STCO) and Stability construction and ground structure (STCG). Other taxes to be considered can, of course, concern STR, GEO and FAT limit states. For example, for a steel turning means of a construction in a tidal area, the FAT limit state should also be considered. If this does not happen, fatigue may cause the actual strength to be smaller than assumed when assessing the water retaining capacity.

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[29] NB: In the Eurocode, the term failure is used here, because in the Eurocode, failure = collapse

[30] In addition, NEN-EN 1990 has the extreme limit state EQU: Loss of static equilibrium of the structure, or of any part thereof, considered as a rigid body. This limit state is not a form of structural failure in the case of a water-retaining work of art burdened by high water.

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7.6 Reliability of a construction

As explained in the previous section, a construction collapses if the strength of the structure is less than the load on the structure. In most cases the strength and the loads are spread and not exactly known. This way the load varies over time. It is therefore impossible to design a structure that can never fail. For that reason, the reliability of a construction is examined. The reliability of a construction is expressed as the probability of failure P_f or a reliability index (β), for a chosen reference period (t_{ref}). The reference period is the period to which the numerical value of the reliability relates, see for further explanation [Ref. 7.2].
The reliability of the artwork is expressed in a failure probability, or the chance of exceeding the limit state:

$$P_f = P(R < E) \qquad 7.2$$

For the relationship between a failure probability and a reliability index:

$$\beta = -\Phi^{-1}(P_f) \quad 7.3$$

At which:

p_f	Failure probability structural failure for a reference period equal to t_{ref} [-]
β	Reliability index for a reference period equal to t_{ref} [-]
Φ	(...) Standard normal distribution

Following on from chapter 2 Design verifications on the basis of flood probability standards, the development of the structural reliability over time is discussed below, through developments of the strength and the loads over time. The focus here is on the probability of failure in a year, given no failure in the previous years. In other words, a reference period (t_{ref}) of 1 year.

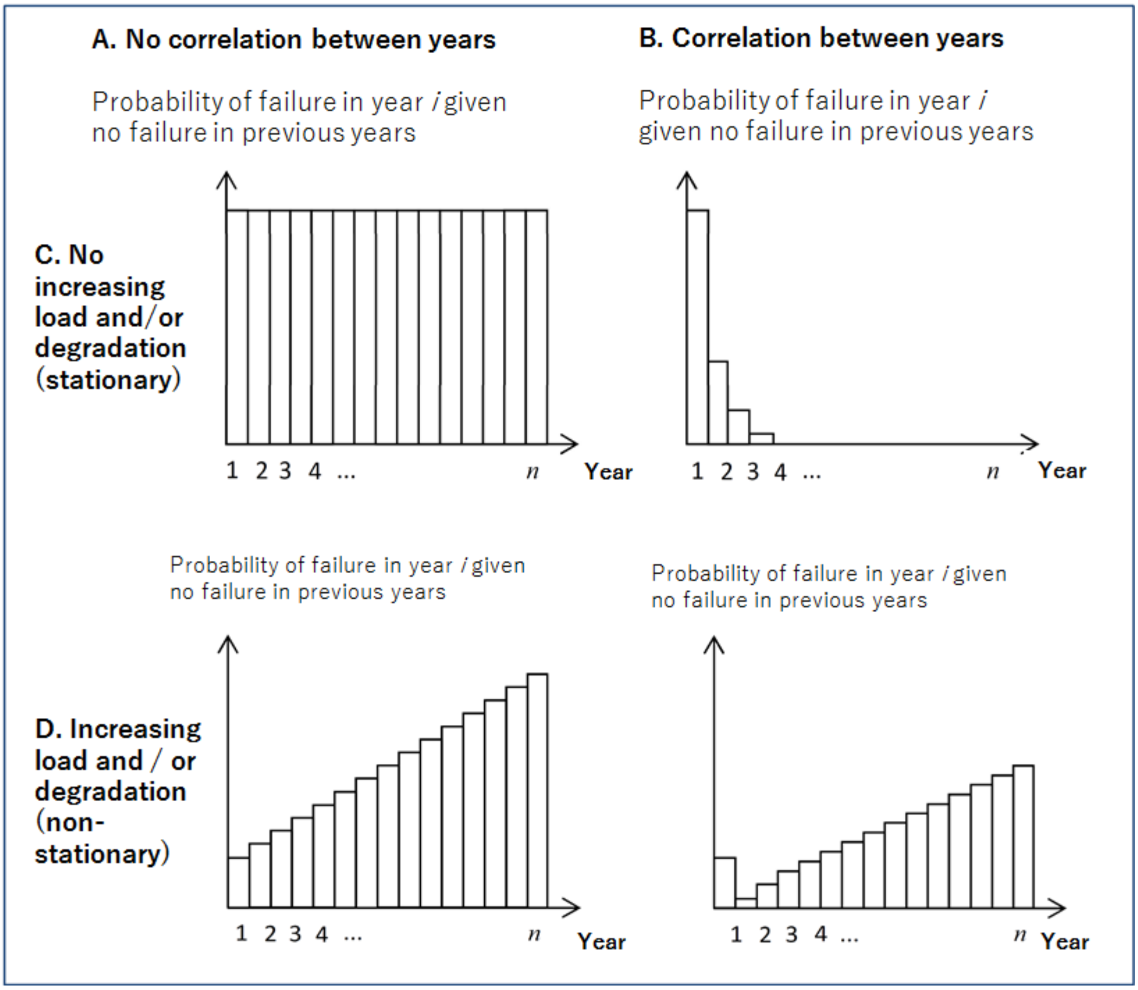
The chance of failure in time is influenced by two factors;

- The correlation between the failure events in the various years, in particular because the uncertain strength parameters are constant over time (relegation aside).
- The course of the strength or load over time. For example: due to aging the strength can decrease and due to climate change the load can increase.

In broad outline, four situations can be distinguished, shown in Figure 29:

- Figure 29 - top left describes the situation without correlation between the failure events in the different years and with constant strength and taxes. In this situation, the probability of failure in year i, given no failure in previous years, will remain constant over time.
- Figure 29 - bottom left also describes a situation without correlation between the failure events in the different years, but with increasing load and / or decreasing strength. In this situation, the probability of failure in year i, given no failure in the previous years, will increase.
- Figure 29 - right above describes the situation with strong correlation between the failure events in the different years and with constant strength and taxes. In this situation, the probability of failure in year i, given no failure in previous years, will decrease over time. **(109 頁)**This phenomenon is also called "proven strength".
- Figure 29 - bottom right describes the most realistic situation with correlation

between the failure events in the different years and an increasing load and / or decreasing strength. Firstly, the failure probability decreases over time by "proven strength", but then increases over time due to a decrease in the strength and / or the increase of the loads (climate developments) over time. This curve is also called the bathtub curve. The initial decrease in structural collapse will only be large with a large proportion of the own weight of the load and / or a very uncertain strength.



※（Rijkswaterstaat, Central government　2018）109 頁より作成。

Figure 29 The influence of time dependency and increasing taxes and / or strength degradation on the probability of failure per year (N = 1) given no failure in previous years (Source: [Ref 7.2] and [Ref 7.4])

High-water art works generally have the failure probability as shown in the bottom right-hand figure in Figure 29, the 'bathtub curve'. In the case of the bathtub curve, the maximum annual failure probability is in the first or the last year of the construction. In case of strong climate change (increase of the load in time) or aging (decrease of strength in time) this will usually be the last year of life, as shown in Figure 29. In case of a dominant permanent load (such as own weight or a relative great minimal decay) this will usually be the first year of life. In most situations, the probability of failure for high-wattage structures (which have survived the construction phase) is at most in the last year of life. **(110 頁)** Only in exceptional cases is the chance of failure maximal in the first year of life.

7.7 Reliability requirements for the construction

To achieve a minimum degree of structural safety, reliability requirements are imposed on a work of art:

$$P_f \leq P_{eis} \tag{7.4}$$

Or:

$$\beta \geq \beta_{eis} \tag{7.5}$$

At which:

P_{eis} Failure chance work of art structural failure for a reference period equal to luck [-]

β_{eis} Failure probability expressed in confidence index for a reference period equal to t_{ref}[-]

Both the Water Act and the Building Decree impose reliability requirements on flood defense structures. The requirements of the Building Decree and the Water Act differ on a number of points. Table 8 provides an overview of the most important differences between the reliability requirements from the Water Act and the Building Decree. Two differences are highlighted below:

- The Building Decree is a constructive failure option for a construction and for its individual construction components. The Water Act provides a flood probability standard for a whole process. A requirement for the risk of structural failure for construction components can be derived from this standard. This difference has already been discussed in section 7.4.
- The standards in the Water Act relate to a reference period of 1 year, which implies that the artwork must meet the standards [31] derived from the standard in every consecutive period of 1 year. The requirements according to the Buildings Decree in NEN-EN 1990 relate to a reference period of 50 years. In

the case of hydraulic engineering objects, a reference period of 100 years is generally applied without adjustment of the reliability indices from NEN-EN 1990.

Table 8 Overview table Water Act versus Building Decree.

Aspect	Reliability requirements Water Act	Reliability requirements Building Decree
Reliability requirements apply to	Only works of art in primary flood defense systems and then only (high) water defense	All water-retaining structures and all load situations/combinations
Reliability requirement relates to	Complete process	Construction component and total construction
Differentiation	Flood probability standards (maximum allowable probability of flooding) from 1/100 to 1 / 1,000,000 per year	Three consequence classes, depending on the consequences. Per class 1 reliability requirement (different for new construction, rejection and renovation)
Reference period	1 year	> 1 year, for design 50-100 years

※（Rijkswaterstaat, Central government　2018）110 頁より作成。

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[31] For the failure mechanisms: Overflow / transshipment, Do not close, piping and Structural failure

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7.7.1　Reliability requirement Water Act

There are no specific reliability requirements in the Water Act for the structural design of hydraulic structures with regard to (high) water defense. However, it is indicated which requirement a standard route must at least meet. From this route requirement a reliability requirement for structural failure can be derived with a reference period of 1 year, for more background see chapter 2 Design verifications based on flood probability standards.

In short, it means that all parts of the artwork must be designed in such a way that they meet this derived reliability requirement in every year:

$$P_{eis,KW,CON} = \frac{P_{max} \cdot \omega_{CON} \cdot c}{N_{dsn}} \qquad 7.6$$

In which:

$P_{eis,\ KW,\ CON}$	Failure probability for structural failure and no failure due to overflow/transsshipment of an individual work of art derived from route requirement from the Water Act for a reference period equal to luck = 1 year [-]
P_{max}	Failure probability for the entire dike section (standard route) based on the maximum permissible probability of flooding from the water law for a reference period equal to the t_{ref} = 1 year [-]
ω_{CON}	Failure probability factor for structural failure [-]
c	Correction factor for the correlation between structural failure and failure through overflow/ transhipment [-]
N_{dsn}	Length-effect factor for structural failure [-]

In the standard failure probability distribution for a standard path, a value of 0.02 is used for the failure probability factor ω_{CON}. This can be deviated from if it is not obvious for structural failure (see also chapter 2 Design verifications based on flood probability standards).

<u>Correlation between structural failure and failure due to overflow/overtopping</u>

The failure mechanisms structural failure and failure due to overflow/overtopping are strongly correlated, because in both cases the hydraulic load is dominant for the failure probability above the uncertainty of the strength and because the failure probability for overflow/transsshipment much greater than that for structural failure. It is therefore likely that a construction has already failed as a result of overflow/transhipment before structural failure occurs. In order not to dimension the artwork unnecessarily conservatively, or to work with water levels far above the crown, the degree of correlation between the two mechanisms is taken into account by the factor c. **(112 頁)** In view of the required reliability, the calculation value of the outside water level can still be higher than the crown height according to this Work Guide. However, excessive water levels high above the crown height are prevented by the factor c.

Table 9 Correction factor c for the correlation between structural failure and failure by overflow/transshipment [Ref. 7.4]

P_{max} [-]	1/100	1/300	1/1.000	1/3.000	1/10.000	1/30.000
c [-]	7	5	4	3	3	3

※（Rijkswaterstaat, Central government　2018）112 頁より作成。

Length effect

The length-effect factor N_{dsn} is formally determined by the number of more or less independent structural components in a range with the same reliability and the number of more or less independent instability mechanisms[32] (STCG). In reality there is a large degree of correlation due to the common load, namely the outside water level. In addition, the construction components rarely have equal reliability within a range:

- There can be a big difference in construction period between artworks in one process, which can lead to large differences in reliability between constructions.
- Material quality was less good in the past, building techniques and building codes change over time and degeneration has occurred.
- Major differences in reliability will generally also exist between main construction components, because for example the high water load is dominant for the high-water-tight closing means and the own weight for the foundation.
- The same applies to the instability mechanisms, in general one mechanism is dominant.

Within the WBI2017, a fixed value of 3 is used for the length-effect factor N_{dsn}. This estimate of N_{dsn} = 3 seems appropriate for most situations, which also followed from an analysis of many VNK2 results. For designs, therefore, N_{dsn} = 3 can be used.

7.7.2 Reliability requirement Building Decree

For the reliability requirements for new buildings, the Building Decree refers to Eurocode NEN-EN 1990. In this, follow-up classes with corresponding reliability requirements are determined. A reliability requirement is stricter if the consequences of failure are more serious. In NEN-EN 1990 three consequence classes have been defined. The reliability requirements for the ultimate limit state are shown in Table 10 for a reference period equal to the life span of the 50-year construction. A lifespan of 100 years is often used for hydraulic engineering constructions. As already explained in the beginning of section 7.7, these reliability requirements can also be used for 100 years.

The reliability requirement in the Eurocode is in principle set to each construction element separately and for each failure mechanism separately. The safety of the total construction as a system can therefore deviate from this. Usually the safety will be higher, because normally a redistribution of the internal forces is possible as soon as the strength is exceeded locally.

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[32] Horizontal or vertical instability and tilt instability

(113 頁) The extent to which this is possible depends on the static system and on the toughness properties of the materials and connecting elements used. Usually the entire system meets the reliability requirement if the components meet[33].

Consequence class	Consequences of failure			Reliability requirement for longevity $\beta_{\text{eis, BB}}$
	Risk of life danger		Risk of economic damage	
CC3	Very big	Or	Very big	4,3
CC2	Significantly		Significantly	3,8
CC1	Excluded / small	And	Excluded / small	3,3

※（Rijkswaterstaat, Central government　2018）113 頁より作成。

Table 10 Reliability requirements NEN-EN 1990 as prescribed in Building Decree (BB)

The relation between confidence index (β) and failure probability (P_f) is given in formula 7.3.

For the reliability requirements for existing buildings, the Building Decree refers to NEN 8700. However, this Work Guide only looks at new construction.

7.8 Design verifications constructive failure

In a design verification the reliability (failure probability) is confronted with the reliability requirement (failure probability). The reliability requirement follows from the Building Decree or is derived from the Water Act. Chapter 7.6 discusses the reliability over time and chapter 7.7 discusses the reliability requirements.

7.8.1 When a Building Decree and Water Act Verification?

The (provisional) design of a water-retaining artwork must always be verified with regard to structural failure based on a reliability requirement from the Buildings Decree (7.7.2). In addition, it may be that the (preliminary) design must also be verified on the basis of the reliability requirement for structural failure from the Water Act (7.7.1). The Water Act Verification is only required in the event of failure of the construction or the structural component, as a result of the considered load situation, a flooding will occur. Below are some examples which are not exhaustive. The manufacturer must determine for himself which verifications are necessary for the combination of the construction (part) and the load situation.

For example 1:

- Construction component: lock gate in the outer head of a lock.
- Load situation: the high water load situation.
- Result in failure: flood.
  Failure of the outer door leads to the loss of water retaining capacity as it is plausible that the strength of the inner door is less than or equal to the outer door and will also collapse. Failure event 3a or 3b (section 7.3.1) then occurs.
- Design verifications: Building Decree and Water Act

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For example 2:

- Construction component: lock door in the outside of a lock.
- Load situation: hanging door after renovation work by means of a floating buck.
- Result in failure: only damage to the door. The probability of a flood is small because it is not suspended in the high water season and only in calm weather conditions. In addition, there is relatively much time for emergency measures, should higher water levels be expected anyway. As a result, the water retaining capacity is not jeopardized.
- Design verifications: Building Decree

For example 3:

- Construction component: wing wall on the inland side of a lock.
- Load situation: the high water load situation.
- Result in failure: local damage artwork. The collapse will not lead to loss of water retaining capacity of the lock.
- Design verifications: Building Decree.

For example 4:

- Construction component: foundation outer head of the lock.
- Load situation: the high water load situation.
- Result in failure: flood. Instability of the foundation will lead to the loss of water retaining power, after which a (progressive) breach will arise (failure event 4 in section 7.3.2).
- Design verifications: Building Decree and Water Act

Restrictions regarding the Water Act verification

Unfortunately, at this moment only the semi-probabilistic design verification with respect to the reliability requirement from the Water Act for the flood water situation is facilitated with calculation rules. The step-by-step plan for going through this verification is included in section 7.9.

However, there are also other loads that cause a significant flood risk in the event of structural component failure. For example, shipping taxes on reversing means where, in case of failure, a large volume of water flows into daily situations. The design must then also comply with the reliability requirement from the Water Act in respect of these loads. However, semi-probabilistic calculation rules based on the Water Act are currently lacking to do this. In line with current design practice, it is sufficient to carry out a semi-probabilistic verification according to the Buildings Decree with the highest reliability requirement (CC3) and corresponding partial safety factors. When the design then meets, the Water Act will practically certainly also be complied with. Explanation of the semi-biblical verification follows in section 7.8.4.2.

Probabilistic verification is always possible, even though it is currently not facilitated. Explanation of the probabilistic verification follows in section 7.8.4.1.

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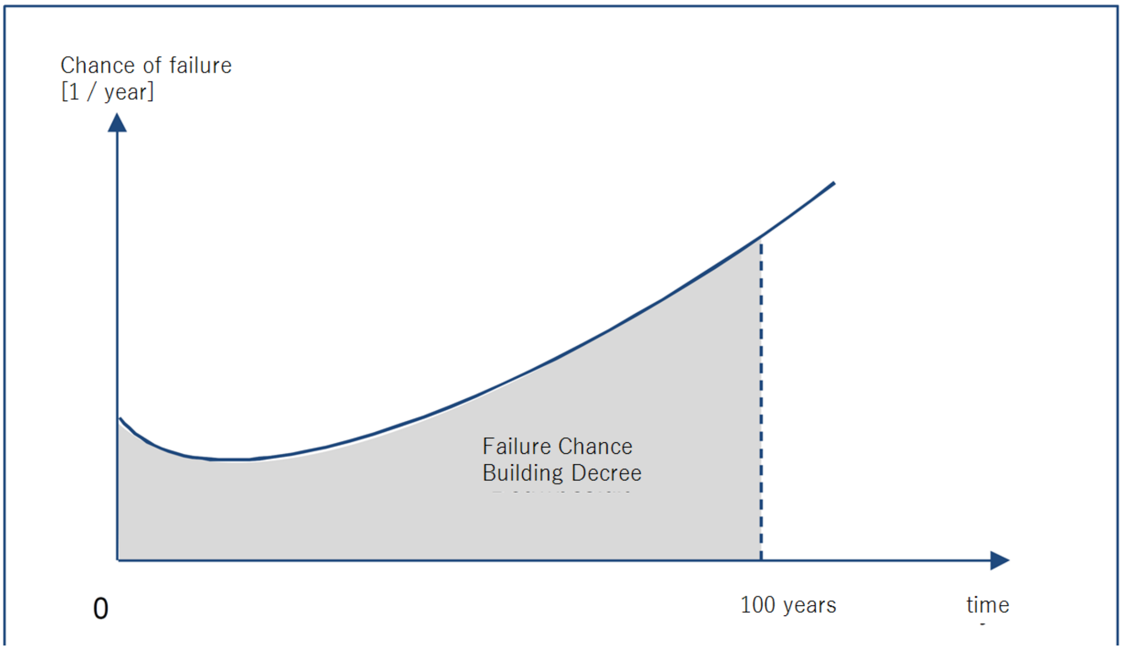
7.8.2 Design verification according to the Building Decree

The construction (component) must meet the reliability requirement or failure probability in the Buildings Decree during the planned service life (see paragraph 7.7.2). The failure probability of the construction (with a lifespan of 100 years) in a continuous period of 100 years to meet the failure probability that has been set for 100 years, see Figure 30.

As discussed in paragraph 7.6, the failure probability of a work of art depends on the tax considered, its development over time and the development of strength over time. In the failure probability analysis of flood defense structures burdened by high water, the probability of failure is generally the greatest in the last year of life. This is the failure probability trend as outlined in Figure 30: the 'bathtub curve'.



※（Rijkswaterstaat, Central government　2018）115 頁より作成。
Figure 30 Verification of failure probability from the Building Decree. The area under the curve is equal to the failure probability. Note: failure probability is specifically associated with (high) water catchment

In accordance with current design practice, the design verification will often be semiprobabilistic. This approach is explained in the following sections:

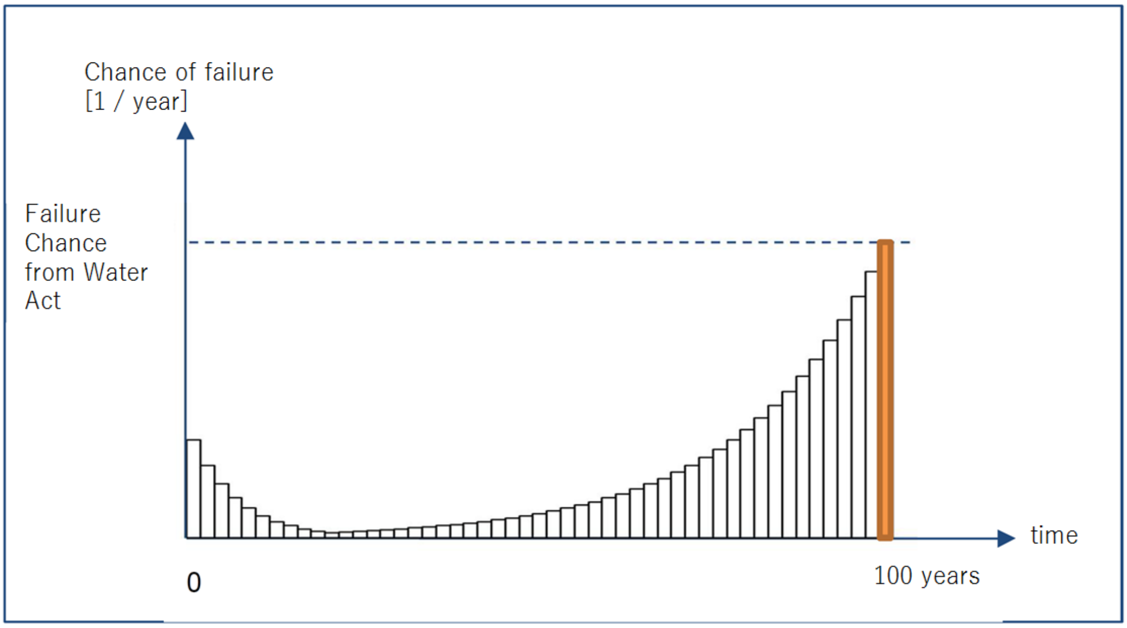
- Section 7.8.4.2 discusses the nature of the semi-probabilistic verification procedure.

- Section 7.9.1 provides the step-by-step plan for completing the semi-probabilistic verification for the flood water situation.
- Section 7.9.2 provides the step-by-step plan for completing the semi-probabilistic verification for other loads, such as wind load or tensile forces ect.

7.8.3 Design verification according to the Water Act

A construction (component) must comply with the failure probability derived from the maximum permissible probability of flooding from the Water Act in each year of the planned lifespan (see section 7.7.1). Calculation rules for semiprobabilistic verifications based on the Water Act are only available for the flood water situation. For the other tax situations that collapse and cause a flood, no semi-probabilistic calculation rules are available from the Water Act and reference is made to section 7.8.1.

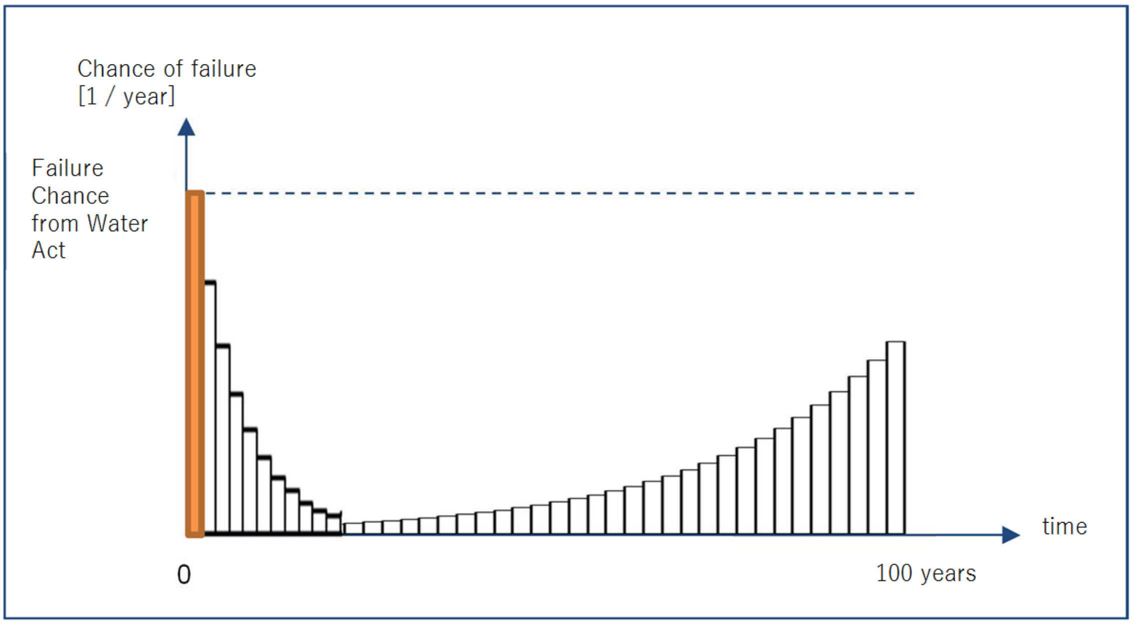
As mentioned, the risk of failure is generally greatest in the last year of life for water-retaining structures subject to flooding. In that case the design in the last year of life must be compared with the failure probability from the Water Act, see Figure 31.（116 頁）



※（Rijkswaterstaat, Central government　2018）116 頁より作成。

Figure 31 Probability failure check from the Water Act, where the realized probability of failure in the last year of life is normative

In rare cases, the probability of failure will be greatest in the first year of life. This is the case if the own weight load is dominant over the hydraulic load during a high water situation. In that case, the probability of failure in the first year of life should be compared with the failure probability for the Water Act, see Figure 32.



※（Rijkswaterstaat, Central government　2018）116 頁より作成。
Figure 32 Verification of failure probability from the Water Act, where the realized probability of failure in the first year of life is normative

Again, in accordance with current design practice, the design verification will usually be semi-biblical, with section 7.9.1 providing the step-by-step plan for completing the semi-biblical verification for the high water load situation.

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7.8.4　Types of verification procedure

A probabilistic or semiprobabilistic design verification can be carried out on the basis of the limit state function discussed in the previous paragraphs and the reliability requirements for a construction. This verifies whether the design meets the reliability requirements from sections 7.6 and 7.7.

7.8.4.1.　Probabilistic verification

This verification method is interesting when very sharply designed, because of (very) high construction or renovation costs.

In a probabilistic verification, the (ultimate) limit state function is fed with probability distributions of strength and loads. The probability distributions of the water level and the significant wave height can be found for every location with Hydra-NL. For the probability distributions of possible other loads, the construction properties and the model uncertainties, it is recommended to use the JCSS Probabilistic Model Code (http://www.jcss.byg.dtu.dk/Publications/Probabilistic_Model_Code); where relevant, the distributions must be brought into line with the prevailing characteristic values according to the Eurocode.

In case of probabilistic verification, the reliability of the structure is expressed as a failure probability P_f and the following condition is met:

$$P_f \leq P_{eis} \qquad \text{zie 7.4}$$

In which:

P_f Failure probability structural failure for a reference period equal to t_{ref} [-]

P_{eis} Failure probability work of art structural failure for a reference period equal to luck [-]

In the case of the Water Act, this concerns an annual verification and, in the case of a Eurocode requirement, a verification for a reference period equal to the life span.

7.8.4.2. Semi-probabilistic verification

In a semi-probabilistic verification, the limit state function is not fed with probability distributions but with calculation values. A calculation value is usually a combination of a representative value and a partial safety factor (see chapter 2 Design verifications on the basis of flood probability standards). A partial safety factor is calibrated so probabilistic that, when used in the Netherlands, the applicable reliability requirements for unity check (UC) are lower than 1, or UC <1 (Building Decree and/or Water Act). The unity check is defined as follows:

$$UC = \frac{E_d}{R_d} \qquad \text{7.7a}$$

At which:

E_d Calculation value tax effect

R_d Calculation value strength construction

(118 頁)

A construction meets the reliability requirement when:

$$UC \leq 1{,}0 \tag{7.7b}$$

Either:

$$R_d \geq E_d \tag{7.7c}$$

1) Calculation value strength

The calculation value for the strength R_d follows from formulas 6.6a to 6.6c in NEN-EN 1990.

2) Calculation value tax effect

For FAT border states (fatigue) NEN-EN 1990 refers to NENEN 1992 and NEN-EN 1999.

For STR and GEO boundary states, a distinction is made in NEN EN 1990 between:

- Permanent and temporary taxes: By far the most taxes or combinations of taxes fall under this category, such as the flood tax. The calculation value for the tax effect (E_d) follows from formula 6.10a ($E_{d,a}$) and formula 6.10b ($E_{d,b}$) in NENEN 1990, where:

$$E_d = \max(E_{d,a}; E_{d,b}) \tag{7.8}$$

- Extraordinary taxes: The calculation value for the tax effect (E_d) follows from formula 6.11a from NEN-EN 1990.

The flood tax falls under permanent or temporary taxes. Section 7.10.2 specifically addresses design verification in this respect. For the other taxes to be considered, the designer himself has to determine in which category they fall.

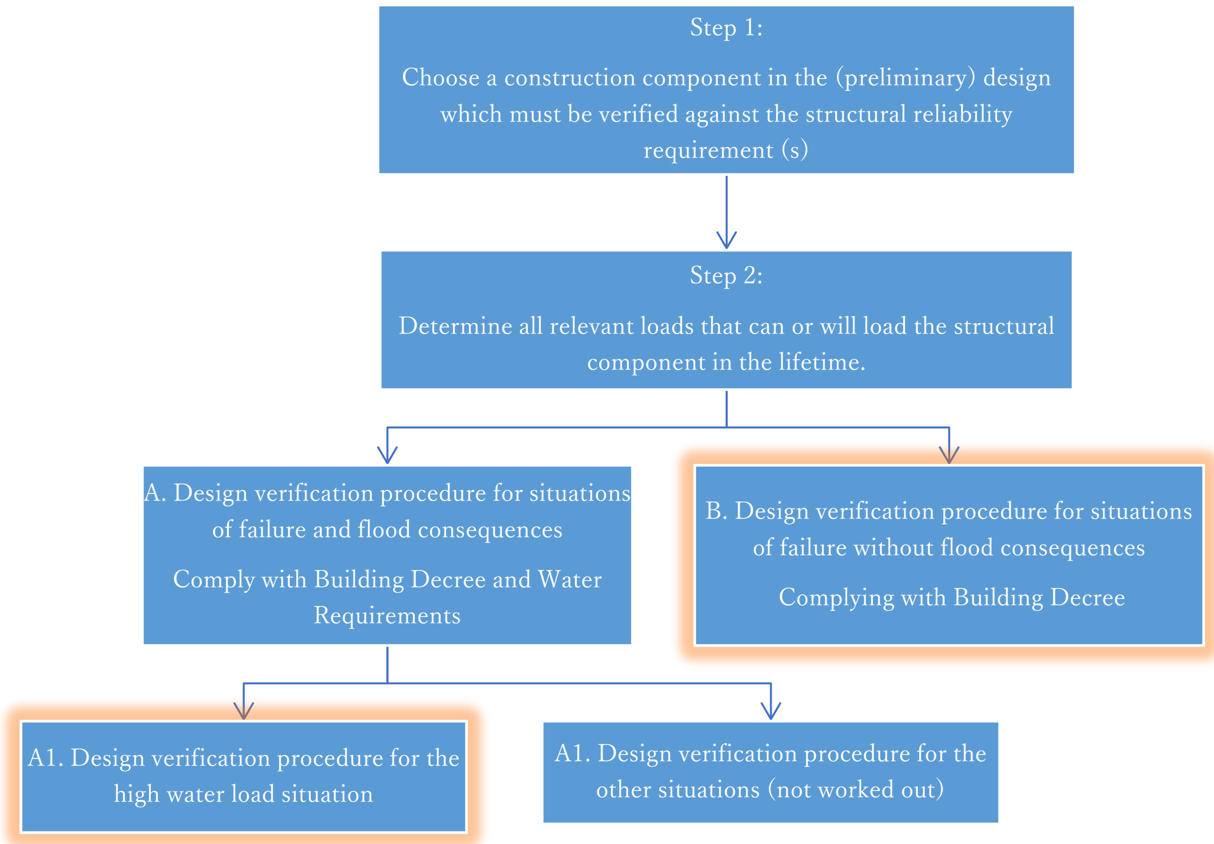
7.9 Step-by-step design verification

The step-by-step plan is included in the figure 33 below. The starting point of the step-by-step plan is a preliminary design of the flood defense artwork. The spatial integration and the design have already been determined, partly on the basis of other functions. In addition, provisional dimensions and material types have been assigned to construction components on the basis of functional considerations or rules of thumb. The following step-by-step plan outlines how, step by step, it can be verified semi-probabilistically whether the construction components are sufficiently strong and stable to meet the reliability requirements from Water Act and Building Decree. Another order can be followed as long as all steps are taken. Of course, there can also be probabilistic verification, but only in limited cases will that have added value. That is why the step-by-step plan for probabilistic verification is included in Appendix E.

Below is a brief explanation of the steps indicated:

Step 1. In this step the construction component or the combination of construction components is chosen which must be verified against the reliability requirements. (119 頁)All construction components must eventually be verified.

Step 2. For the structural component selected in step 1 or the chosen combination of components, it must be checked which loads are important. For example, a foundation pile will not be loaded by a screw radius load and a wooden point door will not be subject to a road traffic load. See section 7.10 for all possible loads.



※（Rijkswaterstaat, Central government　2018）119 頁より作成。

Figure 33 Roadmap for design failure mechanism structural failure

Continuation explanation on step-by-step plan:

A or B　A distinction is made between the verification procedure for situations where, in addition to collapse, there is also a flood (A) and for other situations where only collapse occurs (B). In case of (A), the reliability requirements from both the Water Act and the Building Decree must be considered. In all other situations (B), only the reliability requirements from the Buildings Decree need be considered.

A1 or A2 As indicated in section 7.8.1, only semi-probabilistic calculation rules are available for the high water load situation for the situations where failure and a flood occur (A). The verification for this has been elaborated in sub-step plan A1 (see Figure 34). For the other situations (A2) where collapse and flooding occur, semi-probabilistic calculation rules based on the Water Act are currently lacking. In line with current design practice, it is recommended that semi-probabilistic verification according to the Buildings Decree with the highest reliability requirement (CC3) and corresponding partial safety factors should be carried out according to procedure B. If the design then complies with the Water Act will practically certainly also be complied with.

Verification procedures A1 and B are further elaborated in Figure 34 and Figure 35, respectively.

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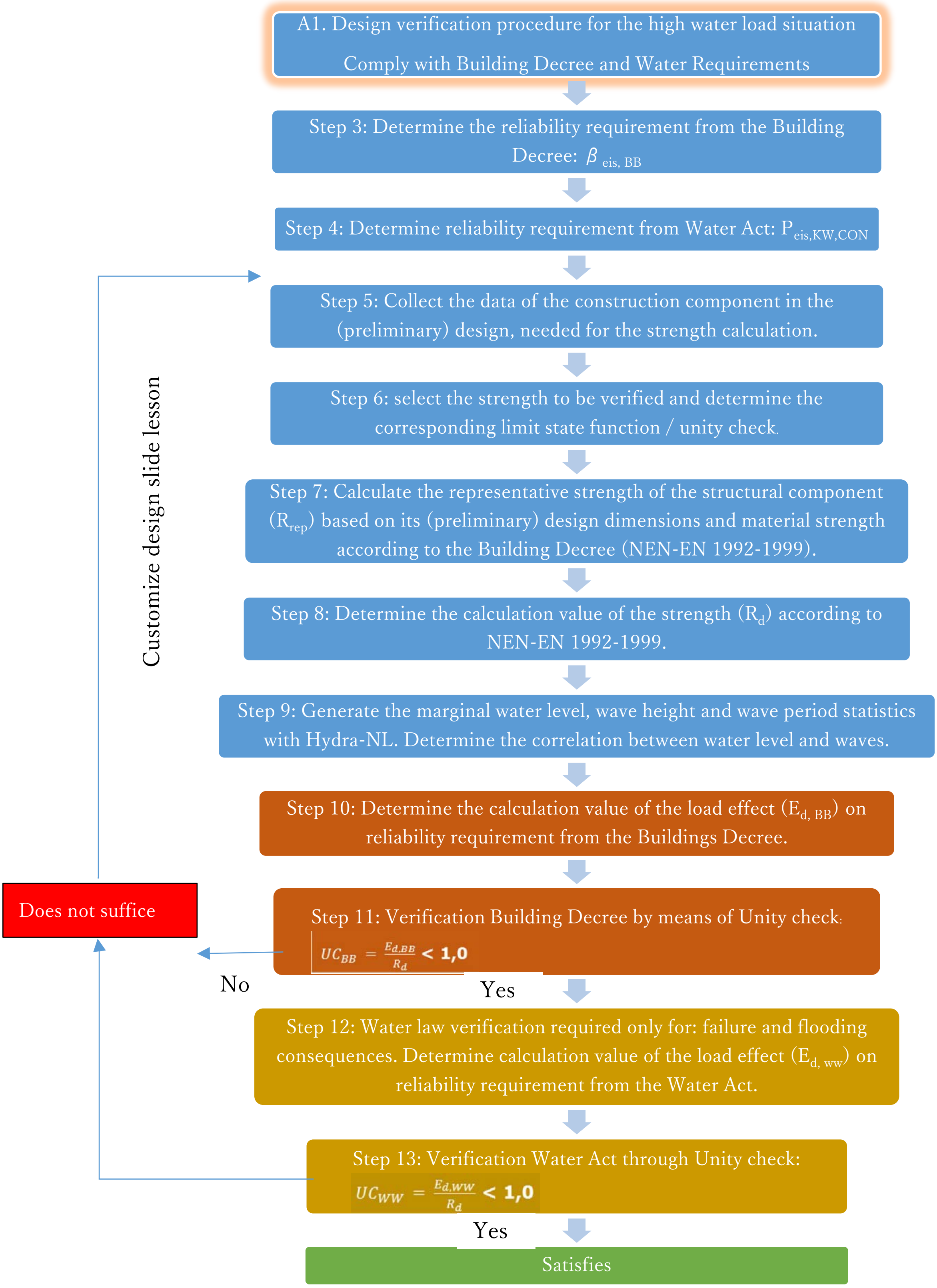

※（Rijkswaterstaat, Central government　2018）120 頁より作成。

Figure 34 Step-by-step design verification A1 for high water load situation

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7.9.1 Design verification procedure for the flood water situation (A1)

Step 3. The reliability requirement from the Buildings Decree depends on the required Consequence Class, see paragraph 7.7.2.

Step 4. The reliability requirement for structural failure from the Water Act is derived from the flood probability associated with the standard route in which the work of art is constructed, see paragraph 7.7.1.

Step 5. For information, see the (preliminary) design. The design verification of the structural component, or combination of components, with regard to the load situation (high) water defense involves checking whether the strength is sufficient to withstand the load. To this end, strength calculations need to be made, for which information about the dimensions and type of material is required.

Step 6. See section 7.5. The structural component or the combination of components can collapse in many ways, for example due to a shortage of moment capacity or due to too little resistance to buckling. The limit state function can also be expressed as a unity check (UC). In some cases the UC is already prescribed in the Eurocodes and the limit state function no longer has to be determined (see paragraph 11.6 of Chapter 11 Case).

- Strength: To be determined with the help of the rules of construction mechanics or with a specialist FEM calculation. This falls outside the scope of the Work Guide.
- Load: The load on the construction has a load effect on the structure. In the case of (high) water defense, the load consists of decay, waves and own weight load, see section 7.10.2 for the tax effect.

> Example
>
> The chosen structural component concerns a purlin in a door, which can be schematized as a beam on two supports with a distributed load q. It must be verified whether the chosen IPE profile is sufficient to withstand the current moment due to hydraulic load.
>
> - The hydraulic load results in a distributed line load q on the purlin
> - The maximum field moment $M_{belasting} = \frac{q \cdot l^2}{8}$
> - The strength of the purlin expressed in moment capacity

$$M_{sterkte} = \sigma_{max} \cdot W$$

- Border status function: $Z = M_{sterkte} - M_{belasting} = \sigma_{max} \cdot W - \frac{q \cdot l^2}{8}$
- Equivalent expression: $Z = 1 - \frac{M_{belasting}}{M_{sterkte}}$
- Construction component satisfies as: Z≥ 0
- Expressed in UC for semi-probabilistic verification: UC=

$$\frac{M_{belasting,d}}{M_{sterkte,d}} \leq 1$$

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Step 7. Using the mathematical relation for the strength from step 6, the type of material and the provisional dimensions of the construction part from step 5, the manufacturer determines the representative value by means of NEN-EN 1992 to 1999 (choice depending on the type of material). strength R_{rep} of the considered strength of strength, such as the moment capacity or the shear capacity.

Example

In this case, R_{rep} is expressed in $M_{sterkte,rep} = \sigma_{rep} \cdot W$

- σ_{rep} = representative bending strength (NEN-EN 1992 up to and including 1999)
- W = resistance moment of the construction part

Step 8. With the help of NEN-EN 1992 to 1999 (choice depending on the type of material), the constructor determines the material factor Υ_m and thus the strength R_d.

Example

In this case R_d is expressed in $M_{sterkte,d} = \frac{M_{sterkte,rep}}{\gamma_m}$

It should be noted that the material factors and formulas with calculation values for the strength (resistance) in NEN-EN 1992 up to and including

1999 are derived for CC2. The factors do not have to be adjusted, as in the past, when performing a Water Act verification. In determining the calculation value of the load (E_d) with the standard method (section 7.10.2) this has already been taken into account.

Step 9. The hydraulic load is location-specific. At this moment it is not yet possible to determine the local statistics of the hydraulic load with load models. For this reason, the manufacturer will have to determine the hydraulic load himself on the basis of the outside and inland water level, the wave height and the wave period. The marginal statistics can be determined with Hydra-NL, see also chapter 2 Hydraulic loads. In order to combine these statistics with the statistics of the hydraulic load on the structure, the mutual correlation must be known, see section 7.10.2.1 for an explanation.

Building decision verification:

Step 10. The tax effect $E_{d,\,BB}$ is determined by means of formulas 6.10a and 6.10b and the reliability requirement from the Buildings Decree, see paragraph 7.10.2. Summary: with regard to the high water load situation, the effect of 6.10a is included in Appendix C and 6.10b in Section 7.10.2 (the 'standard method').

Example
In this case E_d is expressed in $M_{belasting,d}$

Step 11. The actual verification concerns the unity check: the construction component or combination of components is sufficient when UC <1.0.

- If UC <1.0: continue with step 12 and / or check whether the design is fine.

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- If UC> 1.0: continue with step 5, adjusting the dimensions and perhaps even the material type. The subsequent steps must then be followed again. One can also choose to proceed first with step 12 and complete the Water Act Verification. It may be that according to the Water Act the component is even more disapproved.

Example

In this case: $UC = \frac{M_{belasting,d}}{M_{sterkte,d}}$

Water law verification:

Step 12. The tax effect $E_{d,WW}$ is determined by means of formulas 6.10a and 6.10b and the reliability requirement from the Water Act, see paragraph 7.10.2. Summary: with regard to the high water load situation, the effect of 6.10a is included in Appendix C and of 6.10b in Figure 36 (the 'standard method').

Example

Same as Building Decree verification.

Step 13. The actual verification concerns the unity check:

- If UC <1.0:
  - The construction component complies with the relevant strength variable. Continue with step 6 where all other relevant strength variables are verified. In addition to, for example, the moment capacity of a beam, the shear force and possibly also the stability of the profile should also be verified.
  - If the structural component satisfies all relevant strength variables, a subsequent component can be selected and we start again in step 1.
- If UC> 1,0: continue with step 5, adjusting the dimensions and perhaps even the material type. Subsequently, the successive steps must be followed again.

7.9.2 Design verification procedure for other load situations (B)

The design verification procedure for 'other taxes' is without Water Act verification. As discussed under option A or B, it is recommended to choose the reliability requirement for CC3 in the event that the relevant load still causes a significant flood risk. The following description of procedure B is limited to the differences with the procedure for the high water load:

Step 4/12/13: N / A.

Step 9. No tax statistics need to be generated in procedure B. The representative value of the tax can be obtained directly from the Eurocodes NEN-EN 1992-1999 for general taxes and for typical loads on hydraulic structures from Table 11.

Step 10. The calculation value of loads can be determined with the baling factors from NEN-EN1990 / 1991 and the representative values from step 9. Where for loads typical of hydraulic constructions use can be made of Table 11.

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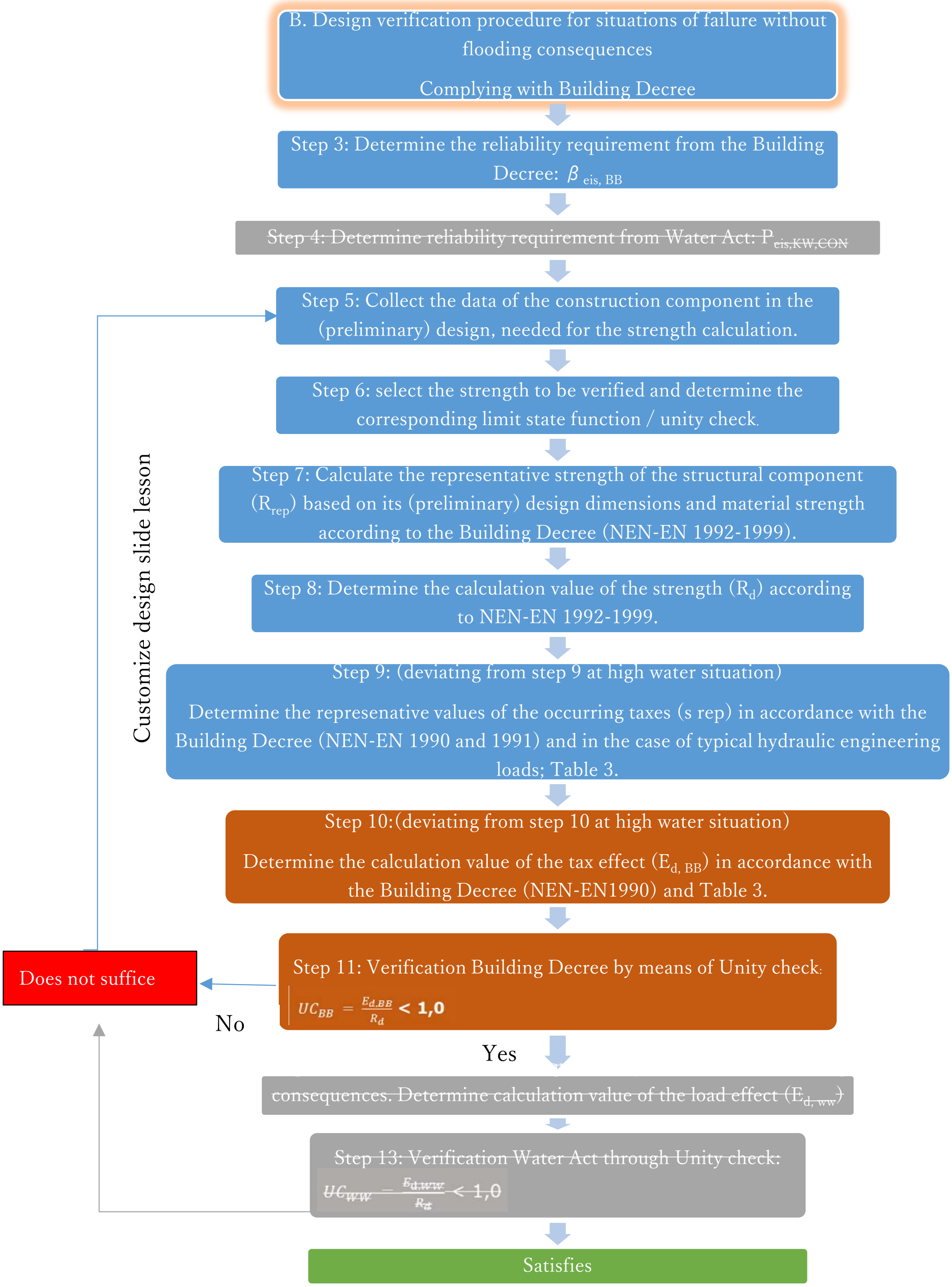

※（Rijkswaterstaat, Central government　2018）124 頁より作成。

Figure 35: Step-by-step plan for design verification B. other taxes

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7.10 Taxes

When determining the load effect, all combinations of loads that may occur during the construction phase, use and maintenance must be taken into consideration. Where relevant, account should be taken of the dynamic nature of taxes and the taxes associated with the situation of "not (timely) closing of weathering".

In accordance with the Eurocode NEN-EN 1990, a distinction can be made between permanent, variable and special taxes. The following paragraphs will discuss the calculation of the calculation values of these loads in more detail. In addition, we also consider (special) usage taxes.

7.10.1 Calculation values for the load

Calculation values of loads are given in NEN-EN 1991. However, hardly any mention is made of hydraulic constructions, hence the calculation values of the most important loads on flood defense structures are included in Table 11. The designer himself must comply with Step 2 of the step-by-step plan in Section 7.9. determine the relevant combinations of taxes,
using Table 11 below and the loads described in NENEN 1991. Some taxes exclude each other, so that they do not have to be considered in combination with each other.

Table 11: Calculation values of loads typical for flood defense structures

Tax	**Calculation value as dominant load (belonging to CC2 from NEN-EN1990)**	**Calculation value as a combination tax**
PERMANENT: Own weight Ground pressure Groundwater pressure Setting	 (1.35 or 1.0 or 0.9) F_{rep} and paragraph 7.10.2 1.0 F_{rep} See section 7.10.4.3 (1,2 or 1,0 or 0,9) u_{rep}	 In accordance with NEN-EN1990 / NB
VARIABLE: Pressure differences (high)		

water retaining. - water levels - wind waves Pressure differences other water retaining situations related to: - water levels - wind waves Flow Ship waves Ship flow Troots Wind load Temperature Traffic tax	 See section 7.10.2 See section 7.10.2 See section 7.10.3 See section 7.10.3 1,3 F_{50} 1,3 F_{rep} 1,3 F_{rep} See section 7.10.5.4 1,5 F_{rep} 1,5 F_{rep} 1,35 F_{rep}	 See section 7.10.2 See section 7.10.2 See section 7.10.3 See section 7.10.3 1,3 F_1 1,3 F_{rep} 1.3 F_{rep} See section 7.10.5.4 1.5 Ψ F_{rep} (Ψ = 0.2) 1.5 $F_{momentaan}$ 1.35 F_{rep}
SPECIAL: Collision Earthquake Explosion Ice Flow (do not close) Vandalism / sabotage	 F_d (see section 7.10.6.2) Customization: see last position of knowledge $F_{nominaal}$ F_{CUR166} F_d (see section 7.10.6.1) -	 0 Customization: see last state of knowledge 0 0 0 -
F_{rep} = Representative value of the load according to the building regulations F_n = Representative value of the tax for variable loads with a repetition time of one year $F_{nominaal}$ = Nominal value of the special tax		
Adjust calculation value for CC1 and CC3: The above factors for permanent and variable loads apply to CC2. For CC1 and CC3, the permanent and variable load factors are multiplied by 0.9 and 1.1 respectively. This does not apply to loads of water levels and wind waves; for these taxes reference is also made to paragraph 7.10.2 for the differentiation.		

※（Rijkswaterstaat, Central government　2018）125 頁より作成。

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In addition to the aforementioned 'external' taxes, in the case of flood defense structures, 'internal' taxes associated with the use of the artwork must be taken into account. Consideration can be given to the forces on reversing means from the guide through the deformation of bearings, the forces from the movement work on the reversing means if the movement is blocked at the bottom by an obstacle, or the holding force from the movement work. This guide does not provide any further information about this, but it must of course be taken into account. Damage by internal forces can ultimately also have consequences for the availability of the artwork and thus for safety.

7.10.2 High water load situation

This section considers the high water load situation, which is a combination of the hydraulic load at high water and the own weight load. The hydraulic load consists of the decay load and the wave load [Ref. 7.3]. Other taxes will generally not be relevant for this tax situation. Also the wind load is not, because in the high water load situation in particular construction components are loaded that turn water. The water in that case is so high that the extra load by wind does not matter or is nil. Other hydraulic load situations, such as in the case of a negative return of water or the drying situation, are discussed in section 7.10.3.

Due to the lack of suitable load models for the time being, the decay and wave loads in the calibration of the semi-probabilistic method are combined in the hydraulic load term (S), where for S a Gum distribution is assumed [Ref. 7.3]. In reality, S is a function of the decay (V) and the wave load (H):

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$$S = f(V, H) \tag{7.9}$$

As stated in section 7.8.4.2, the calculation value of the tax effect E_d is determined by the maximum of $E_{d,a}$ and $E_{d,b}$ (formulas 6.10a and 6.10b from the NEN-EN 1990). When formulas 6.10a and 6.10b are completed for the high water load situation, only the hydraulic and self weight load is therefore relevant and follows:

Own weight dominant: $E_{d,a} = k_{FI} \cdot \gamma_G \cdot G_k + \Psi_0 \cdot S_d$ 6.10a

Hydraulic load dominant: $E_{d,b} = k_{FI} \cdot \gamma_G \cdot \xi \cdot G_k + S_d$ 6.10b

At which:

$E_{d,a}$ Calculation of the tax effect when the own weight is dominant

$E_{d,b}$ Calculation value of the load effect when the hydraulic load is dominant

G_k Characteristic value own weight tax

K_{FI} Factor that in the case of the assessment in accordance with the Building Decree depends on the chosen consequence class CC1, CC2 or CC3 and in case of verification in accordance with the Water Act is set equal to 1.0

Y_G Partial factor for permanent loads

ξ Reduction factor for unfavorable own weight tax

S_d Calculation value of the hydraulic load at high water

Ψ_0 Tax combination factor

In the vast majority of cases 6.10b will be decisive for the high water load situation. Only in case the own weight covers 80% or more of the total load, can 6.10a give the normative tax effect.

For the own weight load, a load factor (Y_G) must be combined with the characteristic value of the own weight (G_k). For the hydraulic load, expressed as water pressure on the structure, it was decided not to apply a partial factor, but to prescribe the exceedance probability of the calculation value of the load P ($S> S_d$). With this exceedance probability and the tax statistic, the calculation value of S_d can then be determined directly. The prescribed exceedance probability P ($S> S_d$) is derived from the constructive reliability requirements from the Water Act (section 7.10.2.5) and the Building Decree (section 7.10.2.4) [Ref. 7.3]. The advantage of this method over the old method with a load factor is that the pressure figure with which it is calculated remains physically correct. The exceedances for the hydraulic load and the load factor for the own weight load have been calibrated with regard to model uncertainties.

In order to be able to determine the calculation value of the hydraulic load, the statistics of the hydraulic load S are required. This is determined by the difference between the load from the outside water (outside water level and waves) and the load from the inland waterway. In the following, the inner water level is assumed to be constant for the sake of convenience. In section 7.10.2.3, the calculation value of the inland water level is discussed in more detail.

