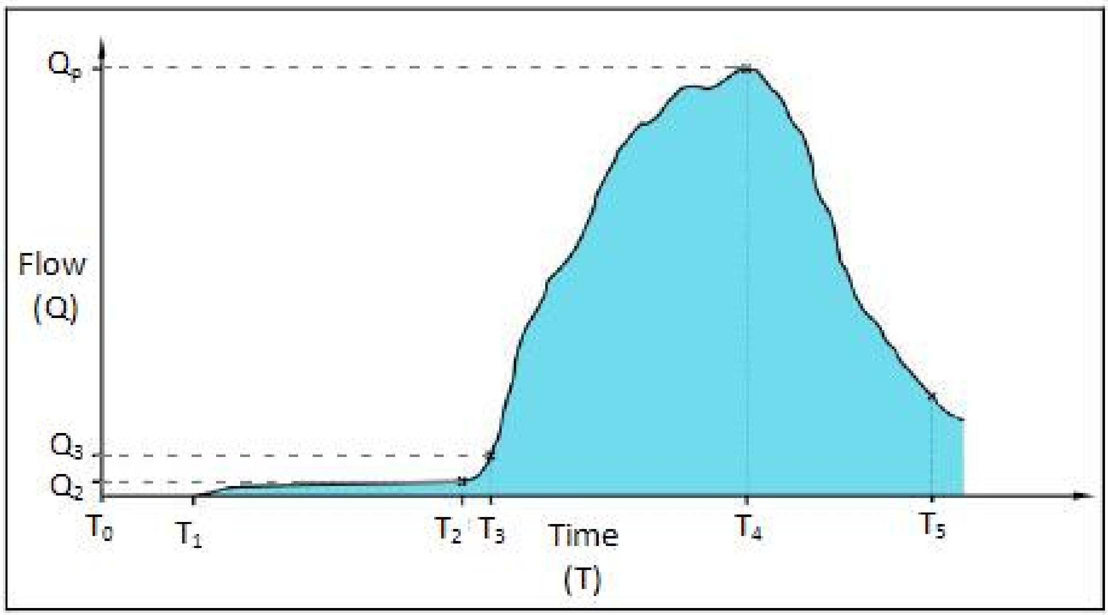
WP4.4: Rapidly assessing breach driven embankment failures



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Summary

The R&D project named *Risk Assessment for flood and coastal defence for strategic planning (RASP)* develops and demonstrates an integrated Flood Risk management approach that assesses the risk associated with flood defence systems instead of the risk of a single embankment failure. To prevent significant errors in the prediction of flood volumes and corresponding flood risks with RASP MDSF2, an improved method for predicting breach volumes was developed. Application of the proposed methodology lead to the development of the AREBA model which rapidly predicts the breach hydrograph of homogeneous embankment failures whereby the discharge through the breach is controlled by the breach dimensions.

AREBA incorporates the effects of a grass cover on the flood hydrograph shape and calculates a flood hydrograph for surface erosion failures, headcut erosion failures or piping failures. The model is based on a simplified description of the physical breach processes and uses simplified physical equations to predict the breach hydrograph shape under a second. So is it assumed that the entire grass cover fails instantaneously, that the flow behaves like a 1D flow, that the breach dimensions are spatially constant, and that the soil parameters are constant in time and space. With respect to surface erosion is assumed that the landside slope retreats under the initial slope angle while the crest erodes downward. New analytical equations have been developed from the assumptions for the calculation of the erosion rate of the landside slope. In headcut erosion mode the embankment is assumed to erode due to cascade formation along the landside slope whereby the crest does not erode downward. For piping failures it is assumed that: the pipe diameter increases until the soil above the pipe slumps down, after which the embankment is assumed to erode according to the surface erosion process. AREBA has been validated against the EU IMPACT experiments and the USDA-ARS experiment and benchmarked against HR BREACH version 4.1. Validation showed that in surface erosion mode AREBA is able to predict the breach hydrograph within the bounds of uncertainty that originate from the uncertainty in model input parameters.

