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Lead author:
Gijs Hoffmans – Deltares
Boussinesqweg 1, 2629 HV Delft, The Netherlands
gijs.hoffmans@deltares.nl

Co-authors:
Andre van Hoven – Deltares
Boussinesqweg 1, 2629 HV Delft, The Netherlands
andre.vanhoven@deltares.nl
Gosse Jan Steendam – Infram
Amersfoortseweg 9, 3951 LA Maarn, The Netherlands
gosse.jan.steendam@infram.nl
Jentsje van der Meer – IHE Delft & Van der Meer Consulting B.V.
Westvest 7, 2611 AX Delft, The Netherlands
jm@vandermeerconsulting.nl

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Summary of research work about erodibility of grass revetments on dikes

Abstract:
Grass prevents erosion of the subsoil and is an effective control measure for overflowing water as well as overtopping waves. This form of protection has long been used for agriculture drainage channels and on the slopes of dikes. Dutch river dikes usually have clay layers covered with grass on the crest and on both the landward and seaward slopes. Lake and sea dikes with hard revetment in the wave impact zone of the seaward slope also have a grass cover on the crest and the landward slope on a clay type of soil. For grass covers, relatively large forces are required to break up clay aggregates within the soil, while smaller forces may suffice to transport pure sand and small clayey aggregates. Therefore, at the onset of dislocation, a grass cover will experience considerable forces, which may be described by turbulence, especially on steep slopes. From 2007-2014 about 50 experiments at several locations on Dutch and Belgian dikes were conducted with the wave overtopping and also run-up simulator for assessing the erodibility of grass revetments. Based on these prototype tests the cumulative overload method has been developed, a model that predicts the damage of grass mats and failure of dikes provided the load of the wave run-up and/or wave overtopping and the erodibility of the soil are known. This paper discusses engineering methods for predicting the erodibility of grass revetments as well as the erosion of grass at transitions, e.g. at the edges of roads, transitions from hard to grass revetments and objects on dikes, for example trees.

1.0 Introduction

In the Netherlands, primary sea and flood defences along the coast and inland waters have to be assessed periodically. Although developments in the safety assessments were realised in the past few years, still gaps in knowledge remain. The Dutch research and development program of flood defence assessment tools aims to improve the physical model relations in failure modes in order to achieve a better estimate of the safety of the hinterland. From 2007 to 2014 erosion of grass on slopes and at objects/transitions was investigated by large scale mobile wave overtopping and run-up tests on real dikes. These tests were useful for a better understanding of the strength of grass covers.

Erosion by wave overtopping at dike transitions is a major cause of dike damage or failure during severe overtopping events. When the flow of the waves is directed from a smooth to a rough bed the load increases. In the direction from rough to smooth the load decreases. Experiments show that erosion at revetment transitions, at geometrical transitions and at objects (e.g. at trees) differs with respect to erosion at e.g. slopes or horizontal grass revetments. This difference can be ascribed to both an increase of the load and a reduction of the grass strength.

2.0 Modelling

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 landward slope. The (wave) overtopping discharge is described as an average discharge of water per unit width and design conditions considered in the Netherlands are from 0.1 ℓ/s per m to 10 ℓ/s per m. For larger discharges severe damage can occur to the grass cover. A wave height of 2 m with a wave period of 6 s might be seen as 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 (cumulative) overload method. Usually the wave overtopping tests were carried out by using fixed hydraulic conditions at the seaward site, that is random waves with a significant wave height of 2 m. Then different overtopping discharges and volumes were simulated, based on the assumed crest freeboard. Next, the theoretical background of the overload method is described.

2.2 Overload method

The 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

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

where D is the damage number, U_i is the front velocity of the overtopping wave at the crest, U_c is the critical flow velocity of the cover (a strength parameter) and N is the number of overtopping waves.