Ideally, the available instruments are suitable for determining the calculation value of the load from the outside water, which is a function of the combined statistics of outside and inland water level, wave height and wave period. **(128 頁)**At this moment the instruments (such as Hydra-NL) are not yet suitable for this. With these instruments, only the marginal statistics of the outside water level, the significant wave height and wave period can be determined (in the form of exceedance frequency lines) [34]. In addition, local waterways are selected for inland waterways, how to deal with them is described in section 7.10.2.3.

In order to be able to determine the calculation value of the hydraulic load S_d, use must be made of calculation values of decay V and wave load H. The calculation values of these variables are defined on the basis of their exceedance probabilities P ($V > V_d$) and P ($H > H_d$) .

When determining the calculation values V_d and H_d, a number of matters are important:

1. Own weight load dominant: If the constructor estimates that the own weight is 80% or more of the total load, then the design must be verified with E_d according to 6.10a and 6.10b. Since the hydraulic load is not always the dominant load, S_d has a relatively high exceedance probability (low return time). This also applies to V_d and / or H_d. Because these are exceptional situations for high-flood works, this verification is not explained in the main text but in Appendix C.

2. Hydraulic load dominant: For high-water works, 6.10b will almost always give the normative load effect, see also [Ref. 7.4]. Then one can suffice with just a verification according to 6.10b. If the hydraulic load is dominant, S_d has a relatively small exceedance probability (large return time). This also applies to V_d and / or H_d. This results in the 'standard method' shown schematically in Figure 36.

3. Fault or wave load dominant within the hydraulic load: In both 6.10a and 6.10b the decay or wave load (V_d or H_d) within the hydraulic load S_d can be dominant. In advance it is not always possible to determine which is dominant, both tax combinations should be considered.

4. Correlation between outside water level and significant wave height:
   If the outside water level and the significant wave height are perfectly positively correlated, then P ($S > S_d$) = P ($V > V_d$) = P ($H > H_d$). In all other cases, it is conservative to determine both V_d and H_d with an exceedance

probability equal to P ($S > S_d$). In order to prevent excessive conservatism, combination values can then be used. There are two possibilities: V_d is dominant or H_d is dominant. The dominant tax variable always has the smallest exceedance probability. See also section 7.10.2.1.

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[34] NB1: with Hydra-NL the joint statistics of outside water level and waves are used to determine the HBN (required crown height). See chapter 3 Hydraulic preconditions. NB2: At the time of writing (July 2018), the WBI is exploring the options for directly determining the calculated value of the hydraulic load S_d associated with a prescribed chance of exceedance with Riskeer for the sight years up to and including ca. This application will not be available until mid-2019 at the earliest.

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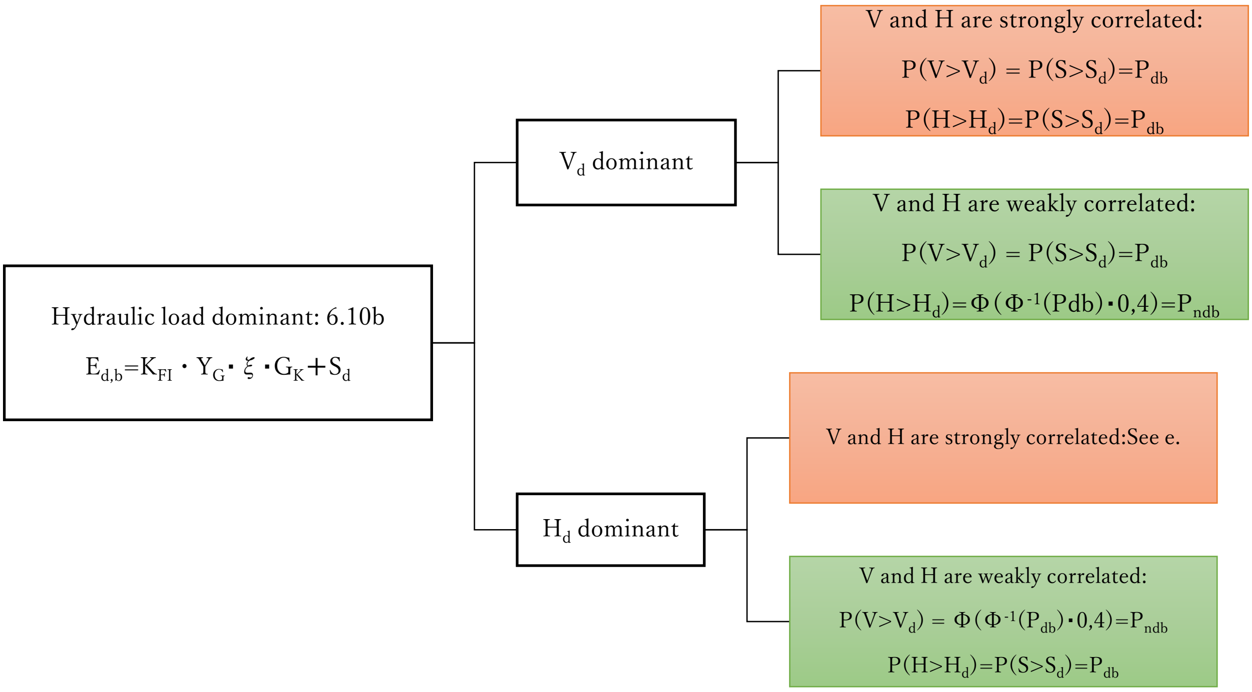

※（Rijkswaterstaat, Central government　2018）129 頁より作成。
Figure 36: 'Standard method' determine exceedance probabilities calculation values for decay and wave load according to 6.10b.

- P_{db} = exceedance probability in case of dominant load
- P_{ndb} = exceedance probability in case of non-dominant load
- Quantification P_{db} and P_{ndb} according to Building Decree and Water Act: see sections 7.10.2.4 and 7.10.2.5.

As stated in section 1.3 in general for the Work Guide, the method discussed here for determining the calculation values of the hydraulic and self-weight load is not readily suitable for longitudinal structures.

7.10.2.1. Correlation between decay and wave tax

In the diagram in Figure 36 an estimate must be made of the correlation between the decay load and the wave load. In the case of a strong positive correlation between outside water level and waves, there is a high probability that a large wave height and period will occur when a high water level occurs. Depending on the orientation of the artwork, this may be the case along the coast where both the water level and the waves of wind are dominated. When the orientation approximately corresponds to the dominant wind direction, decay and wave loads will be strongly correlated. If the orientation of the artwork deviates a great deal from the dominant wind direction, which determines the water level along the coast, then the decay and wave loads on the artwork are not or hardly correlated.

In the case of a weak correlation between water level and waves, the occurrence of a certain water level says little about the wave height and period. This is the case, for example, in the upper river area where the water level is heavily discharged while the waves are determined by local wind. In this situation, the orientation of the artwork no longer matters for the correlation between the decay and wave load.

Of course, the correlation between water level and waves can also be between strong and weak. At this moment it is not possible to make a statement about the correlation with Hydra-NL. Hence, it is recommended to assume full correlation, except for the above-mentioned exception of the upper river area.

When wave heights are small compared to the outside water level, the degree of conservatism is small in this assumption. But when it is expected that this is unnecessarily conservative in this way, advice can be requested through the Helpdesk Water.

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7.10.2.2. Calculation value of wave tax

Wave loading is caused by wind. The wave load varies over the height of the artwork. The wave load can be calculated with several models and depends, among other things, on the wave height, wave period and angle of incidence. In WBI2017 the wave load model of Goda is implemented, the most used model at the moment to determine wave loads on a vertical wall. The model, to be used for the semi-probabilistic application, is included in Appendix D.

To determine the calculation value of the wave load H_d, use must be made of a load model (for example, model of Goda), the exceedance probability of the calculation value P (H> H_d) (see Figure 36 and paragraphs 7.10.2.4 and 7.10.2.5) and the combined statistic of water levels and wave characteristics.

The probability distribution of the wave load on a work of art on the location of a bank location cannot currently be determined with the current instruments. As already indicated in section 7.10.2, the marginal statistics on the water level and the wave characteristics (Hydra-NL) are forced to work. The marginal statistics for the significant wave height (H_s) and wave period ($T_{m-1.0}$) are omni-directional. This means that no account has been taken of a specific direction of wave incidence.

There is a dependency between the wave height and the wave period. It is recommended to assume full correlation between the wave height and the wave period. This means that the calculation values of these variables must be determined with the same exceedance probability. A precondition here is that this correlation does not lead to unrealistic wave losses. Thereby it can be assumed (conservatively) that the resulting wave load is associated with waves that are perpendicular to the structure.

The method for determining the calculation value of the wave load H_d falls apart in the following steps:

1. Determine the calculation value of H_s by means of its marginal statistics and $P(H_s > H_{s,d}) = P(H > H_d)$, see Figure 36 and Table 12 and Table 13:

2. Determine the calculation value of $T_{m-1.0}$, by means of its marginal statistics and $P(T_{m-1,0} > T_{m-1,0,d}) = P(H > H_d)$

3. Assume a frontal wave attack

4. Calculate the calculation value of the wave load H_d using a wave load model, for example the model of Goda (Appendix D). This load is expressed in terms of a

pressure gradient across the height of the structure.

With this method the upper limit of the calculation value of the wave load is determined. A calculation with a probabilistic wave load model always yields a lower calculation value.

An important fact is that in many cases the wave load is relatively small compared to the expiry tax. In the river area the waves are usually small because the length of the wire is often short and where large waves are to be expected, often breakwaters are used. When it is therefore expected that the above method is unnecessarily conservative, the advice of the Helpdesk Water can be requested. (131 頁)Finally, it should be noted that no reduced reliability index applies to wind waves, such as wind pressures in the Dutch National Annex to EN-EN 1990.

7.10.2.3. Arithmetic value inside water level

In some situations, the inland water level is also uncertain with a certain degree of dispersion, depending on the water management conducted just before and during a flood and the orientation of the inland waterway. In many situations the inland water level is limited, such as with an underlying regulated channel. The calculation value of the inland water level $h_{bi,\,d}$ can be determined as follows.

If the inland water level during high water is not known, or not checked, the average water level during the winter season can be taken as the starting point because the water level of the water system during a high water due to seepage and possible decrease of discharge capacity will almost always be higher. If there is a limited summer and winter level can be assumed the winter level. However, it must be verified whether the water level at the work of art cannot be reduced as a result of fanning and grinding. Coordination with the manager regarding the experiences with recent flood periods is desirable in determining the calculation value of the inland water level.

If the inland water level is uncertain and one has statistics during high water, it is also possible to work with an exceedance probability of $h_{bi,\,d}$, whereby the inland water level is considered as the non-dominant water level within the decay load:

$$P\left(h_{bi} > h_{bi,d}\right) = \Phi(-0{,}4 \cdot \Phi^{-1}\left(P(V > V_d)\right) \qquad 7.10$$

Here is:

h_{bi}	Lowest inland water level compared to NAP in a period of one year [m]
$h_{bi, d}$	Calculated value of the inland water level in relation to NAP [m]
Φ (...)	Standard normal distribution
Φ^{-1} (...)	Inverse of the standard normal distribution
P ($V > V_d$)	Exceeding probability of the calculation value of the expiry tax in a period of one year [-]

This approach assumes that the load effect of the outside water level is dominant, which is almost always the case.

7.10.2.4. Recommended calculation values for verifications based on the Building Decree

The 'standard method' (Figure 36) and the table below can be used for the purposes of the verification in accordance with the Building Decree with 6.10b from the NEN-EN 1990. Here, the exceedance probabilities of the calculation values of the decay and wave load must be determined with the 1-annual tax statistics for the last (mostly 100^{e}) year of life, depending on the reliability requirement $\beta_{\text{eis, BB}}$ (see paragraph 7.7.2). This is also the case with a dominant own weight tax.

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Table 12 Prescribed exceedance probabilities and parameter values according to 6.10b for verification Building Decree

Follow-class	$\beta_{\text{eis, BB}}$ for reference period equal to lifetime	$P(S>S_d) = P_{db}$ [-] Involve on 1-year statistics in the last year of life	P_{ndb} [-] Involvement in 1-year statistics in the last year of life	KFI*	ξ *	Y_G*
CC1	3,3	$5{,}0 \cdot 10^{-4}$	$9{,}4 \cdot 10^{-2}$	0,9	0,89	1,35 of 0,9**
CC2	3,8	$9{,}0 \cdot 10^{-5}$	$6{,}7 \cdot 10^{-2}$	1,0	0,89	1,35 of 0,9**
CC3	4,3	$1{,}0 \cdot 10^{-5}$	$4{,}4 \cdot 10^{-2}$	1,1	0,89	1,35 of 0,9**

※（Rijkswaterstaat, Central government　2018）132 頁より作成。

* value in accordance with NEN-EN 1990 / NB

** 1.35 in case of unfavorable working and 0.9 in case of a favorable working weight

The exceedance probabilities in the table have been calibrated taking into account model uncertainties

In case of a verification in accordance with 6.10b, the application of CC3 from the Buildings Decree and the use of the fixed failure probability budget (Table 1) and the default N value (= 3); a verification based on the Water Act is no longer necessary. In that case, the Building Decree is always decisive for the tax effect $E_{d,b}$.

As mentioned earlier, for a verification in accordance with 6.10a, reference is made to Appendix C. The Casus in chapter 11 contains a practical elaboration of the recommended calculation values based on the Building Decree, see section 11.6.10.

7.10.2.5. Recommended calculation values for verifications based on the Water Act Tax Statistics in which year of the construction? During the verification on the basis of the Water Act, when determining the tax statistic, a distinction must be made between the situation that the own weight load is dominant (6.10a) and the situation where the hydraulic load is dominant (6.10b):

- When checking with a dominant self-weight tax (6.10a) must are based on the tax statistics in the 1st year of life of the construction. This is different from verifications based on it Building Decree.
- At a verification with a dominant hydraulic load must be based on the tax statistics for the last (usually 100 th) year of life of the structure. Here the subtle difference between failure and failure must be be observed, see section 7.4.

Hydraulic load dominant
If the hydraulic load is dominant, the load effect only needs to be determined according to 6.10b. The exceedance probabilities from the 'standard method' (Figure 36) P_{db} and P_{ndb} and the parameter values for the self-weight tax are quantified in Table 13. P_{db} and P_{ndb} must therefore be included in the 1-year tax statistics in the last (mostly 100th) year of life.

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The values of P_{db} and P_{ndb} to be maintained and the partial factor for the own weight load are given in Figure 37 and Figure 38 respectively as a function of the failure probability $\beta_{\text{eis, KW, CON}}$:

$$\beta_{\text{eis,KW,CON}} = -\Phi^{-1}(P_{\text{eis,KW,CON}}) \quad 7.11$$

Here is:

$\beta_{\text{eis, KW, CON}}$ Failure Chance expresses in a reliability index for structural failure and no failure by overflow/transshipment for a reference period equal to t_{ref} = 1 year [-]. See section 7.7.1.

Φ-1(...) Inverse of the standard normal distribution

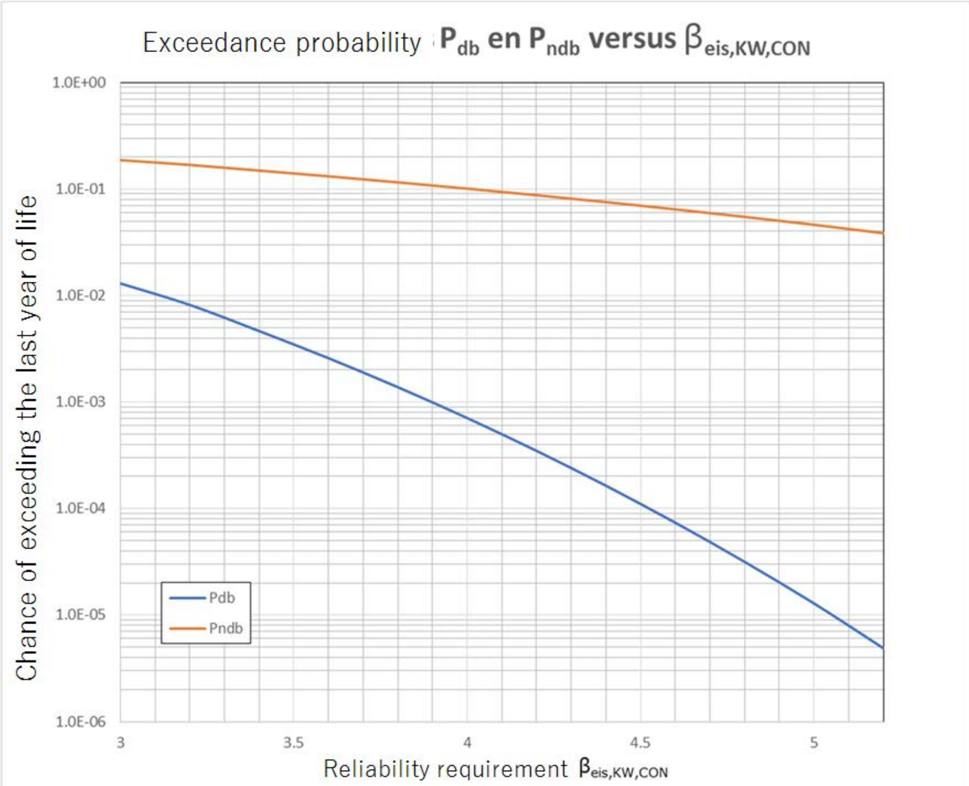
Table 13 Recommended exceedance probabilities and parameter values according to 6.10b when verifying Water Act

Parameters	**Recommended values**
K_{FI}	1,0
ξ	0,89
Ψ_0	0,6
Y_G	See Figure 38
P_{db} and P_{ndb} [-] Involve on 1-year statistics in the last year of life	See Figure 37

※（Rijkswaterstaat, Central government　2018）133 頁より作成。

NB1: The exceedance probabilities and the tax factor for the own weight load are calibrated taking into account model uncertainties.

NB2: The correction of the material factors is incorporated in the P_{db} and P_{ndb}. The material factors in NEN-EN 1992-1999 are based on CC2.



※（Rijkswaterstaat, Central government　2018）133 頁より作成。

Figure 37 Recommended exceedance probabilities for decay and wave load P_{db} and P_{ndb} according to 6.10b for inspections based on the Water Act

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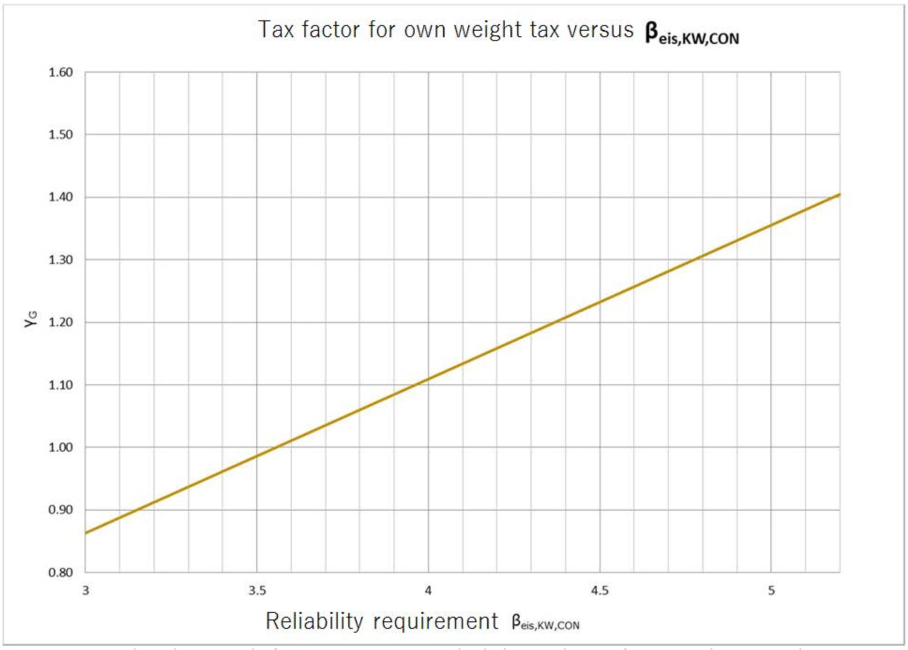


※（Rijkswaterstaat, Central government　2018）134 頁より作成。

Figure 38 Recommended partial factor for self-weight tax during inspections based on the Water Act in case of a dominant hydraulic load.

In case of a verification in accordance with 6.10b, the application of CC3 from the Buildings Decree and the use of the fixed failure probability budget (Table 1) and the default N value (= 3); a verification based on the Water Act is no longer necessary. In that case, the Building Decree is always decisive for the tax effect $E_{d,b}$.

Own weight tax dominant

When the constructor estimates that the own weight is 80% or more of the total load, the verification must also be carried out in the first year of life, whereby the tax effect is determined by 6.10a. Because these are exception situations, this verification is not explained in the main text but in Appendix C.

In the case study in chapter 11 a practical elaboration of the recommended calculation values based on the Water Act is included, see section 11.6.12.

7.10.3　Other hydraulic loads

In this section, other hydraulic loads are treated, namely the loads associated with turning water in daily situations, when there is a negative gradient and when turning water in incidental situations. The method for deriving calculation values for these hydraulic loads has been elaborated on the basis of a fictitious example of a lock. In the elaboration, use has been made of [Ref. 7.6].

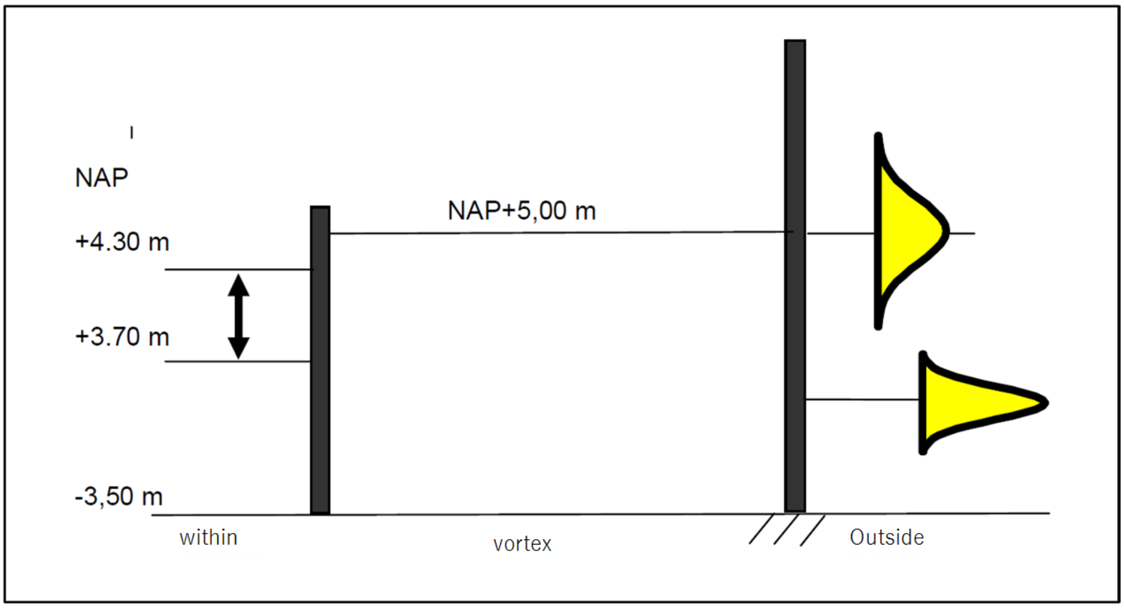
7.10.3.1. Example of lock lock

The lock to be designed in this example is part of a standard route which, according to the Water Act, has a lower limit value of 1 / 10,000 per year. In the question specification, CC2 is required with a design life of 50 years.**(135 頁)**The floor height of the lock should be at NAP + 3.50 meters. The artwork will connect a channel (inside) with the outside water (outside). The fencing process is stopped at water levels higher than NAP + 5.00 meters and lower than NAP + 3.00 meters. The probability of not closing per closing question (P_{ns}) of the outside head is equal to 10^{-3} per question.

Table 14: lock levels in the example lock locks

Parameters	**Values**
Lock level minimum	NAP+3,00 m
Lock level minimum	NAP+5,00 m

※（Rijkswaterstaat, Central government　2018）135 頁より作成。



※（Rijkswaterstaat, Central government　2018）135 頁より作成。

Figure 39 Schematic representation of water levels example of the lock. The probability distributions shown are the probability distributions of the annual minima and the annual maxima of the external water level.

Channel levels

The channel concerns a controlled water system with maintained levels according to Table 15. It is assumed that there is little to no uncertainty regarding these maximum and minimum channel levels, or the calculation value of the water level load by channel water can be derived directly from these channel levels.

Table 15 Channel levels in the example lock locks

Parameters	Values
Channel level minimum	NAP+3,70 m
Channel level minimum	NAP+4,30 m

※（Rijkswaterstaat, Central government　2018）135 頁より作成。

The outside water

The outside water is a fictional river with an uncertain water level. The lock is in a sheltered location so that the wave load is zero. In this example, the annual maxima and annual minimums of the outside water level are described by Gumbel distributions. Due to climate developments, the outdoor water distribution in the last year of life will be the worst, hence the Gumbel distributions for the 50th year of life are shown in Figure 40.

In a design assignment in practice, the water level statistics can also have a different distribution, or as in most cases a data series from, for example, Hydra-NL. The way in which the water level is determined has no consequence for the present method.

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Water level statistics for annual maximum, for example, lock

Parameters	Values *
Outside water annual maximum	μ h,max = NAP+5,00 m Bmax = 0,50 m

* Associated with years of metric from the 50th year of life

The annual maximum is described using a Gumbel distribution for maxima (chance of fall):

$$F_{h;max}(h) = \exp\left(-\exp\left(-\frac{2{,}3(h-u_{max})}{B_{max}}\right)\right)$$

At which:

$$u_{max} = \mu_{h,max} - \frac{\gamma \cdot B_{max}}{2{,}3} = \mu_{h,max} - 0{,}25 \cdot B_{max} = NAP + 4{,}88\, m$$

With:

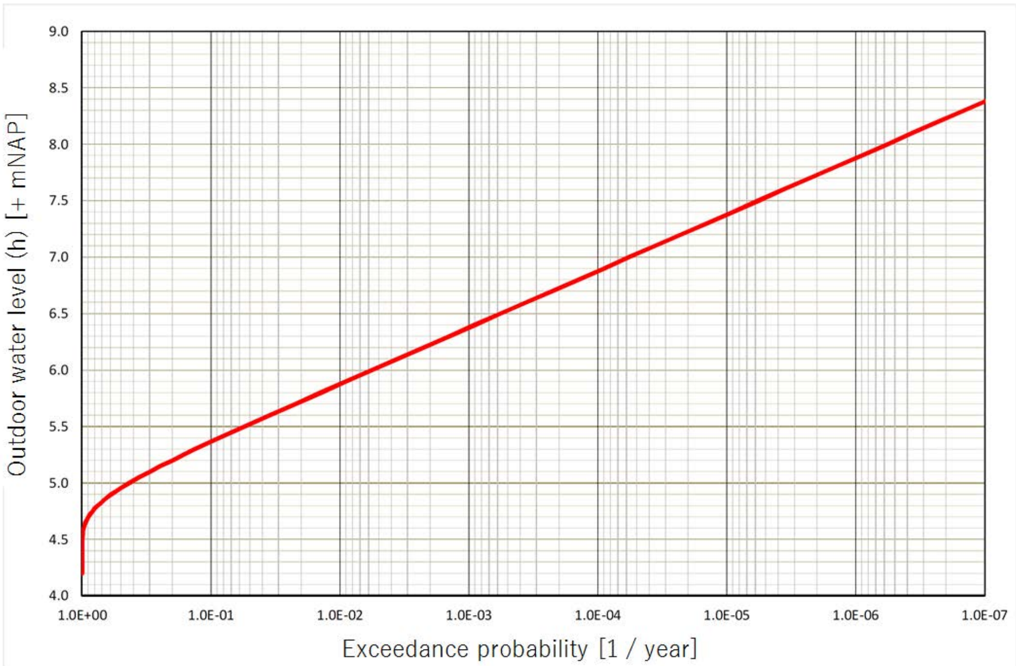
h Outside water level

u_{max} Location parameter of outside water level annual maximum

B_{max} Decimation height annual maximum

γ Constant van Euler (= 0.577)

$\mu_{h,\,max}$ Average value of the annual maximum



※（Rijkswaterstaat, Central government　2018）136 頁より作成。

Figure 40 Annual maximum water level statistics in the 50th year of life

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Water level statistics for annual minimum, for example, lock

Parameters	Values *
Outdoor water annual minimum	$\mu_{h,min}$ = NAP-0,30 m B_{min} = 0,20 m

* pertaining to annual statistics in the 50th year of life

The annual minimum is described with the help of a Gumbel distribution for minima (probability of failure):

$$F_{h;min}(h) = 1 - \exp\left(-\exp\left(\frac{2{,}3(h - u_{min})}{B_{min}}\right)\right)$$

At which:

$$u_{min} = \mu_{h,min} + \frac{\gamma \cdot B_{min}}{2{,}3} = \mu_{h,min} + 0{,}25 \cdot B_{min} = NAP - 0{,}25\ m$$

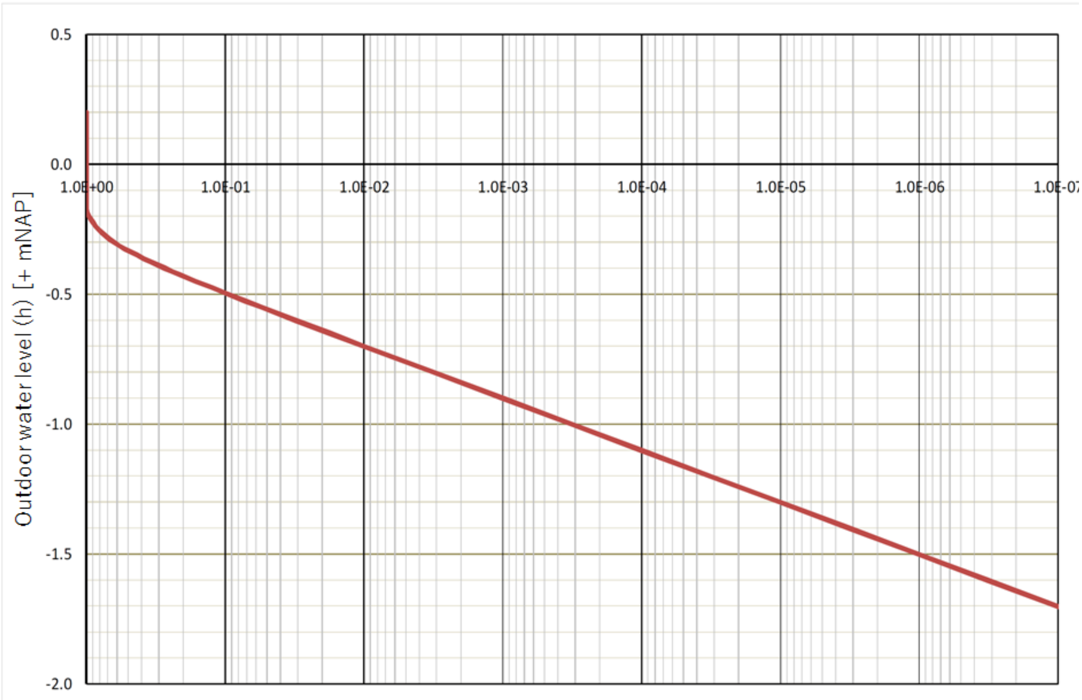
With:

u_{min} Location parameter of outside water level annual minimum

B_{min} Decimation height annual minimum

$\mu_{h,\ min}$ Average value of the annual minimum

$\mu_{h,\ min}$ Average value of the annual minimum



※（Rijkswaterstaat, Central government　2018）137 頁より作成。
Figure 41: Annual minimum water level statistics in the 50th year of life

7.10.3.2. Calculation values hydraulic load for daily situations

In water situations in daily situations here are frequently occurring load situations caused by water level differences and waves, where the outside water level is higher than the inland water level, for example during the protection process.

For flood defense works in primary flood defenses, the outside water level (and the wave conditions) are uncertain with a large spread, as in the example. In daily load situations, such as during the locking, the outside water level and the wave conditions are limited, however, with a maximum and minimum safety level in the case of locks. **(138 頁)**Since the introduction of the Eurocode in 2012, Rijkswaterstaat (Directorate-General for Public Works and Water Management) has signaled that these daily loads (which are limited by the maximum and minimum levels of the ship's level and channel level) are interpreted as variable taxes, to which the tax factors in the Eurocode are applied (for CC1: 1.35, for CC2: 1.5 and for CC3: 1.65). Such verification is unnecessary and incorrect. The daily water levels and wave conditions are part of the collection of all possible water levels and wave conditions that can occur in a year. The calculation value of the expiry tax that can occur in a year is already given in section 7.10.2. Another calculation value of the outside water level does not have to be considered.

7.10.3.3. Calculation value positive decay on inner head

The decay load on the inner head is caused by the difference between the gully and the channel level. For the calculation value of the vortex level, the probability of not closing per closing request (P_{ns}) of the outer head must be taken into account. At positive times, the calculation value of the channel level can be selected as NAP + 3.70 m.

Because the hydraulic load is dominant in relation to the own weight load, a design verification in accordance with 6.10b is sufficient (see section 7.10.2). As in section 7.10.3.2, therefore, use is made of the 'standard method' for determining the calculation value of the decay load from Figure 36. Because the probability of failure of the outer head has an influence on the calculation value of the hydraulic load on the inner head. , this situation will be discussed in detail.

Since the wave load is negligible in the example, the calculation value of the decay load is equal to the calculation value of the hydraulic load: P ($V> V_d$) = P ($S> S_d$) = P_{db}.

In addition, the channel level is regulated so that the uncertainty of the decay load is fully determined by the outside water level. This leads to: P ($h> h_d$) = P ($V> V_d$) = P ($S> S_d$) = P_{db}.

The reliability requirement associated with CC2 is a reliability index β_{eis}, = 3.8 (over the lifetime), see section 7.10.3.1. In accordance with the 'standard method' and its elaboration for the Building Decree in section 7.10.2.4, the following calculation values are recommended:

P_{db} =P ($h> h_d$)	$9{,}0 \cdot 10^{-5}$	per year
Y_G	1,0 x 1,35 = 1,35	-

※（Rijkswaterstaat, Central government　2018）138 頁より作成。

The reliability requirement from the Water Act follows from paragraph 7.7.1:

$$P_{eis,KW,CON} = \frac{P_{max} \cdot \omega_{CON} \cdot c}{N_{dsn}} = \frac{{}^{1}/_{10.000} \cdot 0{,}02 \cdot 3}{3} = 2{,}0 \cdot 10^{-6}\ per\ jaar$$

$$\beta_{\text{eis,KW,CON}} = -\Phi^{-1}\left(P_{\text{eis,KW,CON}}\right) = 4{,}6\ per\ jaar$$

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In accordance with the 'standard method' and its elaboration for the Water Act in section 7.10.2.5 (Table 13), the following calculation values are recommended, given the above requirement:

P($h>h_d$)	$7{,}0 \cdot 10^{-5}$	per year
Y_G	1,25	-

※（Rijkswaterstaat, Central government　2018）139 頁より作成。

The calculation values resulting from both reliability requirements are close to each other. In this example only those from the Water Act will be further elaborated.

For the calculation value of the vortex level, the probability of not closing per closing request (P_{ns}) of the outer head is taken into account, as follows:

$$P(h > h_d) = P_{ns} \cdot P(h > h_{d,kolk})$$

Where $h_{d,kolk}$ is the calculation value of the vortex level. In the example, the exceedance probability of the calculation value of the vortex level is equal to:

$$P\left(h > h_{d,kolk}\right) = \frac{7{,}0 \cdot 10^{-5}}{10^{-3}} = 7{,}0 \cdot 10^{-2}\ per\ jaar$$

In the example, the water level statistics are described by the Gumbel distribution in Figure 40. The calculation value of the outside water level thus follows:

$$h^*_{max} = u_{max} - B_{max} \cdot \log(7{,}0 \cdot 10^{-2})$$

$$h^*_{max} = NAP + 4{,}88m - 0{,}50m \cdot \log(7{,}0 \cdot 10^{-2}) = NAP + 5{,}5m$$

The calculation value of the decay over the inner head is therefore equal to:

$$V_{d,binnenhoofd} = h^*_{max} - h_{d,kanaal} = (NAP + 5{,}5) - (NAP + 3{,}70) = +1{,}8m$$

The model uncertainty has already been discounted in the 'standard method' in section 7.10.2, so that the load effect E_d as a result of the hydraulic load equals the calculation value of the calculated decay load.

$$E_{\mathrm{d,binnenhoofd}} = V_{d,binnenhoofd} = +1{,}8\mathrm{m}$$

It is wise to consider the situation in which the outer door has been closed successfully for the structural design of the inner door. In this case, it makes sense to make the inner door the decay, whereby the vortex level can rotate constructively at the same height as the crown height of the inner door. There is a good chance that in case of extreme conditions and a successfully closed exterior door the water level rises due to wave overtopping over the outer door. The vortex level can rise up to the crest height of the inner door, after which overflow occurs. The water level equal to the crown height can be used as a calculation value without extra safety, since the water level cannot rise.

7.10.3.4. Calculation value negative decay on the outside of the head

In the case of a negative decline, the water pressure on the outer water side is smaller than on the inland water side. In many cases, other turning means will be used to turn a large negative gradient than for a positive fall. **(140 頁)**The calculation value of the decay load is the result of a low outside water level in combination with a high vortex, with the vortex being maximized at NAP + 4.30 meters. This situation does not have to be considered from the Water Act, because the chance of the combination of the occurrence of a high water within the recovery time (or time for measures) after an extremely low water is considered negligible. There is a big difference between the risk of failure and the chance of a flood. We therefore only look at the Building Decree below. Since waves do not play a role in the example, again the calculation value of the decay load is determined at an exceedance frequency P (V> V_d) = P (S> S_d) = P_{db}. Here again, the uncertainty of the decay load is fully determined by the outside water level. Given the above, the calculation value of the decay V_d occurs at the calculation value of the outside water level h_d with an undercutting probability P (h <h_d) = P (V> V_d) = P (S> S_d) = P_{db}.

In the example it is assumed that the hydraulic load is very dominant over the own weight load, so that only the load effect according to 6.10b needs to be considered. As already elaborated in section 7.10.3.3, for the reliability requirement associated with CC2, the reliability index $\beta_{\text{eis, BB}}$ = 3.8 (over the lifetime) and the following calculation values:

P_{db} = P(h<h_d)	$9{,}0{\cdot}10^{-5}$	per year
Y_G	1,0 x 1,35 = 1,35	-

※（Rijkswaterstaat, Central government　2018）140 頁より作成。

Based on the Gumbel distribution in Figure 41, the calculation value of the external water level is as follows:

$$h^*_{min} = u_{min} + B_{min} \cdot \log(P(h < h_d))$$

$$h^*_{min} = NAP - 0{,}25m + 0{,}20m \cdot \log(9{,}0 \cdot 10^{-5}) = NAP - 1{,}1m$$

Of course, this calculation value can also simply be read in the exceedance frequency line of the outside water level. The calculation value of the decay load, expressed in meters water column, in case of negative times is equal to:

$$V_{d,negatief\ keren} = h^*_{min} - h_{d,kanaal} = (NAP - 1{,}1m) - (NAP + 4{,}3m) = -5{,}4m$$

In view of the application of the 'standard method' for determining calculation values from section 7.10.2, the calculation value of the tax effect applies again:

$$E_{\mathrm{d,negatief\ keren}} = V_{d,negatief\ keren} = -5{,}4\mathrm{m}$$

7.10.3.5. Calculation value negative decay on inner head

Negative times are based on the maximized channel level of NAP + 4.30m and a low vortex level. There will be artillery to NAP + 3.00m. As a matter of convenience it is assumed that it is certain that the outer head is closed at outside water levels below NAP + 3.00 meters (P_{ns} = 0 per question), then the calculation value of the decay load is:

$$V_{d,negatief\ keren} = h_{d,kolk} - h_{d,kanaal} = (NAP + 3{,}00m) - (NAP + 4{,}30m) = -1{,}3m$$

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Because we assume a guaranteed closure of the outer head, the uncertainty with regard to the model uncertainty is relatively large in this case and that with regard to the water level is small. The 'standard method' from paragraph 7.10.2 is therefore not suitable for this tax situation. For this reason, use is made of the method in Annex C of NEN-EN 1990 which works with the ISO-standardized influence coefficient for the load α_E = -0.7. Conservatively it is assumed that the model uncertainty is the only uncertain tax quantity, so that the full influence coefficient of the load α_E = -0.7 is applied for the model factor[35]:

$$\gamma_{S_d} = 1 - \alpha_E \cdot \beta \cdot V = 1 + 0{,}7 \cdot 3{,}8 \cdot 0{,}1 = 1{,}26$$

The total calculation value for the expiry tax follows from:

$$E_{d,negatief\ keren} = \gamma_{S_d} \cdot V_{d,negatief\ keren} = 1{,}26 \cdot -1{,}3 = -1{,}6m$$

7.10.3.6. Calculation value hydraulic load in incidental situations

Incidental situations are, for example, maintenance work and inspections, where for example a lock chamber is drained.

Major maintenance work sometimes requires the drying of the lock chamber. In the design phase of the artwork, careful thought should be given to how this should be done. It is possible to opt for special retarding agents that have been specifically designed for drying, such as bulkheads, to ensure that the sluice gates do not need to be 'unnecessarily' heavily dimensioned. It can also be decided to flood the gully (or construction pit) under extreme conditions in order to relieve the construction during an approaching high water during the maintenance work or a threatening calamity. One has to be sure that the deployment of this emergency measure will succeed. A risk analysis is also required for this. For example, by using a lower screed barrier, the chance of success of the emergency measure can be increased. In addition, restrictions on the maintenance period can also be included in the operating manual of the object, for example that the lock may only be drained outside the storm season. Extreme water levels occur less frequently outside the storm season. The statistics of the extreme outside water levels outside the storm season do not follow from Hydra-NL or other instruments, an initial estimate can be obtained by increasing the exceedance probabilities in winter statistics by an order factor (factor 10). Ideally, however, use is made of local water level data.

In the present example, it was decided to dry the lock with the normal lock gates, without restrictions at the time of maintenance. In the example it is assumed that the hydraulic load is very dominant over the own weight load so that only the load effect according to 6.10b needs to be considered (see section 7.10.2). The analysis focuses on the outside of the head and as far as possible aligns with the 'standard method' for determining the calculation values of the hydraulic loads in section 7.10.2. The method for the inner head is approximately the same as in section 7.10.3.4. In this example, it is assumed at the design stage that the lock should be drained for 1 month on average every 5 years.

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[35] In the case of CC3, γ_{sd} should be multiplied by K_{FI} = 1.1 and in the case of CC1 divided by K_{FI} = 1.1.

(142 頁) The probability of failure given drying requires a calculation with the probability distribution of the extreme water level in a random span of 1 month. Based on independence between months, the probability distribution for a year can be related to that of the monthly extremes as follows:

$$P(H > h)_{1jr} = 1 - \{1 - P(H > h)_{1mnd}\}^{12}$$

For small opportunities, the following applies:

$$P(H > h)_{1jr} = 12 \cdot P(H > h)_{1mnd}$$

And vice versa therefore applies to the probability distribution of the monthly extremes:

$$P(H > h)_{1mnd} = \frac{1}{12} \cdot P(H > h)_{1jr} \qquad 7.12$$

Applied to the example with a Gumbel distribution in Figure 40:

$$F_{h;max,1mnd}(h) = \exp\left(-\exp\left(-\frac{2,3\left(h - u_{max} + \frac{B_{max} \cdot \ln(12)}{2,3}\right)}{B_{max}}\right)\right)$$

$$u_{max;1mnd} = u_{max} - \frac{B_{max} \cdot \ln(12)}{2,3} = NAP + 4,34\ m$$

When the external water statistics are described by a data set from, for example, Hydra-NL, the same procedure as in formula 4.12 can of course be used to obtain the monthly statistics. B_{max} is the same for the statistics of the month and year extremes (the exceedance probabilities run parallel to small exceedance probabilities).

Verification according to the Water Act

The Water Act Verification is based on a failure probability for a reference period of 1 year ($P_{eis,\ KW,\ CON}$), or in each year the failure probability for the drying situation must be smaller than the failure probability. For the Water Act verification, it does not matter whether the lock is being drained once every 5 years, 10 years, 20 years, etc. The failure probability for structural failure of the lock in the example concerns:

$$P_{eis,KW,CON} = \frac{P_{max} \cdot \omega_{CON} \cdot c}{N_{dsn}} = \frac{1/10.000 \cdot 0,02 \cdot 3}{3} = 2,0 \cdot 10^{-6} per\ jaar$$

Translated into a reliability index:

$$\beta_{\text{eis,KW,CON}} = -\Phi^{-1}\left(P_{\text{eis,KW,CON}}\right) = -\Phi^{-1}(2,0 \cdot 10^{-6}) = 4,6\ per\ jaar$$

To determine the calculation value of the decay, use can now be made of the 'standard method' in Figure 36 and its quantitative elaboration in Section 7.10.2.5 (Table 13). From Figure 36 follows P ($V > V_d$) = P_{db}. In the example, the waves have been neglected.

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From Table 13 follows:

- P ($V > V_{d,droogzet}$) = P_{db} = $7.0 \cdot 10^{-5}$ per year; this is the exceedance probability of the decay load when the lock is set dry in a consecutive period of 1 year. Because the lock is dried for a maximum of one month in a year, the overflow load on drying with this exceedance is found at a water level with a probability of $7.0 \cdot 10^{-5}$ in a span of 1 month.

- $\gamma_G = 1{,}25$.

The decay load consists of the water level differential pressure as a result of the outside water level and the empty lock chamber. The bottom of the lock chamber is measured and is at NAP-3.50 meters. The uncertainty of the decay tax is therefore fully determined by the outside water level. Using the probability distribution for the monthly extremes follows at P (V> V_d) = P_{db} = $7.0 \cdot 10^{-5}$ and the calculation value of the outside water level is:

$$h^*_{max} = u_{max;1mnd} - B_{max} \cdot \log(7{,}0 \cdot 10^{-5})$$

$$h^*_{max} = NAP + 4{,}34m - 0{,}50m \cdot \log(7{,}0 \cdot 10^{-5}) = NAP + 6{,}4m$$

What results in:

$$V_{d,droogzet} = (NAP + 6{,}4m) - (NAP - 3{,}5m) = 9{,}9m$$

Since the model uncertainty has already been incorporated in the 'standard method':

$$E_{d,droogzet} = V_{d,droogzet} = 9{,}9m$$

Verification according to the Building Decree

The reliability requirement in this example is CC2, which corresponds to a reliability index $\beta_{\text{eis, BB}}$ = 3.8 over the life of 50 years, see section 7.10.3.1. This requirement can be translated into a failure probability in the following way:

$$P_{eis,BB} = \Phi(-\beta_{eis,BB}) = \Phi(-3{,}8) = 7{,}24 \cdot 10^{-5}\ in\ 50\ jaar$$

On average, the probability of failure per year must therefore be less than:

$$P_{eis,BB} = \frac{7{,}24 \cdot 10^{-5}}{50} = 1{,}5 \cdot 10^{-6}\ per\ jaar$$

On average, the construction must comply with this requirement every year with regard to the drying situation. This means that in a random year the probability of failure per year may be somewhat greater, as long as the failure probability is undershot in other years. For water-retaining constructions this is a pleasant method of verification, because the probability of failure varies considerably over the years, see the bathtub curve in Figure 31. Due to climate development, the hydraulic load and thus the probability of failure in the last years of life is greater than in the first part of the life span. An averaging of failure probabilities over the years makes the verification less stringent.

If this permissible averaging is ignored and the construction must comply with the annual probability every year, the verification is stricter, but the 'standard method' can be used in 7.10.2 (Figure 36). Because the failure probability for the lifespan from the Buildings Decree has been converted to an equivalent annual probability, use can be made of Table 13 (with Figure 37 and Figure 38) from Section 7.10.2.5. **(144 頁)**This shows the relationship between the calculation value of the decay and the failure probability per year. This relationship has been developed for verifications based on the Water Act, but in this case also for a verification based on the Building Decree.

To this end, the failure probability must be translated per year into a reliability index for a year:

$$\beta_{eis,BB} = -\Phi^{-1}(P_{eis,BB}) = -\Phi^{-1}(1{,}5 \cdot 10^{-6}) = 4{,}7\ per\ jaar$$

From Table 13 follows:

$P_{db} = P(V>V_d)$	$5{,}0 \cdot 10^{-5}$	per year
Y_G	1,28	-

※（Rijkswaterstaat, Central government　2018）144 頁より作成。

For the calculation value of the decay tax follows:

$$h^*_{max} = u_{max;1mnd} - B_{max} \cdot \log(5{,}0 \cdot 10^{-5})$$

$$h^*_{max} = NAP + 4{,}34m - 0{,}50m \cdot \log(5{,}0 \cdot 10^{-5}) = NAP + 6{,}5m$$

What results in:

$$V_{d,droogzet} = (NAP + 6{,}5m) - (NAP - 3{,}5m) = 10{,}0m$$

Since the model uncertainty has already been incorporated in the 'standard method':

$$E_{d,droogzet} = V_{d,droogzet} = 10{,}0m$$

7.10.4 Permanent taxes

Permanent taxes are taxes that do not vary or only vary in size during the reference period.

7.10.4.1. Own weight construction (parts)

The method described with regard to the permanent load by own weight in combination with a high or not dominant high water load is described in section 7.10.2.

A permanent load as a result of the own weight is taken into account with a load factor Y = 1.35 (CC2, burdensome) if the own weight is the only load source, and there is no question of a geotechnical construction or the own weight of a liquid. A factor Y = 1.2 (CC2, burdensome) may be used for the permanent load due to (permanent) liquid pressure. If the permanent load is considered in combination with other loads (other than high water, see section 7.10.2), a factor Y = 1.2 (CC2, burdensome) must be taken into account. If the load works favorably (relieved), a factor Y = 0.9 must be used in all the cases mentioned.

7.10.4.2. Ground pressure

For the permanent load due to earth pressure on geotechnical constructions such as dikes, embankments and sheet piles, where the weight of the ground occurs as a load, but also plays a role in the soil mechanical strength calculations, the load factor must be Y = 1.0 (stressful and relieving) be asked. (**145 頁**) It is important that both the final situation and the construction phases are considered.

7.10.4.3. Groundwater pressure (also increasing force)

The groundwater level near flood defense structures will generally vary considerably with the outside water level. In that case the calculation value of the groundwater pressure is derived from the calculation value of the occurring decay in accordance with paragraph 7.10.2.

7.10.4.4. Deformations of subsurface/settlement

The characteristic settlement is calculated by assuming the characteristic values for the loads and the soil properties. Locally, at least the difference in settlement must be taken into account in the order of 50% of the maximum settlement.

7.10.5 Variable loads

Variable taxes are taxes that vary substantially over time. For loads due to internal and external water levels and wind waves see 7.10.2.

7.10.5.1. Flow (including any vibrations caused thereby)

In this section the currents are treated over the artwork as a result of a decline. Flow caused by ships is dealt with in section 7.10.5.3. Flow due to a decay on the flood defense in the situation that it should have been closed (after failing closure) is regarded as a special tax. This flow load is dealt with separately in section 7.10.6.1.

The load by flow is linked to the flow rate. Flow loads can occur during the opening of the barrier, the closing of the barrier, and during high water during leakage gaps and during transfer and overflow. This concerns the load on the work of art, on soil protection as well as on gates and doors. Vibrations due to flow must be avoided as much as possible. In particular feedback vibrations (liquid - construction interaction) can lead to very rapidly increasing loads on gates and doors and on moving works. The prevention of vibrations as a result of too large an overflow rate is discussed in section 5.3.4.

The calculation value of the flow load at high water must be determined on the basis of the calculation value of the decay in accordance with paragraph 7.10.2. The calculation value of the flow load as the dominant load in the other situations is determined by combining the representative value, the flow rate that is exceeded on average once every 50 years, with a load factor of 1.3. The representative value of the combination value of the flow load is determined at the flow rate that is exceeded on average once a year.

7.10.5.2. Ship waves

Ship waves are relatively short-lived and are not expected to generate dominant loads during high water. Under standard conditions no or only very limited shipping takes place. **(146 頁)**After the characteristics of the waves caused by passing ships can be caused in the same way as for wind waves representative load on the structure. For the calculation of the wave load is referred to Annex D. The calculation value of the ship wave load is calculated after multiplication by the tax factor. In Table 11 a value of 1.3 is given for both the dominant tax as the combination tax.

7.10.5.3. Shipping current

Currents caused by ships are included in the assessment of the water retention capacity not considered because it is under normative shipping is barely involved. For normal However, in business situations, these taxes can play a major role.

Regarding currents caused by ships, there are two types distinguished, namely the return flow and the screw-jet flow. Both become expressed in terms of flow rates. For the calculation of this movements is referred to the Construction service publications Design of locks [Ref. 7.20] and the Granular Design Manual Soil defenses [Ref. 7.19]. The flow rates thus calculated can be considered as calculation values for both the dominant and the combination load.

7.10.5.4. Tensile forces.

The truss forces of ships may, via bollards, provide a tax on the water-retaining parts of works of art. If the (local) collapse of a structural component due to bollard forces the water retaining function of the can jeopardize the artwork, also serves with design inspections of bollards water safety.

The truss forces are mainly determined by the size of the ship, the water flow and wind load. CUR report 166 Sheet pile structures are for both seagoing vessels and inland navigation vessels, calculation values for bollard forces given as a function of a measure of ship size (for seagoing vessels the water displacement and for inland vessels the class of ships). Account must also be taken of the design of Rijkswaterstaat (Directorate-General for Public Works and Water Management) objects taken into account the Guidelines for Designing Artworks [Ref. 7.11].

7.10.5.5. Traffic taxes

Traffic taxes are viewed from the point of view of water safety alone relevant insofar as they can be present under high water conditions, then can lead to damage to water defenses that cannot be repaired quickly structural components or movement works. The starting point is that all occurring traffic taxes are covered in the design and under normal circumstances they therefore do not need to be verified. Supplementary must go to maintenance situations. For example, the traffic load of a crane in case of a door change of a lock.

7.10.5.6. Temperature

Temperature loads do not have to be considered when assessing the flood defense capacity, because under normative circumstances none extreme temperatures (both high and low) are to be expected.

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7.10.5.7. Wind pressures

The calculation of the representative value of wind load by wind pressure, wind suction and over- and under pressure takes place according to NEN-EN 1991-1-4.

7.10.5.8. Taxes from the movement work

Taxes with standard opening/closing movements will be surrounded with little uncertainty. Restoration measures from chapter 4 Not closing can result in certain loads under (near) high water conditions. These are highly situation-dependent and must be specified by the designer.

7.10.6 Special taxes

Disasters are events that can occur with a relatively small chance and that can directly or indirectly lead to the occurrence of special taxes. If the chance of a calamity is sufficiently small (smaller than the failure probability for structural failure of the artwork), these special taxes do not have to be explicitly taken into account. When rare taxes can be covered without or at a small additional cost, then that is obviously to be considered.

7.10.6.1. Flow in case of non-closing

In the event of failure of a high water seal, high flow rates through the artwork can lead to damage to the soil protection and thus ultimately to undermining and the structural failure of the artwork. If the design has been verified against the failure probability for not closing in accordance with Chapter 4, this load situation is correctly given in the correct manner.

7.10.6.2. Accept

Hydraulic constructions can be hit by ships where parts of the structure collapse. For the flood risk, specifically the acceptance situation on water-retaining construction components is of importance, as a result of the collision a flooding can occur. A flood can occur if a high water occurs within the recovery time of damage, but also when no high water is present and the hinterland is a deep polder.

The chance of accepting water retaining structures during or just before a high water level is very small because waterways are usually blocked in time. In some cases, however, lock locks at higher water may still be in operation. When the second turning means is hit during high water, a flood can occur.