Table of Contents

1 Introduction

System risk models for assessing flood risk at a catchment or larger scale are being developed to determine how investment in flood control measures might be optimised. The system risk approach requires a rapid and accurate method of assessment of the impact of a possible flooding for a given system of flood protection. One part of that process is the prediction of breach through a flood embankment. This report gives a description of the new AREBA model which gives A Rapid Embankment Breach Assessment. The speed at which AREBA runs makes it suitable for an application in system risk models. AREBA can be applied to model embankment failures arising from piping and overflowing. In case of an overflow failure, the global physical process needs to be user defined, whereby the user can choose between surface erosion failure and headcut erosion failure mechanisms. With surface erosion, small scale slope instabilities are assumed to limit the steepness of the landside slope. With headcut erosion, the cohesive properties of the embankment soil are assumed to prevent slope instabilities from occurring and the continuous steepening of the landside slope results into the formation of steps. The cascading flow over the steps causes the steps to increase in size and to retreat through the embankment. This report gives a description of the model. A more extensive review is given by the technical report on AREBA.

2 The modelling methodology

AREBA has been designed to predict the breach hydrograph, breach width and breach depth as a function of the upstream and downstream conditions, embankment geometry and soil parameters. The model is limited to predicting failures of homogeneous, trapezoidal shaped flood embankments. The flow rate through the breach thereby has to be controlled by the breach dimensions and not by the shape of the upstream reservoir.

2.1 Flow calculations

A box shaped reservoir with an inflow and outflow was assumed to allow the use any type of triangular or trapezoidal hydrograph as upstream boundary condition for the breach model. The downstream boundary condition consists of a user-defined flood area which for simplicity has been assumed constant over the height of the embankment. AREBA can however easily be adapted to deal with more complicated boundary conditions like stage-area curves or specific river hydrographs.

In surface erosion or headcut erosion mode, AREBA calculates the discharge over the embankment with the broad crested weir equation (Equation (2.1)) to account for the effects of non-hydrostatic pressures over the crest due to flow contraction. The pressure is assumed to be hydrostatic along the landside slope. The flow width is assumed to be spatially constant and only to vary in time.

$$Q_b = c_w \cdot b \cdot \boldsymbol{h_2} \cdot \sqrt{2g(H - \boldsymbol{h_2})} \tag{2.1}$$

Q_b is the discharge over the crest in m^3/s, c_w is the weir coefficient, b is the flow width in m; h_2 is the water level over the crest in m. H is the difference in water level in the reservoir and the height of the hydraulic control, which is assumed to be located where the crest meets the waterside slope. The water depth d over the crest is expected to be equal to $2/3H$. h_2 is assumed to be equal to the depth d, for non-drowned weir flow, and equal to the downstream water level for a drowned weir flow [1]. Every time step, the upstream and downstream water levels are updated assuming an equal drop or rise in water level everywhere in the reservoir or flood plain.

In AREBA three weir coefficients are applied. The first weir coefficient deals with the effects of vertical flow contraction as illustrated by the first image of Figure 2. The second weir coefficient represents the effects of horizontal flow contraction and is incorporated in the calculations when the retreat of the landside slope has reached the waterside slope. An example of the horizontally contracting flow is given by the second image in Figure 2. The third weir coefficient replaces the first two once the vertical erosion process has stopped and purely represents the effects of the horizontal contraction at this stage as illustrated by the third image in Figure 2. In AREBA this has been calculated as:

$$c_w = \begin{cases} \boldsymbol{c_1}, & \text{for crest width} \geq \boldsymbol{0} \\ \boldsymbol{c_1} \cdot \boldsymbol{c_2}, & \text{for crest width} < 0, \text{ and crest height} > 0 \\ \boldsymbol{c_3}, & \text{for crest height} = \boldsymbol{0} \end{cases} \tag{2.2}$$

Figure 1: Three stages of breach flow (source EU IMPACT project)

The failure and erosion mechanism need to be user-defined. A choice can be made between surface erosion, headcut erosion and piping. The following sections describe the assumptions and constitutive equations underlying each of the three choices.

2.2 Surface erosion

In surface erosion mode, AREBA uses the assumed water level profile from Figure 1 to calculate the erosion rates along the embankment surface. The flow depth is assumed equal to the critical depth d_c at the top of the landside slope and assumed to approach the normal depth d_n further down the landside slope.

