Effects of transitions and objects on the erodibility of grass revetments on dikes

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ABSTRACT: This study deals with the effects of transitions and/or objects such as roads and trees in grass revetments on dikes/levees under (wave) overtopping conditions. Three types of impact categories are defined, that is 1) little impact, 2) moderate impact and 3) large impact. Based on the grass erosion model (i.e., the cumulative overload method) a relation is deduced that describes the critical overtopping discharge as function of the load–increase factor for both a sea and a river regime. This impact factor represents the load increase near transitions and objects which is here expressed by a decrease of the critical overtopping discharge. Lognormal distributions for the critical overtopping discharge are used to calculate the failure probability due to overtopping with and without transitions and objects. For several dike–sections in the Netherlands the differences are calculated with the model PC-Ring. The study shows that the differences are largest for dikes along the coast and lakes. For dikes in a river regime where the significant wave height is usually low the erosion differences are marginal.

1 INTRODUCTION

In order to protect the Netherlands against flooding, the resistance of dikes, hydraulic structures, dams and dunes has to be assessed. These assessment regulations are summarized in technical reports and software and are being developed in various research programs (in the Netherlands, such an example is the WTI-2017 program). Not only desk studies, but also practical experiences show that transitions and (non-water retaining) objects (such as buildings and trees) are usually weak spots in flood defenses (e.g. Seed et al 2005). With the presence of transitions and objects on dikes it is expected that the probability of flooding increases. This paper discusses some computational results obtained from the PC-Ring model.

2 THEORETICAL BACKGROUND

2.1 *General*

Under extreme conditions when the water level is high and the largest waves reach the crest of the dike, waves run over the crest to the inner slope. The (wave) overtopping discharge is an average amount of water per unit width and in normal conditions varies from 0.1 ℓ/s per m to 10 ℓ/s per m; otherwise severe damage or flooding can occur. A wave height of 2 m with a wave period of 5.7 s is characteristic for dikes along the Dutch coast and estuaries. In rivers the wave height is clearly smaller.

Tests on the Vechtdijk in the Netherlands were conducted simulating wave conditions with significant wave heights of 1 m, 2 m and 3 m. Each test lasted 6 hours in which the wave overtopping discharge ranged from 0.1 ℓ/s per m to 75 ℓ/s per m. These experiments were used for calibrating the unknowns in the overload method, as described in section 2.2 .

Usually the wave overtopping tests were carried out by using a fixed hydraulic condition, that is, a significant wave height of 2 m. Below the theoretical background of the overload method is described.

2.2 *Overload method*

The (cumulative) overload method determines the damage on the inner slope of the dike provided that the load of the waves as function of time and the grass strength are known. Based on the force balance the overload method can be described as (Fig. 2.1)

$$\sum_{i=1}^{N}(U_i^2 - U_c^2) = D \quad \text{for} \quad U_i > U_c \tag{2.1}$$

where D is the damage number, U_i is the depth-averaged velocity of the overtopping wave, U_c is the critical flow velocity and N is the number of overtopping waves.

Figure 2.1 Initial damage, damage at multiple locations and failure of embankment

Each wave gives a contribution to the damage/ erosion as long as the flow velocity of the wave (U_i) is greater than the critical flow velocity (U_c). The damage number (D) determines the extent of damage on the slope, ranging from 'no/early damage' to 'failure revetment' and is determined after N waves of the considered storm. To include the effects of transitions, objects, the flow acceleration and the grass strength Eq. 2.1 is adjusted as follows

$$\sum_{i=1}^{N} (\alpha_M \left(\alpha_a U_i\right)^2 - \alpha_s U_c^2) = D \quad \text{for}$$

$$\alpha_M \left(\alpha_a U_i\right)^2 > \alpha_s U_c^2 \tag{2.2}$$

where α_a is the acceleration factor which represents the increase of the flow velocity on the inner slope (α_a lies in the range of 1.0 to 1.6), α_M is the load–increase factor which depends on the type of transition or the form of the obstacle (α_M varies from 1.0 (when there are no transitions/objects) to 2.0 (i.e. theoretical maximum value; more information is given by Hoffmans et al. 2014), and α_s represents the strength of the grass e.g. at a transition which varies from 0.8 (theoretical minimum value) to 1.0 (see also WTI-2013 for more information).