During the verification, the chance of attacking P (accepting) is combined with the chance of constructive failure given an acceptance tax P (Settling | Accepting). In the case of a verification based on the Water Act, the likelihood of exceeding the maximum permissible inflowing volume of water, either from the compilation criterion or from the soil protection criterion, after the collapse of water-retaining components P (Vin> Vmax | Succession) be taken away. So there must be:

$$P(Overstromen\ en\ Aanvaren) =$$
$$P(Aanvaren)\ \cdot P(Bezwijken \mid Aanvaren)\ \cdot\ P(V_{in} > V_{max}\ |Bezwijken)\ \leq\ P_{f,CON,WW}$$

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If the chance of a collision is already smaller than the constructive reliability requirement, the construction meets the requirement immediately.

The parameters in the above formula are a function of many stochastic variables, such as the sailing speed, angle of attack, mass and dimensions of the ship, etc. The arrangement of the outer port and lock chamber is an important precondition for the chance of acceptance. The acceptance tax is therefore unique for every situation. This should therefore be considered tailor-made, with location-specific data. A generic method for determining the calculation value of the acceptance tax is not available.

Where a real acceptance risk is expected, the solution can also be sought in measures to reduce the chance of (serious) collision damage. One can think of another establishment of the outer harbor so that ships are forced to lower sailing speeds or to create an acceptance protection.

For additional information, please refer to NEN-EN 1991-7 with national appendix, Guidelines for Design Artworks (ROK) [Ref. 7.11], Recommendations of the Committee for Waterfront Structures Harbors and Waterways - EAU 2012 [Ref. 7.13], Manual for the quantitative determination of the acceptance risk of movable objects in the waterway (part of the PRA, RWS) [Ref. 7.14] and Handbook Design of locks [Ref. 7.20].

7.10.6.3. Earthquake

For guidelines for the determination of earthquake loads, use must be made of the latest state of knowledge in this area.

7.10.6.4. Explosion

Explosion safety of operating rooms and the like can be verified according to Eurocode EN 1991-1-7. The load caused by explosions of ships with hazardous substances is normally neglected.

7.10.6.5. Ice

In every design situation an estimate of the possibility of an ice load has to be made. Ice-load is not explicitly mentioned in the Buildings Decree, but here, the design of locks and similar constructions must be taken into account. Indicative calculation values can be found in CUR Report 166 Sheet pile constructions [Ref. 7.10]. In the Guidelines for the Design of Artworks (ROK) [Ref. 7.11], further draft recommendations for works of art by Rijkswaterstaat (Directorate-General for Public Works and Water Management) are given.

7.10.6.6. Obstacles during closing

If an obstacle prevents the movement of the door, this can give rise to tensions in the construction that deviate strongly from the normal (extreme) loads. This tax should be considered in relation to the movement work. From the point of view of the Water Act, it must be assessed whether this does not give rise to long-term damage, non-availability and possibly a flooding.

7.10.6.7. Vandalism and sabotage

Attention should be paid to measures against vandalism and sabotage.

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7.11 Strength

7.11.1 Calculation value strength construction

As indicated in section 7.8.4.2, the calculation value of the strength R_d from formulas 6.6a to 6.6c in NEN-EN 1990 follows. The partial material factors (Y_R)

that are used in this process follow from the material-bound Eurocodes (NEN-EN 1990 t / m 1997) and in some cases from other directives. An overview is given below.

Table 16 Overview of the most important construction elements and mechanisms (excluding movement works)

Part of hydraulic structure	Mechanism of limit state	Prescription/guideline
Turning means, moving works and superstructure		
-concrete	strength/stability	NEN-EN 1990-1991-1992
-steel	strength/stability	NEN-EN 1990-1991-1993
-wood	strength/stability	NEN-EN 1990-1991-1995
-brickwork	strength/stability	NEN-EN 1990-1991-1996
Foundation and subsurface		
-construction	strength/stability	NEN-EN 1990-1997
-steel foundation	strength/deformation	NEN-EN 1990-1997
-pile foundation	strength/deformation	NEN-EN 1990-1997
-ground body	stability/deformation	NEN-EN 1990-1997/Guideline River dikes
-dam wall	strength/stability	NEN-EN 1990-1991- CUR report 166 [Ref. 7.10]
- retaining wall	strength/stability	NEN-EN 1990-1991- CUR report 166 [Ref.7.10]
Edge constructions		
-well screens	strength	CUR report 166 [Ref. 7.10]
-soil protection	wash away	Rock manual, Scour manual
-filters/membranes		Internal RWS documents

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7.11.2　Background of the partial material factor

The partial material factors are partly based on probabilistic considerations and partly on historical or empirical. For materials such as steel and steel concrete, extensive background documents exist at European level with a description of test results and a related statistical elaboration. For other materials, including soil, the link between the intended and proven safety is often less strong. For each material and each load (combination) there is the possibility to calibrate the specific partial factors or coefficients, in order to achieve the required level of reliability. However, this requires specific expertise in this area.

7.11.3 Degeneration

When designing, one should take into account aging and damage, such as:

- Steel: fatigue and corrosion
- Concrete: damage, carbonation, chloride penetration, alkali-silica reaction, freezing/thawing, et cetera
- Wood: fatigue, creep, deterioration

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Under aging means mechanisms that slowly reduce the strength. On balance, the reliability also decreases. Because the artwork must still meet the reliability requirements at the end of its design life span, the design must be taken into account especially in the case of hard-to-replace steel parts.

Common methods to prevent, inhibit or obviate aging include preservation (anti-corrosive coating), cathodic protection (steel) and dimensioning with a certain over-thickness. In the case of steel, the corrosion rate can vary greatly depending on the environment and steel quality. A first indication can be derived from CUR report 166 sheet pile constructions [Ref. 7.10]. For Rijkswaterstaat (Directorate-General for Public Works and Water Management) objects are in the Guidelines for Designing Artworks (ROK) [Ref. 7.11] included even greater corrosion surcharges.

The decrease in reliability due to the aging mechanisms can be reduced by inspection and maintenance. If this pays off, maintenance can also be organized on a probabilistic basis. One must then search for state parameters (or guide parameters). These are measurable construction parameters that describe the condition of a flood defense. During the lifespan of the construction, further information can be obtained about these variables by inspection. The reliability index can then be adjusted via Bayesian processing of this data in a reliability analysis. Based on this, it is then possible to decide on maintenance measures.

7.11.4 Fatigue

Fatigue can occur if a load, such as pressure, tension, bending, twisting or combinations thereof, occurs repeatedly. In general, a number of tax changes that are roughly between a thousand and a few millions in life expectancy should be considered. In hydraulic structures, fatigue damage can occur, especially in doors and gates, and in connection with the movement works thereof. This is due to loads caused by wind waves or vibrations through the flowing water. Loads by wind waves can be limited by, for example, the screening of an outer harbor. Loads caused by wave impacts can be prevented by the design of the structure, for example by avoiding angles enclosed with a sliding construction. Vibration by flow can be prevented as much as possible by, for example, choosing the geometry of the sliding edges correctly. Further information can be found in, for example, Dynamic behavior of hydraulic constructions [Ref. 7.9].

In order to assess whether or not a construction will collapse under a variable load, the following aspects are important:

- The nature of tax changes. For example, a construction generally better withstands a variable load if the load direction does not change sign (a so-called jump load) than if it does (a varying load). Incidentally, this does not apply to steel constructions;
- The number of tax changes. Fatigue damage can occur in small numbers of changes (≥ 1000), but this usually does not immediately lead to failure. Furthermore, it appears that if a construction can withstand several million load changes, there is no risk that the construction will collapse purely due to fatigue if the conditions remain the same;
- The amplitude of the load changes: the smaller the amplitude, the smaller the risk of fatigue;

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- The material and the design of the structure. The design and the surface condition can give rise to stress concentrations. Sharp cross-section transitions in constructions, certain types of welds etc. are places where the construction will collapse as a result of fatigue.

In 2017 the version of NEN 6786 part 1 was released which gives the requirements and preconditions with regard to fatigue for movable bridges. This standard can also be used for the design of hydraulic constructions, together with NEN-EN1993 parts 1-9 and the additional ROK provisions [Ref. 7.11].

In addition, the design must also be verified in accordance with the Water Act, whereby the reliability verification must take place in the last year of life. NEN 6786 Part 2 will be released in the near future and will be written specifically for hydraulic constructions.

7.12 Concrete design recommendations

The foundations and concrete structures of hydraulic constructions are generally designed for a planned lifetime of 100 years. Because in the past robust design has been developed, many constructions that are already much older function constructively just fine. In general, the costs of more robust dimensioning are then strictly required for small and medium-sized structures, for which this manual is written, limited compared to the earlier replacement. For structural components such as steel doors or movement works, however, a design life that is considerably shorter than 100 years can be optimal.

At the moment there is a lot of uncertainty about the way in which the climate will develop and with which seawater rise and extreme discharges we have to take into account in the future. In addition, safety standards are updated periodically, such as recently with the introduction of the new Water Act (2017) and the new Building Decree (2012), which in most cases led to more stringent constructive requirements. The use of objects can also change. In this way ships are becoming larger and the sailing intensity of waterways changes over time, so larger dimensions or more efficient systems may be needed.

It is recommended to design on the basis of a life-cycle approach, taking into account as much as possible these future developments. Because there is a long time horizon, the changes in load, strength and use are often difficult to estimate. It therefore seems sensible to replace or strengthen components that are not easy to use, such as foundation, sluice floor and some superstructure components, to make robust choices and to make construction components larger and stronger than at present, with current knowledge is considered strictly necessary as long as it does not require disproportionate investments.

7.13 References

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8 Connection constructions

8.1 Introduction

8.1.1 Purpose and definitions

The purpose of a connection construction is to prevent external erosion at the interface of artwork and subsequent ground body as a result of wave and flow loads. This will prevent the stability of the flood defense at the location of the connection between the artwork and the ground body will be reduced, possibly resulting in a progressive breach and flood consequences.

External erosion at the location of the connection of the artwork and the adjacent ground body can occur on the outer slope, the crown and the inner slope. In principle, the possible failure mechanisms are equal to the dike failure mechanisms, grass erosion crown and inner slope and all dike failure mechanisms with respect to (outer slope) coatings.

The connection construction is understood to mean the entire transverse and longitudinal profile of a ground construction in its deviating shape, at the transition from the work of art to the undisturbed dike. Figure 42 shows the most important components.

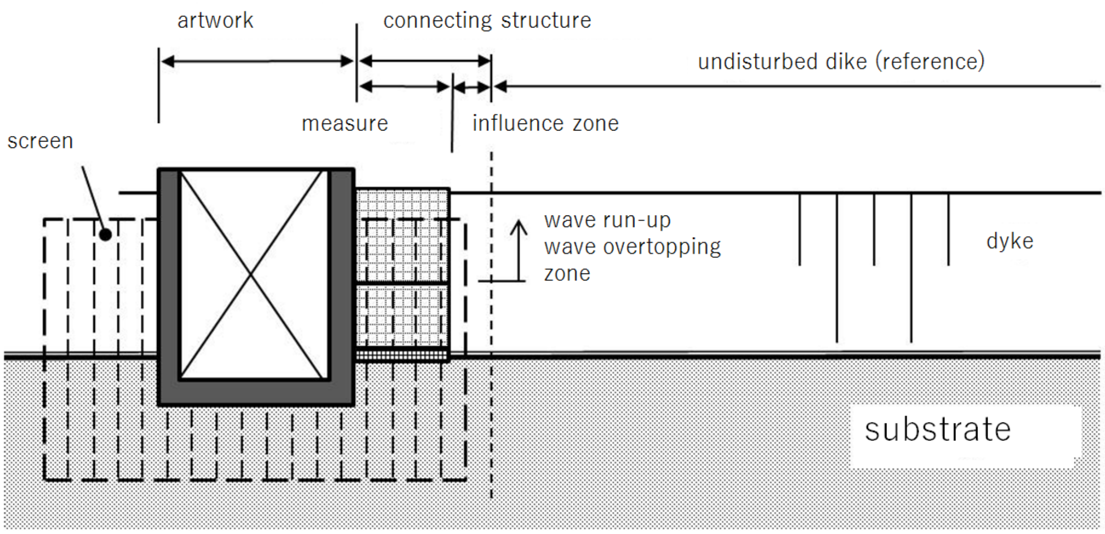

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Figure 42 Principle diagram of connection construction

The connection construction is a physical constructive measure. It can contain visible parts and invisible parts, for example a cover and a screen. The impact zone to the adjoining dike cladding (for example a grass cladding), where the effects of the physical measure on the subsequent cladding are still significant, also belongs to the connection construction.

8.1.2 Delimitation

For the safety verification the higher connection constructions are important, which are necessary for the proper functioning as high water barriers at high outside water levels over the flood defense and / or high waves. These are usually above water in normal conditions, in the wave run-up zone and in the wave transfer zone. Figure 43 gives some examples.

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Lower connection constructions (often under water or in the wave impact zone with more or less daily loads) usually do not have a high-water function and / or are not subject to significant stress during high water. However, care must be taken to ensure that damage that has occurred during high water cannot be aggravated and lead to flooding.

In addition to connecting structures between the dike and the artwork, the transitional constructions between the artwork and the front or bottom of the soil are also important. This concerns, for example, the transition construction from a lock floor to the soil protection structure, or from the soil protection structure to the unprotected soil. Such transitional constructions (soil protection constructions) are discussed in chapter 9 Soil protection.

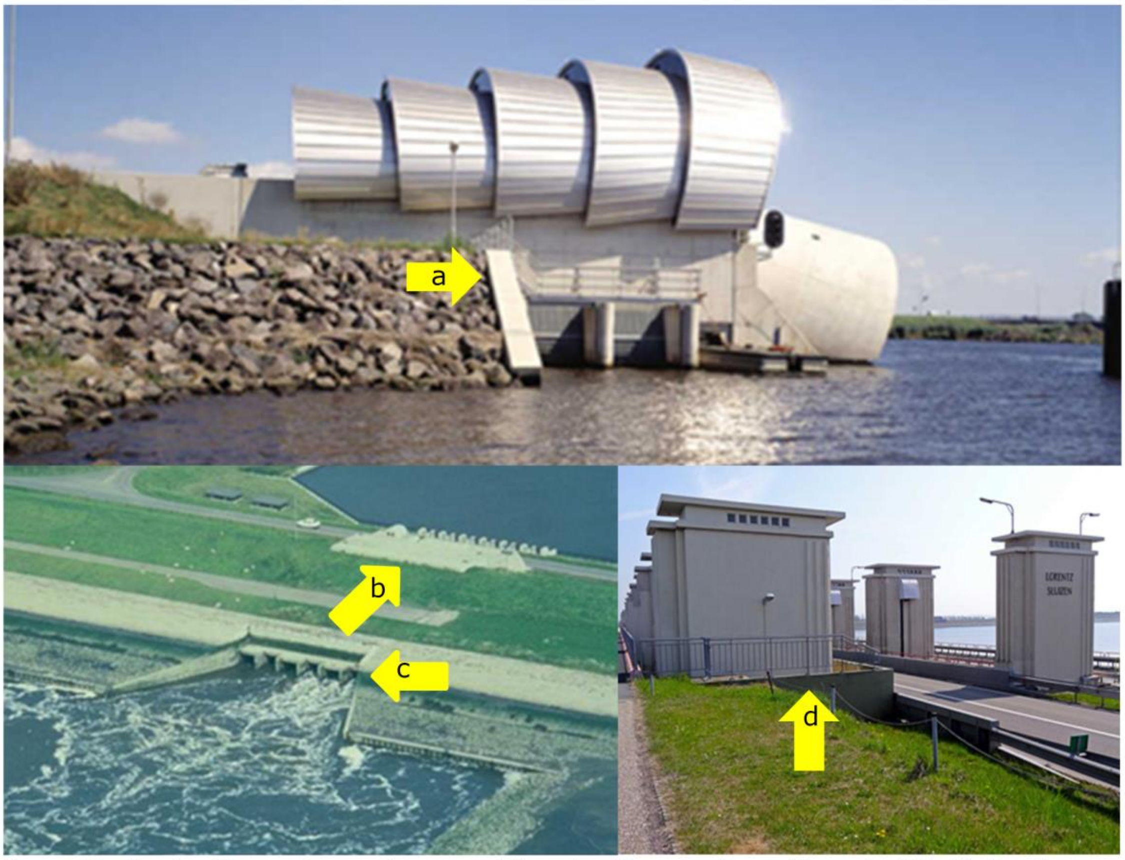

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Figure 43 Examples of high-lying connection structures to the adjacent dike cladding (source: Guide to connection constructions ([Ref 8.1])):
a - Connection wall construction and stone covering bellows Ramspol.
b - Connection to the top wall protruding wall construction, sluice at Bath.
c - Connection of wing wall to slope covering, sluice at Bath.
d - Connection with height difference around artwork, Lorentz locks.

The connection of the dike body to non-water retaining objects, such as stairs, is not treated in this manual.

8.2 Requirements

A connection construction is part of the dyke project and therefore has a flood probability contribution. If use is made of the fixed failure probability budget (Table 1) for assigning failure probability to failure mechanism, however, no failure probability space for connection constructions appears to be defined. As explained in section 8.1.1, connection constructions fail according to the same failure mechanisms with regard to external erosion as normal ground dikes, only in many cases the geometry is different from ground dikes. (**157 頁**) As a result, the failure mechanisms of connection constructions can be accommodated in the mechanisms - grass, erosion, crown and inner slope and all dike failure mechanics with respect to (outer slope) coatings - and the corresponding failure probability.

The reliability requirement from the Buildings Decree can also be relevant for connection constructions when it concerns construction components or a retaining structure.

For the design verification of a connection construction to an imposed failure probability, the schematization of the limit state function (test or design rules) must also be known, or the relation between the strength (R) of the connection construction and the occurring hydraulic load[36] (S). Because of the many possible types of connection constructions and lack of knowledge and research with regard to these constructions, no default limit state functions are available, as is the case in structural mechanics.

In practice, a connection construction is not subject to a failure probability from the Water Act but a relative requirement is used:

> A connecting structure must be designed, dimensioned, executed and maintained in such a way that it is not attenuated during the intended planning period with respect to the adjacent dike body, assuming that this adjacent dike body at least meets the applicable requirements during the planning period.

The failure definition is as follows:

> The initiation of the collapse process by local external erosion of the outer contour of the dyke body, as a result of which (progressive) breach formation occurs leading to substantial damage and / or casualties (flooding). This breach can consist of the instability of the artwork, the washing away of dyke material or a combination of these two.

The above definition means that there must be such an erosion of the outer contour that this may lead to flooding consequences. The beginning of failure can be seen as the first erosion of the connection (for example eroding lining at the location of the connection), after which actual failure occurs if this leads to further erosion of the dike or the instability of the work of art. This means that the positive contribution to the strength of a (back-walk) screen can be included in the considerations.

Incidentally, it should be noted that erosion of the connection construction is something other than erosion as a result of backwardness (internal erosion), in which erosion is initiated in the dike body.

8.3 Schematisation

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[36] Indicated as F in NEN-EN1990.

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8.3.1 Conflict processes

Connections between ground dikes and rigid structures such as structures are vulnerable to external erosion processes, caused by locally increased transhipment rates, irregular geometries and the local effects of very turbulent flow. With connection constructions, there is often a question of:

- Surface erosion and gully formation as a result of run-off surface water that concentrates on the interface between the structure and the dike.
- Surface erosion at the connection due to local turbulent flow caused by the geometry of the structure or by roughness differences.

Initial damage to connection structures can lead to higher loads and thereby initiate a self-accelerating damage process, as a result of which connection structures are maintenance and failure-sensitive. For example, due to insufficient compaction during execution or by entry by sheep, height differences between artwork and adjacent soil body can occur, as a result of which wave run-up and overtopping will concentrate, causing stronger erosion. The erosion holes cause more turbulence to occur, as a result of which the erosion increases further, et cetera.

8.3.2 Design verification

The robustness of the design of a connecting structure can be determined by a relative comparison of load (S) and strength (R) with respect to local external erosion of the outer contour with the load and strength of a reference cross-section of the dike body, provided that this reference cross-section at least meets the design requirements. In this context reference is made to the cross section of the basic body outside the area of influence of the artwork (see Figure 42). As long as the relative increase in load at the location of the connection construction (DS) is less than or equal to the relative strength increase of the connection construction (DR), the connection construction is sufficient.

The approach for the robustness determination of a connection construction contains the following two components:

A. Effects that are the direct result of the design, being the design and dimensioning of the connection construction (also called primary effects).
B. Effects entailing the indirect consequences of execution, management and maintenance as a result of the detailing (also referred to as secondary or long-term effects).

Both effects affect both strength and taxes. The theoretical approach for design verification is shown in the diagram in Figure 44.

It should be noted that the theoretical verification models are missing for substantiation of design choices. From that point of view, expert judgment must then be used. Use can be made here of experiences with existing connection constructions.

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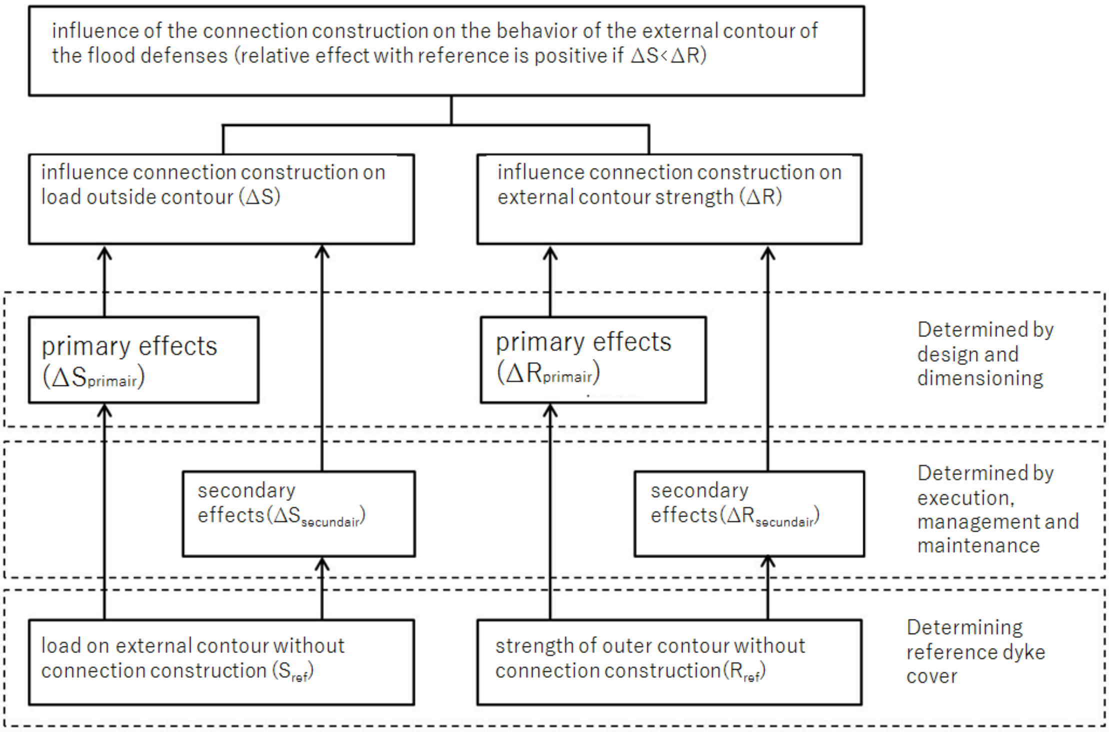

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Figure 44 Conceptual approach of robustness determination of connection construction. Based on [Ref. 8.1]

Explanation:

- **Design**

  For the verification of the connection construction, both the transverse profile and the longitudinal profile are important, including the impact zone to the adjacent cross-sections of the dike covering. Points of attention:

  - Provide sufficiently detailed detailing of the connection construction, including the transition to the work of art and to the adjacent dike covering.
  - Provide clear basic principles and requirements for implementation, management and maintenance.

- **Finding definition**

  The initiation of the collapse process by local external erosion of the outer contour, as a result of which further breaching will occur. This breach can consist of the instability of the artwork, the washing away of dyke material or a combination of these two. Points of attention:

- Check whether this approach can be applied here. Be alert to other or additional failure mechanisms that can occur at transitions caused by water-related loads or secondary loads.
- Consider possible additional effects due to concentrated groundwater flow and / or internal erosion.

- **Reference**

  The reference is the adjacent dike construction (including coating), located outside the impact zone of the connecting structure. Points of attention:

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- Provide a clear, well-founded choice for the reference profile and the reference situation.
- Find out what the dominant design load is at the location of the reference profile.
- Determine in advance what is taken as reference strength: the theoretical design strength or, in case of excess strength, the actual strength.

- **Adjustment of taxes by primary effects ($\Delta S_{primary}$)**

  The overall design of the artwork is often of great influence on the loads on connection constructions. Due to an unfavorable design of the discontinuity, considerable load-increasing effects can occur. Conversely, due to a favorable design, the load on the connecting structure can also be considerably reduced. Points of attention:

- Check how the load is reduced or with which tax increase must be taken into account in relation to the reference.
- Take care when kinking in the slope (for example, the connection to a vertical wall). The load here is both high due to wave run-up and the concentration of the power attack at the interface (jet erosion).
- Simple analytical methods to determine the increase in flow rates at connections are missing and the current design guidelines are mainly based on practical experience and field observations. Simple approaches without waves already show that the flow rates in the zone next to a vertical wall can almost be doubled ([Ref 8.4]). The waves also make the

flow pattern even more complex. Due to this complexity theoretically in many cases advanced modeling is necessary to determine the occurring flow velocities and the stability of the connection construction. In practical terms, it means that the strength of the protective structures in the zone next to the structure must often be considerably greater than the reference strength of the lining on the adjacent dike.

- **Adjustment of strength due to primary effects ($\Delta R_{primary}$)**
  The primary effect of the connection construction on the strength of the outer contour is caused by the interruption of the coating next to the structure. In terms of strength, the starting point in the design practice is that the strength of the solution must be greater than the strength of the outer contour that is interrupted. Points of attention:
  - Avoid geometrical discontinuities, such as sudden height changes.
  - Avoid discontinuities in properties, such as sudden roughness / or permeability transitions.

- **Adaptation of loads by secondary effects ($\Delta S_{secundair}$)**
  A faulty execution or poor maintenance can lead to secondary effects on the load on site of a discontinuity. Points of attention:
  - Ensure that turbulence is prevented by settling differences in linings, by making the heights close to each other and sufficiently compacting the substrate.
  - Provide a durable sand-tight connection. Lack of or premature failure can, for example, cause extra, increased pressure of exiting groundwater over time.

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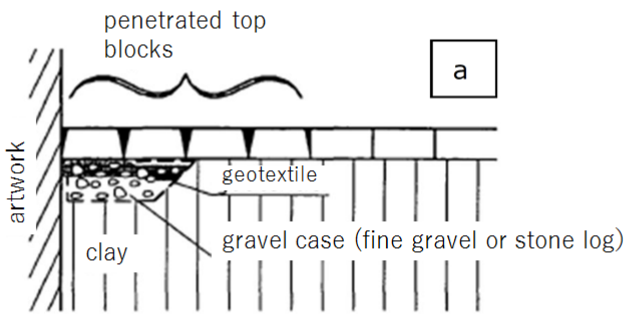
- **Adjustment of strength due to secondary effects ($\Delta R_{secundair}$)**
  Interrupting the outer contour with a connecting construction can indirectly have additional negative effects on the strength of the outer contour, for example through long-term effects such as settlement. This is largely determined by the detailing. Points of attention:
  - Take into account long-term effects, such as settlement. Ensure that the connection construction can follow the resulting difference, without loss of connection.

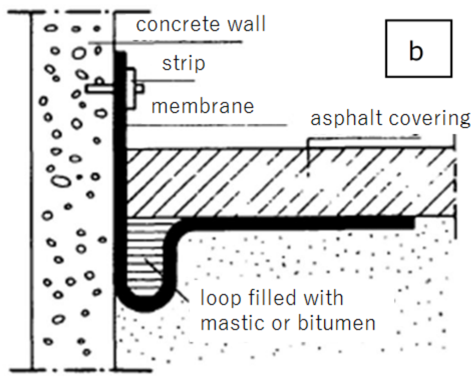
– Pay sufficient attention to the implementation. Prevent poorly compacted clay, insufficient clamping or other imperfections during construction, by proper execution instructions and inspection.
– Consider possible complications for management and maintenance. For example, the grass mat next to the connecting structure is more difficult to reach for mechanical mowers, so that the compaction of the top layer will remain less good in the long term.
– Zones in the shelter of objects are often attractive to animals, so that weakening can occur through grasslands (mice, moles) or through more intensive access (sheep).
– View the possibilities to respond to any adverse effects of the design in the design. For this it is necessary to know the way of implementation.

- **Termination or influence zone**
  The termination is formed by the influence zone. The location and design requires separate attention. Points of attention:
  – Ensure that the transition between connecting structure and undisturbed dike body is carried out carefully, so that this cannot lead to a weak spot at the location of this transition.
  – Carefully consider how this can become a weak spot when designing this transition.

8.4 Principle solutions

Figure 45 gives an overview of some available principle solutions. See below [Ref. 8.1]. There are no examples available where the robustness has been determined in a concrete application.





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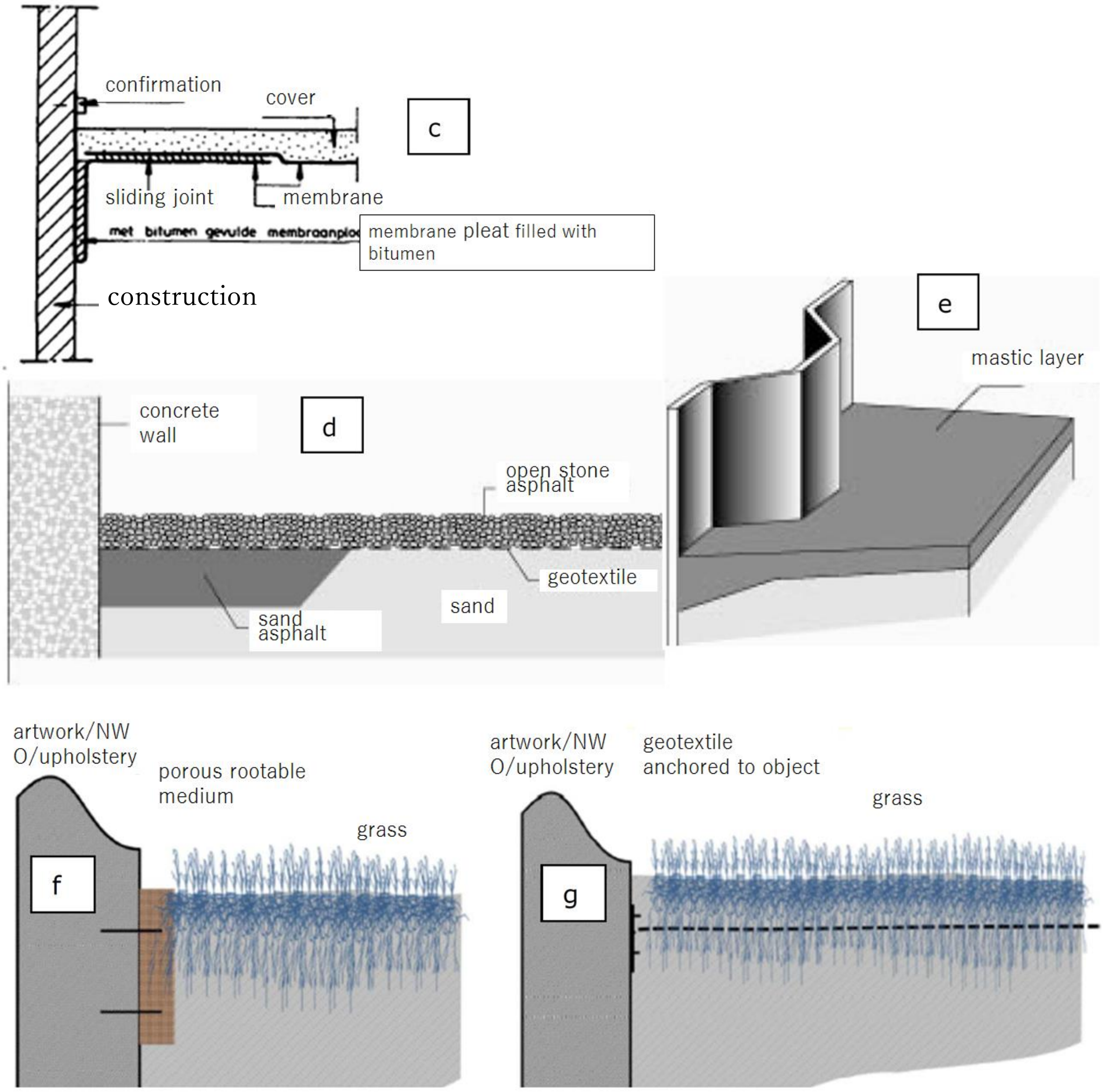

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Figure 45 Examples available principle solutions for connection constructions (source: [Ref 8.1]):

a - on brick set, penetrated tapered blocks on geotextile and gravel suitcase

b - on asphalt cladding, membrane loop filled with mastic or bitumen

c - on upholstery, with bitumen-filled membrane fold

d - on open stone asphalt cladding, geotextile on sand asphalt

e - from concrete wall or steel sheet pile on asphalt cladding, mastic padding

f - concrete wall on grass cladding, anchored porous permeable medium

g - concrete wall on grass cladding, anchored geotextile

8.5 References

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9 Soil protection

9.1 Introduction

A bottom defense protects the soil against erosion and is located on both the outside and the inland side of the hard construction. In relation to high water safety, the soil protection is taxed in the event of flow through the (unjustified) barrier (see chapter 4 Do not close) or the closed barrier (see chapter 5 Height). Soil erosion on the inside of the artwork can ultimately lead to the undermining of the artwork. In order to guarantee the stability of the artwork within the lifetime and given a permissible failure probability, any erosion must take place sufficiently far from the structure or be prevented entirely. In order to realize this, soil protection is being constructed in practice which must meet a number of requirements.

The most important constructive requirements for soil protection relate to:

- Stability (see section 9.4)
- Horizontal dimensions (see section 9.5.2)
- Vertical structure (see section 9.5.3)
- Flexibility in relation to transitions and connections (see section 9.5.4).

Other requirements of less constructive nature are for example:

- Inspection and maintenance requirements
- Environmental requirements
- Implementation requirements

In this chapter, the focus is on the constructive aspects. It should be noted that in a design verification to the Water Act requirement, only the soil protection on the inside of the structure is considered to be under the specific load due to a high outside water level and/or high wind waves. Other taxes, however, are often dominant and usually determine the dimensions of soil protection. However, this falls outside the scope of this manual; for this reference is made to other design documents. In this Work Guide the soil protection is considered as a given; the required dimensions are determined on the basis of the requirements from other functions of the artwork than water-retaining.

9.2 Types of soil protection

In practice, a multitude of types of soil protection and materials are used. The table below gives an overview of the most common types of soil protection.

Table 17 Overview of various common types of soil protection

Type of soil protection	**Material type (most common)**	**Calculation method**
Granular (whether or not on geotextile)	Rubble stone	Shields, extensive Shields for rays Pilarczyk Escarameia and May
	Snails (steel snails, phosphorescent slag)	
	Gravel	
Coherent	Block mats	Pilarczyk Escarameia and May
	Open stone asphalt mats	No model available; model research necessary per application
	Granular material penetrated with asphalt or colloidal concrete	
	Asphalt mastic slabs	
Stone structures	Concrete pillars	Pilarczyk Escarameia and May
	Basalt columns	
Composites	Gabions	Pilarczyk Escarameia and May
	Stone mattresses	

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Different types of soil protection are often calculated with the same stability formula, whereby the type of soil protection is modeled by a value associated with that soil protection for a specific parameter in this formula. There are several stability formulas, the outcomes of which are not always in line with each other. This has partly to do with the sensitivity of the turbulence factor in the different formulas, but can also be the result of possible extrapolation (or the values to be filled in of the different coefficients do not correspond to the situations that have been tested and for which the formula is developed). The final choice for a stability formula is therefore often based on 'engineering judgment' in practice. It should be noted that with numerical models better quantitative information can be derived for flow rates and turbulence intensities and thus more certainty can be obtained. However, this requires a substantial effort.

9.3 Taxes

9.3.1 Causes

The soil defense can be charged by various causes, intentionally or unintentionally. The tax may come from, for example, flow due to:

- Filling/emptying or draining
- Non-closing doors/valves
- Overflow

They can also come from ships as a result of

- (Bow) Screw jets
- Ship waves and return flow

And they can come from wind waves, where the combination of simultaneous occurrence of current and waves can be decisive.

For a design verification to the Water Act requirement, only the load due to a high outside water level and/or high wind waves is important. In the remainder of this section, therefore, only this tax situation will be discussed.

9.3.2 Types of tax

The stability of the top layer of soil protection is in principle calculated on a load as a result of the following three load cases:

1. Flow
2. Wave load, with or without flow
3. Overflow/splash load

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1 Load by flow

The load due to currents is often not uniform, but can be better referred to as radius. Depending on the mechanism, different types of beam forms can occur:

- Flat rays

  The flat beams are created where, for example, doorways do not close (well). The beam is wide and therefore has a large surface area. This kind of radiation breaks off relatively easily because the beam makes contact with the surrounding water over a larger surface area.

- Rectangular beams

  Rectangular beams occur when leveling locks as a result of the square openings in the doors. Usually there are several adjacent openings. This allows the beams to be drawn towards each other and will be less easily broken down by the lesser presence of surrounding (standing) water.

- Round beams

  Round beams are generated by (bow) screws and are accompanied by high turbulence intensity. The round beam has the least surface that is adjacent to the surrounding water and is therefore the least degraded.

The type of jet, and the degree of turbulence ultimately determines to what extent a certain load can be felt behind the artwork. This is important for the length of the soil protection. For a design verification to the Water Act requirement, flow due to transfer/overflow and decay flows due to the (unintentionally) opened artwork is important. These are generally flat rays.

The load on soil protection by flow can be both pulsating and continuous. A pulsating load occurs when waves (partly) play a role in the flow over or through the artwork. Two load cases can be distinguished here:
A. Waves result in an extra volume of inflowing water with an opened artwork
B. Waves lead (partly) to an overflowing ray behind the closed work of art.

Ad A) In an open work of art, waves usually hardly contribute to the inflowing flow because the inflowing flow due to the decline over the work of art is dominant. Nevertheless, situations are conceivable in which waves do contribute to the inflowing flow. Think of a situation with a relatively small water depth above the threshold in combination with large waves. In that case the load has a strong pulsating character.

Ad B) This situation is further elaborated below under 3.

To calculate the flow rate, numerical models can be used. Although often focused specifically on a type of artwork, there are also available formulas to determine the flow rate. In section 16.4 of the Manual for Designing Granular Soil Defenses [Ref. 9.8] some examples are given.

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2 Load by waves

The soil protection for flood defense structures is usually located below the waterline and also behind the work of art. As a result, the soil protection is not directly attacked by waves. As a result, taxes due to waves play a limited role and are included under 1) and 3).

3 Overflow / splash load

A deviating flow pattern occurs as a result of transhipment and / or overflow over a closed work of art. In that case the load consists of an overflowing beam over the reversing means / construction of the artwork, which ends up in water of limited depth. Often this takes place within the contours of the concrete work of the construction, after which the overflowing jet spreads over the wet surface of the artwork. At the location of the soil protection, the soil protection is then still taxed by a flow load.

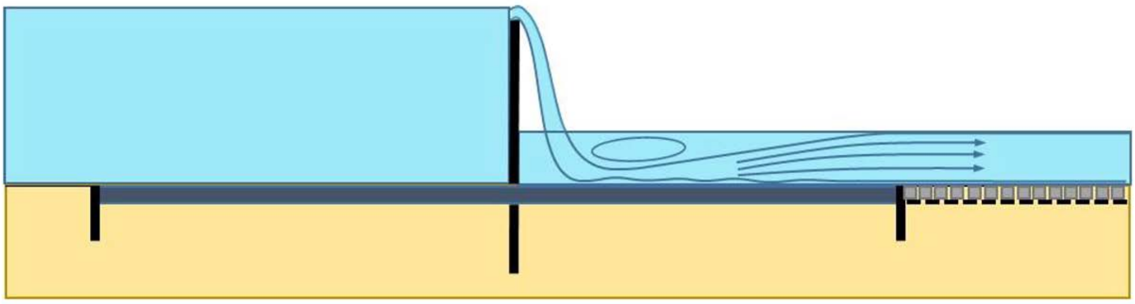

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Figure 46 Overflowing beam within contours of concrete work

It is also possible that the overflowing jet directly charges the soil protection. Consider, for example, transfer and / or overflow over a denomination. This is discussed in more detail in section 9.4.2.

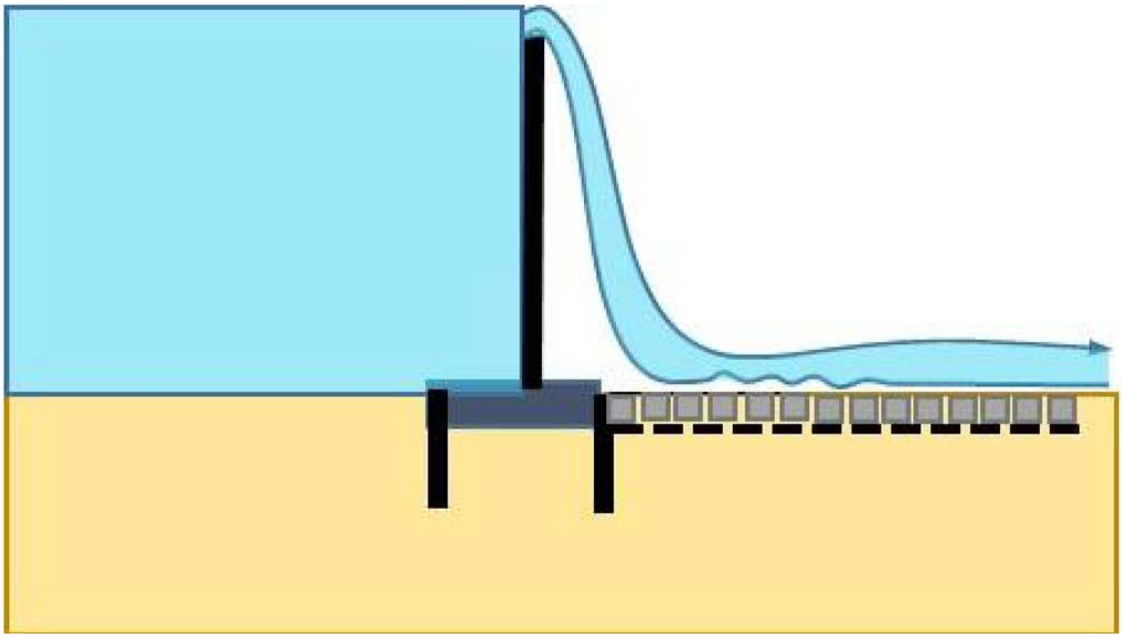

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Figure 47 Overflow jet directly on soil protection

Wave transfer by definition leads to a strong pulsating load on the soil protection behind the artwork. Because no specific relations are known that distinguish between pulsating flow and continuous flow, the stability relationships presented in the next section relate to soil protection structures that are continuously loaded.

9.4 Strength

The strength of the soil protection is determined by the underwater weight of the elements that are used in soil protection. This underwater weight is determined by the size of the elements (usually expressed as D) and their specific gravity (often expressed as Δ being the specific gravity of the element minus the specific gravity of the water divided by the specific gravity of the water). (**169 頁**) Various stability relationships are available for determining the required combination of specific gravity Δ and element dimension D. In this chapter, the following models are discussed; all applicable for a situation with flow load:

- Shields, extended (applicable for granular soil protection)
- Shields for blasting (applicable for granular soil protection)
- Pilarczyk (applicable for almost all types of soil protection, see Table 17)

- Escarameia and May (applicable to almost all types of soil protection, see Table 17)

Although Izbash's formula is also often mentioned, it is not included because this formula only provides an indication and can be made a better calculation with the help of the other theories.

9.4.1 Stability relationships for flow load

The models are presented in the classic way to calculate suitable values for Δ and D at given flow velocities. In this guideline, the use is just the other way around: given a design of a rubble, the critical speed U_{cr} (sometimes other notation) is determined which serves as input parameter for the safety verification in the sections Height, Non-closing and Structural failure. The calculation is not probabilistic; the expected critical speed can be interpreted as a calculation value because the stability relations presented are usually design formulas in which some (sometimes hidden) safety is present.

For the type of 'granular soil protection' the aforementioned models can all be applied. For the other types of soil protection, for the stability relations of Pilarczyk and Escarameia and May a specific coefficient can be entered that applies to the type of soil protection. However, no coefficients are available for the asphalt mastic slabs, stone asphalt slabs and material penetrated with asphalt or colloidal concrete (see also Background report on the track section Height II - Determination of critical transfer / overflow flow rate [Ref 9.4]). In those cases where model-based verification is not possible, physical model tests can be an option.

9.4.1.1 Stability formula of Shields (extended)

The basic relationship for granular material under flow is the relationship of Shields. The relationship of Shields was originally derived for uniform flow over a horizontal bed of loose-grained materials. The comprehensive formulation contains correction factors for slope and non-uniform flow:

$$\Delta \cdot D = \frac{k_t^2 \cdot U_{cr}^2}{k_{sl} \cdot \Psi_{cr} \cdot C^2} \qquad 9.1$$

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Here is:

D	Characteristic element dimension [m]
	For granular materials: $D = D_{50}$
	D_{50} = Screen size so that 50% of the weight of a batch can pass stones.
U_{cr}	Depth-average critical flow rate [m/s]
ψ_{cr}	Critical shear stress parameter, also called Shield parameter [-]
Δ	Relative density [-]
	$\Delta = (\rho_s - \rho_w)/\rho_w$
	ρ_s = mass density stones [kg/m^3]
	ρ_w = mass density water [kg/m^3]
C	Chezy coefficient [m$^{0.5}$/s].
	Different formulas are available for this, the most commonly used is $C = 18 \cdot \log(1 + 12 \cdot h/k_s)$ with Nikurad roughness parameter $k_s = 4 \cdot D_{50}$ and h the water depth for soil protection
k_{sl}	Talus factor [-]

$$K_{sl} = \frac{cos\psi \cdot sin\beta + \sqrt{cos^2\beta \cdot tan^2\theta - sin^2\psi \cdot sin^2\beta}}{tan\theta}$$

ψ = angle of the flow with the upward incline direction (°)

β = angle of soil protection with horizontal (°)

θ = angle of internal friction (°) (for crushed stone 40 °)

For flow parallel to the slope (ψ = 90 °): $K_{sl} = cos\beta\sqrt{1 - \left(\frac{tan\beta}{tan\theta}\right)^2}$

For flow in a downward direction of the slope (ψ = 180 °): $K_{sl} = \frac{\sin(\theta-\beta)}{sin\theta}$

k_t	Turbulence factor [-]
	$k_t = (1+3r_0)/1,3$
	r_0 = depth average relative fluctuation intensity due to turbulence [-] In case of normal turbulence above a flat bed (for example block mat, asphalt mats) r_0 = 0,1 (10%). In the case of normal turbulence above a rough bed (e.g. waste stone), r_0 = 0.15 (15%). In case of high turbulence $r_0 \approx 0.3$ ($k_t^2 = 2$), with extremely high turbulence $r_0 \approx 0.45$ ($k_t^2 = 3$) applies.

The size of the critical shear stress parameter ψ_{cr} depends on the permissible damage. The table below shows the corresponding behavior for 3 different values. The value of this shear stress parameter has been derived in the past for sandy material and uniform flow. Although this value becomes greater with larger stone diameters under uniform flow, use should be made of these values below because other coefficients (such as the relative fluctuation intensity due to turbulence but also other parameters) are probably calibrated in the different formulas using these values of the shear stress parameter.

Table 18 Values for shear stress parameter ψ_{cr}

Shear stress parameter ψ_{cr}	Observable behavior
0,03-0,035	Beginning of movement of stones
0,045-0,05	Some transport of stones
0,060	Continuous transport of stones

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For the design of soil protection it is recommended to use the criterion 'start of movement' and for the assessment the criterion 'some transport of stones'. The final choice of the value of the shear stress parameter should be based on the accepted risk of failure of the artwork and the contribution of the soil protection herein.

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9.4.1.2. Stability formula of Shields for blasting

The stability formula of Shields can be made applicable for concentrated blasting above a rock slab covering by using the flow velocity above the soil (U_b), while the influence of the turbulence of the beams themselves and of the soil roughness is taken into account in a more direct way. The formula is then:

$$\Delta \cdot D = \frac{U_b^2 \cdot (1 + 3 \cdot r_0)^2}{k_{sl} \cdot \Psi_{cr} \cdot C^2} \qquad 9.2$$

Based on the definition for k_t, as given in the extensive Shields formula, it can be deduced that for $U_{cr} = 1.3 \cdot U_b$ both formulas yield the same results.

9.4.1.3. Stability formula of Pilarczyk

A widely used formula for calculating soil protection is the Pilarczyk formula. The advantage of this formula is that the different influencing factors are described separately. This allows the formula to be tailored to specific situations. Essential here is a correct estimate for the different parameters. However, as the application becomes more specific, less information about these parameters is known and the uncertainty about the estimate increases. The spread around the different parameters is decisive for the spread around the calculated stone diameter. The Pilarczyk formula reads:

$$\Delta \cdot D = \phi_{sc} \cdot \frac{0{,}035}{\Psi_{cr}} \cdot k_h \cdot \frac{k_t^2}{k_{sl}} \cdot \frac{U_0^2}{2 \cdot g} \qquad 9.3$$

Here is:

D Characteristic element dimension [m]

For granular materials: $D = D_{n50}$

D_{n50} = nominal stone diameter of a stone piece with a mass M_{50}.

$D_{n50} = (M50/\rho_s)\ 1/3$.

M_{50} = mass [kg] of a stone from the stone grading for which 50% of the mass of the stone grading consists of bricks that are lighter than this mass.

ρ_s = mass density stones [kg/m^3]

For gabions and stone mattresses: D = thickness element

D = block thickness applies to block slabs.

ϕ_{sc} Stability parameter to take into account the influence of transitions and the deviating hydraulic loads that occur here. For ϕ_{sc} the following values are used:

- Termination of directly infused gabions/stone mattresses: $\phi_{sc} = 1.0$
- Termination of directly poured loose rubble stone: $\phi_{sc} = 1.5$
- Rubble stone in a continuous layer (minimum two layers of stones): $\phi_{sc} = 0.75$
- Stone settlements, continuous (brick or block) mat constructions: ϕ_{sc} = 0.50

Δ Relative density [-]

Granular materials, stone settlements, block slabs: $\Delta = (\rho_s - \rho_w)\ \rho_w$

Gabions, stone mattresses: $\Delta = (1\text{-}n) \cdot (\rho_s - \rho_w)/\rho_w$

ρ_s = mass density stones [kg/m^3]

ρ_w = bulk density water [kg/m^3]

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n = open space content (including pores) [-]; for gabions and mattresses n ≈ 0.4

ψ_{cr} Shear stress parameter [-]

Compares the stability of the system with the critical shear parameter of loose stones according to Shields, for which a value of 0.035 is used for the Shields parameter.

ψ_{cr} = 0.035 for granular materials

ψ_{cr} = 0.070 for gabions, stone mattresses

k_h Depth parameter [-]

$k_h = 2 / (\log^2 (1 + 12h / k_r))$ for fully developed speed profile

$k_h = (1 + h / D)^{-0.2}$ for not fully developed speed profile

h = water depth [m]

$k_r = D_n$ for hydraulically smooth elements, concrete blocks

$k_r = 2 \cdot D_n$ for rough elements, such as quarry stone

The factor K_h can be neglected if instead of the average flow rate with the local velocity near the soil is counted.

k_{sl} Talus factor [-]

$$K_{sl} = \frac{cos\psi \cdot sin\beta + \sqrt{cos^2\beta \cdot tan^2\theta - sin^2\psi \cdot sin^2\beta}}{tan\theta}$$

ψ = angle of the flow with the upward incline direction (°)

β = angle of soil protection with horizontal (°)

θ = angle of internal friction (°) (for cru $K_{sl} = cos\beta \cdot \sqrt{1 - \left(\frac{tan\beta}{tan\theta}\right)^2}$

For flow parallel to the slope (ψ = 90 °): $K_{sl} = \frac{\sin(\theta-\beta)}{sin\theta}$

For flow in a downward direction of the slope (ψ = 180 °):

k_t Turbulence factor [-]

k_t^2 = 0.67 in case of low turbulence and uniform flow

k_t^2 = 1.0 at normal turbulence (e.g. flow in rivers)

k_t^2 = 1.5 with increased turbulence (common, non-uniform flow)

k_t^2 = 2.0 at high turbulence (downstream of a water jump or in a sharp bend)

k_t^2 = 3 at very high turbulence (for example jet, screw jet load or near water jump)

U_0 Depth-average flow rate [m / s]

g Gravitational acceleration ($g = 9.81$ m / s^2)

For the stability parameter φ_{sc} to be used, it is important to know whether edges are directly supplied or not. This refers to a situation as shown in Fig. 48, situation a, with the direction of flow in the direction of the overlap. For edges where the flow travels in the same direction as the overlap, the stability parameter for a continuous layer / mat construction can be used (see also situation b in figure 48).

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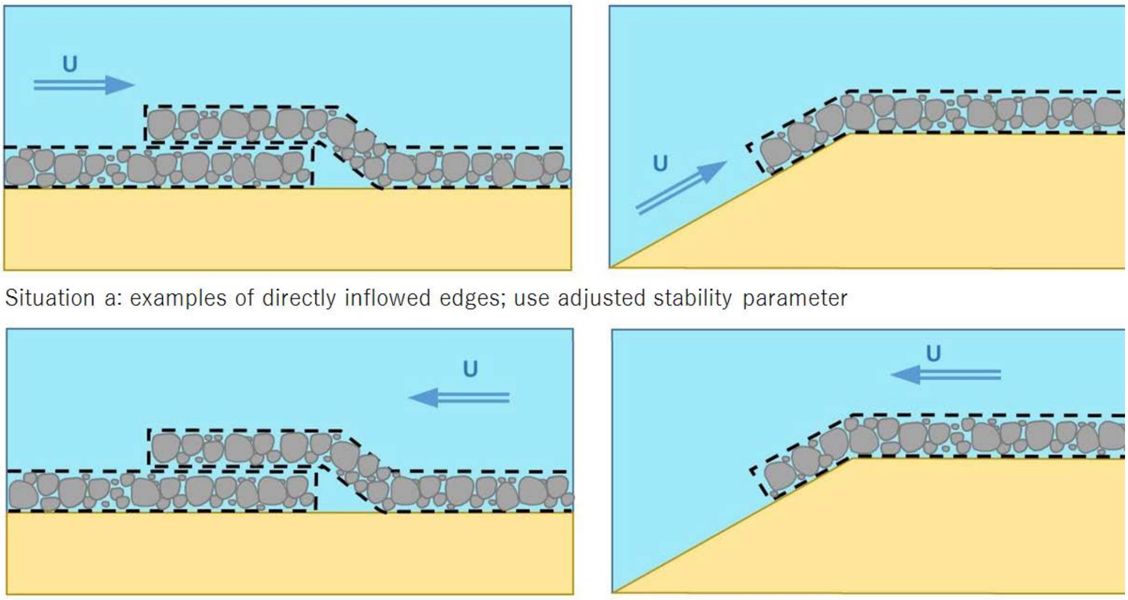

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Figure 48 Examples of direct and non-direct edges

9.4.1.4. Stability formula of Escarameia and May

With the formula of Escarameia and May can be designed in addition to bricks for stone settlements and stone mattresses / gabions. The formula of Escarameia and May is as follows (River and channel revetments - A design manual [Ref 9.11]):

$$\Delta \cdot D_{n50} = K_t \cdot \frac{U_b^2}{2 \cdot g} \qquad 9.4$$

Here is:

D_{n50} 　D_{n50} = nominal stone diameter of a stone piece with a mass M_{50}.

$D_{n50} = (M_{50} / \rho_s)^{1/3}$.

M_{50} = mass [kg] of a stone from the stone grading for which 50% of the mass of the stone grading consists of bricks that are lighter than this mass.