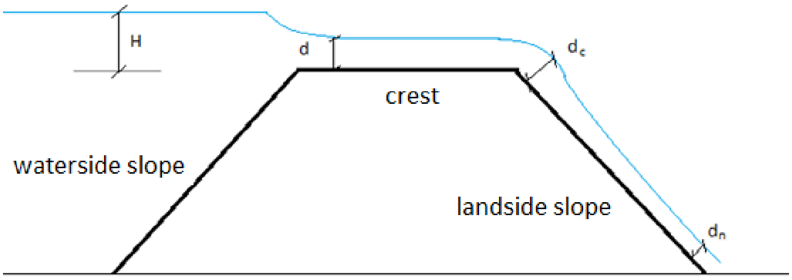

Figure 2: Depth profile over the embankment

The flow over the crest and landside slope causes the crest to erode downwards and the landside slope to retreat towards the waterside slope. Before the retreat of the landside slope reaches the waterside slope, the height of the hydraulic control is affected by the downward erosion of the crest, and thereafter by the rate of retreat of the landside slope. The crest height and location of the landside slope are updated every time step. It is assumed that the gradient of the landside slope remains equal to its initial value. The downward erosion rate of the crest is calculated using Equation (2.3).

$$E = K\left(\rho u^2 \frac{n^2 g}{d^{\frac{1}{3}}} - \tau_c\right) \tag{2.3}$$

Where E is the erosion rate in m/s, K is the erodibility coefficient in m^3/Ns, ρ is the density of water in kg/m^3, n the Manning coefficient in $s/m^{1/3}$, g the acceleration due to gravity in m/s^2, and d the depth over the crest in (m). The downward crest erosion leads the crest width to increase with a rate equal to the downward erosion rate multiplied with factor S to account for the effects of the waterside and landside slopes. S follows from

$$S = \frac{1}{i_{bi}} + \frac{1}{i_{bo}} \tag{2.4}$$

where i_{bo} *and* i_{bi} are the waterside and landside slope gradients. The equation of Belanger (2.5) [2] was used to describe the water level profile along the landside slope.

$$\Delta d = \Delta d_0 exp\left(\frac{x - x_0}{L}\right) \tag{2.5}$$

Δd is the difference between the normal depth and the local water depth in *m*, Δd_0 the difference between the critical depth and the normal depth in *m*, and *L* the adaptation length in *m*, defined as the length along the slope where Δd is equal to $^1/_e\, \Delta d_0$ where *e* is the natural logarithm. *L* follows from

$$L = \left(\frac{1}{3i_{bi}} - \frac{d_n^{\frac{1}{3}}}{3gn^2}\right) d_n \tag{2.6}$$

where d_n is the normal depth in (m) following from Equation (2.14). The velocity profile along the landside slope now follows from combining Equations (2.4) and (2.1) whereby it is assumed that the depth on top of the landside slope is equal to the critical depth d_c following from Equation (2.8). A shear stress profile now follows from replacing the velocity *u* in Equation (2.3), with the expression for the velocity profile along the landside slope. This leads to Equation (2.7)

$$\tau(x) = \rho\left(\frac{1}{\left(-ae^{\frac{x-x_0}{L}} + a + e^{\frac{x-x_0}{L}}\right)^2}\right) u_c^2 \frac{n^2 g}{d_n^{\frac{1}{3}}} \tag{2.7}$$

where d_n (m) replaces the value of the hydraulic radius. *L* is the adaptation length in (m), and u_c (m/s) is the velocity corresponding to the critical depth d_c (m) that follows from

$$d_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} \tag{2.8}$$

where q is the width averaged discharge in m^2/s . *a* follows from

$$a = \frac{g^{\frac{1}{3}} n^{\frac{2}{3}}}{d_n^{\frac{1}{9}} i_{bi}^{\frac{1}{3}}} \tag{2.9}$$

where i_{bi} is the landside slope gradient. The erosion rate normal to the landside slope follows from

$$\bar{E} = K(\bar{\tau} - \tau_c) \tag{2.10}$$

where $\bar{\tau}$ is the mean shear stress along the landside slope and τ_c is the critical shear stress [3]. The rate at which the crest width reduces, follows from multiplying the mean erosion rate along the landside slope with a factor to account for the slope effects. The rate at which the crest width reduces (R_c) is