Each overtopping wave volume gives a contribution to the damage/erosion as long as U_i is larger than U_c. The parameter 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 (Fig. 2.1). To include the effects of transitions, objects, the flow acceleration and the grass strength Eq. 2.1 has been adjusted

$$\sum_{i=1}^{N}(\alpha_M\left(\alpha_a U_i\right)^2 - \alpha_s U_c^2) = D \qquad \text{for} \qquad \alpha_M\left(\alpha_a U_i\right)^2 > \alpha_s U_c^2 \tag{2.2}$$

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

where α_a is the acceleration factor which represents the increase of the flow velocity on the landward slope (α_a lies in the range of 1.0 to 1.6), α_M is the load factor which depends on the type of transition or the shape 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 spot on the landward slope, the larger the magnitude of the acceleration factor ($\alpha_a > 1$) and thus also the local front velocity (EurOtop 2016, section 5.5.5).

At transitions and/or obstacles, the grass revetment is interrupted because on one side no roots grow. Therefore, the strength at these locations is smaller ($\alpha_s < 1.0$) than elsewhere ($\alpha_s = 1.0$) 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 been calibrated with prototype experiments.

The damage numbers were analyzed in relation to flow acceleration and load–increase effects due to transitions and objects (see also WTI-2013 for more information).

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

The turf element model describes the forces acting on a turf cube with a length scale of 10 cm. Working out the vertical force balance and by using both the equation of motion and the definition of the bed shear stress the critical flow velocity can be estimated by (Hoffmans et al. 2008):

$$U_c = 2r_0^{-1}\sqrt{\Psi_c\left(\sigma_{grass,c}\left(0\right) - p_w\right)/\rho} \qquad (2.3)$$

where r_0 is the relative (depth-averaged) turbulence intensity, ρ is the water density, $\sigma_{grass,c}(0)$ is the critical mean grass normal stress at the ground level (i.e. perpendicular to the ground surface), Ψ_c is the critical Shields parameter and p_w is the pore water pressure.

Three grass qualities have been distinguished, specifically closed turf, open turf and patchy turf. For closed turf a design strength is deduced, which measures for closed turf 4.3 m/s and for open turf on clay 6.3 m/s. Based on the Vechtdijk experiments the critical flow velocity is 3.5 m/s for closed turf on sand. 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.

Up to now, the critical flow velocity is assumed to be a constant strength parameter along the slope. However, this assumption is only correct for a uniform flow, e.g. in open channel flow. If the flow increases on the landward slope of the dike then the turbulence decreases. Hence, the critical flow velocity increases (see also Eq. 2.3). 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 along the slope.

Figure 2.2 On the left: Area directly adjacent to the stairs has no grass, nor roots and is called patchy grass; Tholen, Section 2, Stair (before testing). On the right: typical layout of a Dutch garden on the landward slope of a river dike

2.4 Heterogeneity

Bijlard (2015) examined the heterogeneity of grass revetments experimentally at several locations by using a sod pulling device as shown in Fig. 2.3. Figure 2.4 demonstrates the frame in which the sod is anchored. The measured forces needed to lift the grass sod can be rewritten into critical grass normal stresses and by using Eq. 2.3 into critical flow velocities.

For determining the representative strength of the grassed slope during wave overtopping conditions, the strength of the weakest sections in the grass sod have to be determined. Bijlard (2015) assumed a normal distribution for the strength of the grass, where the 2.5% tail value is used as the governing strength of the sod. In order to determine the parameters of the normal distribution, sufficient tests are needed.

Figure 2.3 Overview of the sod pulling device

Figure 2.4 Small pull frame which anchors the sodIn the

2.5 Transitions and obstacles

A transition is defined as a separation between two revetments, where for the example given now, waves flow from the hardened material (e.g., asphalt) to turf. Such a horizontal transition can be in reality flow from an inspection road or a bike path on top of the dike (or bank) to turf (see also Hoffmans et al. 2014).

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 may be the result of poor management and/or inadequate performance of work were identified by (Van Steeg 2014, 2015):

- Indirect impact on the 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, less sunlight by shade;
- Indirect impact on the strength of clay (other type of clay during constructing/laying (sometimes even sand instead of clay), low density of clay, small thickness of clay layer;

In the next chapter the effect of transitions and objects on the erodibility of closed turf is discussed.