ρ_s = mass density stones [kg/m^3]

For gabions and stone mattresses the D_{n50} applies to the stones in the gabion/ stone mattress

For blocking, D_{n50} = block thickness applies

K_t Turbulence coefficient [-] For this applies:

Stone: $K_t = 12,3 \cdot r_{u,b} - 0.20$ *

Gabions and stone mattresses: $K_t = 12.3 \cdot r_{u,b} - 1.65$ **

Stone settlements: $Kt = 0,75 \cdot (12,3 \cdot r_{u,b} - 0,20) = 9,22 \cdot r_{u,b} - 0,15$ ***

Block mats: $K_t = 0.05$ ****

([Escarameia, 1995]) $K_t = 1.79 \cdot r_{u,b} - 0.72$ *****

* Valid as $r_{u,b} > 0.05$ and slope slope 1: 2 or faint

** Valid as $r_{u,b} > 0,12$ and slope slope 1: 2 or faint

*** Valid as $r_{u,b} > 0.05$ and slope slope 1: 2.5 or faint

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**** Valid as $r_{u,b} < 0.43$ and slope slope unknown

***** Valid as $0,43 < r_{u,b} < 0,90$ and slope slope unknown

$r_{u,b}$ = turbulence intensity at 10% of the water depth above the soil protection [-]

Normal turbulence: $r_{u,b} = 0.12$

Increased turbulence: $r_{u,b} = 0.20$

Moderate to high turbulence: $r_{u,b}$ = 0.35 to 0.50

Very high turbulence: $r_{u,b} = 0.60$

U_b Flow rate at 10% of the water depth above the soil protection [m/s]

$U_b = (-1.48 \cdot r_{u,b} + 1.04) \cdot U$ Valid if $r_{u,b}$ £ 0.5

$U_b = (-1.48 \cdot r_{u,b} + 1.36) \cdot U$ Valid if $r_{u,b}$ £ 0.5

U = average flow rate [m/s]

Δ Relative density [-]

For this applies: $\Delta = (\rho_s - \rho_w)/\rho_w$

ρ_s = mass density stones [kg/m^3]

ρ_w = bulk density water [kg/m^3]

g Gravitational acceleration ($g = 9.81$ m / s^2)

The Escarameia and May formula implicitly includes a safety factor because it is intended for designs. It is not known how great this safety factor is. Furthermore, the water depth is not included in the formula of Escarameia and May, which leads to larger elemental dimensions compared to Shields and Pilarczyk, particularly in deeper waters.

9.4.2 Stability calculation for overflow/splash load

Almost no literature is available for overflow/splash load. In Background report test track Height II - Determination of critical transfer/overflow flow rate [Ref. 9.4] the model of Free castle is cited. Although in [Ref. 9.4] it is concluded that the reliability of this model is not so great, it can be used to obtain a first indication. Use can also be made of the guide values as included in table 7-4 of the schematization manual for the height of the artwork [Ref. 5.4].

Depending on the contribution of soil protection to the total risk of failure, consideration can be given to physical model research.

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Figure 49 2D Physical model research soil protection (source: Deltares)

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9.5 Other verifications

9.5.1 General

In general, the verifications in this section do not apply to the draft verification to the Water Act requirement because the dimensions of the soil protection are usually not determined by the flood water situation but by other load situations. However, it is possible that the high water load situation is decisive for the dimensions of soil protection. That is why this section briefly discusses other aspects than the dimensions of the top layer.

9.5.2 Horizontal dimensions

The required length of the soil protection is determined by analyzing the geotechnical stability when a drainage pit develops downstream of the protection as a result of erosion. When the depth of the pit is large or the setting of the pit becomes very steep, shearing or settlement flow can take place. Landing flow is a soil mechanical instability in which a originally loosely packed granular soil mass becomes softened and flows out due to the slope present. The density and the permeability of the soil package are of great influence.

Shear is a soil mechanical instability in which no softening occurs, but the soil mass as a whole undergoes a downward displacement. Depending on the storage length, a shear or a settlement flow can threaten the stability of the artwork. The chance of instability therefore becomes smaller with increasing length of soil protection. The flow velocity also decreases at the edge with increasing length. In addition to the extension of the soil protection, it is also possible to opt for defending the supply slope by recording it. This can be realized, for example, as 'falling apron'.

The minimum length of the soil protection is therefore dependent on the pit depth and steepness of the pit. In addition, the type of soil material is of influence. If settlement flow is possible, the gradient of the end profile will be very faint, for example 1:15, while with shear this gradient will be 1: 6 to 1: 8.

This section describes a method for assessing the length of the soil defense. This method is based on a salvage model, in which the two situations, immediately before and after geotechnical instability, are compared with each other. In the Scour Manual ([Ref 9.5]) the salvage model is described and various mean and maximum values are given that can be used for a general design relationship in which the length of the bottom defense is derived.

The collapse of soil protection as a result of geotechnical macro-instability is initiated by the formation of excavations at the edge of soil protection. If here too steep slopes in combination with too deep pits occur, settlement flow or shear can occur depending on the base.

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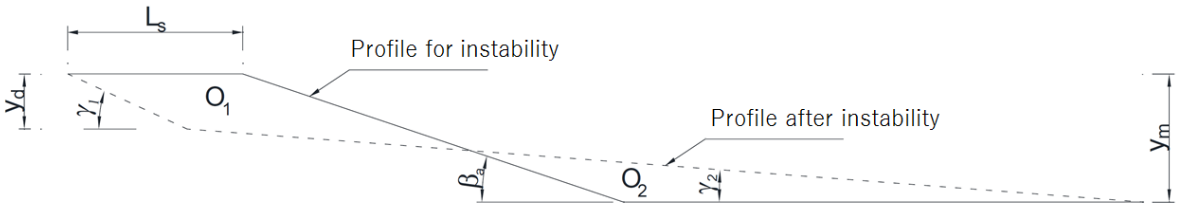

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Figure 50 Storage model (2D) for shear and settlement flow.

$$L_s = y_d\left(\frac{1}{2}\frac{y_d}{y_m} - 1\right)\cdot(\cot\gamma_2 - \cot\gamma_1) + \frac{1}{2}\gamma_m(\cot\gamma_2 - \cot\beta_a) \qquad 9.5$$

Where $O_1 = O_2$ applies

Here is:

L_s	Entry length [m]
y_d	Erosion depth at the transition from the steepest slope to the second slope [m]
y_m	Maximum erosion depth [m]
β_a	Average slope for instability [m]
γ_1	Slope steepest slope after instability [m]
γ_2	Slope angle second slope after instability [m]
O_1	Slid surface [m^2]
O_2	Deposited surface [m^2]

The storage length must be less than the length of the soil protection applied. When y_d is set equal to 0, the acceptance length is the greatest.

The acceptance length depends on the maximum erosion depth. The erosion depth development is a time-dependent process. Reference is made to the Scour Manual, Chapter 3 ([Ref.1.5]) for the calculation method.

The equilibrium depth achieved after a long time load can be estimated with the following formula (Scour Manual [Ref 9.5]):

$$y_e = \frac{(1 + 3 \cdot r_0) \cdot U_0 - U_{cr}}{U_{cr}} \cdot h \qquad 9.6$$

If the average flow velocity U_0 is a number of times greater than the critical flow velocity U_{cr} then the formula yields unrealistic values. Limitation of the equilibrium depth to twice the water depth can then be used. The size of U_{cr} depends on the surface. In the Scour Manual ([Ref 9.5]) a relationship is given to calculate this value as clay particles and thus cohesion must be included. For granular materials the previously given formulas can be used. **(177 頁)** In addition to the length, the width is also important. CUR 197 Quarry Stone in Practice [Ref. 9.2] gives a first impulse for this when the current starts to widen, as with a diver. The total required width (B) is determined here on the basis of the assumption that the flow spreads at an angle of 1: 6. B can then be calculated on the basis of the length, so B = 1/3 · L + b_u.

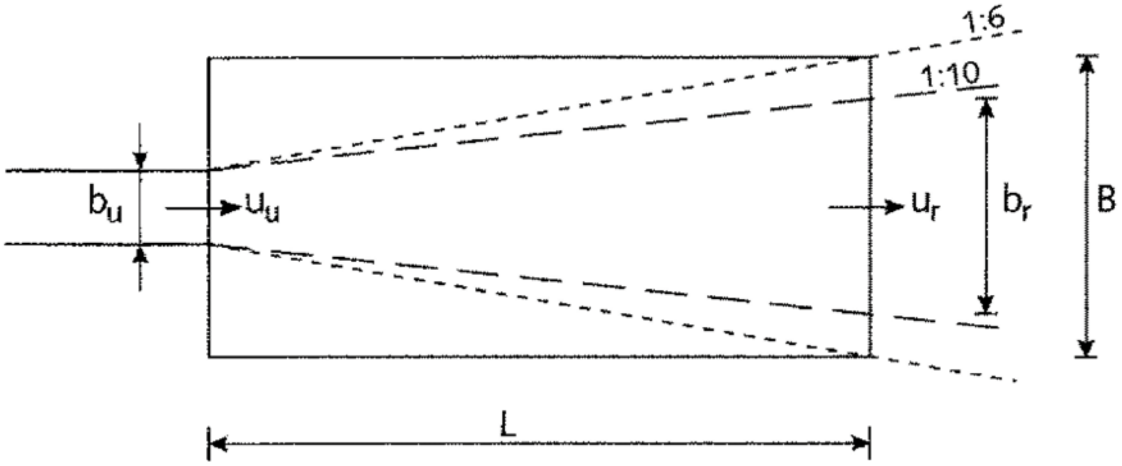

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Figure 51 Determination of width of soil protection on the basis of spreading angle (source: CUR 197)

9.5.3 Vertical structure

The material that forms the top layer of soil protection is usually too large to lay down directly on the existing soil. Often one or more filter layers are applied, which together have the function of preventing erosion of the soil and thus forming a geometrically dense structure. A geometrically dense structure can also be realized by placing a geotextile directly on the bottom material and thereby saving one or more filter layers. To meet geometric-tight filter layers, there are requirements for specific maximum ratios between the layers. These requirements relate to surface stability, internal stability and water permeability. For the formulation of the requirements reference is made to the Handbook Design of lock locks ([Ref 9.8]), section 16.7.3.

There are also geometrically open filters. The top layer should then have a sufficient thickness such that the hydraulic gradient becomes too small to set the soil material in motion. However, existing calculation rules are currently based on more or less uniform flow. The effect of turbulence on the minimum thickness of the top layer is not known. Physical model research must first be done for this. The costs of model research usually far outweigh the costs of an extra filter layer and thus a geometrically open filter can be economically a good alternative for geometrically tight filters. Additional information on geometric-open filters is available in CUR 161 Filters in hydraulic engineering [Ref. 9.1], CUR 233 Interface stability or granular filter structures [Ref. 9.2], Validation and optimization of a design formula for geometrically open filter structures [Ref. 9.9] and Granular open filters on a horizontal bed under wave and current loading [Ref. 9.10].

Both the top layer and the filter layers must have a minimum thickness. For the minimum thickness, a distinction is made between fine grading and light grading. In addition, a distinction is made in the method of application, namely from land in the dry, from land in the wet and from water by dumping. For geometrically dense filters, a fixed surcharge on top of the 2xD_{n50} layer thickness is used for fine grading (Table 19). **(178 頁)** For light grades, the supplement is equal to 0.5 to 1xD_{n50} of the grading (Table 20). The tables below show the tolerance and average thickness for geometric-tight filters for fine grading and light grading. The minimum thickness is set equal to 2xD_{n50}.

Table 19 Tolerance and average thickness in geometrically tight filters fine grading

			Tolerance			Average thickness		
		D_{n50}*				$D_f = 2.0\ D_{n50}$ + tolerance		
			dry	wet	deposit	D_{f_droog}	D_{f_nat}	$D_{f\_storten}$
		[m]	[m]	[m]	[m]	[m]	[m]	[m]
Fine grading	oud: 40/100 mm	0.063	0.1	0.15	0.2	0.23	0.28	0.33
	oud: 80/200 mm	0.127	0.1	0.15	0.2	0.35	0.40	0.45
	45/125 mm	0.067	0.1	0.15	0.2	0.23	0.28	0.33
	63/180 mm	0.097	0.1	0.15	0.2	0.29	0.34	0.39

	90/250 mm	0.135	0.1	0.15	0.2	0.37	0.42	0.47
	45/180 mm	0.080	0.1	0.15	0.2	0.26	0.31	0.36
	90/180 mm	0.114	0.1	0.15	0.2	0.33	0.38	0.43

※（Rijkswaterstaat, Central government　2018）178 頁より作成。

* Average D_{n50} at a density of 2650 kg/^{m3}

Table 20 Tolerance and average thickness for geometrically tight filters for light sorting

Light Sorting			Accuracy			Average thickness		
		D_{n50}*	f x D_{n50}			D_f = 2,0 D_{n50} + accuracy		
			f_{droog}	f_{nat}	$f_{storten}$	Df_{droog}	D_{fnat}	$Df_{storten}$
		[m]	[-]	[-]	[-]	[m]	[m]	[m]
	5-40 kg	0.20	0.5	0.8	1.0	0.50	0.56	0.60
	10-60 kg	0.24	0.5	0.8	1.0	0.60	0.68	0.72
	40-200 kg	0.36	0.5	0.8	1.0	0.91	1.02	1.09
	60-300 kg	0.42	0.5	0.7	0.9	1.04	1.13	1.21
	15-300 kg	0.38	0.5	0.7	0.9	0.94	1.02	1.09

※（Rijkswaterstaat, Central government　2018）178 頁より作成。

* Average D_{n50} at a density of 2650 kg/^{m3}

9.5.4 Flexibility

Wherever the soil protection ends, erosion occurs. The edge of the soil protection must be so flexible that it can follow the excavations without major problems. The connection of the soil protection against a hard construction must be carefully carried out in order to prevent possible erosion of the base material. An example is given in Figure 52.

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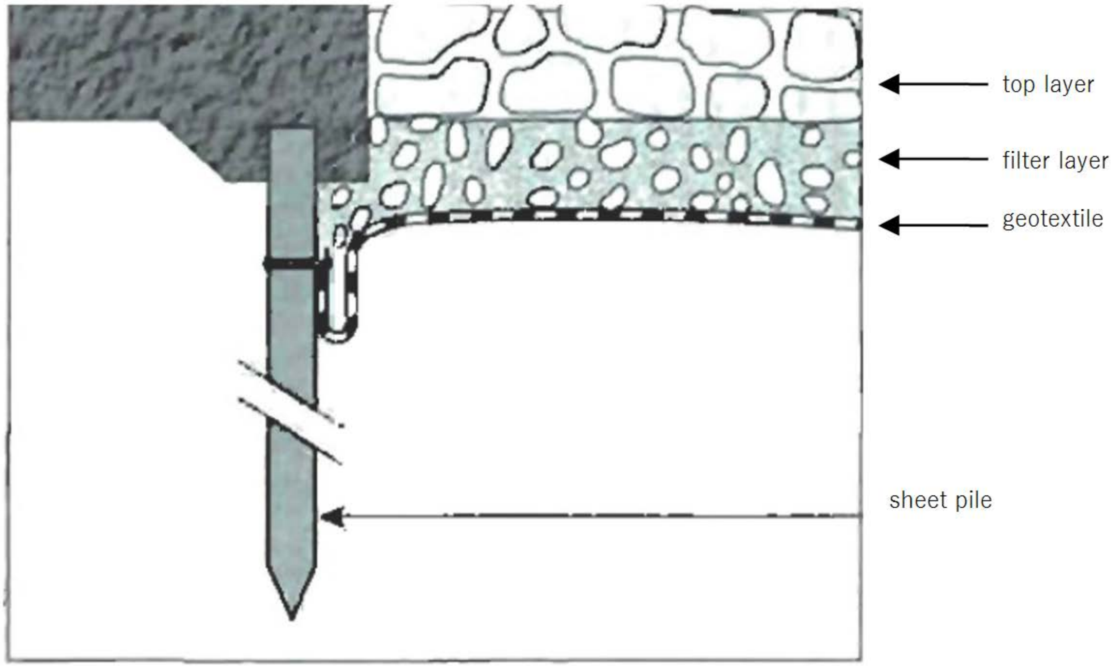

※（Rijkswaterstaat, Central government　2018）179 頁より作成。
Figure 52 Detail of soil protection connection to a vertical construction

9.6 Dealing with empirical formulations and uncertainties

Stability relationships are available for the various tax types, each valid for a limited scope and each with uncertainties due to uncertainties in the input parameters and uncertainties in the quality of the derived stability relationships. These formulations can serve as a starting point for the conceptual design of soil protection. Due to the limited validity and accuracy, physical model research can be considered. This is partly dependent on the contribution of soil protection to the total risk of failure.

9.7 References and background documents

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[Ref. 9.9] Van de Sande, S.A.H., Uijttewaal, W.S.J., Verheij, H.J., 2014. Validation and optimization of a design formula for geometrically open filter structures, Proc. 34th International Conference on Coastal Engineering (ICCE), Seoul, Korea.

[Ref. 9.10] Wolters, G. and Van Gent, M.R.A., 2012. Granular open filters on a horizontal bed under wave and current loading, Proc. 33rd International Conference on Coastal Engineering (ICCE), Santander, Spain.

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10 Cup storage

10.1 Introduction

The size of the cup storage is directly important in determining the failure probabilities of the artworks. The definition of the combining capacity is already set out in section 2.2. This chapter discusses the various aspects of cup storage.

10.2 Size of available cup storage (V_c)

The size of the available storage space depends on various aspects that are discussed in this section. In its most basic form, the combining capacity is nothing more than the permissible water level increase multiplied by the surface of the storage area. In formula form this is as follows:

$$V_c = A_{kom} \cdot \Delta h_{kom} \qquad 10.1$$

In which:

V_c	Maximum available volume of storage capacity in the hinterland with no substantial consequences [m³]
A_{kom}	The available surface in which water can be recovered [m²]
$\triangle h_{kom}$	Permissible water level rise in the combing area [m]

10.2.1 Permissible water level rise in the catchment area ($\triangle h_{kom}$)

The permissible water level increase in the flood risk approach is related to the effects that occur in the hinterland. This water level rise in a (water) system behind a work of art concerns the difference between the inner-dike level at the beginning of the high-water situation and the inner-dike level where there are no substantial consequences and / or victims. This last level is also referred to as open turning height (OKH).

<u>Open Reverse Height (OKH)</u>
The OKH concerns the maximum permissible inland water level where there is just no failure of the inner dike water barrier. The Open Turn Height is always related to the properties of the area or the structure (s) on the inside of the flood defense. This level, together with the other characteristics of the underlying (water) system (surface area, inland water level, etc.), is responsible for the size

of the storage capacity. It forms part of the 'strength' of the work of art with regard to not closing.

When determining the water level at the beginning of the high water situation, account must be taken of what precedes a high water situation. In many cases, this initial level will be higher than the average level or the target level as a result of, for example, precipitation before and during the high water period. But also the policy of the flood defense manager in an approaching high tide is of influence. In connection with, for example, seepage, an administrator can decide to set up the inside water. On the other hand, he can also decide to just grind down an inland waterway, so that additional cup storage is available.

This additional cup storage is then mainly intended to be able to recover from the hinterland during a high water period. (182) The maximum permissible indoor level is primarily related to damage and / or casualties. For underlying barriers the starting point can be that for as long as the water level is lower than the test water level associated with the standardization of these (regional) flood defenses, they will not fail. This means that it is explicitly assumed that the underlying flood defenses are in order and meet the requirements that apply to them. This also means that the maximum permissible level may be lower than the deflecting height of the rear defenses, because these fail, for example, as a result of piping or stability at lower water levels than the deflecting height.

Other criteria may also be linked to the maximum permissible internal level. An example of this is salty, when too much salt water leads to substantial damage to nature or agriculture.

There is a different situation for denominations. There is no underlying water passage in which water can be salvaged immediately. In the case of failure of a denomination, water flows directly over the ground level of the hinterland. If this consists of grassland, the consequences can still be limited. If the denomination is located in a residential core, inflow of water will immediately lead to nuisance. In that case direct use can be made of the term 'substantial economic damage' as defined in section 2.2. It is recommended to take account of cup storage as much as possible for denominations in urban areas (residential areas) and to comply with the requirements by means of the retaining capacity of the denomination.

10.2.2 Surface area storage area (A_{kom})

Determining the storage surface can be done from very coarse to very accurate. This depends in particular on the specific situation. In the figure below, a fictitious situation is outlined on the left, involving a complex system of water surfaces. In addition, the cross section of the waterways is not constant and runs with the water level. Also a low-lying pasture present, of which the flooding is very limited has consequences.

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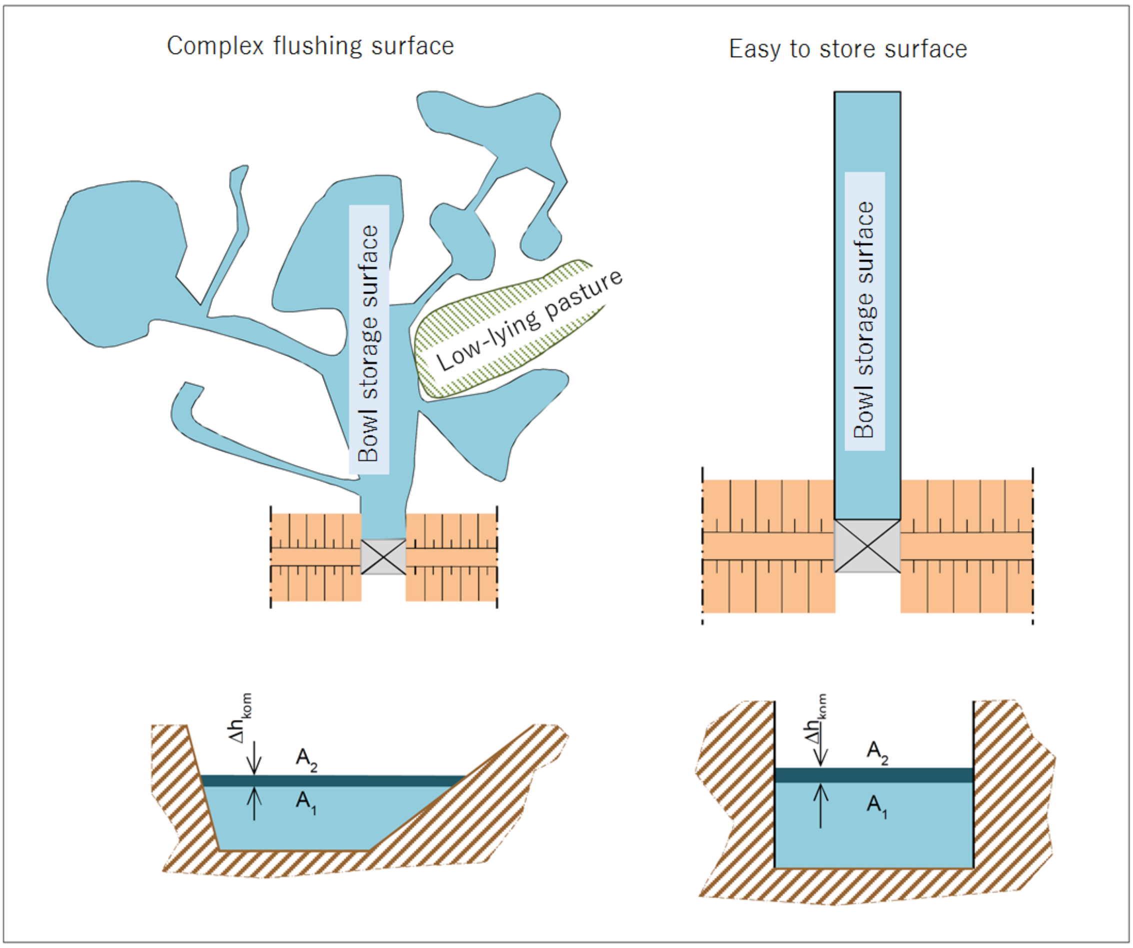

※（Rijkswaterstaat, Central government　2018）183 頁より作成。

Figure 53 Compressed area complex (left) and simple (right)

In the left situation above, there is no single storage surface. It is clear that one immediately has to think of accumulating volume (s).

The situation can also be fairly simple (see figure 53, right); for example, when the waterway behind the artwork is a channel with clear quays and an unambiguous transverse profile.

Tools that can be used in determining the storage surface are GIS applications and calculation software for model calculations.

It is advisable to determine the surface of the area in the first place (conservatively). A lot of uncertainties play a role as far as the combining capacity is concerned, but also with regard to the inflowing volume when the artwork fails. The accurate determination of the storage surface (capacity) is therefore only useful if the other aspects are also taken into account in an accurate manner. Certainly for the design of a work of art, an overall, somewhat conservative way of determining the storage capacity is recommended in the context of a robust design.

10.2.3 Other influencing factors of storage capacity

In addition to the surface area and the critical level increase, there are a number of other factors that influence the available storage space for failure, such as: **(184 頁)**

- Rain in the underlying system or the influx of rivers and canals.
- Pumps and pumping stations that are used during high water to curb the indoor level have a positive effect on the storage capacity.
- Pumps and pumping stations that are used to discharge water from the polders onto the waterway (s) have a negative influence on the storage capacity.

10.2.4 Distribution of storage capacity over several works of art

When the combative capacity is known, it should be considered which part of this is available for the artwork to be designed. To this end, a list is drawn up of works of art that use the same cup storage. These works of art can be located in the same dike section, but can also belong to another dike section and even to another dike ring.

A first conservative approach could be to distribute the available storage space proportionally over the present and the artwork to be designed (V_c/n). A sharper division can be made by looking at the failure mechanisms of the various artworks and determining to what extent these are independent or dependent. When the outside water level is strongly dominant for the failure probabilities of the artworks, there is a considerable dependency and when the failure probabilities of the structures are the same, the distribution is proportional to the cup storage.

To make a more accurate distribution, a (probabilistic) analysis of the artworks involved is therefore necessary. It is then important to analyze the chances that when the artwork to be designed fails, another work of art also fails. If the preconditions and structure of two works of art are completely identical, it is clear that this opportunity is high. In that case, a part of the cup storage can be allocated to each work of art. If, however, the works of art have very different failure probabilities and the circumstances in which failure occurs are not comparable, this is too conservative and the room storage can be fully attributed to the artwork to be designed. All this can take the necessary effort. Using the conservative approach is therefore recommended in the first instance.

A specific point of attention concerns the transhipment/overflow at the dikes. For dykes, there are no guidelines and/or recommendations regarding storage. However, it is clear that with larger overflow/transhipment rates of dykes (not yet a breach), the capacity of the area to absorb is also called for. When designing a new work of art, it is therefore recommended to pay some attention to this. This is particularly the case with high outside water levels where water flows over the artwork and possibly over the adjacent dike sections.

10.3 Inflowing volume

When the water retaining capacity of the artwork has reached its limit and there is therefore an appeal to the storage capacity, it depends on a number of things how quickly (flow) and how much (volume) outside water flows in through/over the artwork.

A. The failure mechanism that occurs

B. Dimensions of the water-carrying element (s)

C. Occurrence of breaches

D. The moment of failure in time

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E. The course of the flood wave over time

F. The course of the inland water level in time

In addition, the course of the floodwaters and the inland water level plays a role in the inflowing flow. The various aspects are discussed in more detail in the following paragraphs.

10.3.1 Emerging failure mechanism

Depending on the occurring failure mechanism, a certain influx situation arises for the artwork. For example, the inflow opening is different in the event of failure of the closure than in the case of overflow/transfer. The way in which water flows in is already described in the various failure mechanisms.

- Height. For height applies that inflow occurs over the reversal / artwork, where there is no direct contact between inside and outside water level
- Do not close. In the case of non-closing, inflow occurs in the first instance by the water-carrying elements present. This can be through a tubular cross-section (for example, inlet diver), but also an open box construction such as a floodgate.
- Piping. With piping, the basic principle is that when a breach occurs, a breach is created in the flood defense. With this failure mechanism, no account is taken of cup storage.
- Strength and stability. For this failure mechanism, it is generally true that an uncontrollable situation will occur in the event of failure, while cup storage hardly affects the failure probability. Failure always takes place at high outside water levels, so that the flow rates of the inflowing water quickly lead to further breach formation. In the failure mechanism strength and stability, therefore, no account is taken of cup storage in designs

10.3.2 Dimensions of water-carrying element (s)

After a failure mechanism occurs, the size of the inflowing flow rate is determined by the physical dimensions of the water-carrying elements. For example, the total transhipment over a narrow denomination is smaller than over a broad denomination. With a broad denomination, the maximum admissible inflow volume will thus have been reached earlier. The physical dimensions of the water-carrying elements are always determined by the primary function of the artwork. The flood defense function has no direct influence on this.

10.3.3 Occurrence of breaching

If after the occurrence of a certain failure mechanism there is a rapid question of further breach formation and breach growth, the inflowing flow is difficult to determine. For those situations, the design of a work of art is recommended to assume that the occurrence of the failure mechanism directly leads to the failure of the artwork (occurrence of flood effects). In particular for strength and stability and piping this is a recommended starting point because these usually take place at high outside water levels and, upon collapse, an uncontrollable situation immediately arises. Possible exceptions to this are inlet/outlet divers and pressure pipes from pumping stations.

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10.3.4 Course of the flood water and the inland water level

The maximum permissible outside water level in the event of the occurrence of a failure mechanism depends on the course of the high water wave, the occurring waves, the method of inflow and the inland water level. This outside water level is referred to as the open timespan (OKP) [37] in the Guidelines for Art Works 2003 (Ref No. 10.2). However, since this outside water level depends on a number of uncertain parameters, such as the course of the flood wave, there is not one OKP, but there are many OKPs. In addition, in the Guidelines for Art Works 2003 [Ref. 10.2] do not close the OKP mainly related to the failure mechanism. Based on the above, the term OKP is no longer used in this manual.

As a result of inflowing water, the inland water level also increases during a high water wave. The extent to which depends on the size and configuration of the cup storage. In a very large bowl the inland water level is hardly influenced, while in a very small basin the inland water level will run along with the outside water level. This means that the inflowing flow varies during a high water wave.

In the chapter Hydraulic Loads (see chapter 3) the gradients of a high water are indicated for the different water systems. These are based on the Water Level Course tool. This contains one form of the flood water per system / area, which is still considered as decisive for the time being. The relevant chapter also indicates how to deal with waves during a flood wave[38].

10.4 Practical approach to inflow volume

In the chapter height (see chapter 5) it is indicated how storage and transfer / overflow can be included in the design for this failure mechanism. For non-closing, the method below indicates how it can be verified whether the available cup storage is adequate for a specific situation. More specifically, which outside water level does not lead to the occurrence of flood effects in the hinterland when the closure fails. For piping and strength and stability, when designing a new work of art, it is recommended not to take account of cup storage. If this is chosen for a specific reason, most of the following steps (see Figure 54) can also be run through.

---

[37] Open Turning Level (OKP): Outdoor water level which, when the valve is open, does not exactly lead to an inadmissible inflowing volume of outside water ([Ref. 5.5])

[38] Within the WBI, the high-water wave is schematized as a block that is initially calculated with a duration of 6 hours. It has been found that this may be suitable for wind-dominated load systems with a wave height up to approximately 2 m, but certainly not for drain-dominated load systems. In the latter case, it is recommended to use the water level gradient from the Water level gradient tool to arrive at an adequate estimate of the inflowing volume.

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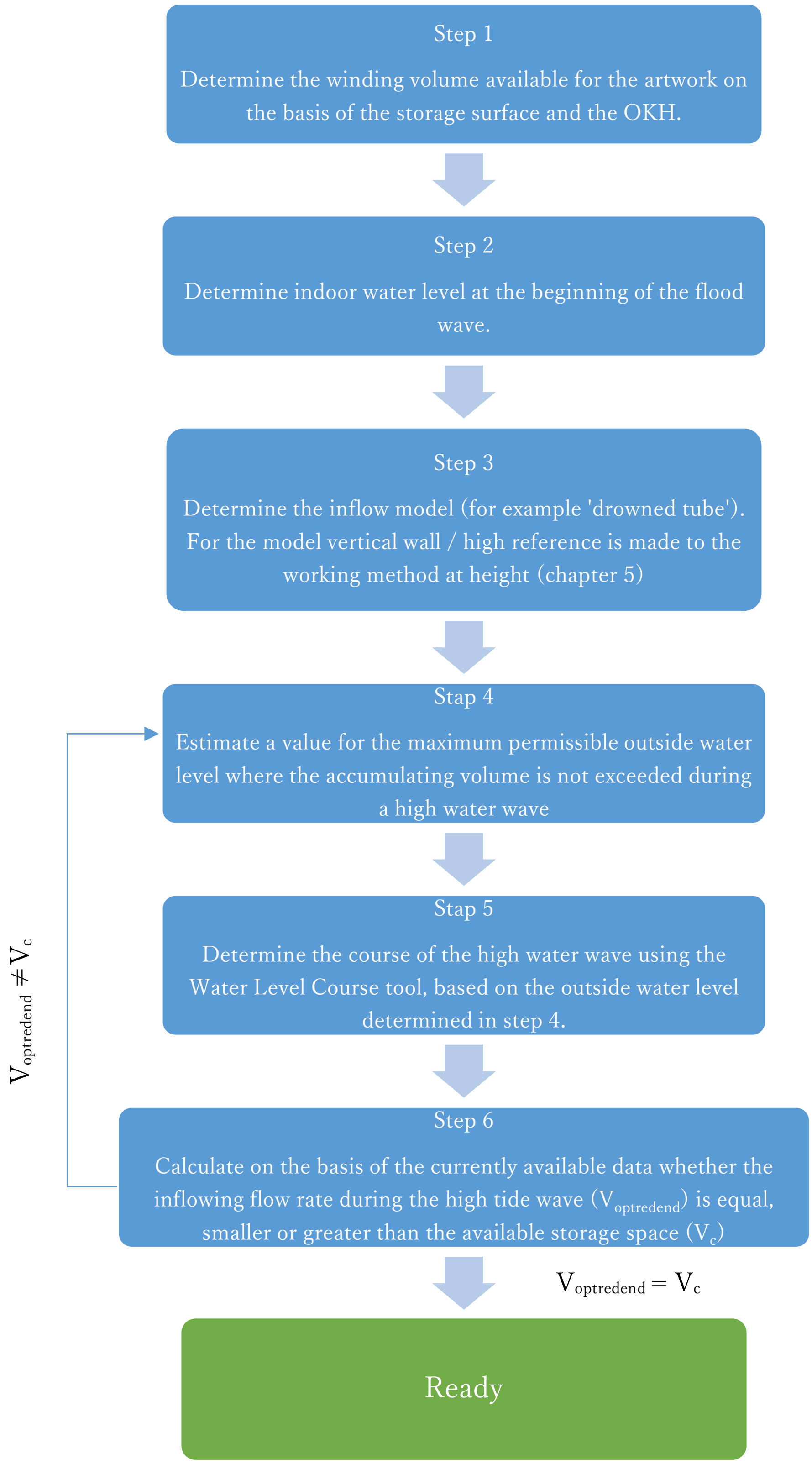


※（Rijkswaterstaat, Central government　2018）187 頁より作成。

Figure 54 Step-by-step approach to combing capacity with reliability of closure

If at step 6 the inflow volume is greater than the available storage space, step 4 can be filled in again by maintaining a lower maximum permissible outside water level. In case the inflow volume is smaller than the available volume, a higher outside water level can of course be maintained at step 4. It may therefore be necessary to make several iteration strokes before finding the desired maximum permissible outside water level.

When there is a (very) small storage room with some simple hands can be viewed whether the entire step plan must be completed. The probability is then that the inland water level can immediately follow the outside water level, so that it is immediately clear which maximum outside water level is permissible from the point of storage. **(188 頁)** Of course, it remains to be seen whether the soil protection is capable of withstanding the flow velocities of the inflowing flow.

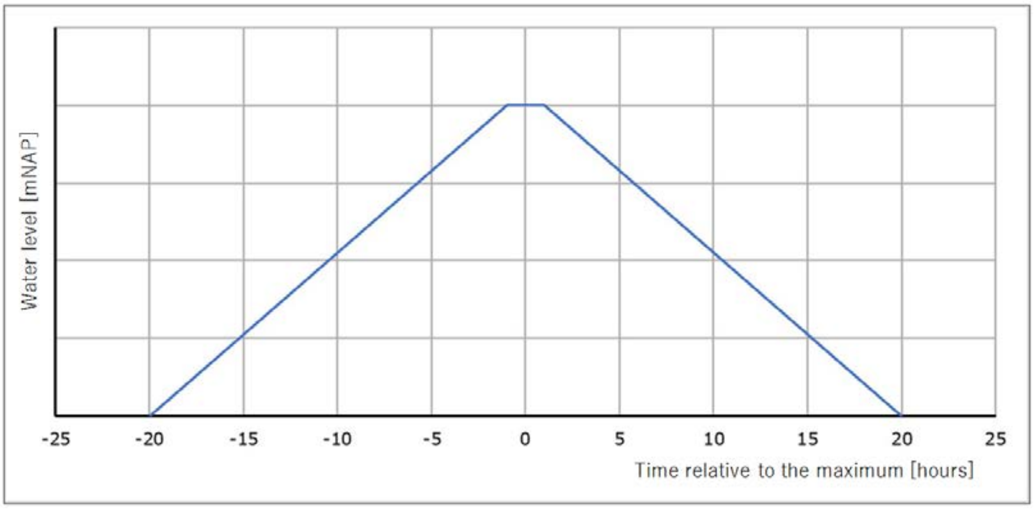
10.5 Example

In this section an example is worked out for the situation of a drainage diver. It is determined which open water level is maximally permissible (peak value of the high water wave) in case the closure of the diver fails. Three approaches are given for this.

10.5.1 Data

The following information is available:

- Immersion model: drowned tube
- Inland water level at closure: NAP +0.0 m
- Permissible rise in water level in the water level = 1.25 m (= NAP + 1.25 m)
- Surface area storage: 400,000 m2
- Storage volume: $V_c = \triangle h_{kom} \cdot A_{kom} = 1.25\ m \cdot 400{,}000\ m^2 = 500{,}000\ m^3$
- Course of flood water: See figure below. The peak period lasts 2 hours with a peak water level H_{piek}.
- Closing level: NAP +0.0 m.
- Surface diver: 1.0 m^2
- No inflow of water from the hinterland and no use of pumping stations.
- Waves do not play a role because inflow occurs via a tube.



※（Rijkswaterstaat, Central government　2018）188 頁より作成。

Figure 55 Course in the time of the external water level during a flood wave

10.5.2　First approach

The first approach concerns a conservative approach, in which the drain wave is schematized as a block with a duration of 40 hours and a constant water level H_{piek}.

In the first instance, a peak water level of NAP + 2.0 m is maintained. This leads to the following figures concerning the course of water levels and inflow.

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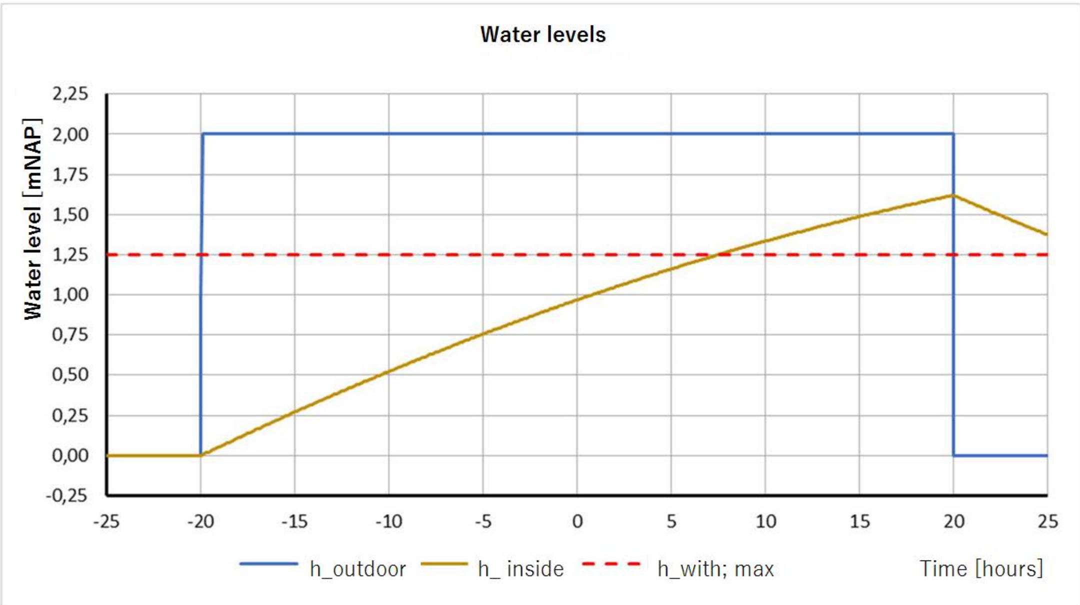
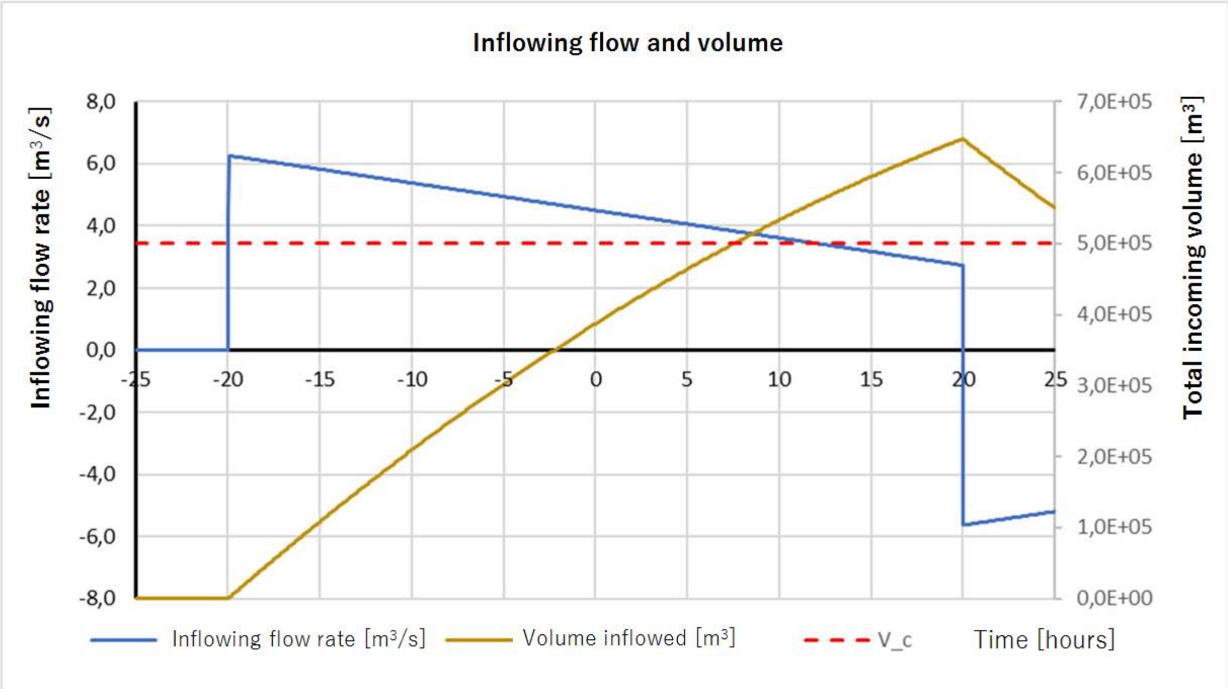


※（Rijkswaterstaat, Central government　2018）189 頁より作成。

Figure 56 Gradation in time of the water levels during high water wave with first simple approach

The red dotted line concerns the precondition that the inland water level may not exceed NAP + 1.25 m. This is exceeded and thus a peak water level of NAP + 2.0 m outside water level is too much for the cup storage. This is also clear in the figure below, in which inflowing flow and volume are displayed.



※（Rijkswaterstaat, Central government　2018）189 頁より作成。

Figure 57 Gradation in time of flow and volume during high water wave at first simple approach

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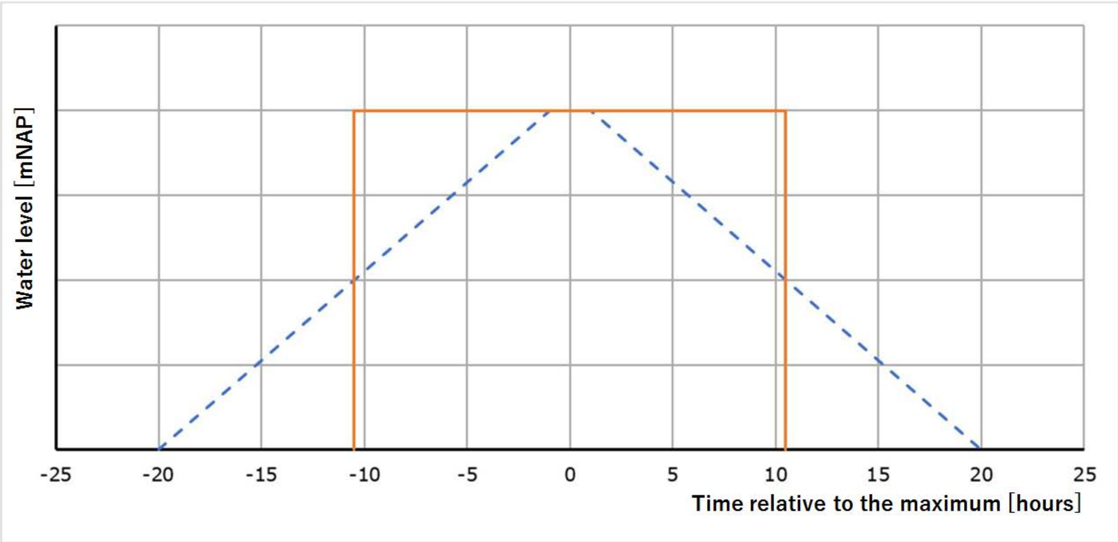
It is visible that the total inflow volume exceeds the critical volume of 5,0·105 m^3. The calculation of the inflow volume has taken into account the increase in the inland water level.

By performing a number of iterations it is finally found that at a maximum outside water level of NAP +1.4 m (H_{peak}) the accumulating volume is not exceeded.

10.5.3 Second approach

The second approach concerns an approach in which the duration of the high water wave is reduced so that it can again be calculated with a block shape, but where the area under this block shape is equal to the surface under the trapezoidal shape of the actual discharge wave (see orange line in the figure below).



※（Rijkswaterstaat, Central government　2018）190 頁より作成。

Figure 58 Schematic course over time of the outside water level for the second approach

So now the duration of the high water wave of 21 hours is calculated. When this is worked out and a number of iterations are carried out, a maximum allowable outside water level of NAP + 2.90 m is found. The graphs below show the course of the water level and the inflow volume.

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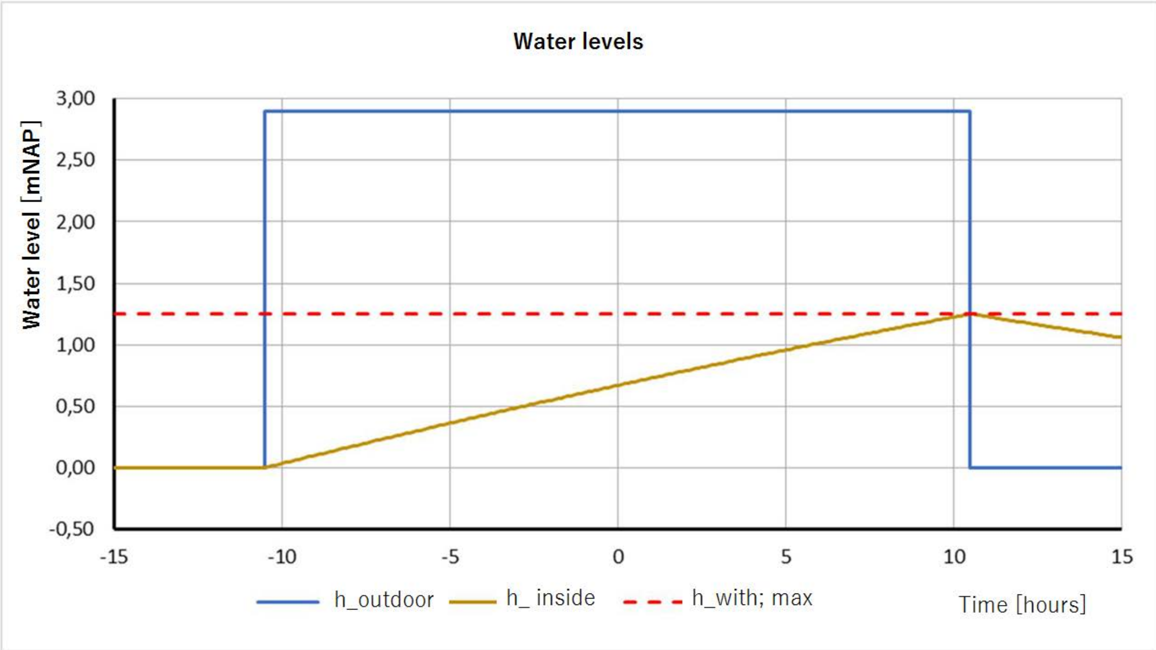
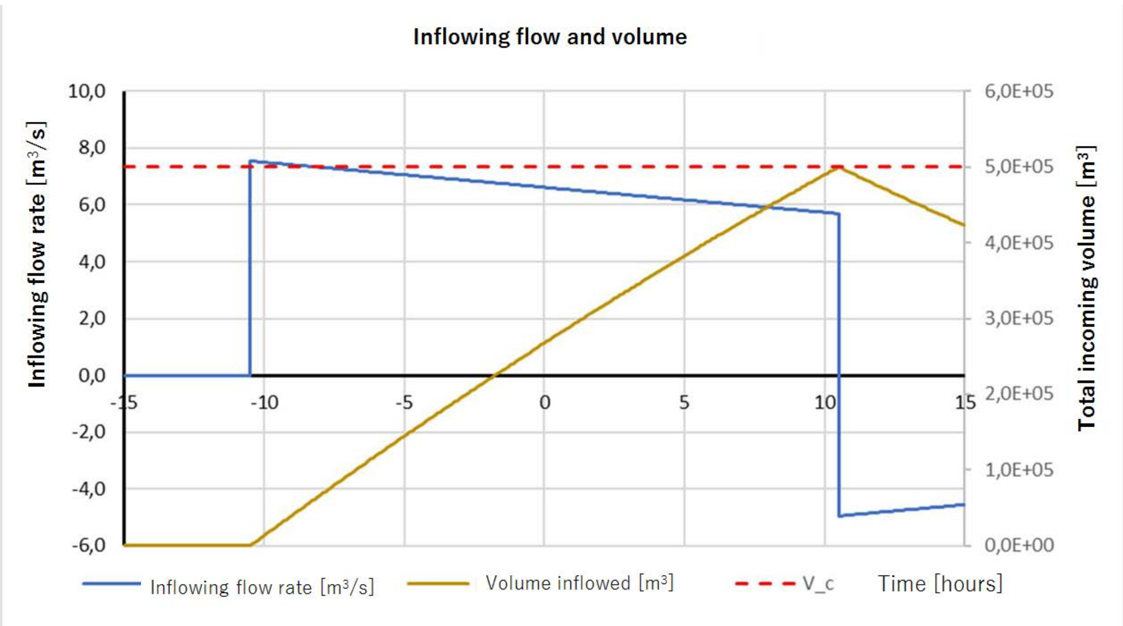


※（Rijkswaterstaat, Central government　2018）191 頁より作成。

Figure 59 Gradation in time of the water levels during high tidal wave in the second approach



※（Rijkswaterstaat, Central government　2018）191 頁より作成。

Figure 60 Gradation in time of flow and volume during high water wave at second approach

10.5.4 Final approach

In the latter approach, the course of the high water wave, as given, is also used in the calculations. If the criteria of a maximum increase of the indoor level to NAP + 1.25 m and 500,000 m^3 are again maintained, a maximum outdoor water level of NAP +2.82 m is found. The diagram below shows the course of water levels and flow rates.

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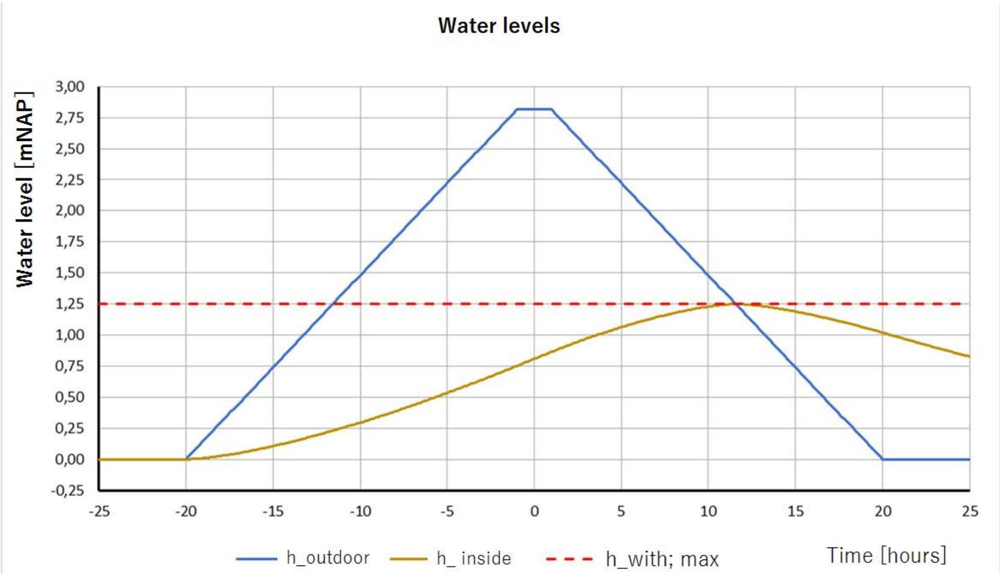
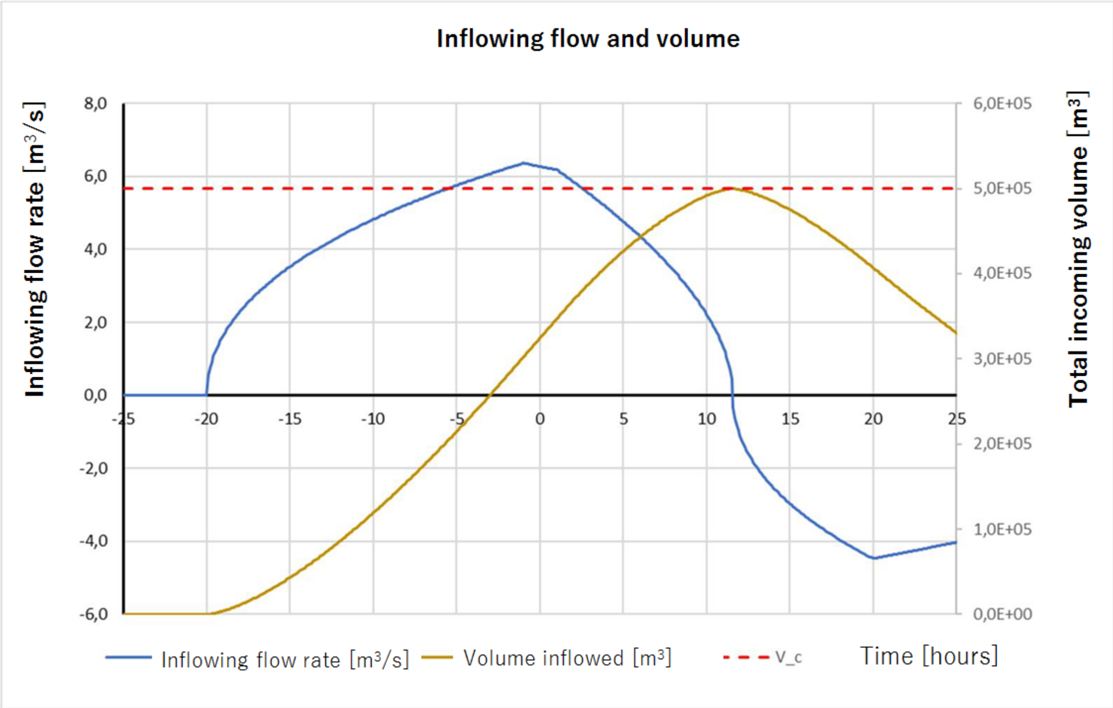


※（Rijkswaterstaat, Central government　2018）192 頁より作成。

Figure 61 Course in time of the water levels during high water wave in the last approach



※（Rijkswaterstaat, Central government　2018）192 頁より作成。
Figure 62 Flow in time of flow and volume during high water wave at last approach

10.5.5 Conclusion

The first rough approach leads to a permissible outside water level of NAP + 1.40 m. The second and third approaches do not differ much from each other. It is clear that the permissible outside water level can be considerably larger in a more accurate calculation. For non-closing, this can make a lot of difference, because the exceedance frequency of the maximum permissible outside water level plays an important role in this failure mechanism. The difference between the whole rough approach and the accurate calculations is about 1.4 m. With a decimation height of 0.5 m, for example, it saves approximately 3 (factor 1000) on the failure probability.