$$R_c = \bar{E}\sqrt{\left(\frac{1}{i_{bi}}\right)^2 + 1} \tag{2.11}$$

where $\overline{E}$ is the numerical averaged mean erosion rate, and i_{bi} is the landside slope gradient. The moment the landside slope reaches the waterside slope the rate at which the hydraulic control lowers is given by

$$R_c = \frac{1}{i_{bo}^{-1} + i_{bi}^{-1}} \overline{E} \sqrt{\left(\frac{1}{i_{bi}}\right)^2 + 1} \tag{2.12}$$

where i_{bo} is the outward slope gradient. With regards to breach widening the rules were adopted from HR BREACH. The breach widening rate is assumed to be proportional to 1.4 times the rate at which the crest lowers. Once the crest height equals the foundation level of the embankment no further vertical erosion is assumed to occur. The breach widening rate now follows from

$$E_w = 1.4 \cdot K \left(\frac{\rho g n^2 u^2}{h^{\frac{1}{3}}_{2}} - \tau_c \right) \tag{2.13}$$

where E_w is the rate of breach widening.

2.3 Headcut erosion

Temple et al. [4] developed a simplified Headcut erosion model (SIMBA) based on empirical equations for the erosion rate and a 1D flow description. In AREBA this method has been adopted to produce a breach hydrograph for headcut erosion failures. Temple et al. identified the following four stages in the headcut breach process:

- Stage 1: Headcut formation on the downstream slope
- Stage 2: Headcut advance through the embankment crest
- Stage 3: Breach formation as the headcut enters the reservoir
- Stage 4: Breach expansion during reservoir drawdown.

The basis of the empirical relationships developed by Temple et al. [4], and adopted in AREBA, was the assumption that the downward crest erosion is negligible during stages one and two. The erosion rate at the landside slope follows from Equation (2.9) whereby the shear stress τ follows from

$$\tau = \rho g d_n S \tag{2.14}$$

where S is the embankment slope defined as the *Sine* of the angle of the slope with the horizontal [4]. d_n is the normal depth in m obtained from Equation (2.12) where i_{bi} the landside slope gradient.

$$d_n = \left(\frac{q^2 n^2}{i_{bi}}\right)^{0.3} \tag{2.15}$$

Stage 1 ends when the calculated erosion depth exceeds the critical depth [4]. During Stage 2, the headcut over fall height increases to a maximum of the embankment height relative to its foundation level, and advances through the embankment crest. Temple et al. [4] gave the following equation for the rate of headcut retreat

$$\frac{dX}{dt} = C(q \cdot h_{hc})^{\frac{1}{3}} \qquad (2.16)$$

where C is the headcut coefficient in $s^{-2/3}$, and h_{hc} is the height of the headcut in m, which multiplied by q represents the dissipation of energy by the falling water [4]. AREBA uses the following relation between the erodibility coefficient (K) and the headcut coefficient.

$$C = 0.000246 * K \qquad (2.17)$$

The coefficient 0.000246 has as units ($\frac{Ns^{\frac{1}{3}}}{m^3}$). The downward erosion of the headcut base follows from Equation (2.16), where τ is given by the maximum of the values following from Equations (2.12) and (2.15). (Temple et al., 2005)[4]

$$\tau = \rho g d_c \cdot 0.011 \left(\frac{h_{hc}}{d_c}\right)^{0.582} \qquad (2.18)$$

With downward erosion of the headcut, the headcut retreats towards the waterside slope. The maximum value for the headcut height h_{hc} is given by the crest height. Once the headcut advance has reached the waterside slope, Stages 3 and 4 commence and breach erosion starts to affects the hydraulic control. The rate of change in height of the hydraulic control h_c during stage 3 follows from accounting for the waterside slope effects in Equation 2.14 and is given by

$$\frac{dh_c}{dt} = i_{bo} \cdot C(q \cdot h_{hc})^{\frac{1}{3}} \qquad (2.19)$$

where i_{bo} is the waterside slope gradient. The increase in width of the hydraulic control is assumed to be equal to the decrease in height [4].When the crest height becomes smaller than the critical depth, the headcut progress stops and the change in crest height is given by.