3.0 Load factor

3.1 Revetment transitions

When the wave flow is directed from a smooth to a rough bed the load increases. In the direction from rough to smooth bed the load decreases. These effects are expressed here by a load factor. The damage at revetment transitions can be predicted by the overload method in which the load is corrected by the load factor**.**

When the flow is directed from a smooth to a rough revetment the load factor (α_M) can be written as (complete derivation is given in WTI-2013)

$$\alpha_M = 2 - \left(\frac{n_{M,s}}{n_{M,r}}\right)^6 \left(\frac{\ln \dfrac{10h}{\left(8\sqrt{g}n_{M,s}\right)^6}}{\ln \dfrac{10h}{\left(8\sqrt{g}n_{M,r}\right)^6}}\right)^2 \qquad (3.1)$$

where g is the acceleration of gravity, h is the flow depth and n_M is the Manning coefficient (subscripts r and s refer to rough bed and smooth bed).

Hence, the load factor varies from 1 (no transition) to 2 (extreme roughness difference at transition). When the flow is directed from a rough to a smooth bed the load factor can be written as

$$\alpha_{M,r\to s} = 2 - \alpha_M \qquad (3.2)$$

Although the load decreases owing to an up-flow near the bed, it is recommended to evaluate Eq. 3.2 with experimental results as the effects of turbulence are neglected in the modelling.

Example 1: Consider the transition of an asphalt revetment to a grass revetment. The Manning coefficient of asphalt is estimated by $n_{M,s} = 0.016$ (or $k_{N,s} \approx 4$ mm), and the roughness of grass is about $n_{M,r} = 0.025$ (or $k_{N,r} \approx 6$ cm), thus α_M varies from 1.7 to 1.8 depending on the flow depth (0.1 m < h < 0.5 m).

Example 2: If a horizontal transition is considered between a grass revetment ($k_{N,r} = 0.05$ m) and a parking area of (smooth) bricks $n_{M,s} = 0.016$ (or $k_{N,s} \approx 4$ mm), thus the flow is directed from the grass revetment to the stones, then the load factor ranges from 0.2 to 0.3 depending on the flow depth. Although the load decreases, damage could occur as the strength or the critical flow velocity of the stones could be less than the grass strength.

Figure 3.1 shows the time dependent erosion at the transition of an asphalt road on the crest of the dike to a grass revetment. This revetment transition was tested on a dike in Millingen aan de Rijn during the winter 2012/2013. At the beginning of the test, sand was washed out forming unevenness at the transition, the influence of which is not included in the modelling.

Most likely a horizontal eddy developed in the track during the testing. Hence, the near-bed velocities in the recirculation zone decrease. However, due to a mixing layer the bed turbulence increases. As the decrease of the near-bed velocities is greater than the increase of the bed turbulence the total load decreases. Although more details are given by Hoffmans (2012) more research is needed to improve the modelling on this topic.

At the transition of the test location Millingen aan de Rijn, damage at various locations (D = 1000 m^2/s^2) was measured at t = 6 h with 10 ℓ/s per m. Failure near the transition (D = 3500 m^2/s^2) was observed at 2 h with 50 ℓ/s per m. It is noted that this analysis was conducted before the damage numbers 'at various locations (D = 3500 m^2/s^2)' and 'failure (D = 7000 m^2/s^2)' were reanalysed. If the following assumptions are made (see also WTI-2013)

Figure 3.1a Less damage at transition from asphalt road to grass revetment (after 6h at the end of q = 1 ℓ/s per m)

Figure 3.1b Multiple open spots at transition from asphalt road to grass revetment (after 12h at the end of q = 10 ℓ/s per m)

Figure 3.1c Failure of transition from asphalt road to grass revetment (after 14 h or after 2h with q = 50 ℓ/s per m)

– Typically, the clay near the edge of the road is more erodible (lower plasticity, higher sand content) than the clay cover on the dike which can be ascribed to the sand foundation below the asphalt road. These effects are not considered here;
– Eddies in the track and the influence of a geometrical transition, i.e. from the horizontal crest to the landward slope are neglected;

– Acceleration factor equals $\alpha_a = 1.0$ (at the crest of the dike);
– Load factor is $\alpha_M = 1.75$ (see also example 1 and Eq. 3.1);
– Critical flow velocity on slope is $U_c = 4.5$ m/s (see also WTI-2013);
– Strength factor is $\alpha_s = 0.9$ thus U_c reduces from 4.5 m/s to 4.3 m/s (see also Eq. 2.2).