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10.6 References and background documents

[Ref. 10.1] Foundations for flood protection, Expertise Network Flood risk management, ISBN / EAN: 978-90-8902-151-9, second revised edition, November 2017

[Ref. 10.2] Guide to Works of Art 2003, TAW, May 2003

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11 Case

11.1 Introduction

This chapter contains an example of the application of the Work Guide for the verification of the design of an artwork to the Water Act and the Building Decree. The example concerns a lock in an existing spatial situation.

11.1.1 Purpose of this chapter

This chapter provides insight into the application of the Work Guide, in which the practical application becomes immediately clear. The aim is to provide insight into the possibilities of using the Work Guide and the role it can play in designing a work of art and the relationship with aspects such as maintenance, management and operation of the object.

11.1.2 Depth of this chapter

This chapter discusses all relevant failure mechanisms that apply to a flood defense artwork. This involves the derivation of the requirements and the way in which these requirements can be converted into a design or taxes for the design. With the help of taxes (specialists) specialists can further develop the design for water defenses, such as determining the provisions for piping or the dimensions of the structural elements of retention devices.

11.2 Description of the situation and the preliminary design

11.2.1 General

Due to the large growth of water sports in the region, it was decided to build an extra waterway between the Black Water and the water sport areas behind it. This is planned in Zwartsluis, where a lock is constructed between the Black Water and the Whaa. From the primary function - passing shipping - a preliminary design was made for the lock. The preliminary design is verified in this case against the requirements of the Water Act and the Building Decree.

This is a fictitious case, because the existing lock the Whaa is already present at the site of the intended object. The case therefore assumes that it does not yet exist.

Since the surge barrier is constructed to enable the passage of ships, the width of the passage opening is already known from this function. The length of the artwork is also a given and is geared to the choice of the turning means (point doors) and the width of the connecting dike bodies.

11.2.2 Location

The object is located in the municipality of Zwartsluis and concerns a work of art in the dike section 9-2 (dike ring 9, Vollenhove. The outside water is the Black Water and on the inside there is a small bowl called the Whaa. The connection of this bowl with the underlying waterways is regulated via the Aremberger lock. Originally this lock was located in the primary flood defense, but in the year 2000 a new primary barrier was installed on the outside.

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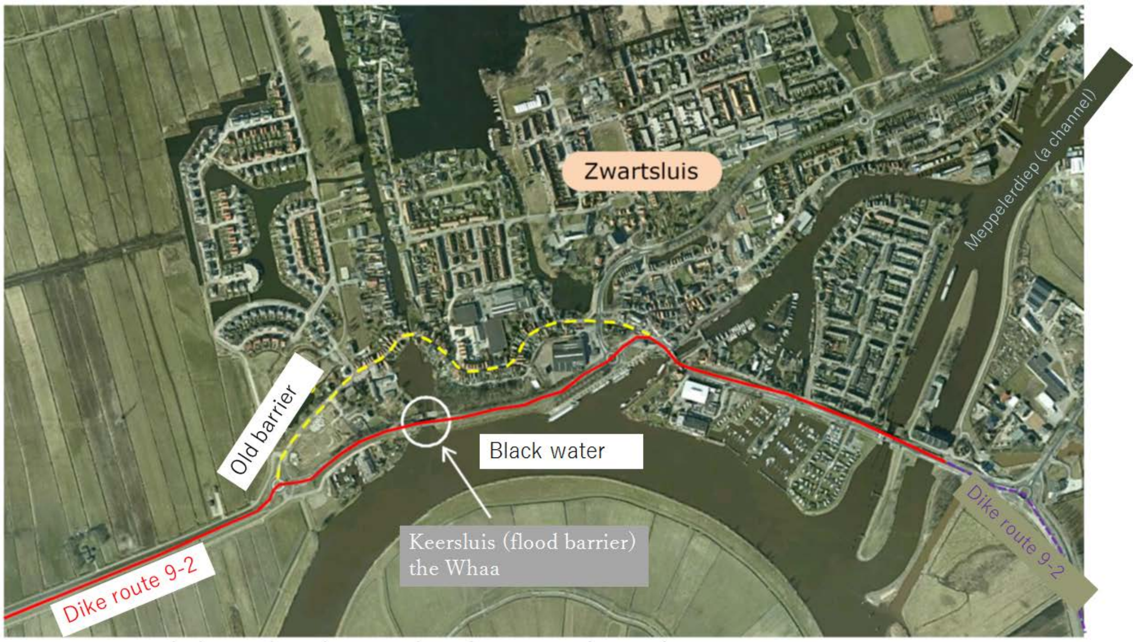

※（Rijkswaterstaat, Central government　2018）196 頁より作成。
Figure 63 Overview of the site lock the Whaa (Source: Google Maps)

The old defense has a vertical height that is at least equal to NAP +2.50 m. The Whaa bowl has an area of approximately 8,750 m^2. When the level of the Whaa rises above the NAP +0.50 m, the bowl is outside its normal banks. At a water level of approximately NAP +1.0 m, there is substantial flooding between the old and new flood defense systems and an area of around 25,000 m^2 is under water.

Behind the Aremberger Lock is the Aremberger Canal, which is in open communication with various large water sport areas, such as the Beulakerwijde, Belterwijde, and the Boschwijde.

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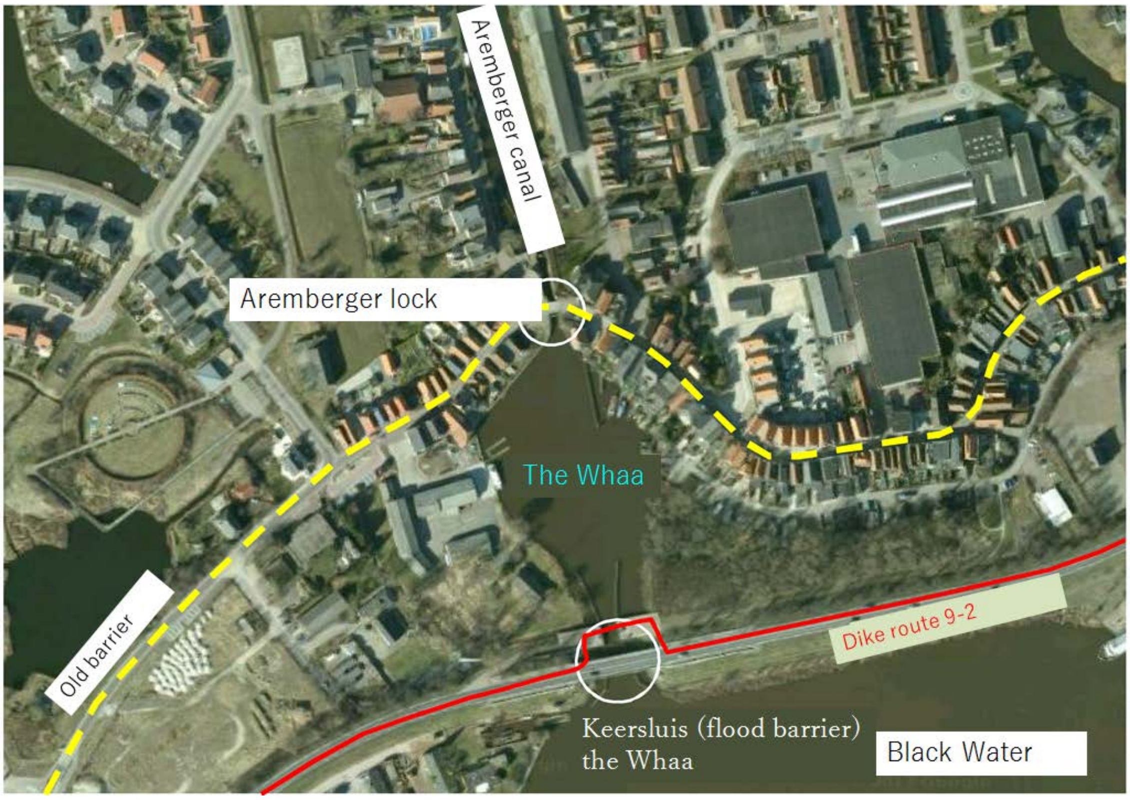


※（Rijkswaterstaat, Central government　2018）197 頁より作成。

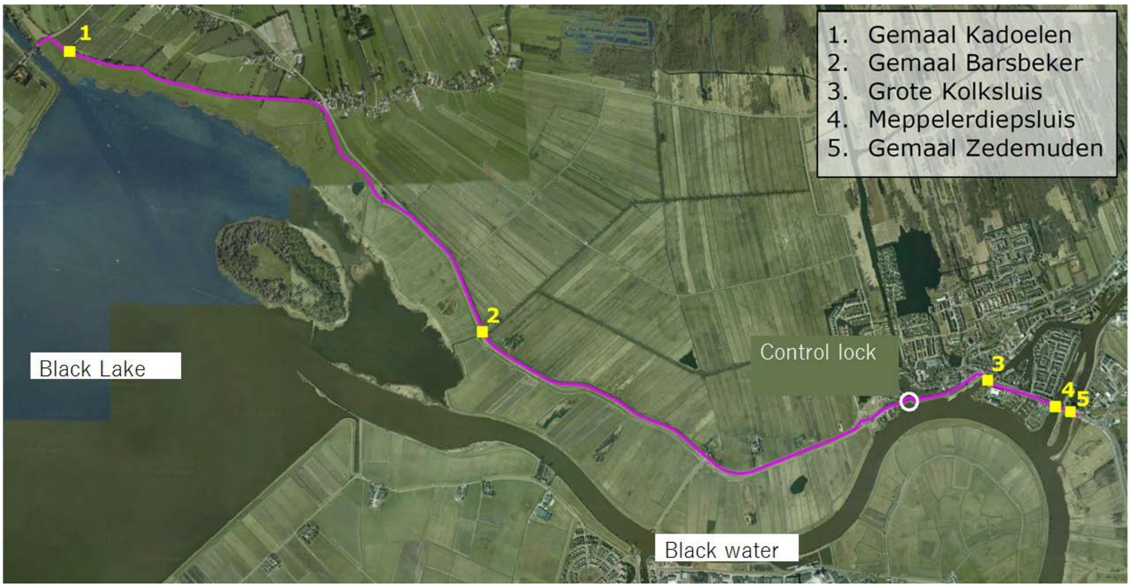
Figure 64 Details location lock the Whaa (Source: Google Maps)

The storage area of the rear area is large and is estimated at $2.0 \cdot 10^7 m^2$. The level rise that can take place here without major consequences is estimated at 0.5 m. Here, the daily level of the rear area is approximately NAP -0.80 m.

11.2.3　Dyke track data

The lock is located in dike section 9-2. The following information applies here:

- Standard according to signaling value: 1 / 3,000 per year
- Standard in accordance with lower limit value: 1 / 1,000 per year
- Artworks in the dyke route: see figure below



※（Rijkswaterstaat, Central government　2018）197 頁より作成。

Figure 65 Artworks in dyke route 9-2 (background Google Maps)

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Of the existing five structures in the dyke section, the following is known globally:

1. Gemaal Kadoelen (a polder pumping station in Amsterdam-Noord: Closing is provided by two independent reversing devices which automatically close when the pump is turned off and automatically open when the pump is switched on. The hinterland consists of pasture.
2. Gemaal Barsbeker (pumping station): This concerns a mortar pumping station with a non-return valve on the outside and an emergency slide in the pumping station that closes automatically in case of high water. This slide can also be operated manually. The lifting point of the auger can be adjusted manually (for example in the case of higher outside water). The pressure cooker of the auger pumping station has dimensions of H × B = 1.25 × 1.5 m. The rear area consists of pasture.
3. Grote Kolksluis (flood defence/lock). This concerns a lock which is open in daily circumstances and thus in particular recreational sailing (commercial vessels only in emergencies) gives the possibility to sail from the Black Water to the Meppelerdiep (a cannal) and vice versa. The lock is permanently closed during the high water season and is therefore not operated. Closing takes place by closing the flood doors in the outer head. The inner head is lower than the outer head.

4. Meppelerdiepsluis (lock). This former lock was reopened in 2017 after four years of renovation, but now as a lock. The outer head is higher than the inner head. With water levels between NAP + 0.47 m and NAP -0.50 m, the lock is just open. Outside of these water levels, the lock is activated and ships are locked.
5. Gemaal Zedemuden (pumping station). This pumping station has several milling operations, each with three reversing devices (check valve and two automatic slides).

11.2.4 Intended dimensions

An inventory of expected shipping (recreational shipping and small vessels) has shown that a passage width of 9.5 m is sufficient. Due to the depth of the passing shipping a bottom height of NAP -3.0 m has been determined.

It was decided to realize the lock with a concrete U-barge with a length of 10 m, with slanted retaining walls on both sides. The lock plateau must be sufficiently high to connect well to the dike bodies on either side. These dike bodies have a minimum deflecting height of NAP + 4.20 m. In order to limit the difference in height with the dike bodies, the lock plateau will have a deflecting height of NAP +3.50 m. The lock plateau has a width of 20 m on both sides of the barrier lock.

11.2.5 Contemplated defense concept

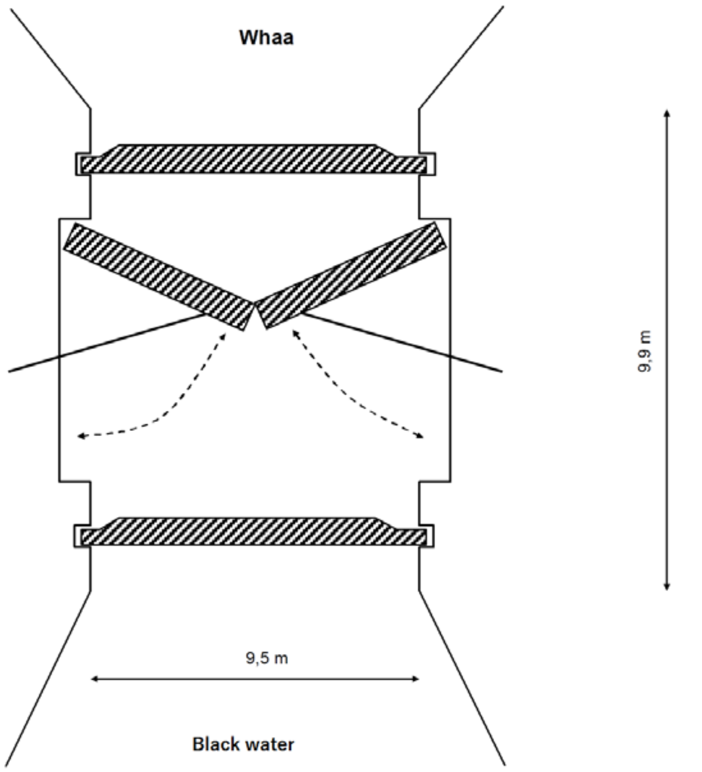
The barrier concept is relatively simple. Under normal circumstances, the site lock is of course open. At higher water levels this is closed with the reversing means. Given that for architectural reasons the reversing means may not be prominently visible. A lifting slide is hereby immediately dispensed with as a turning means.

The Aremberger canal lock normally functions as a lock so long as the lock does not have to be closed. Shunting with the Aremberger lock takes place with the aid of rinkets (lock doors). **(199 頁)** With these rinkets (lock doors), controlled water can also be drained from the bowl towards the Aremberger canal.

The lock should automatically close in case of approaching high water, but the option of manual operation (both locally pressing the button and possibility with the help of a pendulum or the like) should also be implemented. Partly for this reason, the choice was made for point doors as a turning means.

11.2.6 Layout of lock sluice in main lines

It has been indicated from management that baffle rebates must be fitted in order to be able to dry the casing sluice for inspection and maintenance work. With this, the layout of the lock will look as follows:



※（Rijkswaterstaat, Central government　2018）199 頁より作成。

Figure 66 Main dimensions of the lock at the Whaa

11.2.7 Construction and soil building

11.2.7.1. Data construction

From a structural point of view, the choice was made to build the lock in a closed sheet piling and to build on the steel sheet piles that are required for this purpose. An underwater concrete floor of 1.0 m thick with an underside at NAP - 4.9 m is installed. The underwater concrete is applied to a layer of supplementary sand that is applied from a level of NAP - 6.0 m, see also Figure 67.

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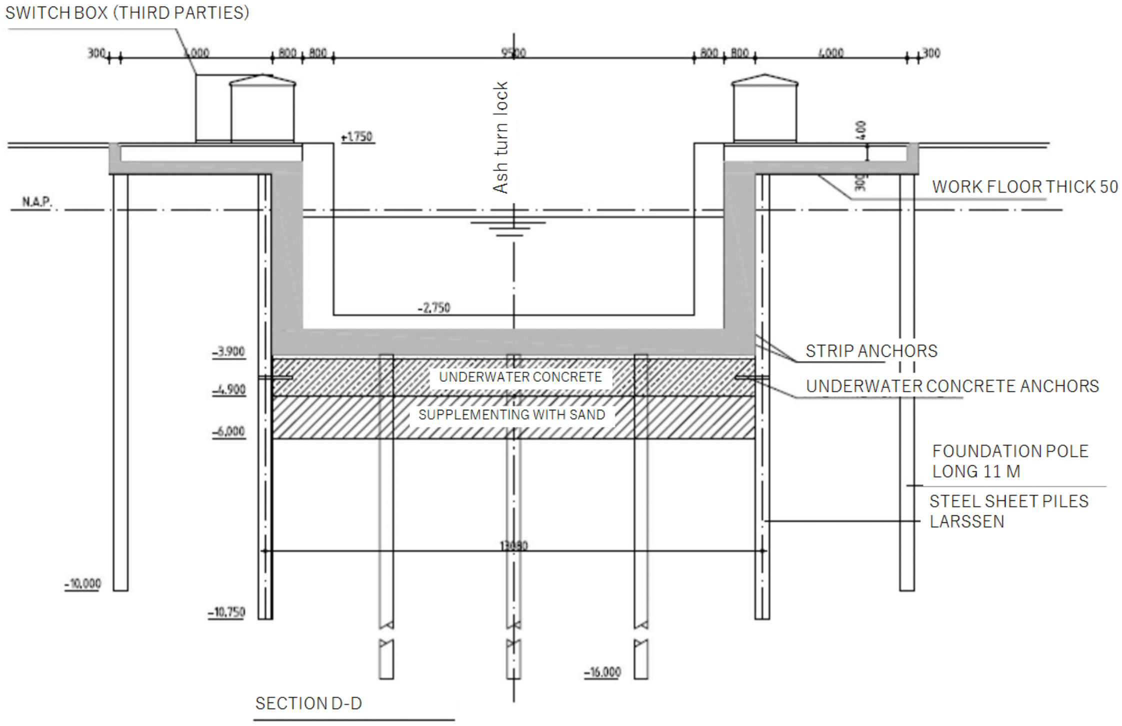

※（Rijkswaterstaat, Central government　2018）200 頁より作成。
Figure 67 Construction pit sluice box lock the Whaa

The abutments are also founded on steel sheet pile screens. The sheet piling screens under the bank lock and the abutments can be seen in the overview drawing below. The required penetration depths were determined by the manufacturer at NAP - 10.75 m for the lock box or NAP - 10.00 m for the abutments.

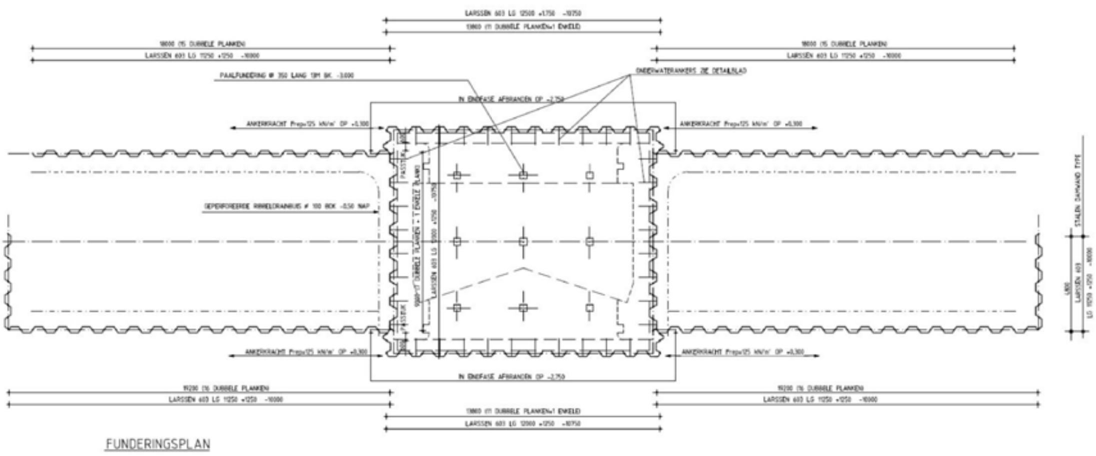

Figure 68 Overview of dam wall screens at the lock the Whaa

11.2.7.2. Soil data

The available soil research does not show a completely unambiguous picture of the subsurface. The most common picture, however, is that a water-retaining layer is present, the bottom of which is at about NAP-5 m to NAP-6 m. Below that, the pleistocene sand layer begins. This consists of moderately fine sand. See also the following probe:

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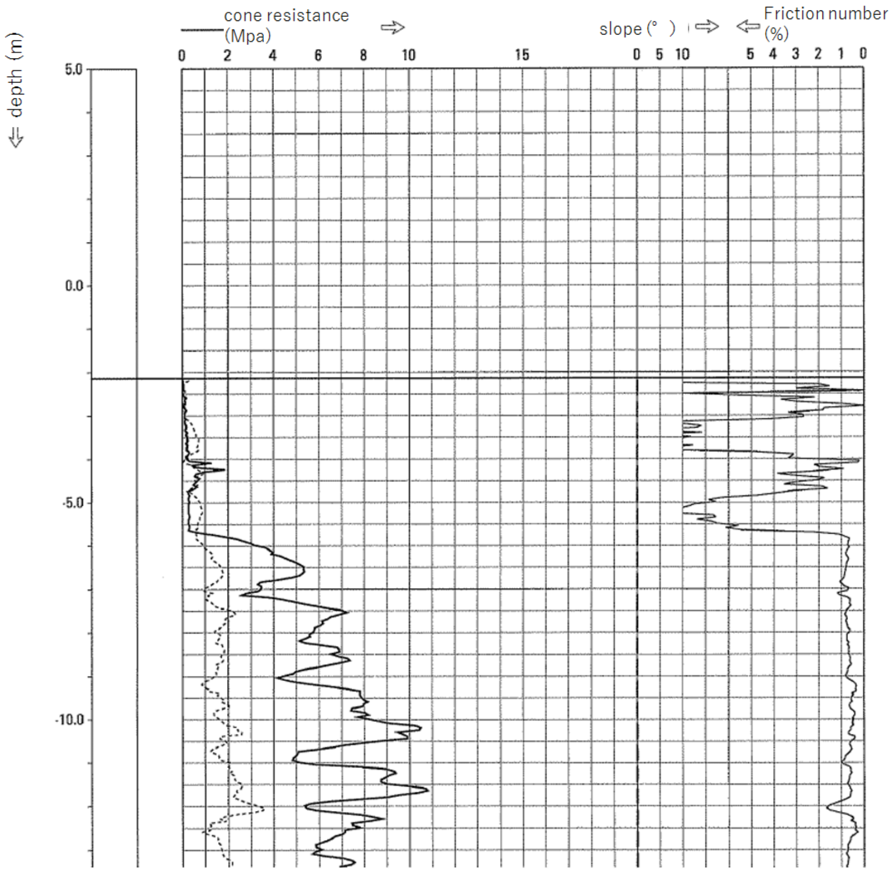

※（Rijkswaterstaat, Central government　2018）201 頁より作成。

Figure 69 Characteristic soil structure around the lock the Whaa

11.2.8　Other data resulting from the preliminary design of the lock

From functions other than water defense, a bottom protection is present both inside and outside the dike of the barrier lock, with a width of 9.5 m at the beginning and 15 m at the end of the soil protection. This soil protection consists of rubble stone with grading 10-60 kg on a zinc piece. The length of this soil protection is 15 meters.

With regard to use, the following is known:

- If a level of NAP + 0.20 m is exceeded on the Black Water, the site lock will be closed automatically with the wooden point doors. After closing the barrier, the internal level is brought back to NAP - 0.20 m. This level is maintained by using the rinkets (lock doors) in the Aremberger lock. This happens automatically.
- From a water level of NAP + 1.5 m, the stability of the Aremberger lock can no longer be guaranteed because the return devices of the Aremberger lock can then no longer reverse the overload.
- The daily water level in the polder behind the Aremberger lock is NAP -0.50 m.

Other starting points are:

- The construction is designed with 2100 visibility year.
- Average outside water level in summer is NAP -0.20 m and winter NAP -0.40 m
- The climate scenario to be taken into account is W $^{+}$
- Capping of the moisture drainage is not taken into account
- Setting and subsidence do not play a role
- Calculated with hydraulic preconditions database WBI2017_Vechtdelta_9-2_v01.sql
- If the export location was chosen: ZW_1_9-2_dk_00389, which is located at a distance of 83 meters from the lock the Whaa

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11.3 Elaboration height (HTKW)

In this section, the required deflecting height of the lock sluice and adjacent abutments is determined. This is done on the basis of the step-by-step plan in section 5.1.3 of the chapter on Elevation.

11.3.1 Step 1 Determine failure probability

The failure probability $P_{eis,\,KW,\,HT}$ is determined using the following formula:

$$P_{eis,KW,HT} = \frac{P_{max} \cdot \omega_{HT}}{N_{dsn}} \qquad 11.1$$

In which:

P_{max} Failure Chance for the entire dike section (standard route) based on the lower limit of the water law = 1 / 1,000 [1 / year]

ω_{HT} Failure probability factor for height = 0.24 [-]

N_{dsn} Length-effect factor for height = 2 for dyke stretch 9-2 [-] (Appendix A of OI2014v4 [Reference 4.2])

This results in a failure probability of 1 / 8.330 per year (1.2E-4 per year).

11.3.2 Step 2 Determine critical flow of soil protection / strength of retardant

Behind the lock a bottom protection is present consisting of rubble stone grading 10-60 kg on a geotextile. The (deep average) critical flow rate u_c is determined here using the Pilarczyk formula (the use of other stability relationships is also allowed, see Chapter 9 Soil Protection):

$$u_c = \sqrt{\frac{2 \cdot g \cdot \Delta \cdot D \cdot \psi_{cr} \cdot k_{sl}}{\varphi_{sc} \cdot 0{,}035 \cdot k_h \cdot k_t^2}} \qquad 11.2$$

Here is:

D [m] Characteristic element dimension. For granular materials: $D = D_{n50}$. A D_{n50} of 0.21 - 0.25 m belongs to a 10-60 kg crushing stone. 0.14 m is used in accordance with CUR 197 ([Reference 11.1]).

φ_{sc} [-] Stability parameter to take into account the influence of transitions and the deviating hydraulic loads that occur here. For φ_{sc}, a value of 0.75 is kept belonging to rubble in a continuous layer with at least two layers of stones.

Δ [-] Relative density $\Delta = (\rho_s - \rho_w) / \rho_w$

ρ_s = specific weight of stones [kg / m^3] = 2650 kg / m^3

ρ_w = specific gravity of water [kg / m^3] = 1000 kg / m^3

Δ = 1.65 [-]

ψ_{cr} [-] Shear stress parameter. ψ_{cr} = 0.035 for granular materials

k_h [-] Depth parameter

$k_h = (1 + h / D)^{-0.2}$ for not fully developed speed profile

Here is:

h = water depth [m] = NAP - 3.0 m - NAP - 0.2 m = 2.8 m

k_r = roughness parameter [-] = 2Dn for crush stone = 0.48 m

Filling yields $k_h = (1 + 2.8 / 0.24)^{-0.2} = 0.60$.

k_{sl} [-] Slope factor = 1 (no incline)

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k_t [-] Turbulence factor. For this, $k_t^2 = 1.5$, which is consistent with increased turbulence (common, non-uniform flow)

g [m / s²] Gravitational acceleration ($g = 9.81$ m / s²)

Filling yields $u_c = 3.4$ m / s.

For the critical transfer / overflow flow q_c that may come from the perspective of the soil protection over the artwork:

$$q_c \cdot B = u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv} \quad ofwel \quad q_c = \frac{u_c \cdot (h_{bi} - h_{bb}) \cdot B_{sv}}{B} \tag{11.3}$$

Here is:

B_{sv} Current carrying width soil protection [m] = 9.5 m

B Width of the crown of the artwork [m] = 9.5 m. In this specific case the transshipment / overflow flow over the abutments does not end up in the lock chamber and thus does not burden the bottom protection behind the lock. So no account needs to be taken here.

u_c Critical flow speed soil protection [m/s] = 3.4 m/s

h_{bi} Inland water level compared to NAP [m] = NAP - 0.20 m

h_{bb} Height top soil protection [m NAP] = NAP - 3.0 m (connect to threshold sluice)

Filling yields $q_c = 9.5$ m³ / s / m.

With such large inflows, dynamic aspects as a result of air inclusions under the overflowing jet may play a role and the stability of the reversing devices cannot be guaranteed. Therefore, the critical inflow rate is maximized at 1.0 m³/s/m (rule of thumb).

11.3.3 Step 3 Determine the return height

With the help of Hydra-NL, the required deflecting height can now be determined. For this purpose Hydra-NL is started up in climate mode. It starts with a sheet pile profile to be defined under the Profile tab. In this screen it is indicated that the angle between north and the axis of the artwork $\psi_{kw} = 170°$. A height does not have to be specified; this is determined by Hydra-NL in the calculation of the hydraulic load level.

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※（Rijkswaterstaat, Central government　2018）204 頁より作成。
Figure 70 Overview of the Profile tab in Hydra-NL

A dike section calculation is then started under the Calculation tab in which the values as determined in steps 1 and 2 are entered (see Figure 71). It should also be noted that if a different visual year is chosen than the hydra-NL 'predefined' visual years 2050 and 2100, the outcomes from Hydra-NL can be linearly interpolated or extrapolated.

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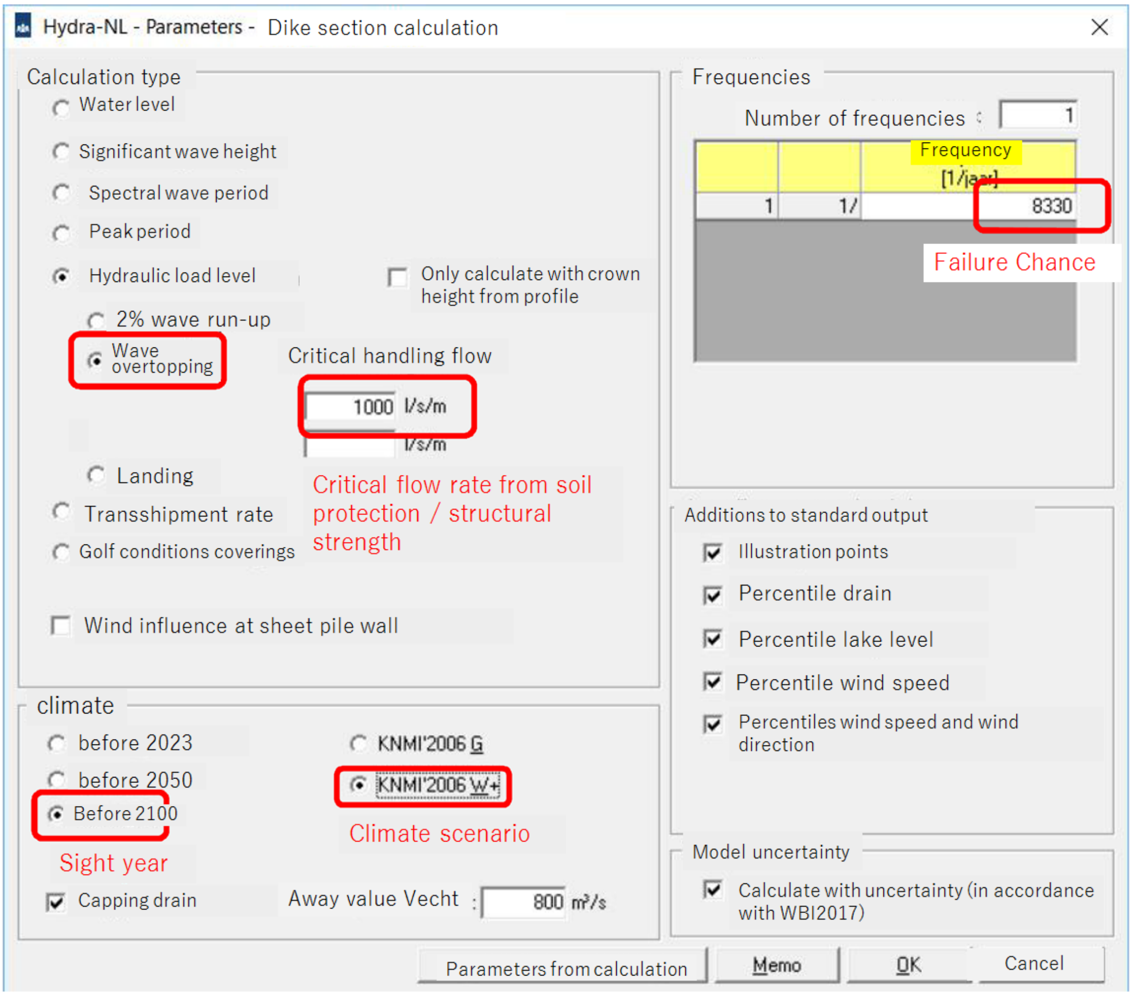
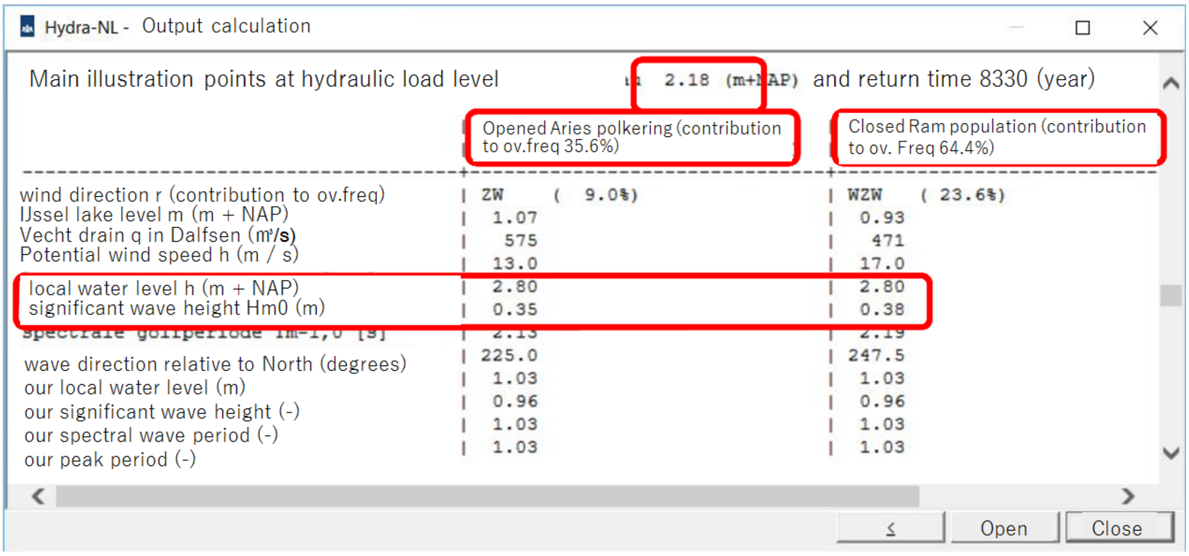


※（Rijkswaterstaat, Central government　2018）205 頁より作成。
Figure 71 Overview input screen calculation Hydra-NL

The result of the calculation is a required crown height of NAP + 2.18 m. Figure 72 shows the main illustration points of the Hydra-NL calculation. This immediately shows that there is overflow, the local water level is well above the flood height. The illustration points are needed in the next step.



※（Rijkswaterstaat, Central government　2018）205 頁より作成。

Figure 72 Main illustration points from calculation Hydra-NL

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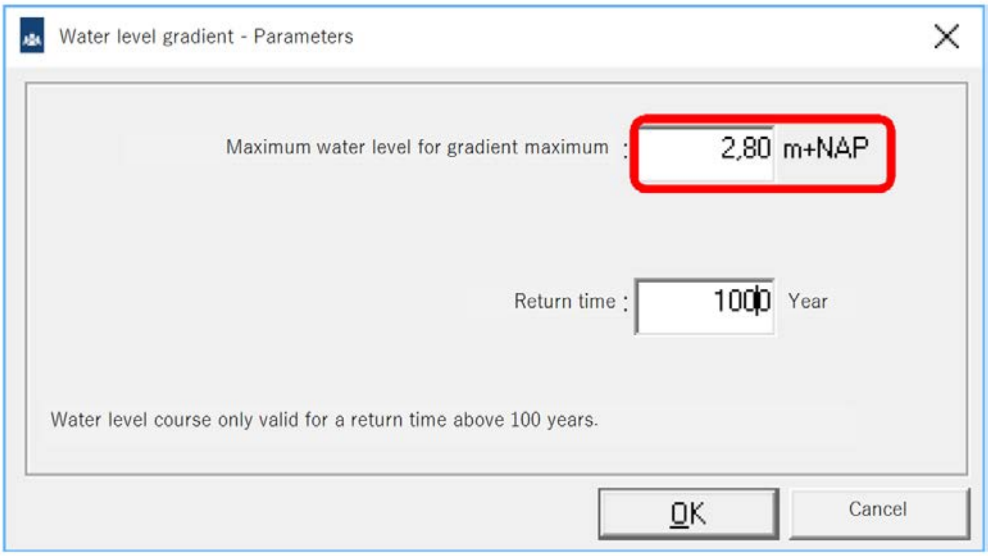
11.3.4　Step 4 Verification of storage capacity

In this step it is verified whether the combining capacity is not exceeded at the calculated turning height from step 3. The data from the illustration point of the Hydra-NL calculation from step 3 is used as input.

With the rinkets (lock doors) of the Aremberger lock, the inflowing flow can be transported to the water sports area behind it. The storage surface is $20 \cdot 10^6$ m^2 and the permissible level rise is 0.5 m, which results in a storage capacity of $10 \cdot 10^6$ m^3.

With the aid of the Water Level Gradient tool, it can be demonstrated whether the storage capacity is not exceeded. This is basically done for both illustration points. Sometimes it can quickly be seen that one of the illustration points is normative. That is not the case here, however. After all, the water level is the same while the wave height is slightly higher (0.35 vs. 0.38 m) with a closed disaster pounding. However, the angle of wave incidence is somewhat less favorable with an opened disaster pounding (77.5° versus 55°). Therefore, the verification is performed for both illustration points. In this case only the normative calculation is presented.

To this end, the water level course is first determined with the aid of the Water Level Course tool. Under the tab preconditions the database VechtIJsseldelta (Vecht IJssel Delta) -WBI2017 is added first. The correct boundary condition is then selected. After this, the water level from the illustration point of the Hydra-NL calculation is entered under the Location - Water level gradient tab (see Figure 73). The return time does not matter for this application.



※（Rijkswaterstaat, Central government　2018）206 頁より作成。
Figure 73 Entry screen tool Water level course

This results in the water level course at the selected output location (see Figure 74). With the Export button the output can be written to an Excel file.

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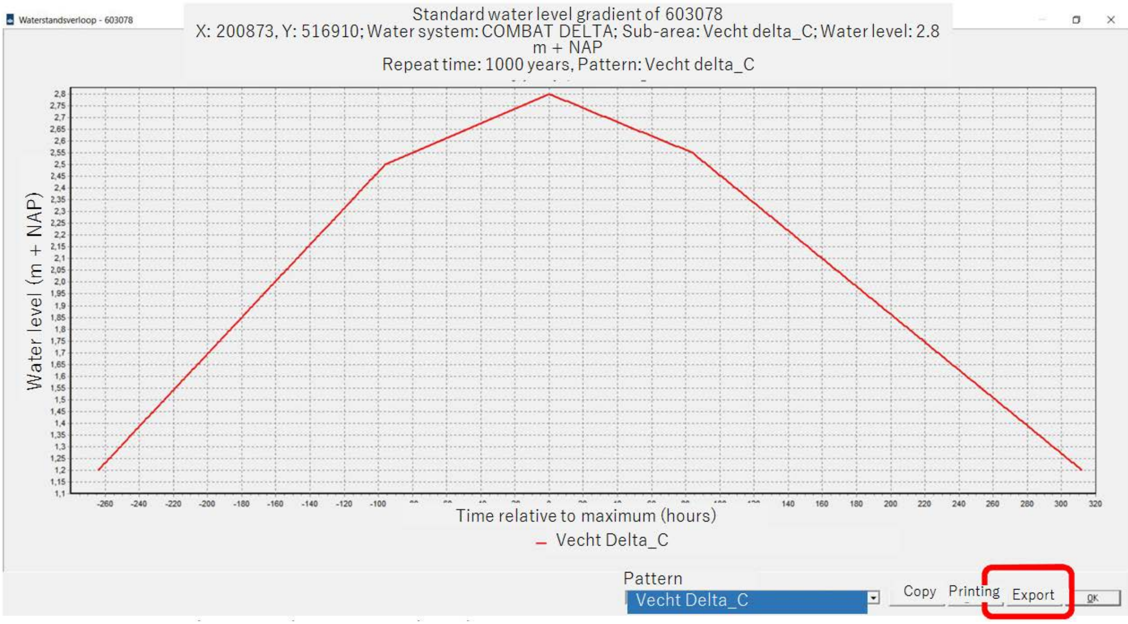

※（Rijkswaterstaat, Central government　2018）207 頁より作成。

Figure 74 Output screen tool Water level course

With the aid of the water levels in this file, an assumption for the course of the wave height during the high tidal wave and the formulas for the transfer/overflow flow rate of section 5.2.3, the inflow volume can be calculated. Spreadsheets have been made available for the Dutch and Zeeland coasts and the Vecht delta at the Helspdesk Water [Ref. 11.3] and [Ref. 11.4] to make these calculations. These can also serve as an example for other areas. Figure 75 shows the completed spreadsheet for the case, with the following workflow:

- Columns A and B contain the water level gradient line as generated with the Water Level Gradient tool
- Row 1-9 contains a block with input data used in the inflow volume calculation:
    - the deflecting height of the retractors is the HBN from the Hydra NL calculation, in this case NAP + 2.18 m
    - the width of the turning means of the artwork (9.5 m)
    - the deflecting height of the abutments, in this case also NAP +3.50 m
    - the width of the abutments of the work of art (2x20 m)
    - the orientation of the artwork (important to determine the angle of wave incidence), in this case 170 °
    - the normative wind direction from the Hydra-NL illustration point (also important to determine the angle of wave incidence), in this case

247.5° . Note: there are load systems (such as the coast) where the wave direction in the illustration point of the Hydra-NL calculation can deviate from the wind direction. The wave direction must be taken over in the spreadsheet and not the wind direction!

  - the wave height from the Hydra-NL illustration point, in this case 0.38 m
  - the gravitational acceleration (9.81 m/s^2)
- Row 11-14 contains a block with calculated parameters based on the input data that are also used in the calculation of the inflow volume (for backgrounds see section 5.2.3):

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  - the angle of wave incidence is calculated on the basis of the orientation of the artwork and the wave direction in the illustration point of the Hydra-NL calculation (note: this does not have to be equal to the wind direction)
  - the reduction factor Y_β is calculated on the basis of the angle of wave incidence (plays a role in the calculation of the overtopping flow)
  - the reduction factor Y_s is calculated on the basis of the angle of wave incidence. This reduces the wave height at the transition from the on-going wave directions.
- In column C the course of the wave height in time is shown. In accordance with section 3.6 of the chapter entitled Hydraulic loads, the wave height follows the time course of the wind set-up. In this case, the wave height of 0 m at T = -24 h increases to 0.38 m at T = 0 and decreases again to 0 m at T = 24 h.
- In columns D and E the inflowing flow per linear meter over the reversing means and abutments is calculated on the basis of the formulas for the transfer / overflow flow rate in section 5.2.3.
- In column F the total flow is calculated that flows in over the entire artwork (reversals + abutments) per second
- In column G, the flow from column F is multiplied by the time duration of a time step (in this case 1 hour but that varies per load system), after which in column H the cumulative flow is given.

Figure 75 shows that the maximum occurring transfer/overflow flow rate is equal to 1.01 m^3/s/m, which according to the imposed transfer/overflow flow rate of 1.0 m^3/s/m is [39]. The total inflow volume over the reversals and abutments amounts to approximately 4,500,000 m^3. This is smaller than the available storage capacity of

20,000,000 m² x 0.50 m = 10,000,000 m³. This completes the design process, a height of (rounded) NAP + 2.20 m is sufficient for this artwork.

The above consideration is based on the assumption that the inflowing flow through the rinkets (lock doors) of the Aremberger lock can be completely removed. Suppose that this is not the case and that a more detailed consideration of incoming and outgoing volumes has shown that the inflow volume may not exceed 3,000,000 m³. In that case, the available cup storage would not be sufficient. For the purpose of this case, this fictitious value of 3,000,000 m³ will be continued and the design process will be continued with step 5 of the step-by-step plan.

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[39] This does not work on the coast because of the phase shift between intent and tide (see section 3.6.1). Here, however, the maximum wave height can be manually combined with the highest water level for verification.

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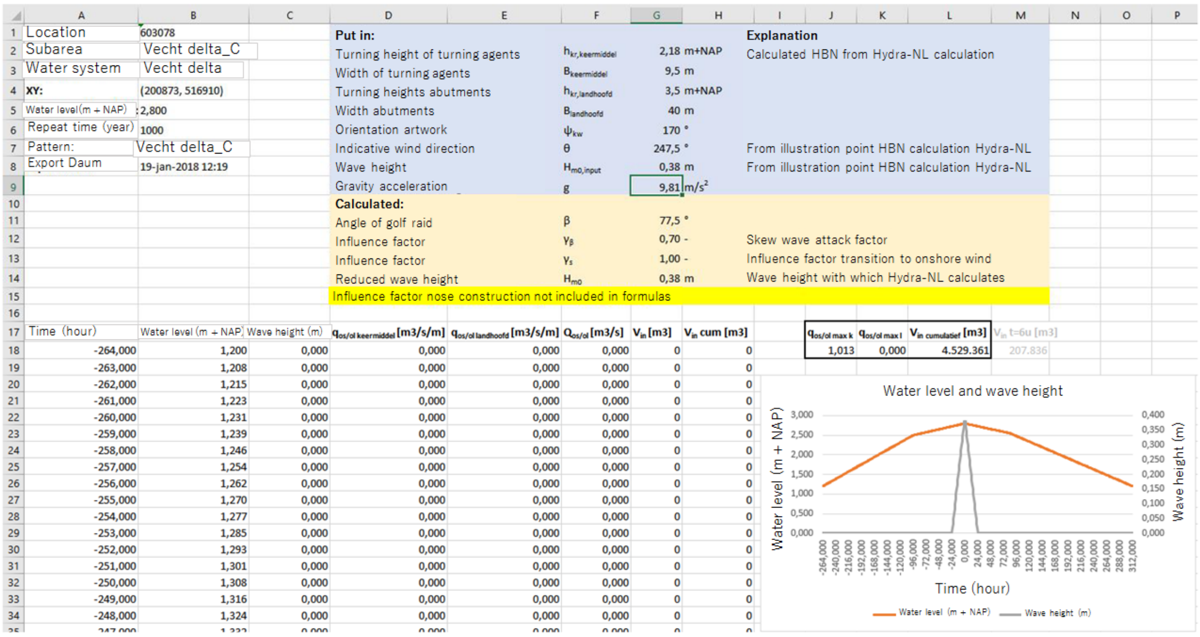

Location	603078	Put in:				Explanation
Subarea	Vecht delta_C	Turning height of turning agents	$h_{kr,keermiddel}$	2,18	m+NAP	Calculated HBN from Hydra-NL calculation
Water system	Vecht delta	Width of turning agents	$B_{keermiddel}$	9,5	m	
XY:	(200873, 516910)	Turning heights abutments	$h_{kr,landhoofd}$	3,5	m+NAP	
Water level(m + NAP)	2,800	Width abutments	$B_{landhoofd}$	40	m	
Repeat time (year)	1000	Orientation artwork	ψ_{kw}	170	°	
Pattern:	Vecht delta_C	Indicative wind direction	θ	247,5	°	From illustration point HBN calculation Hydra-NL
Export Daum	19-jan-2018 12:19	Wave height	$H_{m0,input}$	0,38	m	From illustration point HBN calculation Hydra-NL
		Gravity acceleration	g	9,81	m/s²	
		Calculated:				
		Angle of golf raid	β	77,5	°	
		Influence factor	γ_β	0,70	-	Skew wave attack factor
		Influence factor	γ_s	1,00	-	Influence factor transition to onshore wind
		Reduced wave height	H_{m0}	0,38	m	Wave height with which Hydra-NL calculates
		Influence factor nose construction not included in formulas				

$q_{os/ol\ max\ k}$	$q_{os/ol\ max\ l}$	$V_{in\ cumulatief}$ [m3]	V_{in} t=6u [m3]
1,013	0,000	4.529.361	207.836

Time (hour)	Water level (m + NAP)	Wave height (m)	$q_{os/ol\ keermiddel}$ [m3/s/m]	$q_{os/ol\ landhoofd}$ [m3/s/m]	$Q_{os/ol}$ [m3/s]	V_{in} [m3]	V_{in} cum [m3]
-264,000	1,200	0,000	0,000	0,000	0,000	0	0
-263,000	1,208	0,000	0,000	0,000	0,000	0	0
-262,000	1,215	0,000	0,000	0,000	0,000	0	0
-261,000	1,223	0,000	0,000	0,000	0,000	0	0
-260,000	1,231	0,000	0,000	0,000	0,000	0	0
-259,000	1,239	0,000	0,000	0,000	0,000	0	0
-258,000	1,246	0,000	0,000	0,000	0,000	0	0
-257,000	1,254	0,000	0,000	0,000	0,000	0	0
-256,000	1,262	0,000	0,000	0,000	0,000	0	0
-255,000	1,270	0,000	0,000	0,000	0,000	0	0
-254,000	1,277	0,000	0,000	0,000	0,000	0	0
-253,000	1,285	0,000	0,000	0,000	0,000	0	0
-252,000	1,293	0,000	0,000	0,000	0,000	0	0
-251,000	1,301	0,000	0,000	0,000	0,000	0	0
-250,000	1,308	0,000	0,000	0,000	0,000	0	0
-249,000	1,316	0,000	0,000	0,000	0,000	0	0
-248,000	1,324	0,000	0,000	0,000	0,000	0	0

※（Rijkswaterstaat, Central government　2018）209 頁より作成。

Figure 75 Example spreadsheet calculation inflow volume

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11.3.5　Step 5 Determine the height at which available storage space is not exceeded

With the aid of the calculation sheet from step 4 it is easy to determine the required deflecting height that is required to not exceed the available storage space. Here, the designer usually has the choice to only increase the turning means or to increase both abutments and headers. In this case, the abutments are already so high that no transhipment takes place here. The only option in this case is to increase the reversals.

If the reversing means are increased to NAP + 2.28 m, the inflow volume is 3,000,000 m^3. This is rounded off to NAP + 2.30 m. With this value, the case is continued.

Put in:			Explanation
Turning height of turning agents	$h_{kr,keermiddel}$	2,28 m+NAP	Calculated HBN from Hydra-NL calculation
Width of turning agents	$B_{keermiddel}$	9,5 m	
Turning heights abutments	$h_{kr,landhoofd}$	3,5 m+NAP	
Width abutments	$B_{landhoofd}$	40 m	
Orientation artwork	ψ_{kw}	170 °	
Indicative wind direction	θ	247,5 °	From illustration point HBN calculation Hydra-NL
Wave height	$H_{m0,input}$	0,38 m	From illustration point HBN calculation Hydra-NL
Gravity acceleration	g	9,81 m/s^2	
Calculated:			
Angle of golf raid	β	77,5 °	
Influence factor	γ_β	0,70 -	Skew wave attack factor
Influence factor	γ_s	1,00 -	Influence factor transition to onshore wind
Reduced wave height	H_{m0}	0,38 m	Wave height with which Hydra-NL calculates
Influence factor nose construction not included in formulas			

$q_{os/ol\ keermiddel}$ [m3/s/m]	$q_{os/ol\ landhoofd}$ [m3/s/m]	$Q_{os/ol}$ [m3/s]	V_{in} [m3]	V_{in} cum [m3]	$q_{os/ol\ max\ k}$	$q_{os/ol\ max\ l}$	$V_{in\ cumulatief}$ [m3]	V_{in} t=6u [m
0,000	0,000	0,000	0	0	0,800	0,000	3.084.920	164.179

※（Rijkswaterstaat, Central government　2018）211 頁より作成。

Figure 76 Adjusting the height of the wall in such a way that the storage capacity is not exceeded

11.3.6 Step 6 Determine recurring height with Hydra-NL

In this step Hydra-NL determines the clearance height corresponding to the maximum transfer/overflow flow rate of 0.800 m^3/s/m from step 5. The Hydra-NL input screen is the same as in Figure 71, only for critical transfer -/overflow rate is now 800 l /s/m entered.

From the Hydra-NL calculation follows an HBN of also NAP + 2.28 m. The illustration point is as follows:

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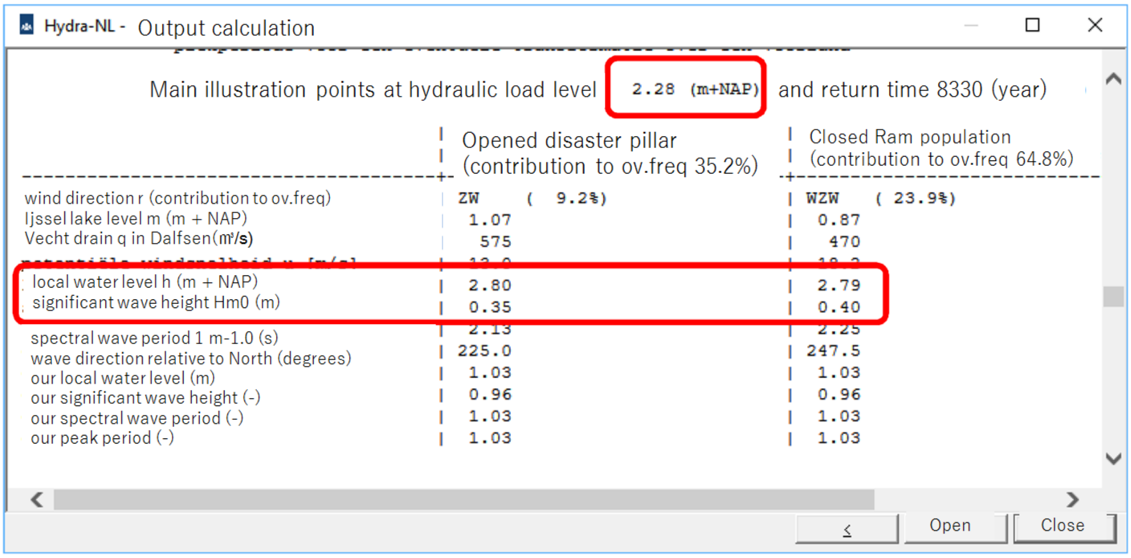


※（Rijkswaterstaat, Central government　2018）212 頁より作成。

Figure 77 Main illustration points from calculation Hydra-NL with adapted critical transfer/overflow flow

It can be seen that the water level in the illustration point is (almost) the same and also the wave height does not differ much (0.38 m to 0.40 m). It can therefore already be assumed in advance that the cup storage is not exceeded (step 7). For the sake of completeness, step 7 will still be completed.