The larger the distance between the crest of the dike and the damage site the greater the magnitude of the acceleration factor or the flow velocity. At the transition, the grass revetment is interrupted because on one side no roots grow. Therefore, the strength at this location is smaller (< 1.0) than elsewhere (= 1.0) on the grass revetment.

The first term on the left hand side of Eq. (2.2) is a measure for the load and the second term characterizes the strength. Equation (2.2) represents a hypothesis, which has recently been validated with prototype experiments (Hoffmans 2014).

Recently the damage numbers were reanalyzed in relation to flow acceleration and load–increase effects due to transitions and objects (Hoffmans 2014) which are here defined as

'no damage' $0 < D < 1000$ m^2/s^2
'early damage' $D = 1000$ m^2/s^2
'damage at several places' $D = 4000$ m^2/s^2
'failure of the grass revetment' $D = 7000$ m^2/s^2

It has been shown that the model has been most reliable for the prediction of the failure of the grass revetment and much less to predict 'early damage' and 'damage at multiple places'.

2.3 *Critical flow velocity*

Three grass qualities are here distinguished, specifically closed turf, open turf and patchy turf. For closed turf a minimal strength is deduced. Based on the Vechtdijk experiments the critical flow velocity is 3.5 m/s. For patchy turf which is comparable with a very poor grass cover (Fig. 2.2) no strength can be granted which was determined experimentally for a sea dike in Tholen. For open turf U_c may vary from 0 m/s to 3.5 m/s.

Figure 2.2 Patchy turf; Tholen, Section 2, Stair (before testing)

Up to now, the critical flow velocity is assumed to be a constant strength parameter. This assumption is only correct for a uniform flow, e.g. in open channel flow. If the flow increases on the inner slope of the dike then the turbulence decreases. The (average) critical bed shear stress represents the (average) strength and is independent of the location for a homogenous grass revetment. As the critical bed shear stress is a function of both the critical flow velocity and the turbulence (e.g. Hoffmans 2012), U_c varies and is at maximum at about halfway the inner slope.

2.4 *Transitions*

In this study, a transition is defined as a separation between two revetments, where waves flow from the hardened material (e.g., asphalt) to turf. Examples of a horizontal transition are flow from an inspection road or a bike path on top of the dike (or bank) to turf. Although vertical transitions are tested, for example, flow on stairs, a reduction factor is needed for angled wave attack. This also applies to oblique transitions on the slopes.

The direct influence of the load is caused by (i) difference between the roughness's of the revetments (turbulence increases from smooth to rough), (ii) geometrical changes at the toe of the dike as a result of a so-called jet or (iii) obstacles that interrupt the flow which yields a different distribution of forces.

Other effects which are often the result of poor management and/or inadequate performance of work, can be identified by (Van Steeg 2014, 2015)

- Indirect impact of grass strength (damage of sheep trails, tire tracks, litter, lower quality of grass at transition, worse grass management where mowers cannot reach, less rooting from too much fertilizer, dog exhaust, less sunlight by shade
- Indirect impact of strength clay (other type of clay during constructing/laying (sometimes even sand instead of clay), lower density of clay
- Indirect effects of load (so-called 'splash load' at greater height, locally stronger flow)

3 IMPACT FACTOR

3.1 *Introduction*

The effect of transitions and objects on the erodibility of closed turf is here modelled by an impact factor which relates the wave overtopping discharge and the load–increase factor. Three categories of impact are here distinguished (low, moderate and considerable impact). In the prototype tests random waves (small and large ones) were used for simulating storms. Based on the volume per wave the flow velocity at the crest, the maximum flow velocity on the inner slope (by using the acceleration factor), and the damage number are calculated. This paper discusses some results of an average sea regime; more details are given by Hoffmans (2015).