Then the predicted and measured times are approximately in agreement on which multiple open spots and failure of the grass cover occur. Table 3.1 gives the calculated damage numbers of these events for three values of α_M. By using the load factor of $\alpha_M = 1.75$ the damage number at $t = 6$ h with 10 ℓ/s per m for multiple open spots is $D = 433$ m^2/s^2. For failure of the grass cover the predicted damage number is $D = 2920$ m^2/s^2. Although these calculated values of D are in agreement with the defined values, a better fit is obtained by using $\alpha_M = 1.85$ (see also Table 3.1).

Table 3.1 Effects of load factor on the erosion as function of time (U_c is related to crest conditions)

α_M (-)	U_c (m/s)	$t_{measured}$ (hours)	$D^{(1)}$ (m^2/s^2)	$t_{measured}$ (hours)	$D^{(2)}$ (m^2/s^2)
1.50	4.0	6h -10 ℓ/s	158	2h -50 ℓ/s	1464
1.85	4.0	6h -10 ℓ/s	595	2h -50 ℓ/s	3649
2.00	4.0	6h -10 ℓ/s	893	2h -50 ℓ/s	4889

Note that the grey marked value of the load factor represents a best guess value;
$^{(1)}$ multiple open spots; $^{(2)}$ failure grass cover

3.2 Geometrical transitions

The situation at a transition of a slope to a horizontal berm can be compared with a jet that normally occurs because of flow under, through or over hydraulic structures. In general, a jet lifts soil and transports it downstream of the impacted area. The jet impact area is transformed into an energy dissipater and a scour hole is formed (see also Fig. 3.2). Note that when a scour hole is formed deeper than 20 cm the grass revetment fails.

Figure 3.2 Geometrical transitions (at toe of dike), on the left: Boonweg, on the right: Kattendijk

For geometrical transitions as shown in Fig. 3.2 the load factor (α_M) can be written as a function of the revetment steepness (θ) (see also WTI-2013 where more details about the modelling are given)

$$\alpha_M = 1 + \sin \tfrac{1}{2}\theta \qquad (3.3)$$

The load factor depends on the steepness of the dike. If there is no geometrical transition or if $\theta = 0^0$ then $\alpha_M = 1$. If $\theta = 20^0$ (steepness is 1V:2.7H) then $\alpha_M = 1.17$. In Millingen aan de Rijn the damage description "multiple damage spots" was observed at the slope and at the geometrical transition simultaneously. Because there was no free run-off possibility at the toe, a shallow pool was formed, which, most likely influenced/reduced the erosion process at the toe. Therefore, the load factor is $\alpha_M \approx 1.0$. For the other test location "Nijmegen", by using Eq. 3.3 the predicted load factor lies in the range of 1.16 (Nijmegen test strip N2) to 1.23 (Nijmegen test strip N1).

The load factor can also be determined by applying both the experimental data and the overload method. In that case, the load factor varies from 1.25 ($U_c = 4.5$ m/s) to 1.35 ($U_c = 3.5$ m/s) for the two test locations in Nijmegen (N1 and N2) (Table 3.2). For the steepest slope at N1 (1V:2H) the load factor is 1.35.