11.3.7 Step 7 Check that cup storage is not exceeded

For the sake of brevity, the wave height of 0.40 m associated with the illustration point Closed disaster pounding is combined with the water level of NAP 2.80 m associated with the illustration point Opened disaster pounding. The most unfavorable angle of wave incidence (225°) is also chosen from both illustration points. This results in an inflowing volume of approximately 2,850,000 m^3. The storage capacity is now not exceeded. A height of NAP + 2.30 m is sufficient.

Put in:			Explanation
Turning height of turning agents	$h_{kr,keermiddel}$	2,3 m+NAP	Calculated HBN from Hydra-NL calculation
Width of turning agents	$B_{keermiddel}$	9,5 m	
Turning heights abutments	$h_{kr,landhoofd}$	3,5 m+NAP	
Width abutments	$B_{landhoofd}$	40 m	
Orientation artwork	ψ_{kw}	170 °	
Indicative wind direction	θ	225 °	From illustration point HBN calculation Hydra-NL
Wave height	$H_{mo,input}$	0,40 m	From illustration point HBN calculation Hydra-NL
Gravity acceleration	g	9,81 m/s²	
Calculated:			
Angle of golf raid	β	55 °	
Influence factor	γ_β	0,82 -	Skew wave attack factor
Influence factor	γ_s	1,00 -	Influence factor transition to onshore wind
Reduced wave height	H_{mo}	0,40 m	Wave height with which Hydra-NL calculates
Influence factor nose construction not included in formulas			

$q_{os/ol\ keermiddel}$ [m3/s/m]	$q_{os/ol\ landhoofd}$ [m3/s/m]	$Q_{os/ol}$ [m3/s]	V_{in} [m3]	V_{in} cum [m3]		$q_{os/ol\ max\ k}$	$q_{os/ol\ max}$	$V_{in\ cumulatief}$ [m3]	V_{in} t=6u [m3]
0,000	0,000	0,000	0	0		0,767	0,000	2.835.671	157.510

※（Rijkswaterstaat, Central government　2018）212 頁より作成。

Figure 78 Check whether the combing capacity is not exceeded

It should be noted that in this case the illustration point in the adapted Hydra-NL calculation deviates little from the first calculation. That does not always have to be the case! That is why an example is given in the next section where this is not the case.

11.3.8 Elaboration with smaller cup storage

Suppose that the Aremberger lock is closed simultaneously with the closure of the barrier lock. In that case the available storage space is determined by the maximum permissible water level of NAP + 1.50 m in connection with the stability of the Aremberger lock. **(213 頁)** The available combing capacity is then NAP + 1.50 m - NAP + 0.20 m (closing level) = 1.30 m x 25.000 m² = 32.500 m³.

Steps 1 through 4 are identical, so that the verification is continued with step 5.

Step 5 Determine the height at which available storage space is not exceeded with the help of the calculation sheet from step 4 the required turning height is again required for the storage capacity not to exceed. A height of NAP + 2.85 m is then required. The case is continued with this value.

Put in:			Explanation
Turning height of turning agents	$h_{kr,keermiddel}$	2,85 m+NAP	Calculated HBN from Hydra-NL calculation
Width of turning agents	$B_{keermiddel}$	9,5 m	
Turning heights abutments	$h_{kr,landhoofd}$	3,5 m+NAP	
Width abutments	$B_{landhoofd}$	40 m	
Orientation artwork	ψ_{kw}	170 °	
Indicative wind direction	θ	225 °	
Wave height	$H_{m0,input}$	0,40 m	From illustration point HBN calculation Hydra-NL
Gravity acceleration	g	9,81 m/s^2	From illustration point HBN calculation Hydra-NL
Calculated:			
Angle of golf raid	β	55 °	
Influence factor	γ_β	0,82 -	Skew wave attack factor
Influence factor	γ_s	1,00 -	Influence factor transition to onshore wind
Reduced wave height	H_{mo}	0,40 m	Wave height with which Hydra-NL calculates

Influence factor nose construction not included in formulas

$q_{os/ol\ keermiddel}$ [m3/s/m]	$q_{os/ol\ landhoofd}$ [m3/s/m]	$Q_{os/ol}$ [m3/s]	V_{in} [m3]	V_{in} cum [m3]		$q_{os/ol\ max\ k}$	$q_{os/ol\ max\ l}$	$V_{in\ cumulatief}$ [m3]	V_{in} t=6u [m3]
0,000	0,000	0,000	0	0		0,065	0,000	32.327	13.408

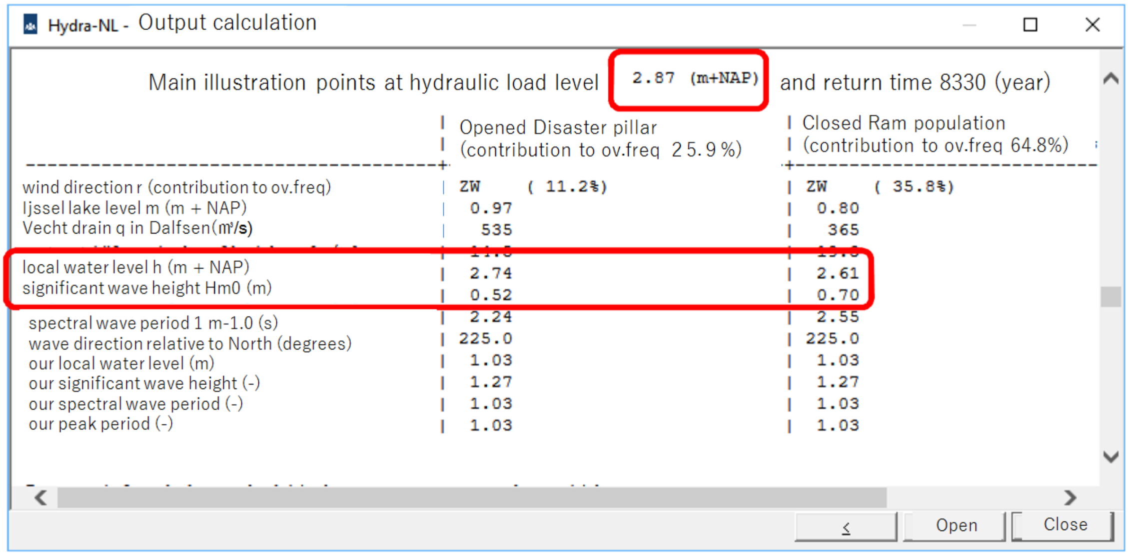
※（Rijkswaterstaat, Central government　2018）213 頁より作成。

Figure 79 Adjusting the height of the wall in such a way that the storage capacity is not exceeded

Step 6 Determine the return height with Hydra-NL

In this step Hydra-NL determines the deflection height corresponding to the maximum transfer / overflow flow rate of 0.065 m³ / s / m from step 5. The input screen of Hydra-NL is the same as in Figure 71, only for the critical transfer - / overflow flow rate is now entered 65 l / s / m.

From the Hydra-NL calculation follows an HBN of NAP + 2.87 m. This is in line with the calculated required crest height of NAP + 2.85 m on the basis of the water level and wave height from the illustration point from step 4. illustration point now looks like this:



※（Rijkswaterstaat, Central government　2018）213 頁より作成。

Figure 80 Main illustration points from calculation Hydra-NL with adapted critical transfer/overflow flow

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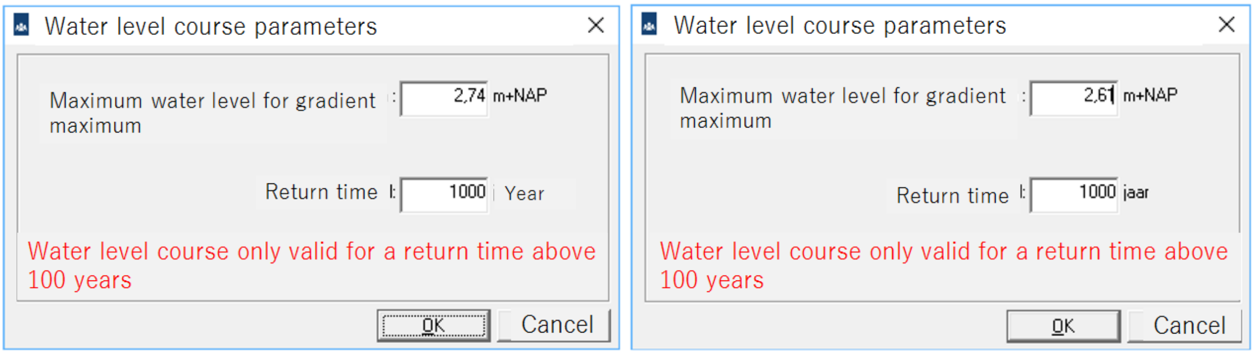
It can be seen that the water level deviates in particular from the illustration point associated with a closed Disaster pounding (NAP + 2.61 m around NPA + 2.80 m). The same applies to the wave height (0.70 m to 0.38 m). The water level (NAP + 2.74 m around NAP + 2.80 m) and wave height (0.52 m by 0.38 m) also slightly deviate from the illustration point that is associated with an opened Disaster pounding.

Note: this step looks a bit different on the coast. Due to the phase shift between the tide and water level set-up (see section 3.6.1), the highest water level must be combined manually with the maximum wave height. This transhipment flow, which is greater than the maximum transhipment flow from the storage container, must be entered in Hydra-NL. In Hydra-NL, the phase shift between tide and water level set-up is not taken into account.

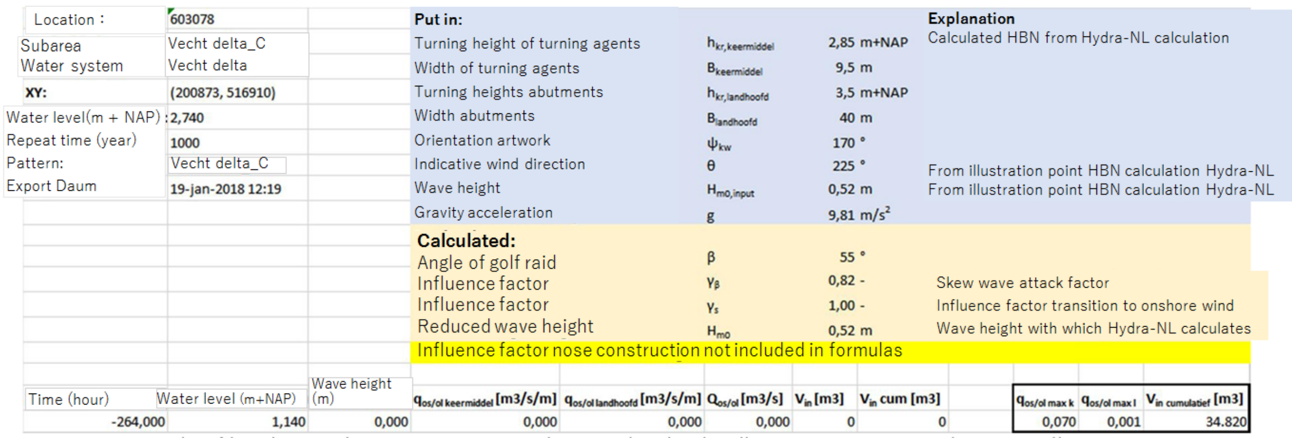
Step 7 Check whether cup storage is not exceeded Started by determining the water level course associated with a water level of NAP + 2.61 m and NAP + 2.74 m respectively. The output is exported to an Excel file.



※（Rijkswaterstaat, Central government　2018）214 頁より作成。

Figure 81 Entry screen new calculation tool Water level course

Started with the illustration point that belongs to an opened Disaster pounding. It can be seen that with a deflecting height of NAP + 2.85 m the capacity of 32.500 m³ is exceeded slightly (inflow volume around 35,000 m³):



※（Rijkswaterstaat, Central government　2018）214 頁より作成。

Figure 82 Checking whether combing power is not exceeded at illustration point opened

Disaster pounding and crown height 2.85 m + NAP at a returning height of NAP + 2.86 m, the available storage space is sufficient. This is almost equal to the calculated HBN of NAP + 2.87 m.

The illustration point associated with a closed disaster pounding is also checked. It can be seen that with a deflecting height of NAP + 2.85 m, the composting capacity of 32,500 m³ is also just exceeded (inflowing volume around 34,000 m³). **(215 頁)** At a returning height of NAP + 2.86 m, the available storage space now also meets the requirements.

Location :	603078
Subarea	Vecht delta_C
Water system	Vecht delta
XY:	(200873, 516910)
Water level(m + NAP)	2,610
Repeat time (year)	1000
Pattern:	Vecht delta_C
Export Daum	19-jan-2018 12:19

Put in:			Explanation
Turning height of turning agents	$h_{kr,keermiddel}$	2,85 m+NAP	Calculated HBN from Hydra-NL calculation
Width of turning agents	$B_{keermiddel}$	9,5 m	
Turning heights abutments	$h_{kr,landhoofd}$	3,5 m+NAP	
Width abutments	$B_{landhoofd}$	40 m	
Orientation artwork	ψ_{kw}	170 °	
Indicative wind direction	θ	225 °	From illustration point HBN calculation Hydra-NL
Wave height	$H_{mo,input}$	0,70 m	From illustration point HBN calculation Hydra-NL
Gravity acceleration	g	9,81 m/s²	
Calculated:			
Angle of golf raid	β	55 °	
Influence factor	γ_β	0,82 -	Skew wave attack factor
Influence factor	γ_s	1,00 -	Influence factor transition to onshore wind
Reduced wave height	H_{mo}	0,70 m	Wave height with which Hydra-NL calculates
Influence factor nose construction not included in formulas			

Time (hour)	Water level (m+NAP)	Wave height (m)	$q_{os/ol\ keermiddel}$ [m3/s/m]	$q_{os/ol\ landhoofd}$ [m3/s/m]	$Q_{os/ol}$ [m3/s]	V_{in} [m3]	V_{in} cum [m3]	$q_{os/ol\ max\ k}$	$q_{os/ol\ max\ l}$	$V_{in\ cumulatief}$ [m3]
-264,000	1,010	0,000	0,000	0,000	0,000	0	0	0,068	0,002	33.991

※（Rijkswaterstaat, Central government　2018）215 頁より作成。

Figure 83 Checking whether combing power is not exceeded at illustration point closed Disaster pounding and crown height 2.85 m + NAP

Given the fact that the formulas from the Guidelines for Art Works 2003 with which the transfer/overflow flow rate is determined to be somewhat conservative, a crest height of NAP + 2.85 m is considered appropriate for this artwork.

11.3.9 Tightening with the help of EurOtop2016

With the rules from the EurOtop2016 the required crown height can be determined even more sharply. This can be done by using Figure 84:

- Determine the relative freeboard of the barrier R_c/H_{m0}. In this case the freeboard is equal to NAP + 2.87 m (the required crest height calculated with Hydra-NL) - NAP + 2.74 m + NAP (water level in the illustration point of the Hydra-NL calculation open disaster pounding) = 0, 13 m. The wave height in the illustration point of the Hydra-NL calculation is 0.52 m. The relative freeboard therefore amounts to 0.13 / 0.52 = 0.25. For the illustration point of the closed Disaster pounding the relative freeboard is 0.26 / 0.70 = 0.37. The latter is normative.
- The relative throughput rate $q/(gH_{m0}{}^3)^{0,5}$ is $0.065/(9.81 * 0.70^3)^{0.5}$ = 3.54E-2 in the illustration point of the closed Disaster pounding and 0.065 / $(9.81 * 0.52^3)^{0,5}$ = 5.87E-2 in the illustration point of the opened Disaster pounding. As a check, these values are shown in Figure 84 (green = situation open Disaster pounding, blue = closed Disaster pounding).
- In the case of a relative overtopping rate of 3.54E-2, a relatively freeboard of 0.17 should be used if the formulas from EurOtop2016 are used (see dotted

blue line from the figure below). This means a freeboard of 0.17 * 0.7 m = 0.12 m. The required crown height is NAP + 2.61 m + 0.12 m = NAP + 2.73 m. This is 0.14 m lower than the crest height of NAP + 2.87 m calculated with Hydra-NL. To check: the difference in the relative freeboard is 0.20, which also amounts to 0.20 * 0.7 = 0.14 m.

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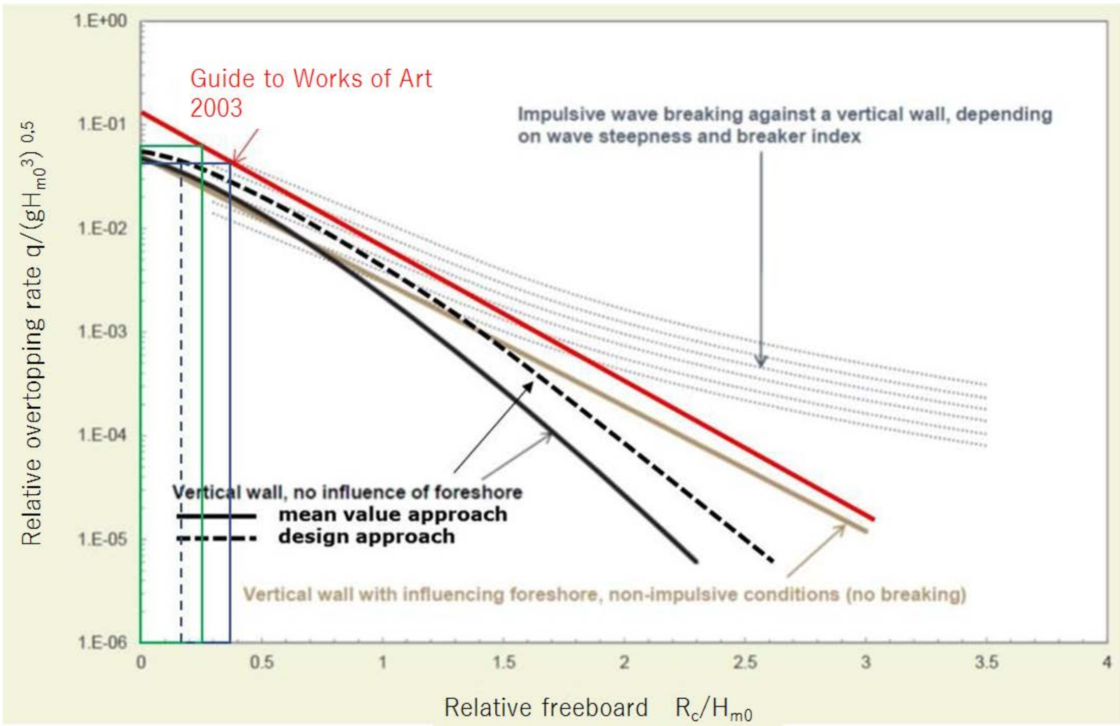

※（Rijkswaterstaat, Central government　2018）216 頁より作成。

Figure 84 Tightening required crown height on the basis of EurOtop2016

The same result is found when formula 7.2 is used from EurOtop2016:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0{,}054 \cdot exp\left[-\left(2{,}12\frac{R_c}{H_{m0}}\right)^{1{,}3}\right]$$

Describing provides the following formula:

$$R_c = 0{,}47H_{m0}\left[-\ln\left(\frac{q}{0{,}054\sqrt{gH_{m0}^3}}\right)\right]^{0{,}77}$$

Filling yields (with q = 0.065 m³ / s / m and H_{m0} = 0.70 m) also here an R_c of 0.17, or a freeboard of 0.17x0.7 = 0.12 m and a required crown height of NAP + 2.73 m.

11.4 Do not close elaboration (BSKW)

In this chapter, the configuration of the reversing devices is determined. This is done on the basis of the step-by-step plan in section 4.1.3 of the Non-closing chapter.

The primary function of the artwork concerns the passage of shipping traffic and from that function the artwork should always be open under normal circumstances. This implies that the function of high-water traffic during the primary function is not filled. In order to activate this function, certain actions are required, such as signaling the closing level, curling shipping and closing the turning gear.

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11.4.1 Step 1 Simple design rules

In this step, it is determined whether the functioning and configuration of the reversing means can be controlled in such a way that the failure mechanism cannot be tapped at the site lock by means of simple design rules. A further analysis of the failure mechanism is no longer necessary in that case, because it is already clear in advance that the artwork with that particular operation and configuration has a negligible contribution to the risk of flooding.

Looking at the primary function of the artwork, the simple design rules do not apply to this site lock. The primary process, the passage of ships, does not allow the artwork to always be closed high water. Specifically, this failure mechanism must be designed.

11.4.2 Step 2 Determine failure probability

The failure probability $P_{eis,\,KW,\,N}S$ is determined with the help of the following formula, where the basis for the values of the various parameters is given below:

$$P_{eis,KW,NS} = \frac{P_{max} \cdot \omega_{HT}}{N_{NS}} = \frac{\frac{1}{1.000} \cdot 0{,}04}{4} = 1{,}0 \cdot 10^{-5}\ per\ jaar \qquad 11.4$$

In which:

P_{max}	Failure Chance for the entire dike section (standard route) based on the lower limit of the water law = 1/1,000 [1/year]
ω_{HT}	Failure probability factor for closing reliability = 0.04 [-]
N_{NS}	Length effect factor for closing reliability = 4

The value for the length-effect factor was determined on the basis of an analysis of the other structures in the dyke section. There are 5 artworks in the process. These are further checked below for non-closing.

1. Gemaal Kadoelen (a polder pumping station in Amsterdam-Noord): Because the closure is provided by two independent retractors and these reversing devices close automatically when the pump stops, the failure probability contribution of this artwork to non-closing is negligibly small.
2. Gemaal Barsbeker (pumping station): The auger pumping station has a non-return valve that automatically closes after jacking and also an emergency slide that automatically closes when a high water is present. As a result, there are no two independent reversing devices that close after each meal request and thus turn the work of art high water twice outside of the time used. However, it is known from VNK's analysis that the probability of failure is not negligible. The reasons for this are the height of the discharge points, the presence of a non-return valve, the presence of an emergency gate that closes automatically in the event of high water or when the non-return valve does not close and the limited flow area between auger and trainer. The artwork is not taken into account in the length effect for not closing due to the above.
3. Grote Kolksluis (flood defence/lock). This lock is closed as standard during the high water season. In the summer season, however, this is not the case and it has one set of high water-pivoting point doors. A second barrier can then be built up with the aid of bulkheads. For this artwork applies that this is included in the length-effect for not closing.
4. Meppelerdiepsluis (lock). This lock is fully open at certain water levels. Outside it is shaken and the outer head can turn a higher water level than the inner head. On this basis it cannot be said that the artwork has a negligible failure probability with regard to non-closing and is therefore included in the length effect.
5. Gemaal Zedemuden (pumping station). Each mill has three reversals, two of which are not entirely independent. This in any case leads to two independent reversing means that close when the pump (s) stop grinding.

(218 頁) The failure probability contribution with a view to non-closing is therefore negligible and the artwork is not included in the length effect.

The lock is artwork number 6 in the dyke section. Finally, the manager indicates that it is quite possible that in the future another work of art (inlet sluice) will be added to the dike section.

On the basis of the above, four works of art with a non-negligible contribution to the failure probability for non-closure are taken into account (Grote Kolksluis (pumping station), Meppelerdiepsluis (lock), Keersluis (flood barrier) the Whaa and the possible future artwork). This makes the length-effect factor NNS equal to 4.

From the above follows a failure probability for not closing 1/100,000 (= 1.0E-5) per year (see formula 10.4).

11.4.3 Step 3 Determine maximum admissible inflow volume (cup storage)

When the casing lock does not close and the outside water level rises, water flows through the casing sluice and enters the bowl between the casing sluice and the Aremberger lock. The contents of this bowl have the following characteristics (assuming a closing level of NAP +0.20 m):

Table 21 Water levels in relation to cup storage

Maximum indoor water level	Coming surface	Compressive Capacity	Effects
NAP + 0,50 m	8.750 m²	2.625 m³	Bowl filled to edge banks
NAP + 1,00 m	25.000 m²	20.000 m³	Substantial flooding in bowl
NAP + 1,50 m	25.000 m²	32.500 m³	The security of the Aremberger lock is no longer guaranteed and substantial flooding in the basin

※（Rijkswaterstaat, Central government　2018）218 頁より作成。

When the rinkets (lock doors) of the Aremberger lock are opened when the closing level is reached, a larger cup-storing surface is available. Initially, only the cup storage in the bowl is assumed, with the maximum admissible water level NAP + 1.50 m in the bowl.

11.4.4 Step 4 Determine critical flow from soil protection

The critical flow from the soil protection on the inside depends in the first instance on the critical flow rate with respect to this soil protection. This critical flow rate has already been determined in section 11.3.2. It follows that $u_c = 3.4$ m / s.

The critical inflow rate depends on the inland water level and the way the water flows inwards. The latter relates to the modeling of the situation. In the present case there is a so-called low threshold, in which the question is whether it is a complete or imperfect flow at the moment when the work of art is not closed high water.

11.4.5 Step 5 Determine maximum permissible outside water level

In this step, this outside water level must be sought, which is maximally present during a discharge wave, without this leading to an overrun of the storage space or the exceeding of the critical flow rate of the soil protection.

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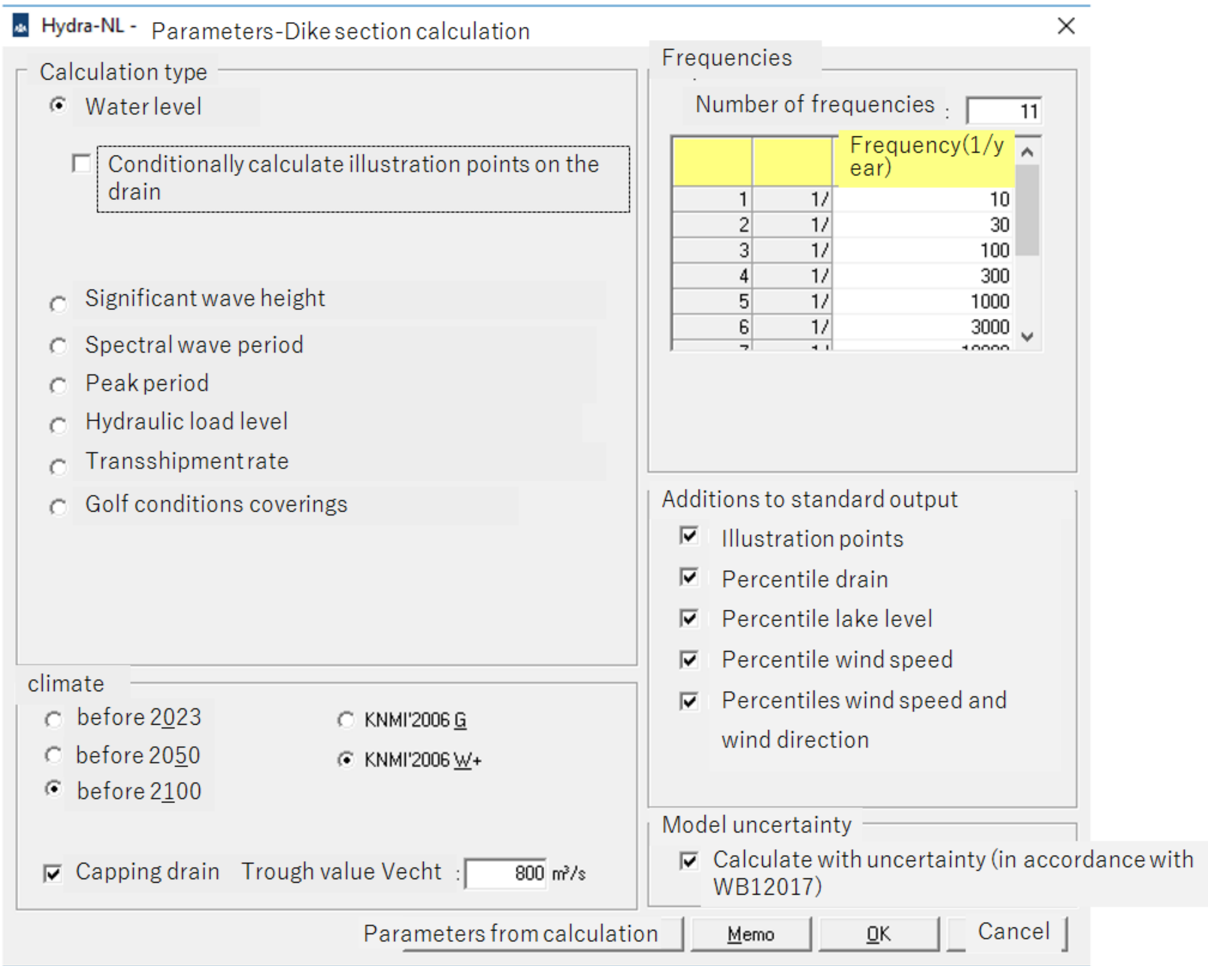
11.4.5.1. Chances of exceeding the outside water level

In order to gain insight into the exceedance probability of the various outside water levels, a line with exceedance probabilities can be made using Hydra-NL. To create an exceedance frequency line, it is not necessary to create a profile in Hydra-NL. After the program has started you can go directly to the tab 'Calculation' and for the type of calculation you have to choose 'Water level'. Subsequently, it can be indicated for how many frequencies and which frequencies the exceedance probability of the water level must be determined (see Figure 85). The following restrictions apply from Hydra-NL:

- Maximum frequency is 1/10 per year
- Minimum frequency from backgrounds Hydra-NL is 1/100,000 per year. At lower frequencies Hydra-NL indicates that the calculation results are less reliable.
- Absolute minimum frequency is 1/1,000,000 per year



※（Rijkswaterstaat, Central government　2018）219 頁より作成。

Figure 85 Entry screen Hydra-NL determination of exceedance opportunities outside water level

In order to make the exceedance frequency line for the future, one of the 'predefined' view years 2050 or 2100 must be chosen. It should be noted that if the design opts for a different visual year than 2050 or 2100, the results from Hydra-NL can be linearly interpolated or extrapolated.

For the insight it is recommended to also make an exceedance frequency line for the visible year 2023.

For the hydraulic boundary condition of the lock sluice, the above analysis has been carried out and this leads to the following graph.

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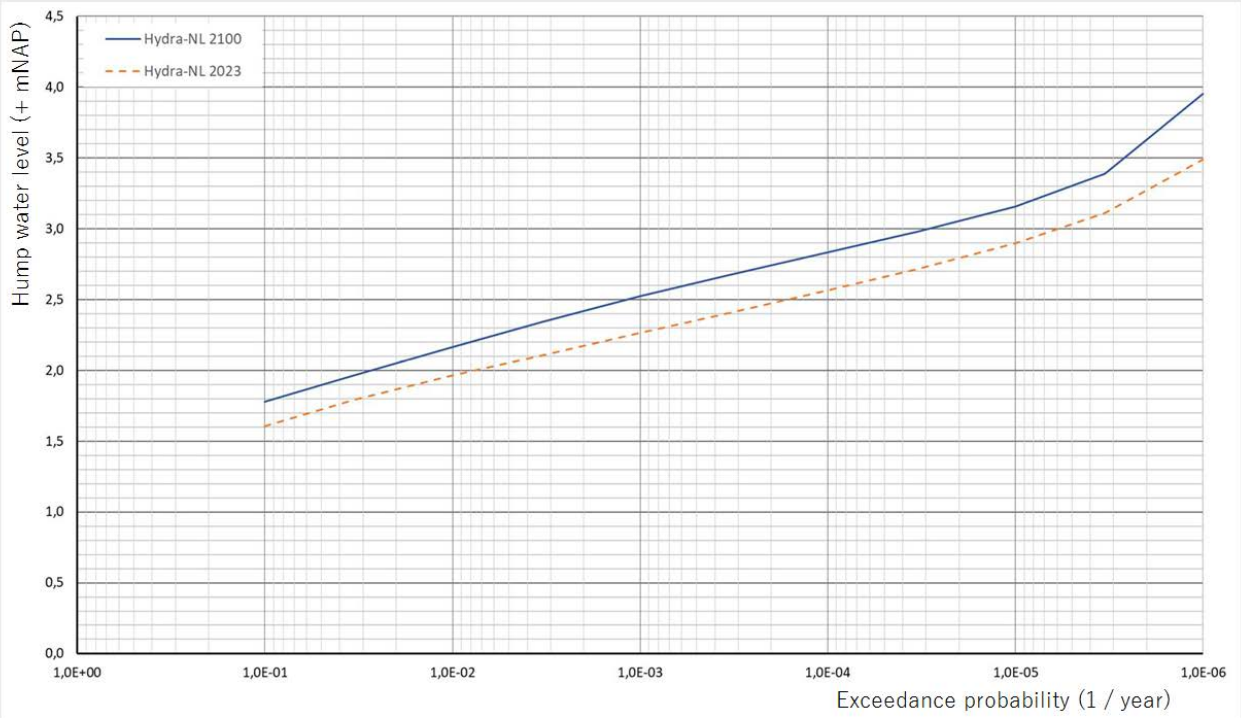

※（Rijkswaterstaat, Central government　2018）220 頁より作成。
Figure 86 Overwater odds outside waterway lock the Whaa based on Hydra-NL

For water levels with a greater probability of exceedance than 1/10 years, local water level statistics may be used if initially available. If enough local measurement data are available, the statistics for larger exceedance frequencies can be derived on this basis. Local measurement data can, for example, be found via the website https://waterinfo.rws.nl for the case of the lock the Whaa, this has taken place because the open turning height is below the outside water level with an exceedance probability of 1/10 per year and thus has a higher frequency and thus falls outside the range of Hydra-NL. On the basis of the local water level statistics, the graph above has been extended to figure 87. Use has been made of 30 years of water level data.

This figure shows that local water level data at the lock the Whaa does not directly match the calculations with Hydra-NL[40]. One of the causes may be that the time period over which the water level data is available is not long enough. A thorough analysis requires a period of at least 50 years if one is interested in exceedance probability of 1/10 per year. In addition, there are a number of other (possible) causes which are not discussed here, but which are known to the Helpdesk Water. The case as this has now been detected at the Whaa lock has been submitted to the Helpdesk Water. In consultation with this helpdesk we finally reached the exceedance opportunity line as shown in Figure 88.

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[40] Possible differences between Hydra-NL and local water level data differ per location. It can therefore also happen that the data connects nicely.

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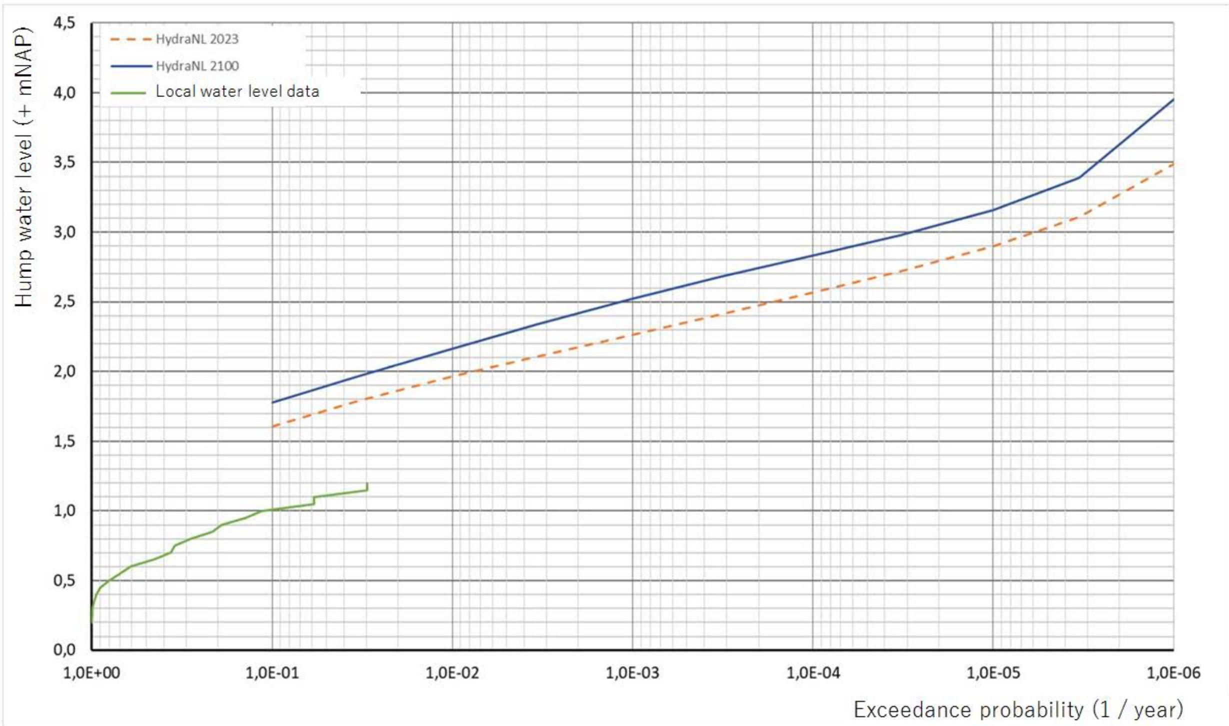
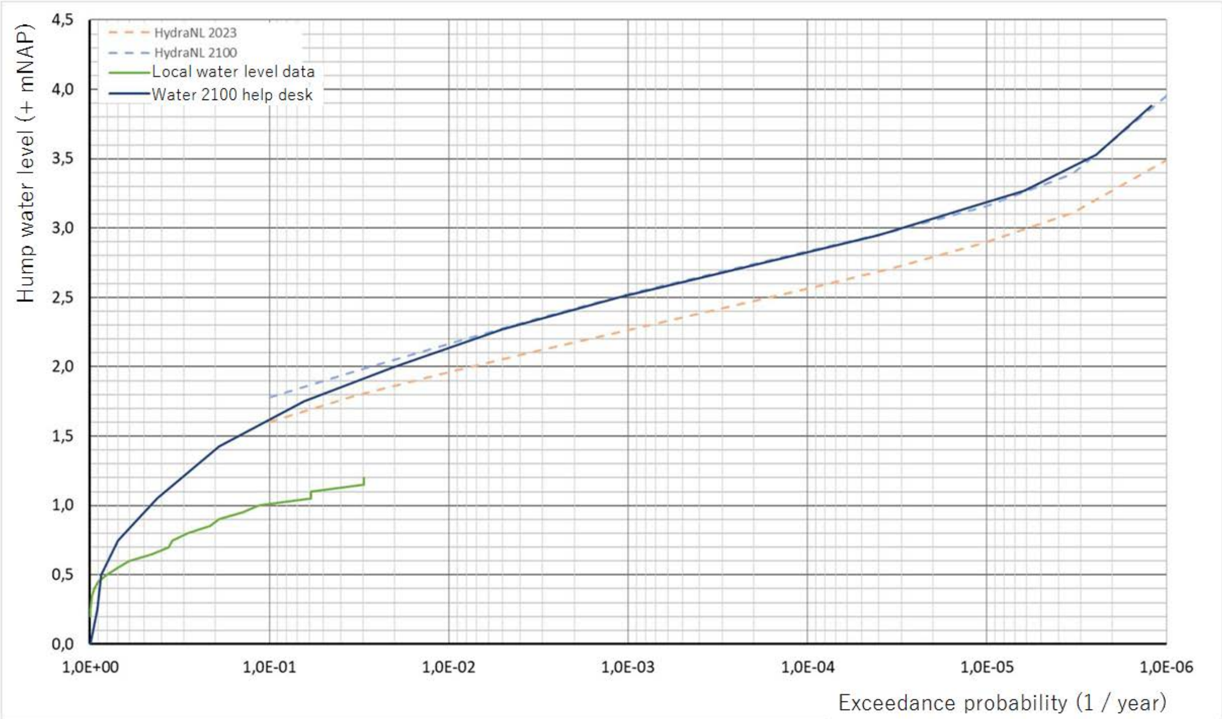


※（Rijkswaterstaat, Central government　2018）221 頁より作成。
Figure 87 Local water level data processed in excess water level



※（Rijkswaterstaat, Central government　2018）221 頁より作成。

Figure 88 Design plan regarding the exceedance opportunities of the outside water level

11.4.5.2. Expiration of high-water wave

To view the course of the inland water level when there is an inflow through a non-opened work of art, use can be made of the discharge wave. This can be determined with the help of the Water Level Course tool. This tool has already been described in chapter 3.

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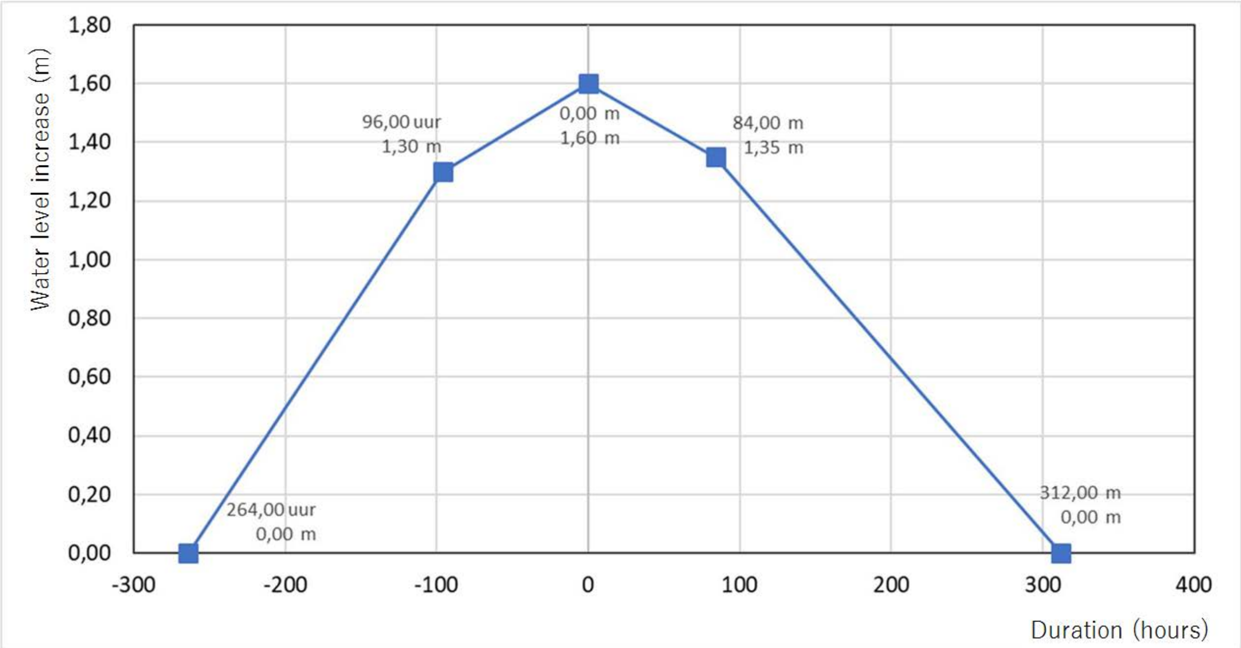
In the Water Level Course, a high water wave is described by means of a period of time and a certain course of the outside water level in this period of time. This course can consist of several routes. For the precondition point at lock-in the Whaa, the following sequence is present in the tool.



※（Rijkswaterstaat, Central government　2018）222 頁より作成。

Figure 89 Expiration of surge wave surge barrier the Whaa in water level course

The outlined process is always present, independent of the peak of the high tide wave. So the water level difference between the beginning and the peak of the wave is always 1.6 m and in addition the duration of the wave is always 576 hours. This means that when the maximum permissible inland water level at a low peak water level in the discharge wave is already reached, the application of the fixed course as shown in Figure 89 leads to a somewhat remarkable situation, which is made clear in the following example.

Suppose that at the peak of the discharge wave of NAP + 0.5 m the criterion of the inland water level is also exceeded. In that case, the outdoor water level starts at NAP + 0.5 m - 1.6 m = NAP - 1.1 m. This water level is well below the average daily water level in the winter period of NAP - 0.40 m. So that is bad unlikely. On the other hand, a high value of the peak water level would also lead to a situation that is not unambiguous. Suppose that at a peak water level of NAP + 2.0 m in the high water wave exceeds the storage space, this leads to an outside water level of NAP + 2.0 m - 1.6 m = NAP + 0.40 m prior to the high water wave. This water level is again 0.80 m above the average daily water level.

In order to avoid the above points with the current instruments, the form of the discharge wave (duration and location of the break points) from the Water Level Course tool is used in this case[41], but the water level increases are scaled relative to the situation with a water level increase of 1.6 m from water level course. This has been made clear in the figure below. The various inland water levels as mentioned in Table 21 are included.

[41] This principle has been used here because there are currently no other tools available to model the flood wave depending on the peak water level. This does not affect the fact that one can also choose another starting point, provided that this is well documented.

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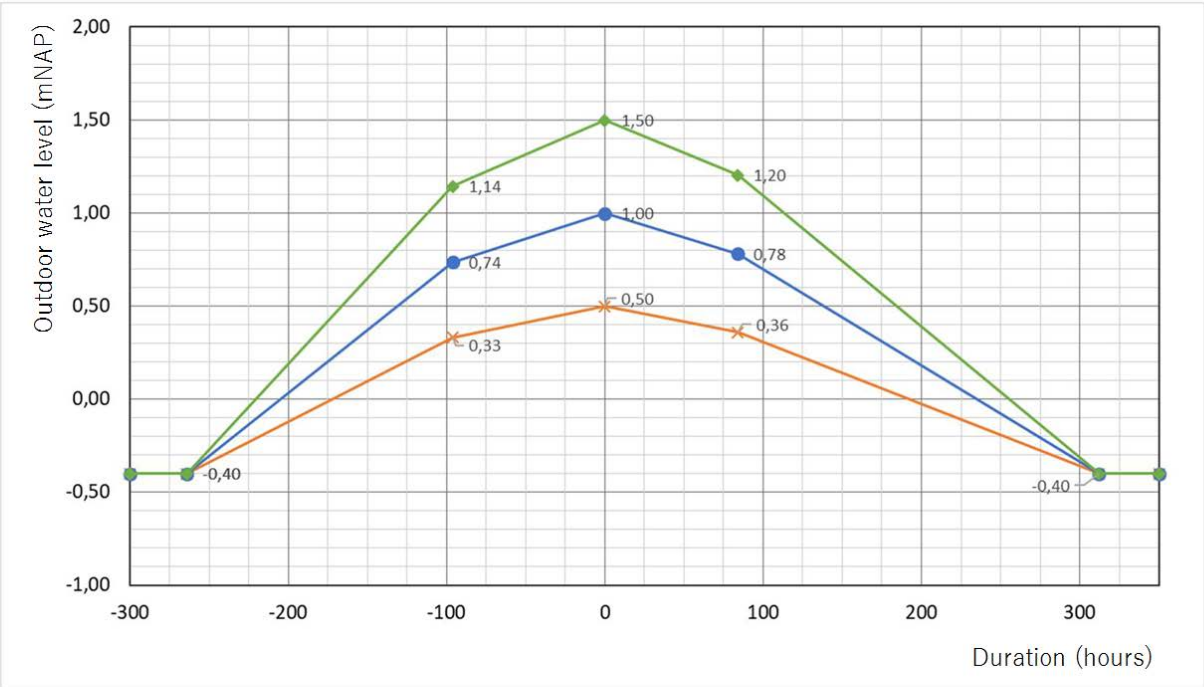

※（Rijkswaterstaat, Central government　2018）223 頁より作成。
Figure 90 Expiration of the high water wave at the various outside water levels

Because the storage capacity of the bowl between the old and new flood defense is limited, an outside water level higher than NAP + 1.50 m quickly leads to an inland water level, given an unsealed lock, to be higher than NAP + 1.50 m. stability of the Aremberger lock can no longer be guaranteed.

11.4.5.3. Course inland water level

Now that the course of the floodwaters is known, it is possible to calculate how the inland water level behaves as soon as water enters the water during a high water wave due to the artwork that has not been closed. In view of the very limited dimensions of the bowl, the inland water level runs directly with the outside water level. The course of the inland water level is therefore equal to that of the outside water level.

The course of the flood water from the above subsection is therefore not used any further when following the next steps. The analysis carried out in this section is in particular included as an example of a possible approach to a cup storage where the course is important.

11.4.5.4. Permissible outside water level

The fact that the inland water level can immediately follow the outside water level (see section 11.4.5.3) means the following:

- A maximum outside water level of NAP + 1.50 m is permissible from storage.
- A critical flow rate of 3.4 m / s is permissible from soil protection. This flow rate is not exceeded because the inland water level is almost the same as the outside water level. With a water depth on the inside of (NAP - 0.4 m - NAP - 3.0 m) = 2.6 m, at a critical flow rate of 3.4 m / s, a critical flow rate of 3.4 × 2.6 = 8.84 m^3 / s / m. When this is stopped in the formula for the flow of a low threshold (imperfect flow), this results in a water level difference of 0.60 m (see below) for the bottom protection to collapse.

$$Q_{in,onvolk.} = m_{onv} \cdot (h_{bi} - h_{dr}) \cdot \sqrt{2 \cdot g \cdot (h_{bu} - h_{bi})}$$
$$8{,}84 = 1{,}0 \cdot (-0{,}40\ -\ -3{,}0) \cdot \sqrt{2 \cdot 9{,}81 \cdot (h_{bu} -\ h_{bi})}\ \ \Rightarrow\ \ (h_{bu} -\ h_{bi}) = 0{,}60\ m$$

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In view of the limited composting capacity, this decay will not occur and so the soil protection will not be decisive. The maximum permissible outside water level will be equal to NAP +1.50 m on the basis of the above considerations.

11.4.6 Step 6 Determine the probability of not being turned upside-down of the artwork

This step involves determining the factor P_{open}. It has to be determined what the chance is that from the primary function of the artwork the artwork is already closed at the moment that a high water presents itself. For the lock sluice it is always open and only closed if this is required from high tide times. This makes P_{open} equal to 1.

11.4.7 Step 7 Determine the required chance of closing the reversing means

The maximum permissible outside water level that may occur during a high water wave when the artwork is open and the chance that the artwork is open at the moment of high water is now known. With this, the required failure probability of closure can be determined from the failure probability for not closing. This does not take into account a possible recovery of a failed closure in an alternative manner ($P_{f,herstel}$= 1). The probability that the outdoor water level will exceed NAP +1.5 m will be set at 0.14 per year with the aid of the exceedance chance line (Figure 88).

$$P_{f,KW,NS} = P_{open} \cdot P_{ns} \cdot P_{f,herstel} \cdot \{P(Z_1 < 0) \cdot P_{f,KW|erosie\ bodem}\ OF\ P(Z_2 < 0)\}$$

$$1{,}0 \cdot 10^{-5} = 1 \cdot P_{ns} \cdot 1 \cdot P(h_{bui} > 1{,}5\ mNAP)$$

$$P_{ns} = \frac{1{,}0 \cdot 10^{-5}}{1 \cdot 1 \cdot 0{,}14} = 7{,}14 \cdot 10^{-5}\ per\ vraag$$

The probability of not closing P_{ns} should therefore be less than or equal to 7.14E 05 per question.

11.4.8 Step 8 Verify that the chosen locking devices meet the requirement in step 7

The first set-up of the site lock consists of the use of a single set of high water retaining reversals. The most obvious solution is to use a set of point doors. In the concept of the foundation (see section 11.2.5) it is indicated that shot-bar rebates must also be made. However, these serve for bulkheads that are used in case of maintenance. For the time being, they are not taken into account for turning a high water, because the placing of the bulkheads in case of strong wind is rather uncertain.

Because the reversing means are only closed when a high water is present, the new score tables ([Ref 4.5] and [Ref 11.2]) can be used to determine the probability of failure of the reversing means. Based on the assumption that in the new situation, as far as organization is concerned, all the conditions in these score tables are fully met, the next score for P_{ns} is feasible when one set of high water-retaining overhead doors is used.

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Assessment aspect	**Score E_i [-]**	**Probably not close P_{ns} [1 question]**
Alarm	7	1,00E-07
Mobilization	5	1,00E-05
Service	5	1,00E-05
Technical failure	3,5	3,16E-04
Total		**3,36E-04**

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This shows that the requirement is not met in step 7 (section 11.4.7). This saves another factor 5.

There are now several options:

1. The area behind the Aremberger lock is also involved in cup storage.
2. The use of the lock is adjusted
3. Additional retractors are placed.

These three options are discussed in the following paragraphs.

11.4.8.1. Increasing cup storage (back to step 3)

By also using the cup storage behind the Aremberger lock, the cup storage is greatly increased. To this end, the flow capacity of the rockets in the doors of the Aremberger lock must be sufficient so that no outside water level higher than NAP + 1.50 m occurs at this lock, which could lead to instability of the lock. When the water level at the Aremberger lock becomes larger than NAP + 1.50 m and this lock collapses, an uncontrollable situation occurs. Due to the large storage surface behind the lock, however, it is still very possible that large consequences are not possible.

A larger cup storage may help to meet the failure probability if the exceedance probability of the maximum allowable outside water level during a high water wave (peak water level) is a factor of 5 smaller than the exceedance probability of NAP +

1.50 m. The latter had an exceedance probability of 0, 14, so the searched outside water level may have a maximum probability of exceedance of 0.14 / 5 = 0.028 per year.
An excess water level (see Figure 88) of approximately NAP +2.0 m is associated with this exceedance. This outside water level does not lead to the overflow of the old flood defense, but the flow velocities around and through the Aremberger lock will be large. At a target level of NAP -0.80 m and an outside water level of NAP +2.0 m, flow velocities occur of approximately 7.5 m / s ($\sqrt{2 \cdot g \cdot \Delta H}$). It is expected that the Aremberger lock and the immediate vicinity will not withstand such flow velocities, which will increase the breach in the old defense.

Based on the above considerations, it is not considered desirable to solve the shortage of failure probability by taking into account the cup storage behind the Aremberger lock.

11.4.8.2. Adjusting the use of the barrier lock (back to step 6)

There are two options for this solution direction:

1. Reducing the chance that the artwork is open
2. The installation of a lock locks instead of a barrier lock. In fact, this is a result if option 1 is implemented very far.

The second option is not acceptable. After all, the starting point is to make a lock, so that shipping can sail in and out freely.

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Option 1 can possibly be realized by closing the floodgate at night as no recreational boating is present. By doing this, the chance of open standing (P_{open}) reduces to 0.5. The requirement of the probability of failure of the closing of the tip doors then reduces from 7.14E-05 per demand to 1.43E-04 per question. In addition, the chance of failure per closure also decreases because the reversing device is now operated every evening and in the morning. This increases the reliability, provided that management and maintenance are carried out correctly. Based on the standard failure probabilities (see Appendix B), there is a risk of not closing 1.0E-04 per question. This means that one set of point doors meets the requirement for not closing.

11.4.8.3. Additional reversals (back to step 7)

By adding another set of retractors, the reliability of the closure is of course increased. There may not be a full correlation between the reversing means. This means, among other things, that there must be sufficient distance between the reversing means.

If the failure probability of closure is determined for two reversals using the score tables, then the following scores are obtained:

Assessment aspect	**Score E_i [-]**	**Probably not close P_{ns} [1 question]**
Alarm	7	1,00E-07
Mobilization	5	1,00E-05
Service	5	1,00E-05
Technical failure	4,5	3,16E-04
Total		**5,17E-05**

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The value for P_{ns} now becomes 5,17E-05 per question, where the requirement was 7,14E-05 per question (see paragraph 11.4.7). The requirement is therefore met.

A solution with two sets of retractors thus leads to a situation that meets the requirements with respect to non-closure.

11.4.8.4. Selectable design concept for not closing

On the basis of the analyzes carried out, it appears that both reducing the open period (adaptation P_{open}) and making a second set of point doors (adaptation P_{ns}) leads to meeting the requirement. Which solution is ultimately chosen should be discussed with the client.