3.2 *Sea and lake regime*

During normative storm conditions, the significant wave height along the Dutch coast varies from 0 m (offshore wind) up to 4 m (onshore wind). To approximate the impact factor for an average sea regime the following assumptions are made: significant wave height is 2 m, wave period is 5.7 s, storm duration is 6 hours, average value of acceleration factor 1.3 (α_a lies in the range of 1.0 to 1.6), strength factor is 0.8 (α_s ranges from 0.8 to 1.0), critical flow velocity varies from 6 m/s to 8 m/s and damage number ranges from 2500 m^2/s^2 to 11200 m^2/s^2.

The calculations start with a mean acceleration factor ($\alpha_a = 1.3$) and a low strength factor ($\alpha_s = 0.8$). Later the values of these parameters are varied to quantify the impact. Consider a storm with a discharge q = 75 ℓ/s per m and a load–increase factor $\alpha_M = 1$. If the critical flow velocity equals $U_c = 6$ m/s then the damage number is $D = 11200$ m^2/s^2, see also the second column of Table 3.1.

Table 3.1 Calculation results of the overload method

	$U_c = 6$ m/s	$U_c = 7$ m/s	$U_c = 8$ m/s
	$D = 11200$ m^2/s^2	$D = 5700$ m^2/s^2	$D = 2500$ m^2/s^2
q ℓ/s/m	α_M (-)	α_M (-)	α_M (-)
75	1.00	1.00	1.00
50	1.23	1.21	1.20
30	1.58	1.53	1.49
10	2.75	2.47	2.28
5	3.97	3.36	2.95
1	11.53	8.04	5.81

If the damage number equals $D = 7000$ m^2/s^2 it corresponds to the failure of the grass revetments, and this occurs only when the wave overtopping discharge exceeds its critical value. As the failure of the grass cover is important (the mechanisms 'initial damage' and 'damage at several places' are not considered) the relation between load–increase factor and the critical overtopping can be estimated by

$$\frac{\alpha_{M,1}}{\alpha_{M,2}} = \beta^{½} = \left(\frac{q_{c,2}}{q_{c,1}}\right)^{½} \quad \text{for} \quad D = 7000\ \text{m}^2/\text{s}^2 \qquad (3.1)$$

where $q_{c,1}$ is the critical overtopping discharge associated with the factor $\alpha_{M,1}$ and $q_{c,2}$ is the critical overtopping discharge associated with the factor $\alpha_{M,2}$ and β is the impact factor (-)

Reducing q from 75 ℓ/s per m to 50 ℓ/s per m, α_M increases from 1.00 to 1.23. By using Eq. 3.1 it follows that $\alpha_{M,1} = \alpha_{M,2}(q_{c,2}/q_{c,1})^{0.5} = 1\cdot(75/50)^{0.5} = 1.22$. If q = 1 ℓ/s per m then α_M (= 11.53) reaches its maximum value. In the calculation the damage number, the wave conditions and the load duration were unaltered. Columns 3 and 4 of Table 3.1 show the computational results for $U_c = 7$ m/s and $U_c = 8$ m/s.

Figure 3.1 shows the computational results of Eq. 3.1 and three curves obtained from the overload method. These combinations are $U_c = 6$ m/s and D = 11200 m^2/s^2 (blue line), $U_c = 7$ m/s and D = 5700

m^2/s^2 (red line), and U_c = 8 m/s and D = 2800 m^2/s^2 (green line). Although there are differences between these curves (green line characterizes 'initial damage - damage at multiple spots' and the blue line represents a hypothetical case, since D > 7000 m^2/s^2) Eq. 3.1 gives a good approximation for the failure of the grass cover; compare the red line (D = 5700 $m^2/s^2 \approx$ 7000 m^2/s^2) and the purple line (D = 7000 m^2/s^2) in Fig. 3.1.