Table 3.2 Effects of load factor on the erosion as function of time (U_c is related to crest conditions)

location		α_M (-)	U_c (m/s)	$t_{measured}$ (hours)	D (1) (m^2/s^2)
Nijmegen	N1-S	1.00	3.5	1.5h -50 ℓ/s	734
Nijmegen	N1-T	1.23	3.5	6h -10 ℓ/s	376
Nijmegen	N1-T	1.35	3.5	6h -10 ℓ/s	613
Nijmegen	N2-S	1.00	4.5	4h -100 ℓ/s	959
Nijmegen	N2-T	1.16	4.5	4h -50 ℓ/s	361
Nijmegen	N2-T	1.25	4.5	4h -50 ℓ/s	600
location		α_M (-)	U_c (m/s)	$t_{measured}$ (hours)	D (2) (m^2/s^2)
Nijmegen	N1-S	1.00	3.5	6h -50 ℓ/s	2636
Nijmegen	N1-T	1.23	3.5	4h -50 ℓ/s	4324
Nijmegen	N1-T	1.35	3.5	4h -50 ℓ/s	6061
Nijmegen	N2-S	1.00	4.5	1.5h -200 ℓ/s	3280
Nijmegen	N2-T	1.16	4.5	4h -100 ℓ/s	1589
Nijmegen	N2-T	1.25	4.5	4h -100 ℓ/s	3977

Note that the grey marked value of the load factor represents a best guess value (not obtained from Eq. 3.3); (1) multiple open spots; (2) failure grass cover; S represents the slope of the dike and T is the toe.

Although there are differences between the measured and calculated load factors the relative error is less than 10%. The range of the load factor obtained from Eq. 3.3 agrees with other prototype measurements (see also WTI 2012-1). The load factor varied from 1.05 to 1.21 (Boonweg: $\alpha_M = 1.05$ and $U_c = 6.3$ m/s; St. Philipsland: $\alpha_M = 1.09$ and $U_c = 5.0$ m/s; Tholen: $\alpha_M = 1.21$ and $U_c = 4.0$ m/s. If the flow is directed from a horizontal crest to a landward slope then the load factor is

$$\alpha_M = 1 - \sin \tfrac{1}{2}\theta \qquad (3.4)$$

Typically, the initial damage and failure started randomly on the landward slope and not at these types of transitions. Therefore, α_M is less than 1. As the exact value of the load factor could not be evaluated by using the Dutch overtopping experiments the computational results obtained from Eq. 3.4 should be considered as rough estimators.

3.3 Flow blocking objects

The flow pattern around vertical objects can be divided into four characteristic features for sub-critical flow, namely the bow wave (or surface roller) due to the up-flow, the down-flow, the horseshoe vortex and the wake zone with the shed vortices (or vortex street). The flow decelerates as it approaches the object and comes to rest at the face of the object. Near the surface, the deceleration is greatest, and decreases downwards. The down-flow reaches a maximum just below the bed level. The development of the scour hole at vertical objects also gives rise to the horseshoe vortex, which is effective in transporting particles and extends downstream, past the sides of the pier (for more information see also Hoffmans and Verheij 1997).

Usually the flow separates at the sides of the object leading to the development of shed vortices in the interface between the flow and the wake. However, practical tests have shown that downstream of thick vertical objects there will be no direct mixing of water for super critical flow as present in wave overtopping conditions. Consequently, the load of the water upstream and along the tree is decisive with respect to the load downstream of the tree. The following starting points are made for modelling the erosion process at vertical objects (for example trees; Fig. 3.3)

– Prototype tests at Dutch dikes have shown that the erosion process of grass covers is negligible at slender vertical objects (diameter is less than 15 cm);
– At relative thick vertical objects, whose thickness varies from 0.15 m to 1 m (e.g. tree on the Vechtdijk), erosion was observed after a series of storms, so these situations are further considered;

Figure 3.3 Erosion at tree (halfway the inner slope of the dike)

Based on expert judgment, the load factor (α_M) upstream of the vertical obstacle with C_D as the drag coefficient can be given by (http://www.aerospaceweb.org/question/aerodynamics/q0231.shtml)

$$\alpha_M = 1 + \tfrac{1}{4} C_D \tag{3.5}$$

Here, two-dimensional objects are considered. The drag coefficient for piles, trees and houses varies from 1.2 to 2.3 depending on the shape. Along the vertical obstacle, that is, in the acceleration zone the load factor is estimated by (basis is also expert judgment)

$$\alpha_M = 1.4 K_s \tag{3.6}$$

where the shape factor K_s varies from 0.8 to 1.2 (e.g. Hoffmans and Verheij 1997). For cylinder shaped objects (e.g. trees) the drag coefficient measures C_D = 1.2 (K_s = 1.0) yielding α_m = 1.3 (Eq. 3.5) and α_m = 1.4 (Eq. 3.6). For rectangular objects, for example for a side wall structure, the drag coefficient is C_D = 2 (K_s = 1.2) thus α_m = 1.5 (Eq. 3.5) and α_m = 1.7 (Eq.