From the primary function of the casing lock, the provision of an additional set of point doors can be the most desirable. The primary function (passing through shipping) is not affected by this and there is the 'certainty' that if something happens to the one set of point doors, one immediately has access to an extra set of doors.

From cost (installation and maintenance) the adjustment of the operating time is preferable.

11.5 Elaboration of piping (PKW)

In this section the required dimensions of the seepage barriers around the barrier and adjacent abutments are determined. This is done on the basis of the step-by-step plan in section 6.1.4 of the Piping chapter.

11.5.1 Step 1 Determine design decay about the artwork

The design water level is determined using Hydra-NL. Here, a water level sum is rotated in which the water level is determined with an exceedance probability equal to the lower limit (maximum permissible failure probability) of the dyke stretch (1 / 1,000 per year).

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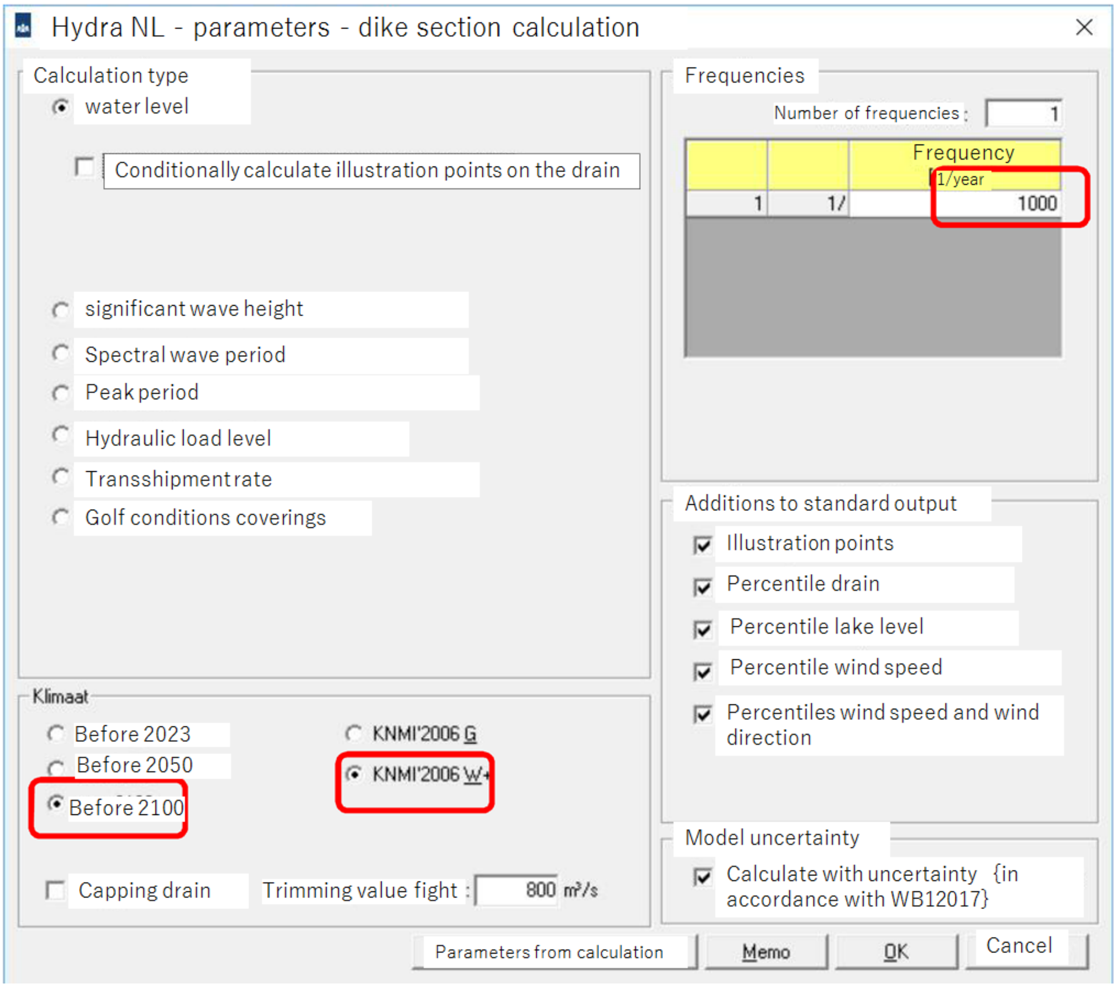

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Figure 91 Overview of Hydra-NL water level calculation input screen

This design water level is 2.52 m + NAP.

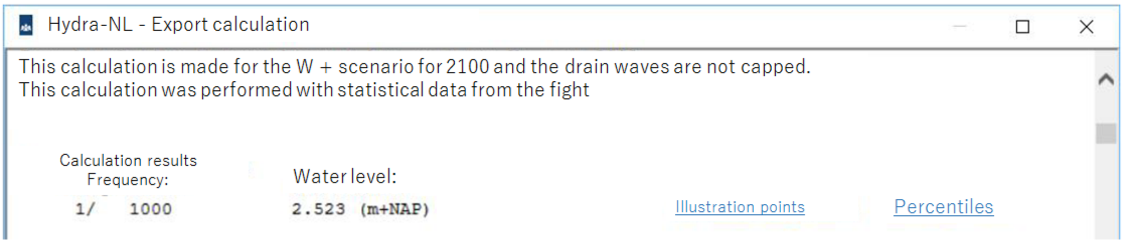

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Figure 92 Overview of the Hydra-NL water level calculation output screen

For the inland water level, a value of 0.20 m-NAP is used, being the water level at which the level after closing is brought by the rinkets (lock doors) of the Aremberger lock. In view of the small area behind the bank lock, no account needs to be taken of the deviation under normative conditions. The design decay thus amounts to 2.52 m + NAP - 0.20 m-NAP = 2.72 m.

11.5.2 Step 2 and 3 Determine whether there is sufficient seepage length sufficient

In this case, many seepage screens already exist from constructive considerations. This section assesses whether the dimensions are sufficient.

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In this case it already seems clear from a qualitative view that piping is not a relevant failure mechanism. The work of art is enclosed by a thick package of impermeable layers (at least between soil level 3.0 m-NAP and 5.0 m-NAP clay / peat is present). However, in order to be able to completely shut off piping, the underlying sand layer must also be assessed. This requires an additional calculation that is more work than performing a simple piping view.

The artwork meets a simple heave test. The design decay is 2.73 m and the length of the downstream seepage screen is NAP - 10.0 m (depth of land screens) - NAP - 3.0 m (bottom level sluice) = 7.0 m. The transition over the downstream seepage screen thus amounts to 2.73 / 7.0 = 0.39. This is smaller than the heave criterion of 0.5 so that underflow can be excluded on the basis of this simple heave consideration.

Only backwardness may therefore play a role. Figure 93 shows a 3D representation of the artwork in which the possible normative seepage route has been included. Here, the dike is schematized as a clay dike (known from the statutory assessment of the dike body). The following seepage is normative:

Table 22 Overview of seepage lengths normative seepage

No.	Description	Length
1	From soil level 3.0 m-NAP to lower level clay layer at 5.0 m-NAP	L_v = 2 m
2	From the side of the lock chamber to the side of the land on level 5.0 m-NAP	L_h = 20 m
3	From the front of the manor to the back of the head of the land at level 5.0 m-NAP. Note: this is a so-called 'short path'. It is unlikely that the	L_h = 9,9 m

	groundwater flow follows the 'notch' in the sheet piling back to the column wall.	
4	From side of land to side of lock chamber at level 5.0 m-NAP	L_h = 20 m
5	From level underside clay layer to 5.0 m-NAP to level exit point (= soil level) 3.0 m-NAP	L_v = 2 m

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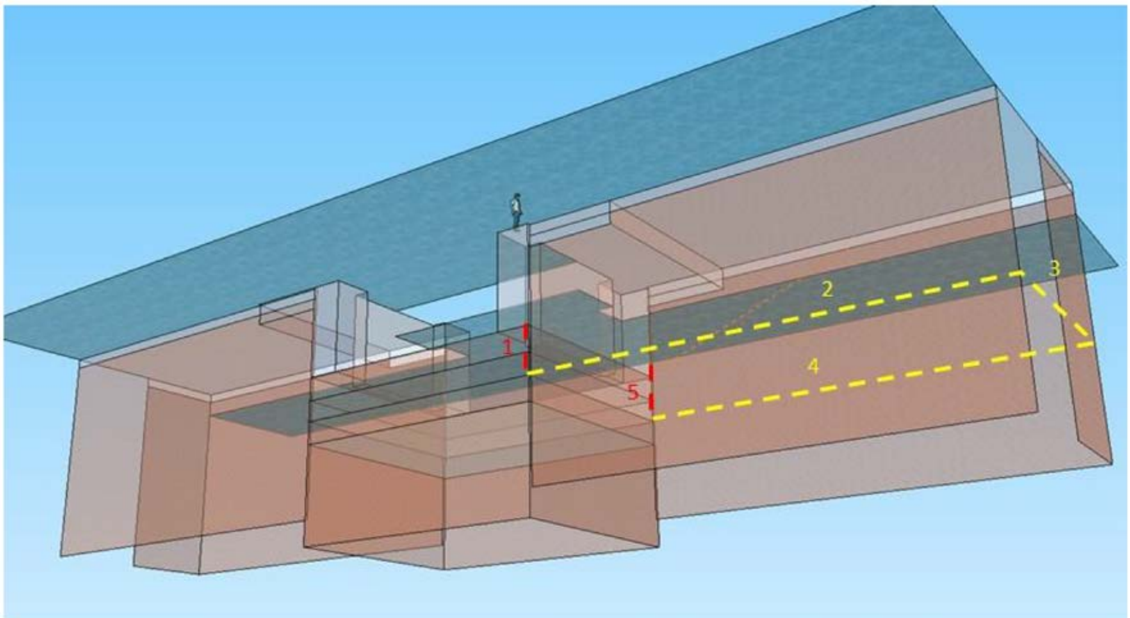

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Figure 93 Overview of normative seepage along the work of art

Because the normative seepage route also contains vertical parts, Lane's formula is used. Filling in (with C_{creep} = 7 belonging to moderately fine sand):

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$$\Delta H_c = \frac{L_v + {}^{L_h}/_3}{C_{w\,creep}} = \frac{4 + {}^{49{,}9}/_3}{7} = 2{,}95 \text{ m}$$

The design decay is 2.72 m smaller than the critical drop of 2.95 m. So no additional seepage screens are needed.

11.6 Effective structural failure

In this section the design of the lock is verified against the reliability requirements with respect to structural failure from the Water Act and the Building Decree. Within the WBI, structural failure is only considered as a result of the high water load and the mechanism Strength and stability is called point constructions (STKWp). In the case of designs, however, the artwork must be verified with regard to all possible loads, so STKWp is not complete. Given the focus of the Work Guide, the example does limit itself to the design verification with regard to the high water load situation. To this end, the strength of all water-retaining components must be verified in practice (event 2, see section 7.3), as well as the overall stability of structure and ground body (event 4, see section 7.3). In the case only the verification of the main water retaining element, the wooden gaps, is discussed. Because this concerns a case, only one structural part of the doors is verified, namely a horizontal beam. The verification is done on the basis of the step-by-step plan in section 7.9 of the chapter Structural failure.

Figure 94 shows a schematic view and cross-section of the wooden point doors. The crest height as determined in section 11.3 is maintained. The wooden pointed doors can be roughly schematized as a number of horizontal girders that are connected by two resins against which planking has been installed.

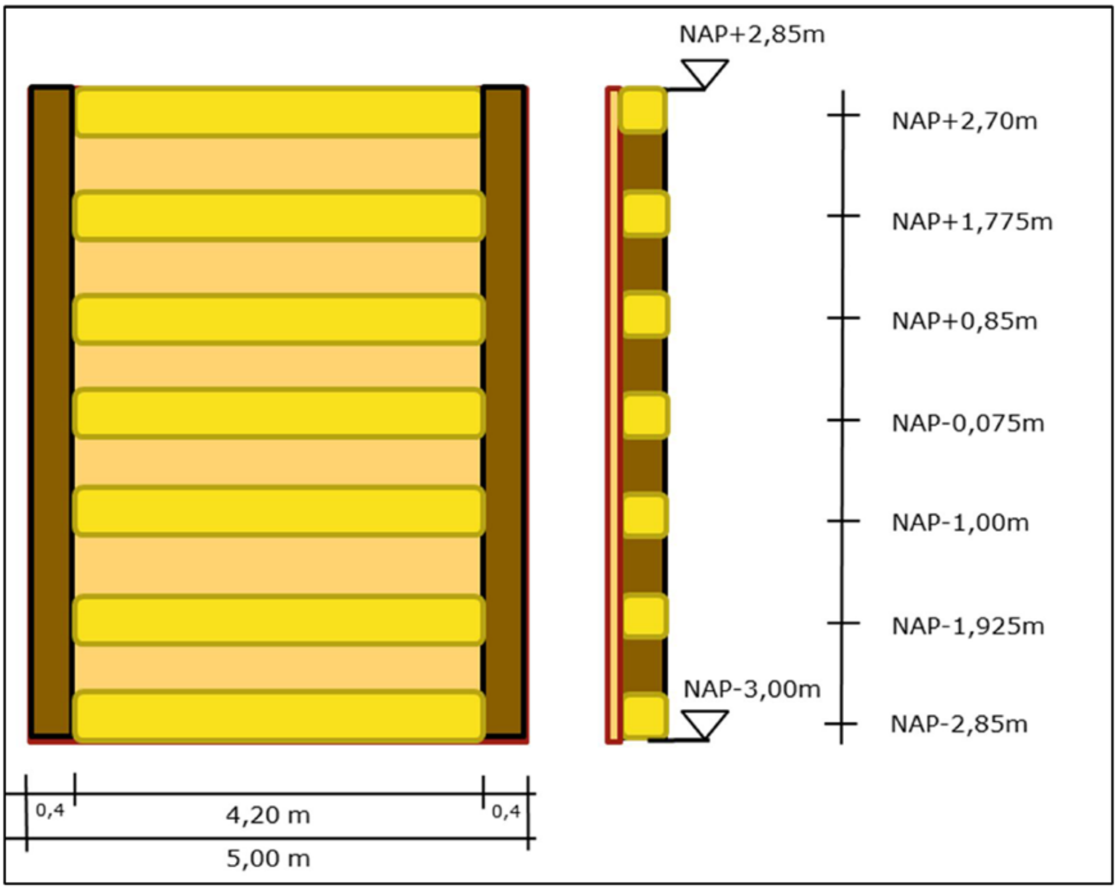

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Figure 94 Schematic representation of structural design wooden point doors of the lock The Whaa

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11.6.1　Step 1: choose the construction component to be verified

In this case, the dimensions of the horizontal wooden beam are verified at a height of NAP -1.00 m.

11.6.　2 Step 2: determine the relevant taxes

The beam must be able to withstand the loads at high water, or the combination of the hydraulic load and the self-weight load. The beam may also have to withstand ice load and temperature load (see Table 11). In this case, however, only the high water tax is discussed.

11.6.3　Choice A or B: design verification regarding high water or other loads

Given the scope of the example: choice A in Figure 33.

11.6.4 Step 3: determine the reliability requirement from the Buildings Decree

The lock is part of the primary flood defense, in practice in this case Consequence Class (CC3) is usually required. However, if the route standard from the Water Act involves a major probability of flooding (a flexible standard) and CC3 requires disproportionate investments, it may be sensible to deviate from this and to bring the reliability requirement from the Buildings Decree in line with that of the Water Act. In that case, for example, CC2 may be required. However, this is up to the client.

In the example, the client requires CC3, or $\beta_{\text{eis, BB}} = 4.3$ for a lifespan of 100 years.

11.6.5 Step 4: determine the reliability requirement from the Water Act

The failure probability $P_{\text{eis, KW, CON}}$ is determined using the following formula:

$$P_{\text{eis,KW,CON}} = \frac{P_{\max} \cdot \omega_{\text{CON}} \cdot c}{N_{\text{CON}}} = \frac{\frac{1}{1.000} \cdot 0{,}02 \cdot 4}{3} \qquad 11.5$$

In which:

$P_{\text{eis, KW, CON}}$	Failure Chance for structural failure and no failure by overflow / transhipment of an individual work of art derived from route law from the Water Act for a reference period equal to t_{ref} = 1 year [-]
$P_{\max}$	Failure Chance for the entire dike section (standard route) based on the maximum permissible probability of flooding from the water act for a reference period equal to the t_{ref} = 1 year [-]. $P_{\max}$ = 1 / 1,000 [1 / year]
ω_{CON}	Failure probability factor for structural failure = 0.02 [-]
c	Correction factor for the correlation between structural failure and failure by overflow / transsshipment = 4 [-]
N_{dsn}	Length-effect factor for structural failure = 3 [-]

This results in a failure probability of 1 / 37,500 per year (2.67E-5 per year). This requirement is formally related to failure, or all events that, from a structural point of view, lead to a flood. As indicated in section 7.4, in the case of 'designs' it is recommended that this requirement with regard to failure as a result of collapsing water-retaining construction components equates to initial failure, or the failure of water-retaining construction components.

11.6.6 Step 5: collect the data from the construction part

The center line of the wooden beam is at a height of NAP-1.00 m and is executed in Azobé (tree of the family Ochnaceae). The beam has a length of 4.2 m, a height (y axis) of 400 mm and a width (z axis) of 300 mm, see Figure 95.

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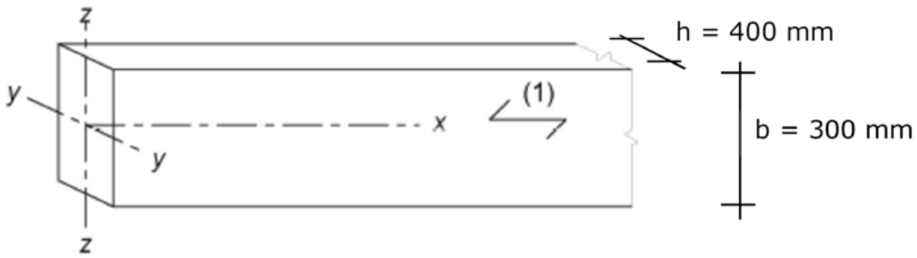

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Figure 95 Dimensions cross-section beam

11.6.7 Step 6: select strength magnitude to be determined and determine the limit state function / unity check in the example, the recordable stress in the middle of the beam, in the design dimensions from step 5, becomes the strength magnitude to be verified.

Load on the girder

In the high water load situation the girder is loaded by the hydraulic load p (acting in the y direction and leading to bending stresses on the z axis), the own weight load q (acting in the z direction and leading to bending stresses around the y -as) and a spatial force N (pressure force in the x-direction). We assume conservatively that the sheet metal only transfers the hydraulic load to the beams and does not add any strength itself. The beam is schematized as a beam on two hinged support points.

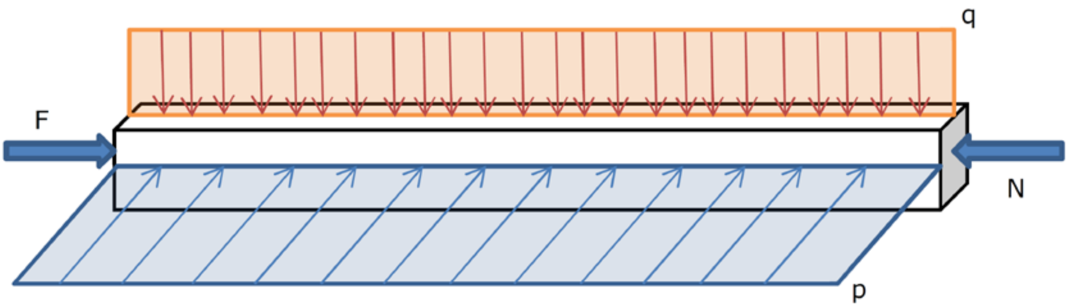

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Figure 96 Loads on the beam

The splash force N is a typical load associated with point doors and results from the hydraulic load.

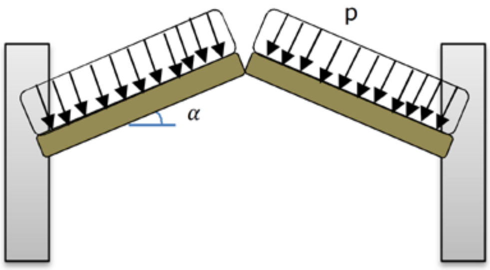

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Figure 97 Distraction of the spatial force I

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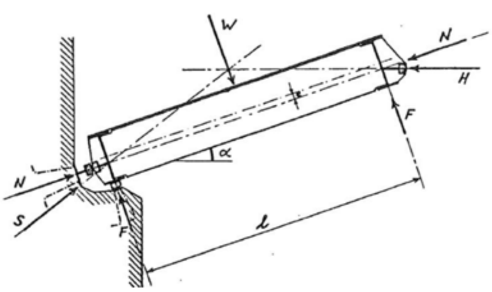

Figure6-5 Forces on point door

The reaction force S can be dissolved in a force parallel to the door (N) and a force perpendicular to it (F). These reaction forces can be written as a function of the water pressure W and the angle α that the doors make with the normal on the lock shaft (Figure 6-5).

$$S = H = \frac{W}{2 \sin \alpha}$$

$$F = ½\, W$$

$$N = \frac{W}{2 \tan \alpha}$$

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Figure 98 Derivation of spatial force II

The point doors of the turn lock the Whaa are supported at an angle of a = 22 ° against each other which leads to the following force:

$$N = \frac{W}{2 \tan \alpha} = \frac{l \cdot p}{2 \tan 22°}$$

Strength of the beam

Wood is an anisotropic material, which means that the material properties differ per direction of orientation. Figure 96 shows that the beam is loaded in three different directions, which means that three different strength properties play a role in the relevant stress verification.

Border status function / unity check

For the verification of the maximum voltage to be absorbed by the beam, article 6.2.4 from the NEN-EN 1995 (Eurocode Wood) is used, which has already translated the limit state function for the load situation to be verified (Figure 96) into two prescribed unity checks (formulas 6.19 and 6.20) expressed in calculation values:

6.19 $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

6.20 $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

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At which:

$\sigma_{c,0,d}$ Calculation value for axial compressive stress [N / mm²]
$f_{c,0,d}$ Calculation value for axial compressive strength [N / mm²]
$\sigma_{m,y,d}$ Calculation value of bending stress around y-axis [N / mm²]
$f_{m,y,d}$ Calculation value of the bending strength when bending around the y-axis [N / mm2]
$\sigma_{m,z,d}$ Calculation value of bending stress around z-axis [N / mm²]
$f_{m,z,d}$ Calculation value of the bending strength when bending around the z-axis [N / mm²]
Km 0.7 [-] (rectangular cross-sections)

In this case, bending around the y-axis is caused by the self-weight load (q) and the z-axis by the hydraulic load (p).

11.6.8 Step 7 and 8: determine the representative value and calculation value of the strength

In this case the strength is expressed in three different strength terms ($f_{c,0}$, $f_{m,y}$, and $f_{m,z}$) for which the representative values and calculation values have been derived with the help of NENEN1995, NEN-EN338 and NEN-EN1912. The tip doors are made of Azobé (wood class D70 in accordance with NEN-EN1912) and climate class 3 is considered applicable. The load duration class for the hydraulic load is short (less than 1 week).

The relationship between the representative and calculation value of the strength is equal to: $R_d = k_{mod} \cdot \frac{R_k}{\gamma_m}$

Table 23: representative strength and calculation values of the strength

Azobé (D70)	**Step 7. Representative strength * [N / mm2]**	**k_{mod}** (climate class 3)**	**Υ_m**	**Step 8. Calculation value strength [N / mm2]**
Compressive strength	$f_{c,o,k} = 36$	0.70 (short)	1,3***	$f_{c,o,d} = 22$
Flexural strength around y-axis	$f_{m,y,k} = 70$	0.5 (permanent)	1,3***	$f_{m,y,d} = 27$
Flexural strength around z-axis	$f_{m,z,k} = 70$	0.70 (short)	1,3***	$f_{m,z,d} = 43$

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* According to NEN-EN338

** According to NEN-EN 1995

*** Associated with sawn timber according to NEN-EN 1995

11.6.9　Step 9: generate the marginal water level, wave height and wave period statistics with Hydra-NL and determine the degree of mutual correlation.

The hydraulic load is a combination of decay and wave load. As discussed in detail in section 7.10.2, combined statistics cannot be derived for designs at this time. For the time being, use should be made of the marginal statistics of the inland and inland water levels, the wave height and the wave period. In this step, the marginal statistic for these parameters in the form of exceedance probability distributions is generated with the help of Hydra-NL.

Statistics outside water level

In paragraph 11.4 the exceedance probabilities of the external water level have already been determined and are shown in Figure 86. In the illustration points of all calculated water levels, wind directions appear to be west (270°), west-south (247,5°) and south-west (225°), respectively approximately 30%, 25% and 15% contribute to the outcome. This means that these wind directions are therefore dominant for the water level statistics in this export point.

Inland water level statistics

There is no inland water statistics for the small bowl the Whaa, so that a safe value for the inland water level is assumed for determining the decay load. The winter level is assumed to be NAP-0.20m.

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Marginal statistics significant wave height and spectral wave period

In the same way as for the outside water level in section 11.4, the exceedance frequencies for the significant wave height (H_s) and the spectral wave period ($T_{m-1.0}$) are generated with Hydra-NL and shown in Figure 99 and Figure 100.

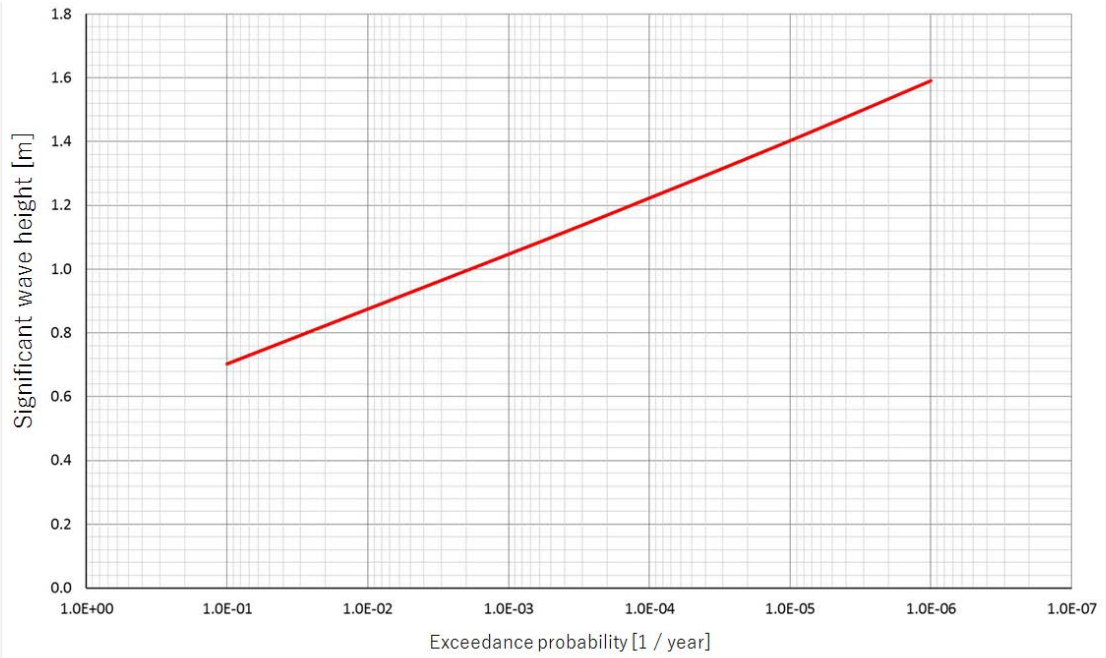

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Figure 99 Excess frequency line significant wave height

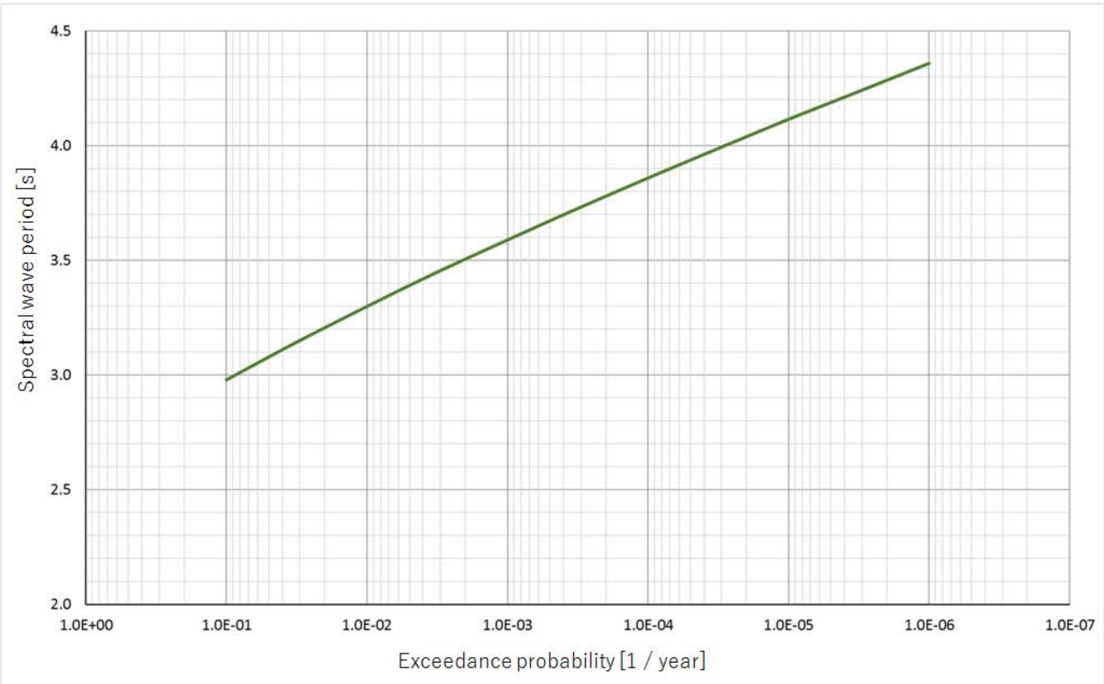

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Figure 100 Excess frequency line spectral wave period

For both H_s and T_m-1.0 it appears that in all illustration points wind direction southwest（225°）contributes about 90% to the outcome. Wind direction southwest is therefore dominant for the wave load.

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Degree of correlation

To combine these parameters into the final hydraulic load, a degree of correlation between the parameters has to be assumed. This cannot, as discussed in paragraph 7.10.2.1, be determined in a model-based manner, but can only be estimated. For the determination of the wave load, H_s and $T_{m-1.0}$ appear strongly correlated on the basis of the above results; we keep full correlation here. From the above results also seems to be a proper correlation between the wave parameters and the outside water level. As described in section 7.10.2.1, it is sensible to assume full correlation in that case for combining the wave load and the expiry tax.

11.6.10 Step 10: Determine the calculation value of the tax effect on the Building Decree

The calculation value of the tax effect E_d consists of the combination of the calculation value of the own weight load and hydraulic load. The hydraulic load in this load situation is dominant over the own weight load, which means that only the load effect needs to be verified according to 6.10b from NEN-EN1990:

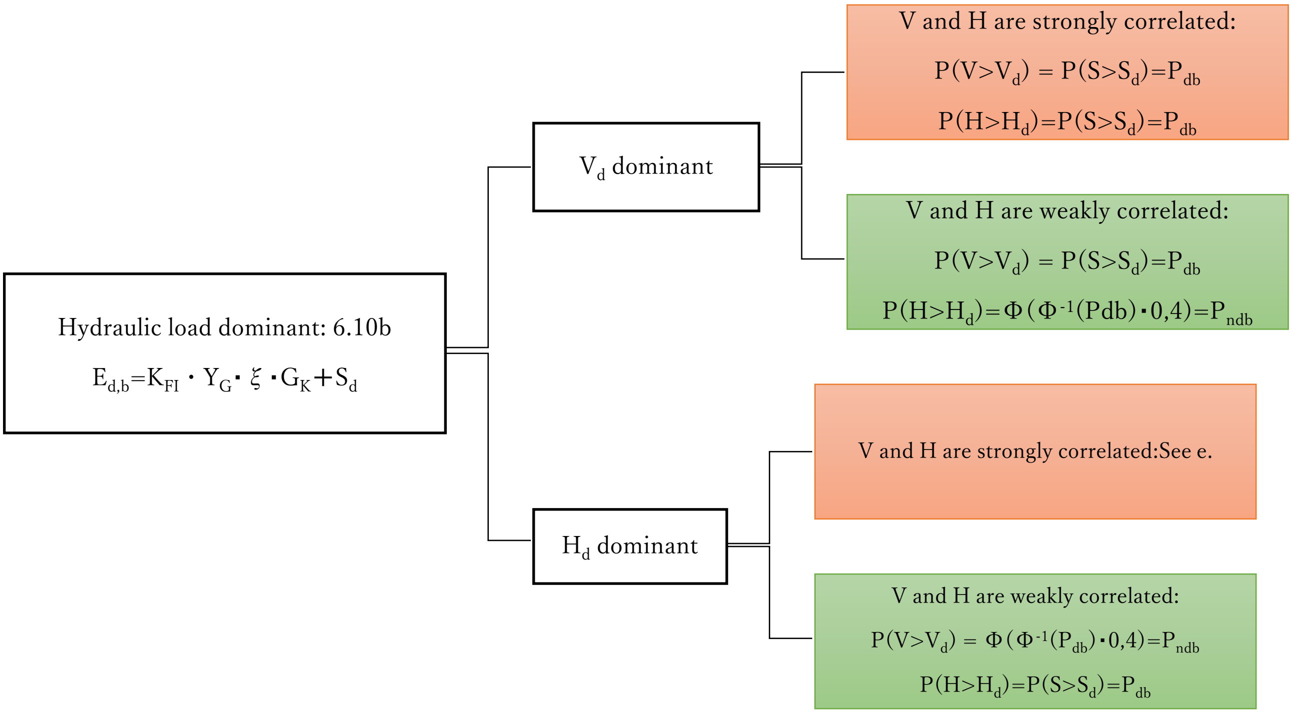
$$E_{d,b} = G_d + S_d = k_{FI} \cdot \gamma_G \cdot \xi \cdot G_k + S_d$$

At which:

G_d	Calculation value of own weight tax
S_d	Calculation value of the hydraulic load at high water
K_{FI}	Factor that in the case of the assessment in accordance with the Building Decree depends on the chosen consequence class CC1, CC2 or CC3 and in case of verification in accordance with the Water Act is set equal to 1.0
Y_G	Partial factor for permanent loads
ξ	Reduction factor for unfavorable own weight tax
G_k	Characteristic value own weight tax

Arithmetic value hydraulic load (S_d)

Since only the load effect according to 6.10b needs to be verified, the 'standard method' for determining the calculation value of the hydraulic load according to section 7.10.2 is applied.



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At which:

P_{db} = exceedance probability in case of dominant load

P_{ndb} = exceedance probability in case of non-dominant load

In step 9 it was concluded that:

- The wave load (H) and the decay load (V) are fully correlated
- The significant wave height (H_s) and the spectral wave period ($T_{m-1.0}$) are fully correlated.

It follows that:

- Exceedance of probability calculation value of decay load: P ($V> V_d$) = P ($S> S_d$) = P_{db}
- Exceeding probability calculation value wave load: P ($H> H_d$) = P ($S> S_d$) = P_{db}
- Abutting probability calculated value of significant wave height: P (Hs> Hs, d) = P ($S> S_d$) = P_{db}
- Exceedance probability calculation value spectral wave period: P ($T_{m-1.0}$> T $_{m-}$

$_{1.0,\,d}$) = P (S> S_d) = P_{db}

The decay load is determined by the inside and outside water level. Since the inland water level (h_{bi}) is a deterministic variable in this example, the outdoor water level applies: P (h_{bu}> $h_{bu,\,d}$) = P (V> V_d) = P_{db}.

In accordance with Table 12 of Section 7.10.2.4, in case of a verification in accordance with the Buildings Decree for CC3: $P_{db} = 1.0 \cdot 10^{-5}$ per year.

Using the marginal statistics from step 9, this results in the following calculation values:

Table 24: calculation values of wave parameters and water levels

$H_{s,d}$	1,40	m
$T_{m-1,0,d}$	4,12	s
$h_{bu,d}$	+3,15	mNAP+
h_{bi}	-0,20	mNAP+

※（Rijkswaterstaat, Central government　2018）236 頁より作成。

From this follows the calculation value of the decay load shown in Figure 101, where for the specific weight of water a value of 10 kN / m^3 is maintained.

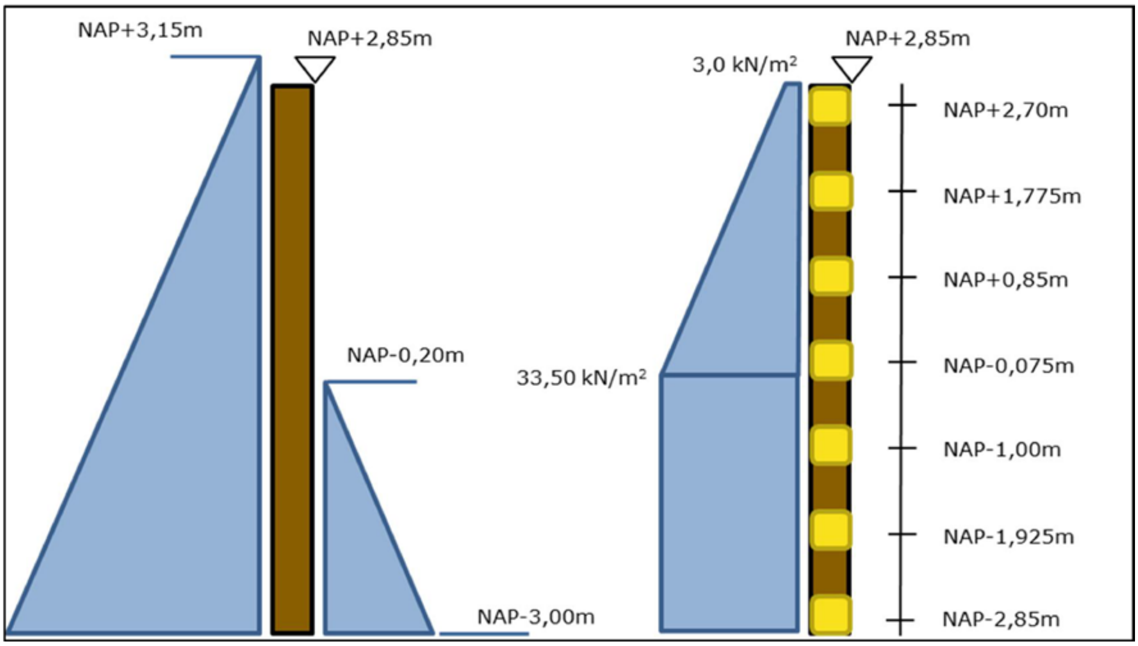

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Figure 101: Calculation value for the expiry tax when verifying Building Decree

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The wave load is determined with the model of Goda (for explanation see Appendix D) and the results from Table 24. The normal of the artwork (θ_{wall}) has an angle of 170° with respect to the north and the heavily dominant wind direction for the wave height (θ_{waves}) turned out to have an angle of 225° to the north. The dominant angle of incidence for the wave load compared with the normal of the artwork is therefore 55°.

Table 25 Input model of Goda

Symbol	Variable	Unit
B_M	0,0	m
g	10,0	m/s^2
$h_{bu,d}$	+3,15	m+NAP
h_{dr}	-3,00	m+NAP
h_{kr}	+2,85	m+NAP
$H_{s,d}$	1,40	m
$T_{m-1,0,d}$	4,12	s
Δh	0,0	m
γ_w	1000	kN/m^3
λ_1	1,0	-
λ_2	1,0	-
λ_3	1,0	-
Θ_{wall}	170	°
Θ_{waves}	225	°

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Table 26: export model of Goda

Symbol	Variable	unit
η	3,63	m
ω_0	n.v.t. L determined by means of formula C1	s^{-1}
d	6,15	m
h	6,15	m

H_d	3,08	m
k	n.v.t. L determined by means of formula C1	m^{-1}
k_0	n.v.t. L determined by means of formula C1	m^{-1}
T_p	4,53	
L	28,2	m
p_1	16,00	kN/m^2
p_3	7,80	kN/m^2
p_4	n.v.t. because it concerns an overflow situation.	kN/m2
α_{imp0}	0,50	-
α_{imp1}	-0,0023	-
α_{mpuls}	-0,0012	-
α_1	0,667	-
α_2	0	-
α_3	0,485	-
α_4	n.v.t. because it concerns an overflow situation.	-
δ_1	-6,55	-
δ_2	-2,52	-
δ_{11}	-0,33	-
δ_{22}	-0,51	-

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The calculation value of the wave load is shown in Figure 102.

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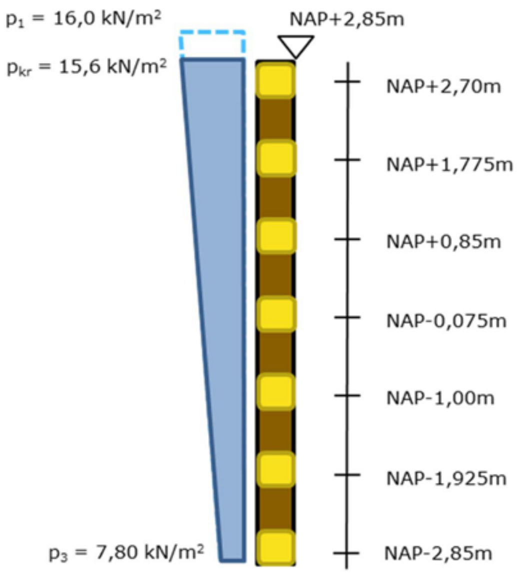

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Figure 102 Calculation value wave load on verification Building Decree

When the calculation values of the decay and wave load are combined, the calculation value of the total hydraulic load in Figure 103 follows.

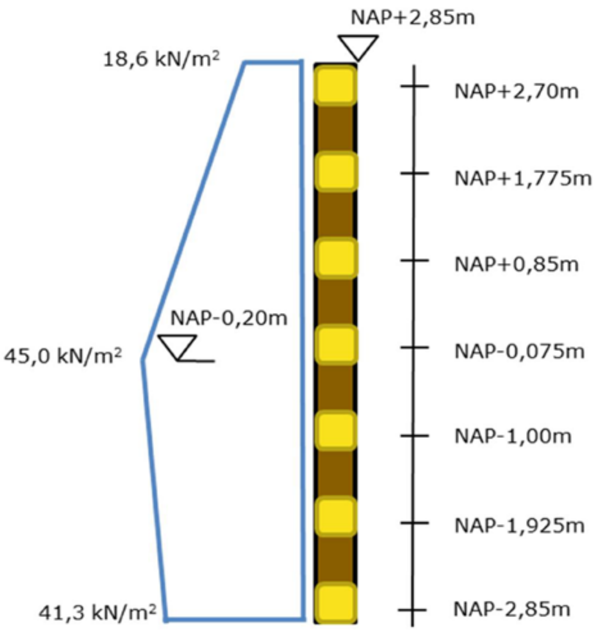

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Figure 103 Calculation value total hydraulic load on verification Building Decree

In the case, the horizontal beam at the height of NAP-1.00 m is verified, the calculation value of the hydraulic load according to Figure 103 must be translated into a distributed load p and spatial force F for the beam. The distributed load p is based on the water pressure that the beam must withstand at a height of NAP-1.00m, see Figure 104. As shown in Figure 104, the average calculation value of the hydraulic load is NAP-1, 00m equal to 43.8 kN / m^2.(239 頁) The calculation value of the resulting distributed load p on the beam is at a h.o.h. distance of 0.925 equals:

$$p = S_d \cdot h.o.h. = 43{,}8 \cdot 0{,}925 = 40{,}5\ kN/m$$

From this follows a calculation value of the field moment due to the hydraulic load for the beam equal to:

$$M_{S,d} = \frac{p \cdot l^2}{8} = \frac{40{,}5 \cdot 4{,}2^2}{8} = 89{,}3\ kNm$$

The axial force in the beam follows from:

$$N = \frac{W}{2\tan\alpha} = \frac{l \cdot p}{2\tan 22°} = \frac{4{,}2 \cdot 40{,}5}{2\tan 22°} = 211\ kN$$

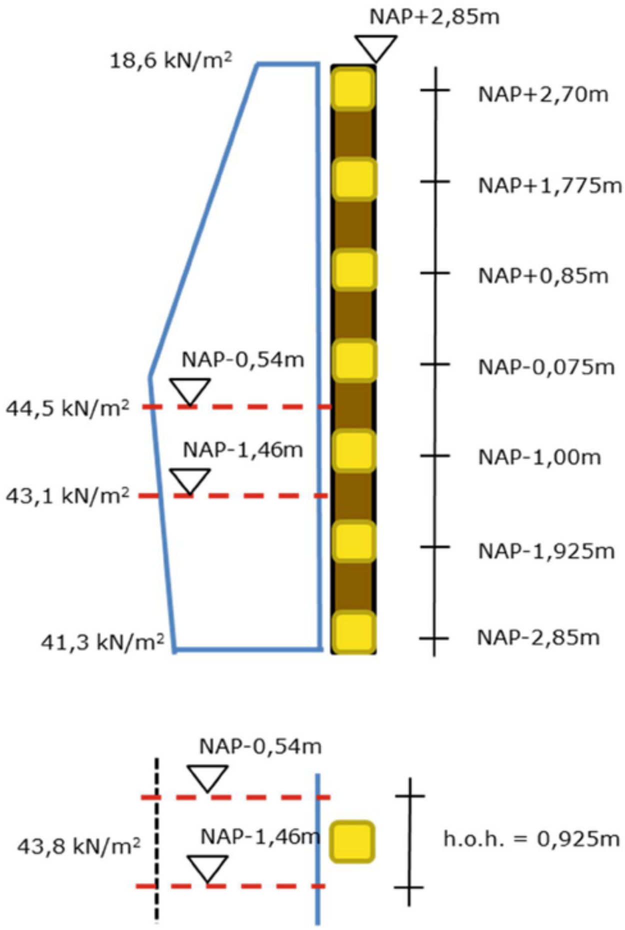

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Figure 104 Hydraulic load at the level of NAP-1.00 m

<u>Calculation value own weight tax G_d</u>

The cross-section of the horizontal beam is equal to:

$$A = b \cdot h = 0{,}3 \cdot 0{,}4 = 0{,}12\ m^2$$

The characteristic density of azobé (strength class D70) (Houtinfo.nl) ρ_k = 1000 kg / m³ = 10.0 kN / m³. This results in a divided own weight load:

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$$q = A \cdot \rho_k = 0{,}12 \cdot 10{,}0 = 1{,}2\ kN/m$$

From this follows a characteristic field moment due to the own weight load for the beam equal to:

$$M_{G,k} = \frac{q \cdot l^2}{8} = \frac{1{,}2 \cdot 4{,}2^2}{8} = 2{,}65\ kNm$$

In accordance with formula 6.10b and Table 12 the calculation value for the field momentum as a result of the own weight load follows:

$$M_{G,d} = k_{FI} \cdot \gamma_G \cdot \xi \cdot G_k = 1{,}1 \cdot 1{,}35 \cdot 0{,}89 \cdot 2{,}65 = 3{,}50\ kNm$$

Combine to total tax effect E_d

The load effect for the high water load situation is a combination of the hydraulic load and the own weight load. As mentioned, the verification in accordance with NEN-EN 1995 consists of two prescribed unity checks, in which the tax effect is expressed in three different terms, namely:

- The calculation value of the axial compressive stress due to the hydraulic load ($\sigma_{c,0,d}$)
- The calculation value of the bending stress due to the own weight load ($\sigma_{m,y,d}$)
- The calculation value of the bending stress due to the hydraulic load ($\sigma_{m,z,d}$)

Arithmetic pressure load calculation value $\sigma_{c,0,d}$:

- As calculated above N = 211 kN = 211,000 N.
- Area A = b · h = 300 · 400 = 120,000 mm^2
- This results in an axial compressive stress of:

$$\sigma_{c,0,d} = \frac{F}{A} = \frac{211.000}{120.000} = 1{,}76\ N/mm^2$$

Calculation value bending load around the y-axis $\sigma_{m,y,d}$:

- The calculation value of the bending stress in the outermost fiber around the z-axis follows from $\sigma_{m,y,d} = \frac{M_{y,d}}{w_y}$
- $M_{y,d} = M_{G,d} = 3.50$ kNm $= 3.50 \cdot 10^6$ Nmm
- Resistance moment around the y-axis: $w_y = \frac{1}{6} \cdot h \cdot b^2 = \frac{1}{6} \cdot 400 \cdot 300^2 = 6{,}0 \cdot 10^6\ mm^3$
- From this follows a bending stress equal to:

$$\sigma_{m,y,d} = \frac{M_{y,d}}{w_y} = \frac{M_{G,d}}{w_y} = \frac{3{,}50 \cdot 10^6}{6{,}0 \cdot 10^6} = 0{,}58\ N/mm^2$$

Calculation value bending load around the z-axis б m,y,d:

- The calculation value of the bending stress in the outermost fiber around the z-axis follows from $\sigma_{m,z,d} = \frac{M_{z,d}}{w_z}$
- $M_{z,d} = M_{S,d} = 89{,}3\ kNm = 89{,}3 \cdot 10^6\ Nmm$
- The moment of resistance on the z-axis:

  $w_z = \frac{1}{6} \cdot b \cdot h^2 = \frac{1}{6} \cdot 300 \cdot 400^2 = 8{,}0 \cdot 10^6\ mm^3$

- From this follows a bending stress equal to:

$$\sigma_{m,z,d} = \frac{M_{z,d}}{w_z} = \frac{M_{S,d}}{w_z} = \frac{89{,}3 \cdot 10^6}{8{,}0 \cdot 10^6} = 11{,}2\ N/mm^2$$

11.6.11 Step 11: verification in accordance with the Building Decree

As mentioned in step 6, NEN-EN1995 provides two unity checks for the verification of the maximal absorbable tension of the beam, when subjected to load according to Figure 96 (formulas 6.19 and 6.20):

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6.19 $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

6.20 $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

All calculation values of the strength and load terms are determined in steps: 7, 8, 9 and 10. From this follows:

6.19 $$\left(\frac{1{,}76}{22}\right)^2 + \frac{0{,}58}{27} + 0{,}7 \cdot \frac{11{,}2}{43} = 0{,}0064 + 0{,}022 + 0{,}7 \cdot 0{,}26 = 0{,}21 < 1$$

6.20 $$\left(\frac{1{,}76}{22}\right)^2 + 0{,}7 \cdot \frac{0{,}58}{27} + \frac{11{,}2}{43} = 0{,}0064 + 0{,}7 \cdot 0{,}022 + 0{,}26 = 0{,}28 < 1$$

The beam complies with a Building Decree verification with regard to the high water load situation.

11.6.12 Step 12: Determine the calculation value of the tax effect on the Water Act

All actions to determine the calculation value of the tax effect in accordance with the Water Act are carried out in the same way as for the Building Decree in step 10. However, there is a different failure probability, so that the calculation value of the load effect with other calculation values of the outside water level, the wave height, wave period and own weight load is determined.

The structural failure probability $P_{eis,kunstwerk,CON}$ is equal to 1 / 37,500 per year (2,67E-5) in accordance with step 4.

Again, the standard method from section 7.10.2 is applied. In step 9 it was concluded that:

- The wave load (H) and the decay load (V) are fully correlated
- The significant wave height (H_s) and the spectral wave period ($T_{m-1.0}$) are fully correlated.

Based on in step 10 it was concluded:

- Exceedance of probability calculation value of decay load: P ($V> V_d$) = P ($S> S_d$) = P_{db}
- Exceeding probability calculation value wave load: P ($H> H_d$) = P ($S> S_d$) = P_{db}
- Abutting probability calculated value of significant wave height: P ($H_s> H_{s, d}$) = P ($S> S_d$) = P_{db}
- Exceedance probability calculation value spectral wave period: P ($T_{m-1.0}> T_{m-1.0, d}$) = P ($S> S_d$) = P_{db}

The decay load is determined by the inside and outside water level. Since the inland water level (h_{bi}) is a deterministic variable in this example, the outdoor water level applies: P ($h_{bu}> h_{bu, d}$) = P ($V> V_d$) = P_{db}.

The constructive failure probability according to the Water Act must be described in a reliability index:

$$\beta_{\text{eis,KW,CON}} = -\Phi^{-1}\left(P_{\text{eis,KW,CON}}\right) = -\Phi^{-1}(2{,}67 \cdot 10^{-5}) = 4{,}0 \; per\ jaar$$

Then, from Table 13 and Figure 37 in Section 7.10.2.5, the probability of exceedance of the dominant tax $P_{db} = 7.0 \cdot 10^{-4}$ per year. From Table 13 and Figure 38 the load factor for the eigen weight tax $Y_G = 1.10$ follows. From Table 13 follows

K_{FI} = 1.0 and ξ = 0.89.

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Using the exceedance frequency lines from step 9 and $P_{db} = 7.0 \cdot 10^{-4}$ per year, the following parameters can be read:

Table 27 Calculation values of wave parameters and water levels when verifying according to the Water Act

$H_{s,d}$	1,08 m
$T_{m-1,0,d}$	3,65 s
$h_{bu,d}$	NAP+2,6 m
h_{bi}	NAP-0,20 m

※（Rijkswaterstaat, Central government　2018）242 頁より作成。

All hydraulic parameters in Table 27 and the load factor for the own weight tax turn out to be less than derived from the verification in accordance with the Buildings Decree in step 10. The calculation value of the tax effect according to the Water Act is therefore smaller than that of the Building Decree and therefore not normative.

11.6.13 Step 13: verification in accordance with the Water Act

The calculation value of the tax effect according to the Water Act is smaller than that according to the Buildings Decree: $E_{d,BB} > E_{d,WW}$.

Since the calculation value of the strength R_d is the same for both verifications, the verification with regard to the Building Decree is dominant. This means that the verification of the high-water load situation with regard to the reliability requirement in accordance with the Water Act for the horizontal beam at NAP-1.00 meters can be terminated with regard to the absorbable bending stress. The recordable shear stress and possibly also the stability of the beam still have to be verified for the high water load situation. The girder must then be checked for other load situations. In case these have already been done, other parts of the lock must be verified.

11.7 References

[Ref. 11.1] CUR 197 - Rubble in practice Part 2: dimensioning of constructions in inland waters

[Ref. 11.2] Rijkswaterstaat (Directorate-General for Public Works and Water Management) WVL, Guide to assurance of reliability of closure in scenarios, Background report for the use of the scoring tables for the failure mechanism not to be closed, November 2017, Definitive

[Ref. 11.3] Spreadsheet inflowing flow rate Dutch and Zeeland coast height, Bob van Bree, October 2018

[Ref. 11.4] Spreadsheet inflowing flow rate Vecht Delta, Bob van Bree, October 2018

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Terms and definitions

This section provides an overview of the most important terms and definitions as used in this Work Guide. Where possible, reference is made to source documents in which definitions are presented in order to prevent possible inconsistencies between the various documents.

Water barriers and works of art

Weir
A flood defense is an artificial height (dike), natural height (dune) or high ground including water-retaining components (for example works of art), or a succession of these, which has the function of separating or turning water over a certain length and which is indicated in, for example, the shelf. A distinction is made between primary flood defense systems (main flood defenses) and non-primary flood defense systems, including the regional water defenses.

Artwork
A work of art in general is a civil engineering work for the infrastructure of roads, water, railways, flood defense systems and / or pipes not intended for permanent human residence.

Water-resistant artwork
A water-retaining structure is a construction that forms part of a water-retaining structure and over a limited length takes over the water-retaining function of the ground body, but has been constructed for another (utilitarian) function that crosses the water-retaining structure (such as fencing and drains, but also a passage for traffic - denomination). In connection with this utilitarian function, these hydraulic structures are usually provided with one or more movable closing means.