If the exponent ½ in Eq. 3.1 remains unaltered and if the acceleration factor increases from 1.0 to 1.5 then the critical flow velocity also increases, compare the red lines with 4500 $m^2/s^2 < D <$ 8000 m^2/s^2 and the purple lines with D = 7000 m^2/s^2 in Figs. 3.2 and 3.3. For failure of the grass revetment the critical flow velocities on the crest and on the slope are U_c = 4.5 m/s for α_a = 1.0 (see Fig. 3.2) and U_c = 8.0 m/s for α_a = 1.5 (see Fig. 3.3). It is noted that U_c = 4.5 m/s is interpolated between U_c = 4.0 m/s with D = 10300 m^2/s^2 and U_c = 5.0 m/s with D = 4800 m^2/s^2.

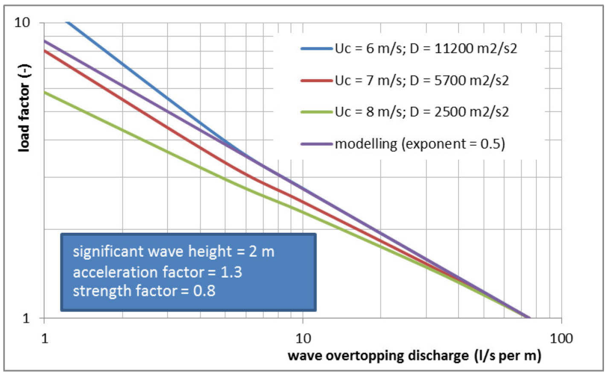

Figure 3.1 Load–increase factor as function of discharge

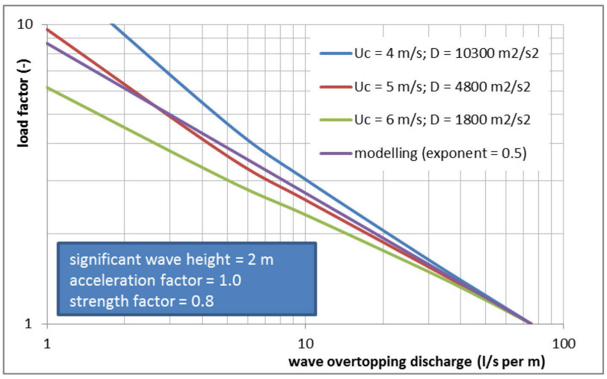

Figure 3.2 Acceleration factor is 1.0 (see also Fig. 3.1)

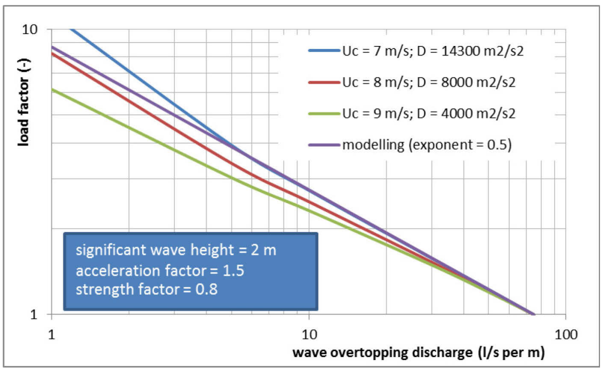

Figure 3.3 Acceleration factor is 1.5 (see also Fig. 3.1)

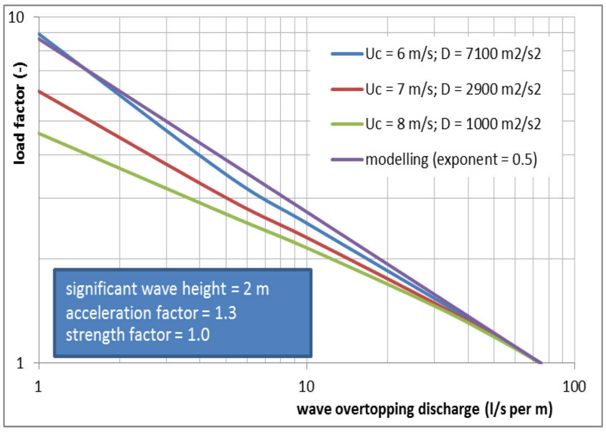

Figure 3.4 Strength factor is 1.0 (see also Fig. 3.1)

Also for geometrical transitions (at the toe of the dike) when the strength factor equals 1.0 (Fig. 3.4) Eq. 3.1 yields satisfactory results, compare the blue (D = 7100 m^2/s^2) and purple (D = 7000 m^2/s^2) lines. Note that the differences between the strength factors 0.8 and 1.0 are negligible (compare Figs. 3.1 and 3.4).