3.6). Consequently, the load factor for vertical obstacles lies in the range of 1.3 to 1.7. Next, these predictors for the load factor are validated by using prototype tests.

In Nijmegen, a side-wall structure was tested on the horizontal berm. In the stagnation zone the grass revetment was reinforced with a concrete protection so that at that location no erosion occurred. However, in the acceleration zone multiple damage spots were observed at $t = 1$ h in $q = 50$ ℓ/s. Subsequently, the acceleration zone was covered with a geotextile to prevent damage escalating to a point where the entire overtopping test would have to be stopped. For simulating the erosion process the following assumptions are made

- Acceleration factor equals $\alpha_a = 1.0$;
- Load factor is $\alpha_M = 1.7$ ($K_s = 1.2$);
- Critical flow velocity is $U_c = 4.5$ m/s (see also WTI-2013);
- Strength factor $\alpha_s = 0.9$.

Hence, the calculated time at which initial damage at several locations occurs, is $t = 1$ h ($q =$ 50 ℓ/s giving $D = 997$ m^2/s^2). This modification of the load factor agrees well with the measurement; see also Table 3.3 where different load factors are presented with corresponding damage numbers.

Following Pijpers (2013) the load factor is related to the wave volume and ranges from 1 (for the smallest waves) to 2.4 (for the largest waves). Although there are differences between his approach and the proposed modelling, this is not analyzed further here. Though the predicted time satisfies the measured time when multiple open spots near the object occurred, it is recommended to validate the approaches by using more observations and/or to deduce theoretical models.

As the erodibility of grass near objects was tested in Nijmegen the dimensions of these objects were relatively small compared to the width of buildings. At present there are still knowledge gaps, e.g. the erosion close to stairs is still not fully understood

Table 3.3 Damage number versus load factor for object at Nijmegen N2

α_a (-)	α_s (-)	α_M (-)	U_c (m/s)	$t_{measured}$ (hours)	D [(1)] (m^2/s^2)
1.0	1.0	1.4	4.3	1h -50 ℓ/s	527
1.0	1.0	1.7	4.3	1h -50 ℓ/s	1402
1.0	1.0	2.0	4.3	1h -50 ℓ/s	2774
1.0	1.0	2.3	4.3	1h -50 ℓ/s	4617

Note that the grey marked value of the load factor represents a best guess value; (1) multiple open spots

4.0 Conclusions and recommendations

The cumulative overload method has been evaluated, including the extensions to predict the load increase or the load decrease at transitions and obstacles. These effects can be expressed by a load factor. When a down-flow occurs the load factor is greater than 1 due to acceleration of the flow. The load decreases provided there is an up-flow close to the bed resulting in a load factor that lies in the range of 0 to 1. The reduction of the strength can be modelled by a strength factor which reduces the front velocity.

The load factor for an asphalt road to a grass revetment varies from 1.5 to 2.0 with a best guess value of $\alpha_M = 1.85$. The reduction of the strength, expressed by α_s is approximately 0.9. This value is about in agreement with the research results of Pijpers (2013).

For geometrical transitions the load factor depends on the steepness of the dike. The range of the load factor obtained from the conceptual model agrees with other prototype measurements. The load factor varies from 1.05 to 1.35.

Two models based on expert judgment are discussed for predicting the load factor at vertical objects. One relation characterizes the load increase just upstream of the object and the other relation represents the relative load increase of the near-bed forces along the obstacle. To determine the erosion at the obstacles the overload method is recommended for use.

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