Wet Artwork
A wet work of art is a civil-construction construction that is part of a waterway or waterway with the aim of regulating the water levels, passage of ships, flood protection, crossing of waterways and / or drainage of water. A denomination therefore does not fall under a wet work of art, but it does fall under a water-retaining artwork

Water-resistant object
A water retaining object is an object in or on the flood defense that takes care of the flood defense function completely independently or in combination with other components that form the barrier. A retaining wall (such as near Harlingen), a cofferdam or a quay wall belong to this category

Non-water retaining object
A non-water retaining object is an object on or in the dike that does not make a positive contribution to the flood defense function of the flood defense, or even has negative effects, such as pipes, houses and trees.

Connection constructions
The entire transverse and local longitudinal profile of a ground construction in its deviating form, at the transition to (in this case) the artwork.

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Reliability

In this working guide, the terms reliability requirement, limit state, limit state function, failure / failure, reference period, design lifetime are frequently used. For the definition of these terms, reference is made to the Foundations for flood protection ([Ref 10.1]). Some additional additions are made below.

Border condition
For the definitions of the different limit states (limit and serviceability limit state), reference is made to the Accounting Principles.

Border states are assessed within the framework of the different design situations. These can have to do with loss of balance, exceeding the (static) strength of the construction or the foundation and fatigue. In addition to the normally present static and variable loads, special design or load combinations (explosion, collision, etc.) must also be taken into account, with a relatively small chance of occurrence but with possibly considerable damage to the structure. With these extraordinary loads one accepts more damage to the construction than in the design situations for ordinary loads. Key concepts are resilience, prevention of continued failure and residual strength with respect to the primary functions.

Furthermore, the Eurocode also knows the seismic load combination; however, this falls outside the scope of this guideline.

Design life

The design service life is the period in which the construction or a construction component is deemed to fulfill its function. The (economically) optimal design life does not have to be the same for all parts of a construction. For example, the optimal design life span of a movement work will often be considerably shorter than that of a pile foundation.

Residual life

The period in which it is expected that an existing work of art will still be able to fulfill its (water retaining) function (from a certain reference point in time, for example a planned inspection), is also called the residual life. By extending service life-extending maintenance, the remaining service life can be increased.

Plan period

The plan period is the period that is considered in planning and plan development. The plan period is not the same as a design life of a construction (component). For example, it can already be foreseen in advance that extensions or modifications to constructions will be required within the plan period.

Failure definition

In assessing the reliability of flood defenses, models are used. Because of the state of the art, these do not always describe the entire process that leads to a flood. In that case, the ultimate limit is not actually tested, but a situation that is still somewhat removed from it. The description of this condition is also called the failure definition. For the failure definitions related to the failure mechanisms of transfer and / or overflow, non-closure, piping and structural failure, reference is made to the WBI test track reports ([Ref 4.1], [Ref 5.1], [Ref 6.1] and [Ref 7.1]).

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Water levels

Water level

Water level with respect to a reference level. In the Netherlands, the NAP (Normal Amsterdam Level) is generally used for this purpose.

Water depth
Difference between water level and soil level.

Expire
Water level difference between indoor and outdoor level.

Atrial level
Water level inside in relation to the reference level.

Outside water level / outside level
Water level outside in relation to the reference level.

Target level bosom
Water level on which the underlying area is controlled. Since this is a target level, this does not mean that the atrial level is a fixed water level. The water level in a chimney breast can fluctuate considerably, for example as a result of precipitation.

Maximum atrial level
Maximum permissible water level on the chimney breast (upper management limit).

Critical atrial level
Atrial level where no damage occurs in the hinterland.

Artwork related gauge

Threshold height
Height of the physical threshold of a work of art compared to the reference level. An example is the top of the bottom construction of a lock.

Turning height reversing device
Top of the closed turning means of a work of art compared to the reference level.

Turning height - abutment
Top of the water retaining part of the artwork compared to the reference level.

Closing level

The level on the outside on which the artwork must be closed (variable of choice determines the on-site operations).

Warning level
Low storm surge - limited mobilization (Definition according to Storm Surge Protection Service SVSD).

Alarming level
High storm surge - full mobilization (approx. Once every 5 years, definition according to Storm Surge Protection Service SVSD (now Water management center Nederland WMCN)).

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Highest and lowest lock level
Highest and lowest water level at which a gun lock is still used.

Open Reverse height
See definition in section 10.2.1.

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Establishment and quality assurance

In 2003, the Technical Advisory Committee on Flood Defense published the Guideline for Art Works 2003. This guideline has since been extensively used for the design and assessment of flood defense structures. Since 2003, knowledge and technological development has taken place and in 2017 the switch from exceedance probability to flood probability standards has been made. This has made it necessary to update the Guidelines for Art Works 2003.

In 2016, under supervision of Ton Vrouwenvelder and Jos Wessels (both TNO) and guidance from Arnaud Casteleijn and Ilka Tanczos (both RWS WVL), a set-up was made for a renewal of the Guidelines for Art Works 2003. In terms of content, the following persons have contributed to this: Ton Vrouwenvelder (TNO), Ruben Jongejan (Jongejan RMC), Raphael Steenbergen (TNO), Hans Niemeijer (Arcadis), Martin van der Meer (Fugro), Jentsje van der Meer (Van der Meer Consulting) and Bob van Bree (independent consultant)), Hessel Voortman (Arcadis), Bob Maaskant (HKV) and Alex Capel (Deltares).

Subsequently, two engineering firms (Royal HaskoningDHV and Movares), a contractor (Van Hattum and Blankevoort) and Rijkswaterstaat (Directorate-General for Public Works and Water Management) applied the new guide to the same case. This did not lead to a sufficiently unambiguous design. This has been a reason for Rijkswaterstaat (Directorate-General for Public Works and Water Management) WVL to turn the guideline into a working guide in which the emphasis has been placed on practical applicability: how can a design be made given the current state of technology and the availability of instruments.

This working guide is written by Arnaud Casteleijn (RWS-WVL), Rob Delhez (Greenrivers), Ruben Jongejan (Jongejan RMC) and Bob van Bree (independent consultant). Here, the results of the previous phases have been gratefully used. The chapter Hydraulic loads has been reviewed by Alfons Smale (Deltares). The chapter 'Structural failure' was realized in close collaboration with Raphaël Steenbergen (TNO Construction). Bas Jonkman (TU Delft) has reviewed the Work Guide in its entirety. The Work Guide was then applied to the same case by three engineering firms: Lievense, Antea and Sweco. In this case, however, an unambiguous design verification was achieved.

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Appendix A　Process of 'Designing' incl. LCC

A.1　Preface

A work of art is built for other reasons that turning water. Just when other functions must be combined with water turning is a work of art. At the design is of great importance to keep in mind from the start functions must be fulfilled by the artwork. It has big advantages to it design process explicitly based on the functions. A method to do that do is known under the name "Systems Engineering" (https://www.leidraadse.nl/). This appendix follow the principles of Systems engineering as much as possible.

A.2　Structure of the design process

The following concepts are important:

- Operational concept: a description in ordinary language of the planned operations with the artwork. Workable is a description of at most a few pages. The operational concept is the starting point of the design process. The description it is an important means of communication between the designer, the future user and the client.
- Function: a description of what the artwork has to do. Inside the operational concept, the artwork must be able to do certain things (for example: turn water, let ships pass).
- Aspect: where the functions arise from the wishes and needs of the future user, aspects arise from the fact that there is a work of art. Example: nobody will want a work of art to do maintenance. Maintenance is however an important aspect when designing a work of art. The same applies for example for landscaping.
- Requirement: a requirement sets a hard limit to a function or an aspect. Example is one prescribed deflectable height: too low does not meet, too high does. For aspects: a manhole for maintenance has a minimum size of 1m at 1m. To small does not meet, too big does.
- Assessment criterion[42]: not all aspects can be formulated in a requirement. Business like Landscaping is often not provided with a hard rejection limit. In the design choices such aspects are scored on a scale of worse to better (multi-criteria analysis). Among other things, the aspects that apply within the framework of an EIA should be treated in such a way.

- Technical solutions (or concepts): these are technical measures (variants) that to fulfill the requested functions and meet the requirements.

Although structuring the design process is of great importance, it is also important not to apply the above concepts too dogmatically. Furthermore, there is no one in practice a continuous line from the operational concept to the technical solution. A well-designed design process has an iterative character. Starting with a first one sketch of the operational concept becomes a first set of functions and aspects defined. From this follows a first set of requirements and assessment criteria. In practice it turns out that with a set of requirements of 10 to 15 requirements the core of the design problem can go well be covered. Such a small set of requirements is workable when the concepts still have to be completed are being developed.

Developing technical solutions is to a large extent a creative process. It is recommended to see the development of variants "separate" from the development of requirements.

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[42] The international literature on systems engineering makes a distinction between "requirements" and "measures of effectiveness”

(250 頁) The conscious development of solutions that apparently fall outside the requirements helps to either get the requirements sharp or to maximize the solution space to make.

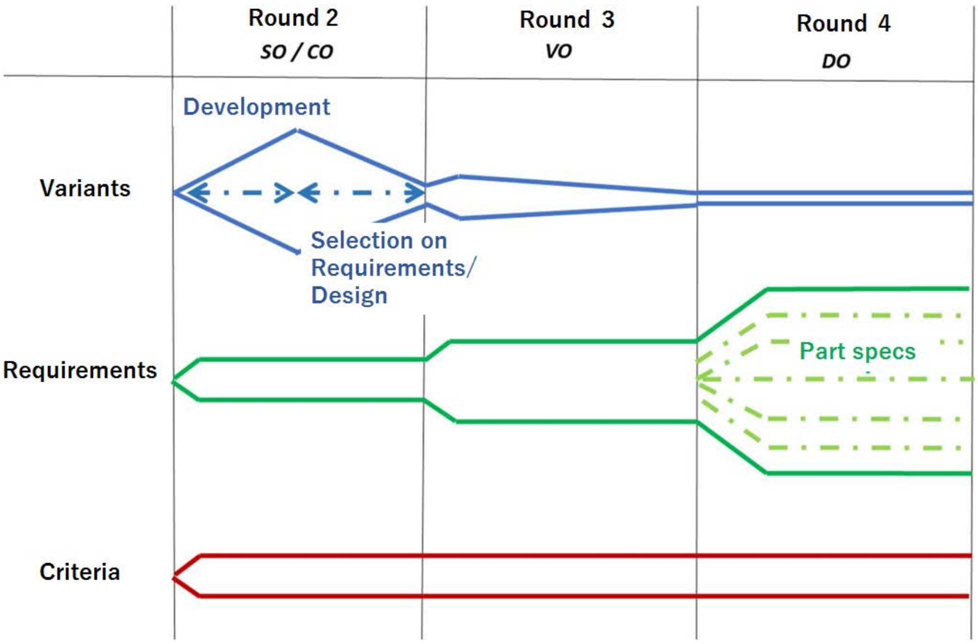
Finding is the process whereby developed variants are assessed on requirements and assessment criteria and the remaining set of variants is further reduced to the final preferred variant. An important principle in funnel is the principle of consistent coarseness. In the early phases of the design, the process is dominated of defining and substantiated choosing (or rejecting) different variants. A very detailed elaboration of all functions and aspects is not necessary, as long as the comparison between variants is done fairly. Because the number of variants is large at first is, a very comprehensive set of requirements and assessment criteria is even undesirable because of this makes the design process uncontrollable and distracts attention from the main issues.

Costs are generally treated as an assessment criterion (lower is better) but stand well apart from the other assessment criteria. By doing that you can make a trade-off between an expensive variant with very good performance and a cheap variant with only the minimum required properties.

As the design process progresses, the number of variants at the level of the artwork itself becoming smaller (until eventually one). Similar considerations then become made for parts of the object (for example, turning gear, movement work, energy supply). This further elaboration is accompanied by the development of variants at part level and the further detailing of the requirements to ultimately partial specifications. With the exception of costs, assessment criteria often play a role minor importance in the weighing of parts.

A final important element of structuring the process is stopping on time. A sub-specification for the energy supply system is still meaningful; a partial specification for a socket is not. With further detailing the number of requirements grows explosively while the added value for the process.

Figure A. 1 shows the structure of the design process.



※（Rijkswaterstaat, Central government　2018）250 頁より作成。

Figure A. 1 Structure of the design process

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A.3 Design of water defense works

A.3.1 Operational concept and functions

The operational concept describes all operations that should be possible with the artwork to be. In the context of this Guideline, one of the functions will always be turning water. The operational concept should (roughly) indicate under which circumstances the artwork is high-watering and under what circumstances not [43]. The operational concept is concise and described in natural language so that it is the basis for a shared view of the object to be designed.

Also for the functions applies that they must all be described, including of course the function water times. In many cases the design is determined by other functions and is turning something "that comes with it".

A.3.2 Environmental analysis

The operational concept and the functions of the artwork come from what it work in the larger system. The environment in which the artwork stands however, often has a dominant influence on the design. Analyze the environment of it artwork and determine which additional requirements and assessment criteria this imposes on it artwork. "Environment" must be interpreted broadly. It concerns the physical environment in which the artwork is realized, but also the administrative environment. Consider, for example, integration in urban areas, accessibility of the artwork for breakdown repair, maintenance but also in emergencies.

If the artwork is part of a long transport corridor, the environment can be over hundreds of kilometers and the environment can even be cross-border.

A.3.3 Aspect analysis

Aspects are intrinsic properties of the system to be designed which, as a rule, do not derive directly from a function to be fulfilled. An aspect analysis is analyzed which aspects are important for the (continued) functioning of the artwork. Examples of aspects are:

- Design
- Availability, reliability
- Maintainability
- Safety

- Future stability
- Demolition
- Ambient nuisance
- Durability

Aspects concern the entire artwork and are not limited to the function of water defense. Well the aspects of availability, reliability and maintainability are possible significant influence on the function of water defense, especially on the failure mechanism "not close".

A.3.4 Requirements

The operational concept, functions, environmental analysis and aspect analysis lead to requirements to which the artwork must comply. A requirement is "hard" and the evaluation or design it meets in principle with yes or no. To be able to do that a requirement for an authentication method (calculation method), a variable in which the performance of the artwork is expressed and a rejection limit.

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[43] An example of this is the lock in Empel. It returns to "outside" about 50 weeks a year (so it keeps the canal water in) and only returns the outside water of the Maas for 2 weeks a year. Only turning the Meuse water falls under the Water Act and therefore under this Guideline. Requirements must also be made for turning the canal water, but these do not arise from the “turning outside water” function

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Requirements must be defined from the beginning of the design process. The detail level of requirements increases as the design process progresses. The extent of the requirement set will grow as the design process progresses.

Example of the development of a requirement at different stages of the design (no. refer to the phases of the design; explanation in italics)

The level of detail of the elaboration of the requirements varies from case to case. A requirement is sooner detailed elaboration (and verified) when the risk that does not become on further elaboration is bigger. That is the case when the requirement is very strict and / or the possibilities to meet the requirements. The designer must at all times keep an eye on all requirements and per design step that evaluate requirements that determine the feasibility of the artwork. For a As a flood defense, sometimes the flood defense requirements are crucial, but sometimes they are not.

1. Concept choice
   Artwork must comply with the requirements of the Water Act (requirement is rather coarse and still bad verifiable; in many cases, however, it is good to make it plausible that the artwork can meet and the choice of concept is determined by other requirements).
2. Sketch design / preliminary design
   The failure probability of the artwork for the aspect of water defense should be smaller than or equal to xxx per year. Concerns an overall probability of failure for the object, derived from the location (dike section) and budgeting of the failure probability within the process. The design team estimates the feasibility of the requirement. When the requirement is relatively mild, experience can be determined at this stage that can be fulfilled. In other cases, an analysis can be done on some parts (such as reliability closure) are necessary.
3. Definitive design / execution design
   The artwork must meet the following task-setting failure probabilities:
   - Transshipment / overflow: xxxx per year
   - Close reliability: xxxx per year
   - Constructive reliability: xxxx per year (alternative description: risk class xxxx from Eurocode)

A.3.5 Assessment criteria

Like requirements, criteria also arise from the operational concept / function analysis, aspect analysis and environmental analysis. A purely assessment criterion differs substantially of a requirement. There is no rejection limit in a purely assessment criterion. An assessment criterion is assessed on a continuous scale from bad to good. In English literature contains criteria known as "measure of effectiveness".

In practice there are many mixed forms of requirements and assessment criteria. For instance in requirements regarding nature and environmental protection: the "soil" is formed by the legal requirements but a design that contributes positively to nature and the environment scores the criteria better.

Costs generally apply as an assessment criterion (cheaper is better), often in combination with a requirement (task setting budget). It is preferable to the costs to be treated separately from the other assessment criteria. This creates one clearer image which provides an extra investment for the performance of the artwork.

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Practical example; storm surge in the mouth of the Göta Älv (or River of the Geats) (Sweden)

The city of Gothenburg in Sweden regularly suffers from flooding caused by storm surges in the Kattegat. For this reason, the city of Gothenburg commissioned a feasibility study in 2015 to a storm surge barrier in the mouth of the Göta Älv (or River of the Geats). Special is that Stena Line with ferries and cruise ships moor in the city. The barrier must therefore be passable for this type ships (requirement). In the end, two variants remained:

- A barrier divides into three openings with cylinder doors; the big ships can join good conditions and passing the barrier at low speed
- A barrier in one opening, with floating sector doors; Two-way traffic is possible with the largest ships

Both options meet the requirements, but it will be clear that the criterion "quality shipping passage "the second option clearly outperforms the first. However, it also cost picture looks completely different. Mixing (adding) these criteria gives a diffuse picture where no clear preferred variant arises. When both criteria are separated held, it is immediately clear that a better nautical situation is possible at the expense of a (many) higher investment. The latter is a clear choice about which a decision can be made by the authorized persons.

A.3.6 Variants

The development of variants is to a large extent a creative process and distinguishes itself that respect of the formal and analytical process of requirement analysis. It is therefore wise to see the development of variants separately from the development of requirements.

Variants begin coarse (main dimensions, number of individual openings, barrier type). On the roughly described variants takes place on a funnel, resulting in a number of variants to lose weight. The remaining variants will be further detailed and then re-evaluated choice takes place.

A.4 Verification of the design

A.4.1 Verification general

Verification is the process in which it is explicitly established or a (designed) object meets the requirements set for it. To be able to carry out the verification is a requirement provided with a verification method and a measure of the performance of the artwork with linked to a rejection limit. A verification method must be appropriate for the requirement. Choosing an appropriate verification method is the responsibility of the designer.

A work of art usually has to comply with several sets of regulations at the same time (for example the Water Act and the Building Decree). It is the responsibility of the designer to ensure that all requirements are verified. Combining / integrating different sets of regulations is not necessary and can sometimes even lead to lack of clarity and "missing" requirements. What is the best way to deal with combinations of to deal with regulations is the responsibility of the designer.

A.4.2 Verification of flood defense requirements

This guide provides guidelines for the verification of requirements from the Water Act the artwork will be asked. Other frameworks for verification are not considered in this work guide. **(254 頁)** The designer is responsible for choosing appropriate verification methods for the function of water defense.

Not all parts of a work of art fulfill a function when turning water. Ten For the purposes of the verification, it must be made explicit which parts of the function water defense systems are involved.

Parts that do not turn water can possibly cause damage if they fail water-retaining components (e.g. bollards and fenders in a lock). The designer must explicitly analyze such failure possibilities. Possibly such parts are still classified as part of the flood defense.

The flood defense does not stop where the artwork stops. The connection to the environment makes part of the barrier and must be verified in the design.

A.5 Special topics

A.5.1 Judge

The manager must always be able to demonstrate that the artwork meets the requirements from the Water Act (duty of care). This is already taken into account in the design, for example by making explicit which parts are part of the flood defense and by to make water-retaining parts inspectable. Once every 12 years, the performance of the flood defense system formally reported to parliament.

For new artworks, the design documentation must specify to which criteria the artwork must meet in order to comply with the flood defense requirements. Employed assumptions are explicitly recorded so that it is easy to determine in the management phase or circumstances have changed in an unfavorable sense.

A.5.2 Management and maintenance

Management and maintenance is necessary to ensure that the artwork continues to comply with the performance requirements. With the design of the artwork, therefore, the maintenance strategy to be co-designed. When detailing maintenance facilities designed in accordance with the maintenance strategy. Special one attention is needed for the sometimes extremely high taxes that may accompany maintenance operations. Consider, for example, the lifting of reversals; the associated loads from the cranes must be able to pass through the construction be included.

For new structures, management and maintenance must be included in the design by:

- Determining the minimum quality criteria per component for each component must comply.
- Determining a maintenance concept for each component and associated with it inspection and maintenance regime.

A.5.3 Reliability closure in the design

The rail reliability closure differs from the track transfer and constructive collapse because the performance is dependent on hard (technical) and soft (organizational) factors. The design therefore belongs to the design of artworks of an operating regime, including mobilization, staffing, et cetera. This applies for both regular use and operation as well as any backup measures and emergency procedures. In a RAMS analysis all these aspects come together in one structured analysis.

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As a rule, a work of art does not meet the requirements when it is first passed on, but is adaptation of the design. The RAMS analysis iterates together with the design the final image and, like other parts of the draft dossier, goes from "gross to nice".

Measured technical measures (backup systems etc.) are clearly visible in the design documentation. However, these measures go hand in hand with measures to be taken on the process side (operation). Often comes the detailed description of the mobilization procedures and operation only later. It is therefore of great importance to be in the draft documentation to establish which procedures are presumed / agreed when making the design.

In practice, the reliability closure aspect appears to be the most conflicting issue deliver between the function water defense and the other functions of the artwork.

Examples of the interaction between the function water barriers and other functions

- A storm surge barrier has a barrier that rotates under water when it is open. With In view of the safety of shipping, the defense is provided with an open position in the open position latch. However, not being able to pull the bolt leads to failure of the flood defense because it cannot be closed (action taken for the benefit of one function leads to poorer performance of water retaining).
- With a view to water safety, it was decided to provide a lock with two heads up to full turning height. Because always one of the heads is closed, the barrier is always closed. However, the locked door can be hit upon when locking (extra failure mechanism) flood defense due to presence of the other function).

- A discharge sluice must meet a strict requirement with regard to flood risk management. To that reason a concept is chosen where the barrier, in case of loss of the energy supply, automatically closes on its own weight. However, this solution goes directly costs of the availability of the "drainage of water" function (better performance as a barrier leads to worse performance on another function).

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Appendix B　Standard failure probabilities

For reversals that are already regular from the primary function of the artwork automatic opening and closing are possible failure processes such as alarm, mobilization and operation not directly applicable. After all, the primary function requires that the reversing means are closed from a different function than high water times. Which means that in the case of a properly functioning work of art these reversing means are closed as soon as the primary function is not exercised. For such reversals are in the table below gives a number of failure probabilities, which are largely based on conservative estimates. An actual substantiation is hardly or not at all available. The numbers are partly taken from the first version of the Guideline Artworks (2003).

Turning means	**Event**	**Failure rate per closing question [1 / question]**
Watergate	Movement work	10^{-4}
	Sand / dirt on the bottom	10^{-4}
	Obstruction on soil	10^{-4}
Check valve	Reject closure	10^{-5}
Wake-up door	Reject closure	10^{-5}
	Obstruction / sand / dirt	10^{-3}
Sliding valve	Movement work	10^{-4}
	Obstruction on soil	10^{-5}
Butterfly valve	Do not close	10^{-5}
Aggregate	Do not start	10^{-4}

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The above numbers are a first indication of possible failure probabilities per closing question. Before one of the above opportunities is applied, it should be considered how real it is that one of the mentioned events occurs in the situation of the design artwork.

Example: Sand / dirt on the floor near a floodgate can cause the floodgate does not close at all (door is blocked in his greenhouse) or that the lock gate does not close completely because sand / dirt does not stop the door can lower its threshold. In the first case, there is a large one opening in the flood defense and this must be taken into account at the remainder of the analysis (large flow opening). In the second case there will be there is only a gap, the flow opening is limited, and this should be taken into account in the remainder of the analysis. When the administrator regularly inspects whether there is a question sand / dirt on the bottom or from the use data over several years that this never occurs, or that there is no sand / dirt during inspections noted, this event does not have to be taken into account at all are held. At a lock that the storm door turns into protective process (several times a day), then it is very likely that sand / dirt on the bottom will not be an issue.

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Appendix C Verification flood water situation with dominant own weight

In the exceptional situation that the own weight tax is very dominant compared to the hydraulic load, the design must also be verified tax effect by means of 6.10a has been determined. The criterion is that the own weight tax should be 80% or more of the total tax.

In Figure 105 is again a translation of the prescribed exceedance probability of the calculation value of the hydraulic load P ($S > S_d$) as prescribed exceedance probabilities of the calculation values of decay and wave load P ($V > V_d$) and P ($H > H_d$) included, but now for the tax effect according to 6.10a.

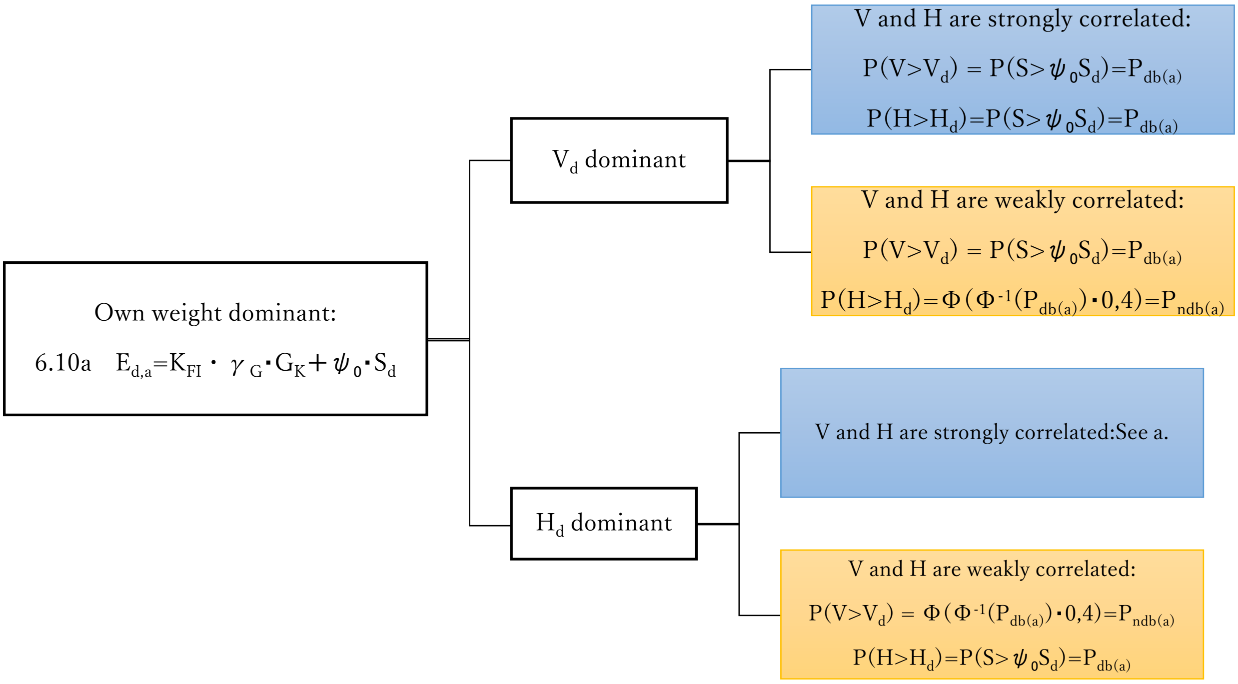

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Figure 105: Exceeding probabilities calculation values of decay and wave load according to 6.10a.

- $P_{db\ (a)}$ = exceedance frequency in case of dominant load
- $P_{ndb\ (a)}$ = exceedance frequency in case of non-dominant load
- Quantification $P_{db\ (a)}$ and $P_{ndb\ (a)}$ according to Building Decree and Water Act: see the tables below.

With a verification according to 6.10a it is not possible to estimate in advance whether the Building Decree or the Water Act is decisive for the tax effect $E_{d,a}$. Both a verification of 6.10a according to the Building Decree and the Water Act must be carried out.

Calculation value according to reliability requirement Building Decree

For the purposes of the verification of the Building Decree, the following table is used, in which the probability of exceeding the calculation value of the hydraulic loads (water level and wave height) must be taken into account from the 1-year tax statistic in the last (mostly 100th) year of life, depending on the failure probability $\beta_{eis,\ BB}$ (see paragraph 7.7.2).

Table 28 Calculation values hydraulic load Building decree

Consequence class	$\beta_{\text{eis, BB}}$ for reference period equal to lifetime	$P(S>\Psi_0 S_d) = P_{db(a)}$[-] Involvement in 1-year statistics in the first year of life	$P_{ndb(a)}$ [-] Involvement in 1-year statistics in the first year of life	K_{FI}*	ξ *	γ_G**
CC1	3.3	$4.0 \cdot 10^{-2}$	$2.4 \cdot 10^{-1}$	0.9	0.89	1.35 of 0.9**
CC2	3.8	$1.0 \cdot 10^{-2}$	$1.8 \cdot 10^{-1}$	1.0	0.89	1.35 of 0.9**
CC3	4.3	$4.0 \cdot 10^{-3}$	$1.4 \cdot 10^{-1}$	1.1	0.89	1.35 of 0.9**

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Calibrated with consideration of $\Psi 0 = 0.6$

* Value in accordance with NEN-EN 1990 / NB

** 1.35 in case of unfavorable working and 0.9 in case of a favorable working weight

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Calculation value according to failure probability Water Act

For the Water Act verification, the tax effect must be determined in 6.10a in the 1st year of life. The exceedance probabilities $P_{db\,(a)}$ and $P_{ndb\,(a)}$ are shown in Figure 105 and the other parameter values are shown in Table 29. P_3 and P_4 must therefore be included in the tax statistics for the first year of life, depending on the failure probability $\beta_{\text{eis, KW, CON}}$ (see paragraph 7.7.1), where:

$$\beta_{\text{eis,KW,CON}} = -\Phi^{-1}(P_{\text{eis,KW,CON}})$$

Here is:

$\beta_{\text{eis, KW, CON}}$ Failure probability expresses in a reliability index for structural failure and no failure by overflow / transshipment for a reference period equal to t_{ref} = 1 year [-]. See section 7.7.1.

Φ-1 (...) Inverse of the standard normal distribution

The parameter values for the own weight load are also included in Table 29.

Parameters	Values in 1st year of life
K_{FI}	1,0
ξ	0,89
Ψ_0	0,6

γ_G	See Figure 107
$P_{db(a)}$ en $P_{ndb(a)}$ [-] Involve on 1-year statistics in the first year of life	See Figure 106

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Table 29: Recommended exceedances and parameter values according to 6.10a when verifying Water Act

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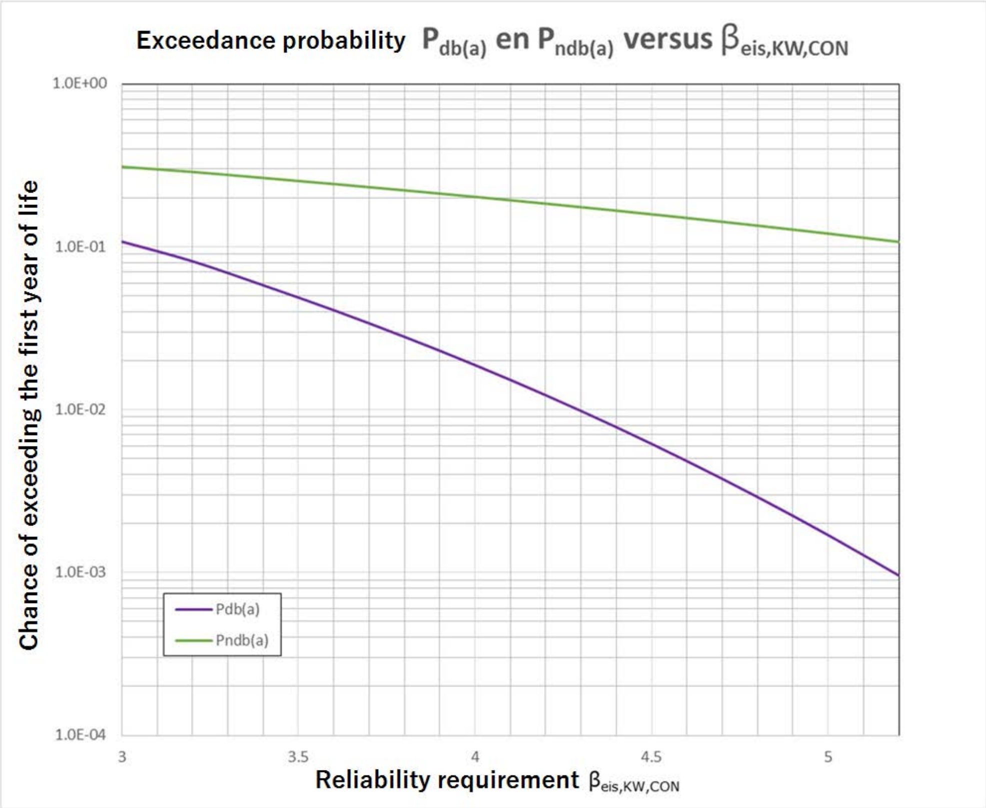

※（Rijkswaterstaat, Central government　2018）261 頁より作成。

Figure 106 Recommended exceedance probabilities decay and wave load $P_{db(a)}$ and $P_{ndb(a)}$ according to 6.10a on verification Water Act

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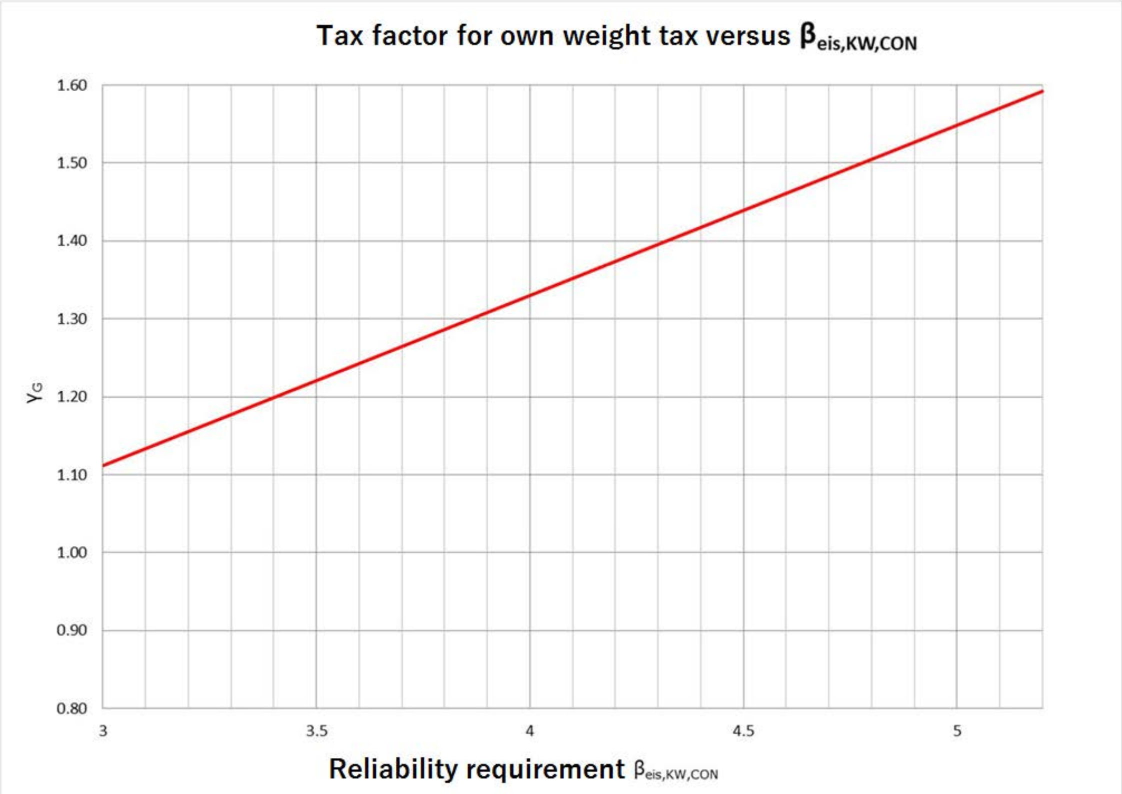

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Figure 107 Recommended partial factor for self-weight tax for inspections based on the Water Act in case of a dominant self-weight tax

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Appendix D Wave tax according to Goda's model

In determining the wave load, three types of waves are usually distinguished know non-breaking, breaking and broken waves. There are many different ones for this models available that can translate one or more types of waves into one wave tax. These models each have their own advantages and disadvantages. In this work guide only the modified model of Goda is discussed.

Goda has a general formulation of the wave pressure on a caisson on a dumping ground threshold. The equations of Goda apply to both non-breaking and breaking waves. It should be noted that the model is in principle 'state of the art', but it is based on curve fitting on the results of experiments. The formulation is through Takahashi et al [Ref. 7.15] adapted to be used for waves that are against break the construction. Goda's comparisons are not only becoming worldwide used in the design of vertical breakwaters, but also for the design of flood defenses. Although the comparisons are derived for breakwaters on a dumping ground threshold, they are also widely used for walls without a threshold.

The equations do not change in case of transfer. The model of Goda is, however, unsuitable when large waves of waves are breaking at heavy waves occur in combination with reversing means in a water-retaining structures. The model is designed for breakwaters, compared with reversing means relatively light. As a measure of heavily breaking waves at retarders the ratio of significant wave height and water depth at construction> 0.5 are used with a slope steeper than 1:50. This situation, however, is not very common in Dutch works of art. A second situation where large wave impacts can occur is when the wall is irregularly shaped, so waves can be 'trapped'. In this in cases, there is no question of a vertical wall and the model of Goda cannot applied. This occurs, for example, with a slide that is in a tube, or where a crossbar (top) protrudes. Such a projection can especially be large wave impact loads if it is around the waterline. In that case, a numerical or physical model to be used or other design choices to be made to prevent such peak loads. A last case involving the use of the formula of Goda must be watched as the waves are a double-headed wave spectrum show what can occur on the coast. This can be an apparently small one the immersion component make a major contribution to the wave force. When using the wave period $T_{m-1,0}$ in the formula of Goda can underestimate the maximum power to become. The period of the low-frequency peak can be used as a safe assumption. Unfortunately, Hydra-NL does not provide this information, so when this situation becomes a real risk Estimated, a numerical or physical model should be used.

Goda's model, moreover, seems somewhat conservative, which is not the case for designs problem. In Van der Meer et. al ([Ref.2.22]) becomes the formula of Goda compared with testing. This showed that the formula averaged the maximum power with 10% overestimated, with a spread (standard deviation) of 25%.

The modified model of Goda is also part of the WBI instruments and programmed in Risk, so that the use for designs fits well with future legal reviews.

This appendix deals with the application of the Goda model for the probabilistic

verification of the design, where the model is fed with calculation values. **(264 頁)**
For more detailed information about Goda's model referred to [Ref. 7.15].

Wave pressure distribution in calculation values

The wave pressure distribution according to the Goda model is included in Figure 108 together with some determining parameters. Only a few parameters are discussed in detail in the text, while the rest are offered in Table 30 and Table 31, where all variables used in the Goda model are explained.

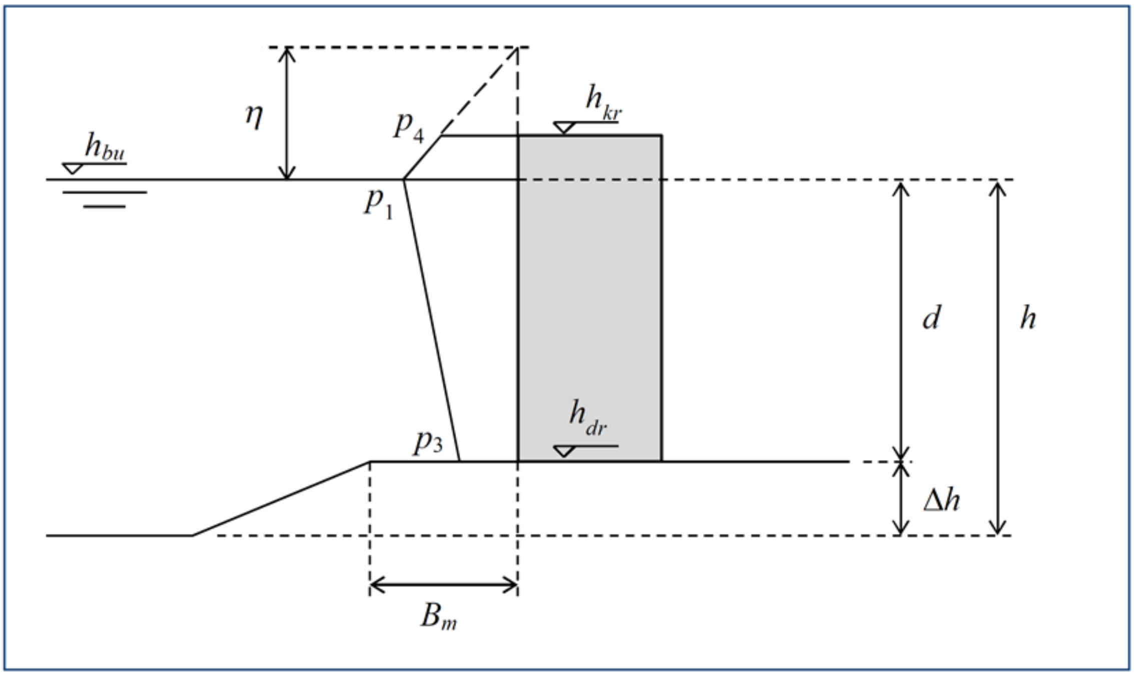

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Figure 108: Wave pressure figure according to the model of Goda

The pressure figure is composed by the values:

$$p_1 = 0{,}5 \cdot (1 + cos(\theta_{wall} - \theta_{waves})) \cdot (\lambda_1 \cdot \alpha_1 + \lambda_2 \cdot \alpha_{max2i} \cdot cos^2(\theta_{wall} - \theta_{waves})) \cdot \gamma_w \cdot H_d$$
$$p_3 = \alpha_3 \cdot p_1$$
$$p_4 = \alpha_4 \cdot p_1$$

There is:

$$d = h_{bu} - h_{dr}$$

In case $h_{bu} \leq h_{dr}$:

$$\eta = p_1 = p_3 = p_4 = 0$$

In the case of $h_{bu} > h_{dr}$, the values of η p_1, p_3 and p_4 are calculated in the following manner, including intermediate steps.

The modification factors λ1, λ2, λ3 depend on the construction shape of the wall. For a straight wall, all factors are equal to 1. In [Ref. 7.18] are some examples for the values of λ1, λ2, λ3 for different construction types. For specific forms, the modification factors can be determined with model research.

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Wavelength and wave period

The wavelength (L) can be iteratively determined in the following way:

$$L = \frac{gT_p^{\,2}}{2\pi}\tanh\left(\frac{2\pi h}{L}\right) \qquad \text{C.1}$$

At which:

$$T_p = 1{,}1 \cdot T_{m-1,0}$$

With:

T_p Peak period wave

$T_{m-1,0}$ Spectral wave period

The spectral wave period $T_{m-1.0}$ follows from the hydraulic boundary conditions, for example derived with Hydra-NL. In that case only the marginal statistic of $T_{m-1.0}$ is known and an estimate of the degree of correlation with the wave height, wave direction and water level. This is discussed in sections 7.10.2.1 and 0.

The wavelength can also be accessed in the following way ([Ref 7.16] and [Ref 7.17]):

$$\omega_0 = \frac{2\pi}{T_p}$$

$$k_0 = \frac{\omega_0^{\ 2}}{g}$$

$$k = k_0 \left(1 - \exp\left(-\left(k_0 h\right)^{5/4}\right)\right)^{-2/5}$$

$$L = \frac{2\pi}{k}$$

Although equation C.1 is preferred, the error of the approximation is less than 1%.

Wave height

The design wave height (H_d) is derived from the significant wave height (H_s), of which the marginal (but exclusively omnidirectional) statistic can be obtained with Hydra- NL. The marginal statistic of the spectral wave height $T_{m-1,0}$ can also be used with Hydra-NL are determined. In the absence of information about the relationship between individual wave heights and For the time being, wave periods are recommended to include the calculation values of H_s and $T_{m-1,0}$ determine the same exceedance. The spectral wave period $T_{m-1,0}$ eventually becomes used in combination with the design wave height H_d . For more information about the Correlation between the wave height, wave direction and water level is referred to paragraphs 7.10.2.1 and 0.

As discussed in chapter 3 Hydraulic preconditions, it is possible are currently the hydraulic preconditions on the site of the design construction is not yet distracted and one only has access to the preconditions on deep water (see Helpdesk Water). In that case the significant wave height and the peak period at the site of a flood defense using the Golf Tax Guide ports and protected areas [Ref. 7.21]. Depending on the bathymetry there can be phenomena such as shoaling, refraction and diffraction. These influence the wave height. The effect is described with:

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$$H_s = K_s \cdot K_r \cdot K_d \cdot H_{s,0}$$

With:

H_s	The significant wave height of the incoming wave just before the construction [m]

$H_{s,0}$ The significant wave height on deep water [m]
K_s The shoaling coefficient [-]
K_r The refraction coefficient [-]
K_d The diffraction coefficient [-]

The first, very simple approach can be based on:

$$H_s = H_{s,0}$$

For the definitive calculation of the wave load, the various coefficients have to be determined more accurately. The refraction and diffraction coefficients can be determined using mathematical models. For further information, please refer to Collegehandleiding Windgolven (Wind waves college manual) [Ref. 7.16] and Collegehandleiding Korte golven (Guideline Short waves) [Ref. 7.17].

The design wave height (H_d) is chosen so that the exceedance probability of this value during the storm peak is approximately 10%. For the average Dutch conditions along the North Sea coast, on the Wadden Sea, on the Ijsselmeer and in the river area, a safe value can be assumed, based on a Rayleigh distribution of wave heights, of:

$$H_d = 2{,}2 \cdot H_s$$

With:
H_d Design wave height of the incoming wave just before the construction [m]

The design wave height can be physically limited by the water depth. In such a case the wave breaks for the construction and the height is limited. For this reason, the following applies:

$$H_d \leq 0{,}9 \cdot D_{0,5L} \qquad \text{C.2}$$

With:
$D_{0,5L}$ Water depth at approximately half wavelength (L / 2) for the construction

[m]

If this condition is not met, the design wave height of the incident wave for the structure must be reduced to the maximum value according to formula C.2 above.

In the model of Goda the reflection coefficient does not occur. The model assumes complete reflection. In the model, the wave height H_d of the incoming wave (without reflection) has to be entered. The reflection is discounted in the expressions for the wave pressures.

Wave pressure coefficient model of Goda

$$\delta_{11} = 0,93 \cdot \left(\frac{B_M}{L} - 0,12 \right) + 0,36 \cdot \left(0,4 - \frac{d}{h} \right)$$

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$$\delta_{22} = -0,36 \cdot \left(\frac{B_M}{L} - 0,12\right) + 0,93 \cdot \left(0,4 - \frac{d}{h}\right)$$

$$\delta_1 = \min\{20 \cdot \delta_{11}; 15 \cdot \delta_{11}\}$$

$$\delta_2 = \min\{4,9 \cdot \delta_{22}; 3,0 \cdot \delta_{22}\}$$

$$\alpha_{imp1} = \begin{cases} \dfrac{cos\delta_2}{cosh\delta_1} & \text{als} \quad \delta_2 \leq 0 \\ \dfrac{1}{cosh\delta_1 \cdot \sqrt{cosh\delta_2}} & \text{als} \quad \delta_2 > 0 \end{cases}$$

$$\alpha_{imp0} = min\left\{\frac{H_d}{d}; 2\right\}$$

$$\alpha_{impuls} = \alpha_{imp0} \cdot \alpha_{imp1}$$

$$\alpha_1 = 0,6 + 0,5 \cdot \left\{\frac{\frac{4 \cdot \pi \cdot h}{L}}{\sinh\left(\frac{4 \cdot \pi \cdot h}{L}\right)}\right\}^2$$

$$\alpha_2 = min\left\{\frac{\left(1 - \frac{d}{h}\right) \cdot \left(\frac{H_d}{d}\right)^2}{3}; \frac{2d}{H_d}\right\}$$

$$\alpha_3 = 1 - \left(\frac{d}{h}\right) \cdot \left\{1 - \frac{1}{\cosh\left(\frac{2 \cdot \pi \cdot h}{L}\right)}\right\}$$

$$\alpha_{\max 2i} = \max\{\alpha_2; \alpha_{impuls}\}$$

$$\eta = 0,75 \cdot (1 + \cos(\theta_{wall} - \theta_{waves})) \cdot \lambda_1 \cdot H_d$$

$$h_{c,corr} = \min\{\eta; \max\{(h_{kr} - h_{bu}); 0\}\}$$

$$\alpha_4 = 1 - \frac{h_{c,corr}}{\eta}$$

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Variables in Goda's model

Table 30: Input parameters model of Goda

Symbol	Variable	Unit
B_M	Berm width	m
g	Gravitational acceleration (9.81 m / s^2)	m/s^2

h_{bu}	Outside water level	m+NAP
h_{dr}	Height of the bottom of the wall	m+NAP
h_{kr}	Height of the top of the wall	m+NAP
H_S	Significant wave height	m
T_p	Peak period wave	s
Δh	Vertical distance between the bottom of the wall and the toe of the dike / berm	m
γ_w	Volumetric weight of water	kN/m^3
λ_1	Modification factor for the geometry of the wall	-
λ_2	Modification factor for the nature of the wall	-
θ_{wall}	Angle between a line perpendicular to the turning means and the north	°
θ_{waves}	Angle of wave fall in relation to the north	°

※（Rijkswaterstaat, Central government　2018）268 頁より作成。

Table 31: Output parameters model of Goda

Symbol	Variable	Unit
η	Distance between the highest point of the wave pressure distribution according to the Goda formula and the outside water level	m
ω_0	Wave frequency on deep water	s^{-1}
d	Vertical distance between the still water line and the bottom of the wall	m
h	Vertical distance between the still water line and the toe of the dike / berm	m
H_d	Calculation value of the wave height (without reflection)	m
k	Wave number	m^{-1}
k^0	Wave number on deep water	m^{-1}
L	Wavelength	m
p_1	Wave pressure at the level of the still water line	kN/m^2
p_3	Wave pressure at the level of the bottom of the wall	kN/m^2

p_4	Wave pressure at the level of the top of the wall	kN/m^2
α_{imp0}	Factor for the effect of the threshold height	-
α_{imp1}	Factor for the effect of the shape of the threshold	-
α_{impuls}	Pulse wave pressure coefficient	-
α_1	Slowly varying pressure component	-
α_2	Wave pressure component	-
α_3	Quotient of p_3 and p_1	-
α_4	Quotient of p_4 and p_1	-
δ_1	Coefficient 1 in the Goda formula	-
δ_2	Coefficient 2 in the Goda formula	-
δ_{11}	Coefficient 11 in the Goda formula	-
δ_{22}	Coefficient 22 in the Goda formula	-

※（Rijkswaterstaat, Central government　2018）268 頁より作成。

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Appendix E Roadmap probabilistic verification of flood water

This verification method is only explained to a limited extent for the high water load, the roadmap is roughly outlined in Figure 109 and is very similar to the semi-probabilistic one variant in Figure 34. Instead of calculating with calculation values and using a unity check verifies whether the design meets, is now explicitly with the probability distributions of load and strength worked and a failure probability verification done. In front of more information is referred to CUR190 Opportunities in civil engineering, part 1: probabilistic design in theory [Ref. 7.7] and the TU Delft dictation Probabilistic Design [Ref. 7.8].

Step VII: The hydraulic load (decay and wave load) must be used are made of the correct hydraulic boundary conditions, see chapter 3 Hydraulic preconditions. In the event that waves play a role playing at the moment (2018) is almost impossible to play probabilistic verification outside Risk to do. For the hydraulic tax should be made use of the combined statistics of water level and wave conditions, which only in Risk is available. For the time being, Riskeer ((Ring test) is a software application that supports the WBI-2017 assessment) is only suitable for assessment and not design, because the program only has the tax statistics with year of view 2023. In case there is no or limited waves can be expected in extreme conditions and the hydraulic load so, due to decay tax is dominated, outside of Riskeer probabilistic. With Hydra-NL in that case for the export location near the artwork to be designed the external water statistics are obtained, on which a probability distribution can be fitted.

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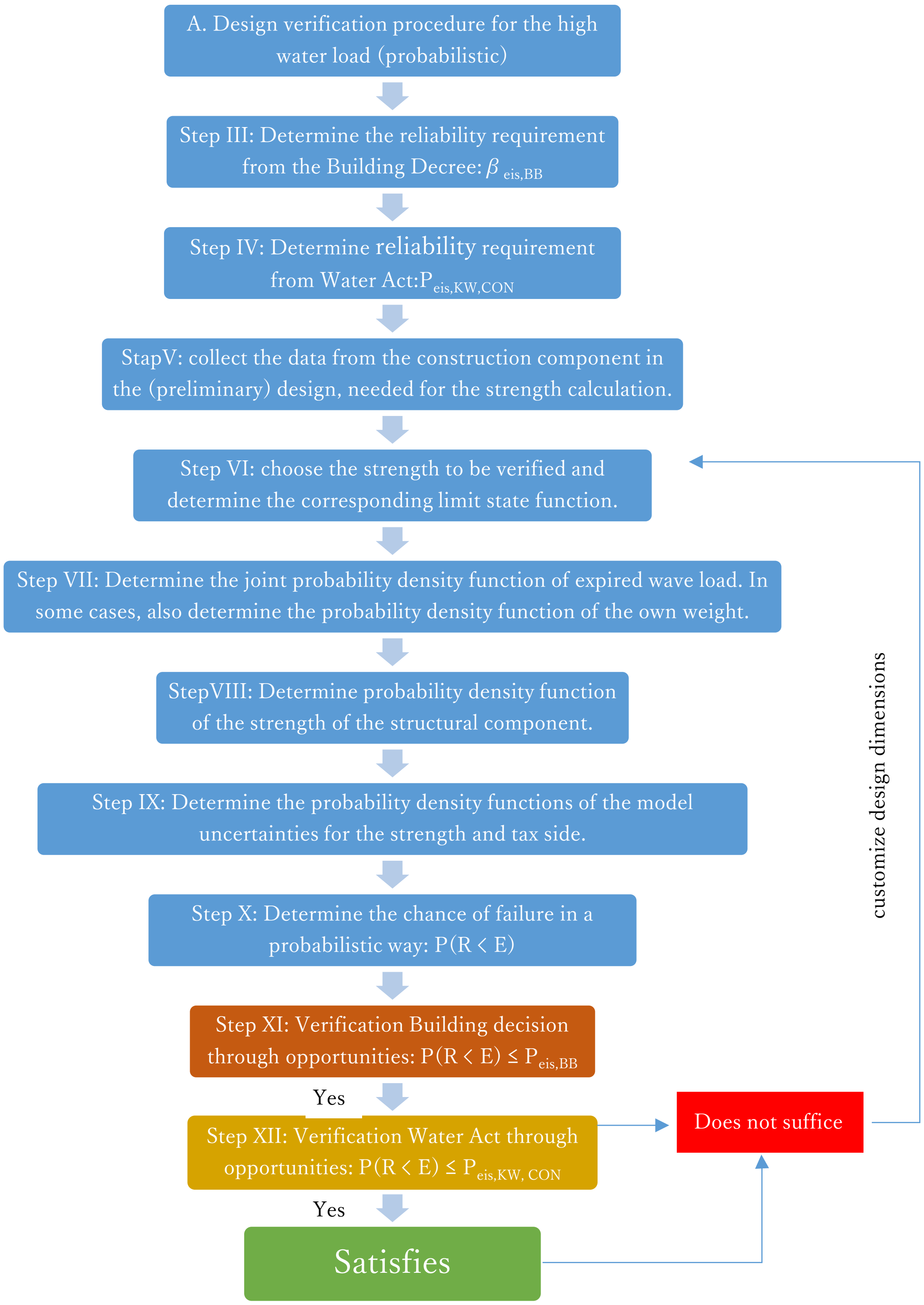

Figure 109: Step-by-step probabilistic design verification A. for high water load

※（Rijkswaterstaat, Central government　2018）270 頁より作成。