Assuming that the load–increase factor increases from 1.0 ($\alpha_{M,1}$ = 1.0) to 1.2 ($\alpha_{M,1}$ = 1.2), the critical wave overtopping discharge (at failure medium) is reduced by about a factor 0.69 (application of Eq. 3.1 gives $\beta = q_{c,2}/q_{c,1}$ = 0.69). In the extreme case, i.e., for $\alpha_{M,1}/\alpha_{M,2}$ = ½, the critical wave overtopping discharge may even decrease by 75% (impact factor is 0.25). It should be remarked that the above example is a simplification of the reality.

Summarizing, this analysis shows that there is a pragmatic relation between α_M on the one hand and q_c on the other hand. If α_M increases, q_c decreases. The reduction of α_s is here modelled as an increase of α_M.

3.3 *Classification transitions/objects*

Table 3.2 shows the impact of three scenarios, viz. geometrical transitions (Fig. 3.5), obstacles such as trees (Fig. 3.6), and inspection roads (Fig. 3.7). The impact factor demonstrates the effect of the load–increase and/or the decrease in strength. A distinction has been made between sea and river regimes. For q > 10 ℓ/s per m the relation between α_M and q is comparable for both regimes. However, if $q \leq$ 10 ℓ/s per m and if small wave heights (less than 1 m) are considered then the difference between the load–increase factor and the critical overtopping discharge is greatest.

In this study, it is assumed that the value of β = 0.1 is most damaging. Because not all conceivable situations are examined on the dike, there is a chance, albeit very small, that the impact factor is less than 0.1. To precisely determine this limit further investigation is required.

Table 3.2 Impact factors transitions/objects

regime	impact factor (β)			
	(1)	(2)	(3)	(4)
sea regime $1\ \mathrm{m} < H_s < 3\ \mathrm{m}$	1.0	0.7	0.5	0.2
river regime $0\ \mathrm{m} < H_s < 1\ \mathrm{m}$	1.0	0.6	0.3	0.1

(1) no impact ($\alpha_M = 1.0$ en $\alpha_s = 1.0$)
(2) less impact; ($\alpha_M = 1.2$ en $\alpha_s = 1.0$) e.g. geometrical transition;
(3) moderate impact; ($\alpha_M = 1.5$ en $\alpha_s = 0.9$) e.g. tree
(4) much impact; ($\alpha_M = 2.0$ en $\alpha_s = 0.8$), e.g. inspection road

Figure 3.5 Geometrical transition (at toe of dike)

Figure 3.6 Erosion at tree (halfway the inner slope of the dike)

Figure 3.7 Inspection road on the crest of the dike

4 PC-RING MODEL

4.1 *Introduction*

The PC-Ring model has been designed specially to calculate the reliability of a flood defense system as a composition of sections and calculates the probability of failure per section and per mechanism. Subsequently, the probability of failure of the complete flood defense system is computed. The failure mechanisms that are included in PC-Ring are overtopping and overflow, uplifting and piping, inner slope failure, damage of revetment and erosion of the dike body, erosion of dunes and failure of hydraulic structures (Vrouwenvelder et al. 1999). In this study the failure mechanisms overtopping and overflow are considered.

The mechanism overtopping occurs when at a certain location the quantity of water overtopping is larger than the crest and inner slope can handle (Fig. 4.1). This leads to erosion, after which a breach can start to grow and flooding may occur.

The mechanism overflow is supposed to occur when at a certain location the water level rises above the critical level of the dike (Fig. 4.2) Note that in these cases not always failure will follow.

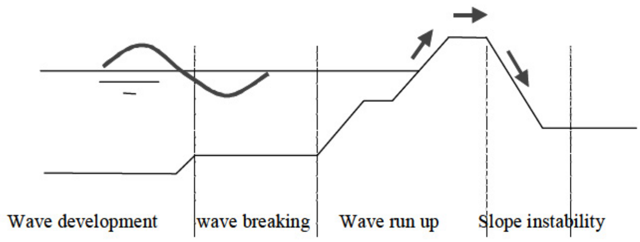

Figure 4.1 Dike failure mechanism: overtopping

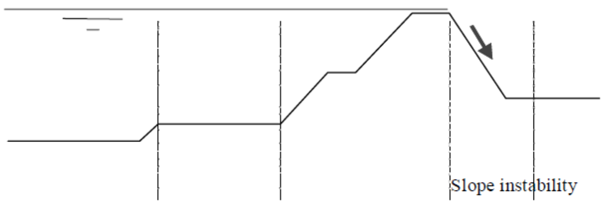

Figure 4.2 Dike failure mechanism: overflow

4.2 *Probability distribution of the critical overtopping*

Experimental results of the overtopping tests and the computational outcome obtained from the PC-Ring model have provided insight into the failure of the grass revetment. First estimates are made for the probability of occurrence as function of the critical overtopping discharge. Based on expert judgment lognormal distributions are fitted to approach two types of turf (closed and open turf) for different significant wave heights (Fig. 4.3).

Figure 4.4 shows the probability distributions of the critical overtopping discharge considering different impact factors (β = 0.1, 0.3, 0.6 and 1.0) given significant wave height of 1 m. These lognormal probability distributions of the critical overtopping discharge are used to estimate the contribution of the probability failure due to transitions and objects.

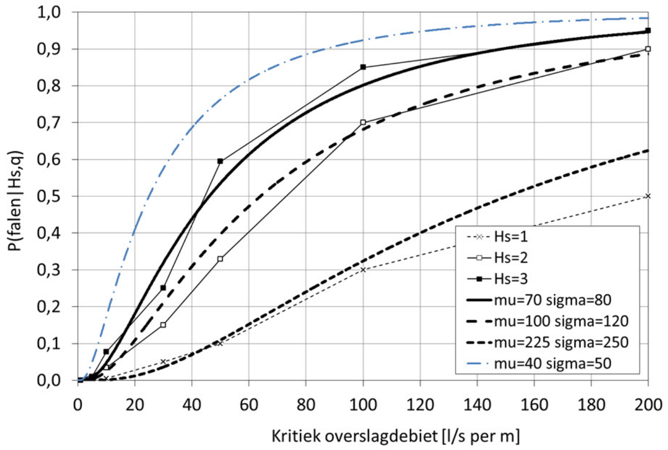

Figure 4.3 Probability distribution of the critical wave overtopping discharge for different significant wave heights including fit-functions (lognormal functions) (RWS et al 2013)

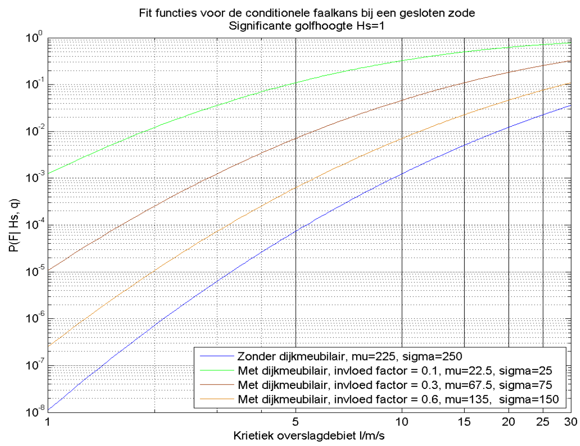

Figure 4.4 Lognormal distributions of the critical wave overtopping discharge for significant wave height of 1 m, for closed turf, zie also Fig. 4.3 (RWS et al 2013)

4.3 *Computational results*

The PC-Ring calculations show that the impact can be large, in an extreme case, a factor 1000 on the failure probability. This is especially the case at dikes which are exposed to large waves. Table 4.1 shows some computational results of PC-Ring for locations were waves play an important role.

Table 4.1 Computed failure probabilities – comparison for dikes with high wave heights

dike section	H_S (m)	failure probability ($\beta = 1.0$)	failure probability ($\beta = 0.1$)	ratio failure
DV_044_12.90_10.95	0.95	5.26E-11	5.47E-08	1041
DV_073_6.35_7.20	2.86	6.50E-10	1.58E-07	244
O-17.24600.24850	1.53	2.69E-08	5.70E-06	212
DV08_9.35km-9.80km	1.24	2.81E-07	6.09E-05	217
DV12_13.20km-14km	1.46	1.93E-10	4.58E-08	238
S053.0-S049.0_rd04	1.11	5.84E-06	4.26E-04	73
HHSK17dp147-dp160	0.87	7.90E-07	3.97E-05	50
DijkvakV-4_km13.531	1.15	6.41E-12	2.21E-09	345
hm 35.940 - 37.720	2.70	4.17E-07	1.39E-05	33

The impact of transitions and objects on the calculated probability of failure is low for river dikes or dikes that deal with a combination of high water and offshore wind. When waves are (very) small, the failure probability is dominated by the risk of overflow

The PC-Ring results show that the probability of failure in Krimpen aan de Lek (Fig. 4.5) increases significantly by including transitions with an impact factor of 0.1 (see also code HHSK17-dp147-dp160 in Table 4.1 and Table 4.2).

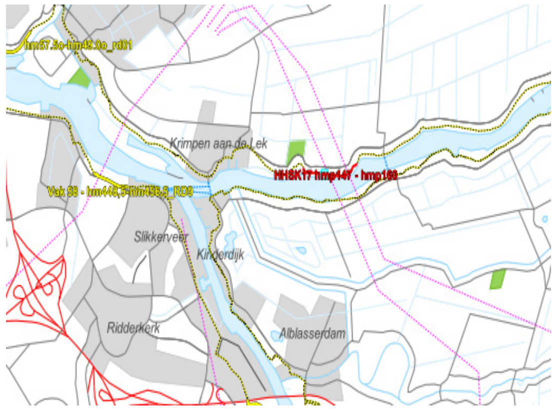

Figure 4.5 Lower basis of river Rhine

Because the critical overtopping discharge decreases (and thus also, the average overtopping discharge), the free crest height increases and the water level decreases. In this case, the free crest height increases, about half a meter from 1.32 m to 1.79 m. The water level decreases from 3.27 + NAP to 2.80 m + NAP m. Because the water level decreases the probability of occurrence is increasing, and thus also the failure probability.

Table 4.2 Overview of PC-Ring results

parameter	Krimpen aan de Lek	
	without objects	with objects
crest height	4.59 m + NAP	4.59 m + NAP
flow level	3.27 m + NAP	2.80 m + NAP
free crest level	1.32 m	1.79 m
significant wave height	0.90 m	0.87 m
impact factor	n.a.	0.1
critical wave overtopping discharge	42.2 ℓ/s per m	4.87 ℓ/s per m
failure probability overtopping	$7.9 \cdot 10^{-7}$	$3.97 \cdot 10^{-5}$
ratio failure	50.2	

5 CONCLUSIONS

For three types of transitions/objects on dikes, impact factors are derived (little impact, moderate impact, theoretical maximum impact). With these factors the decrease of the critical overtopping discharge is calculated with respect to the situation without transitions and objects.

The lognormal probability distributions of the critical overtopping are adjusted, in such a way that the influence of the transitions and objects are included in the calculation of the probability of failure.

The analysis by using the PC-Ring model has shown that the influence of transitions and objects is negligible for the upper basin of the river Rhine and for the river IJssel. If the significant wave height is small (say < 50 cm), then there is hardly an increase of the probability of failure. In such cases, the mechanism overflow is dominant.

At some dike sections, the impact was viewed in more detail (Tables 4.1 and 4.2). Though the Dutch flood defenses will be assessed (in 2017) with one impact factor (i.e. a conservative value of $\beta = 0.1$) more research is needed to evaluate these first estimations regarding the effects of transitions and objects.

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