$$\frac{dh_c}{dt} = -K(\tau - \tau_c) \qquad (2.20)$$

where τ is obtained from Equation (2.18). The breach widening rate is now assumed to be proportional to the rate at which the crest lowers by a factor of 1.4 [4]. Once the waterside slope has fully eroded, the breach will only widen since the embankment's foundation is assumed to be non-erodible. For the calculation of the breach widening rate, Equations (2.20) and (2.21) are used without calculating the new breach height.

$$\tau = \frac{\rho g^2 n^2 (h_1 - h_2)}{h_2^{\frac{1}{3}}} \qquad (2.21)$$

In Equation (2.21) h_1 is the upstream water level in m, and $h_2 = 2/3h_1$ for a non-drowning breach and equal to the downstream water level for a drowning breach [4].

2.4 Piping

Piping failure is modelled with the method proposed by Mohamed [5] and applied in HR BREACH. It is assumed that a pipe has already formed an open connection between the landside and waterside. Mohamed [5] calculates the discharge through the pipe with

$$Q_b = A\sqrt{\frac{2g(\Delta H)}{h_f}} \tag{2.22}$$

where Q_b is the discharge through the pipe in m^3/s, A is the wet cross sectional area of the pipe in m^2, and ΔH the head difference over the pipe in m. h_f is a factor representing the contraction losses and friction losses and follows from Equation (2.23), with L the pipe length in m, and D_p the pipe diameter in m. f is a friction coefficient determined from Equation (2.24) [5] where D_{50} , is the grain diameter that 50% of the soil mass exceeds. The energy losses due to outflow from the pipe have been neglected, which is valid in case of free outflow from the pipe. The pipe is assumed to have failed before drowning of the pipe takes place.

$$h_f = 1.05 + \frac{fL}{D_p} \tag{2.23}$$

$$f = 0.216 \cdot \left(\frac{D_{50}}{D_p}\right)^{1/6} \tag{2.24}$$

The cross sectional area of the pipe *(A)* is calculated from assuming a circular shaped pipe that does not grow beyond the embankment's foundation level. Once the invert of the pipe reaches the embankment foundation, the pipe shape becomes that of a truncated circle. The surface area of the pipe then follows from

$$A = \frac{1}{4}\pi D_p^2 - \frac{\alpha}{4}D_p^2 + h_p^2 tan\alpha \tag{2.25}$$

in which

$$\alpha = \frac{\arccos(2h_p)}{D_p} \tag{2.26}$$

where h_p is the distance between the centreline of the pipe and the foundation level of the embankment in m. The pipe is assumed to grow uniformly along its length according to equation 2.7. The wall shear stress in the pipe (τ) follows from

$$\tau = \rho g R i_w \tag{2.27}$$

whereby R is the hydraulic radius of the pipe, which for a submerged circular pipe is equal to $D_p/4$, with D_p representing the pipe diameter [1]. i_w is the gradient in water level over the pipe and follows from

$$i_w = f\frac{u^2}{2gD_p} \tag{2.28}$$

In AREBA the assumption has been adopted from Mohamed [5] that the mass, above the pipe fails when the weight of the soil mass above the pipe exceeds the friction at its sides. Mohamed [5] gives the following relationship for the factor of safety *(FOS)*

$$FOS = \frac{c \cdot A_c}{W} \tag{2.29}$$

where c is the cohesion of the soil in N/m^2, A_c are the areas of the sides of the soil volume calculated with respect to the centre of the pipe. For simplicity the weight of the soil W in N is purely determined for the soil above the top and follows from.

$$W = f_a \rho g D_p A_s \tag{2.30}$$

where A_s is the surface area of the sides of the soil wedge in m^2 calculated from the top of the pipe upwards. Mohamed [5][5] gives the following relationship for f_a which is an empirically based reduction factor representing the arching of the soil.

$$f_a = 0.0003757 + 0.0000986 \cdot r^2 - 0.0002918 \cdot r \tag{2.31}$$

r represents the ratio of the distance between the top of the pipe and the crest, and the pipe diameter. r has a minimum value of 0.1 and a maximum value of 1.6. [5]
When the factor of safety is smaller than 1, *(FOS)* <1, the pipe collapses and the embankment crest is assumed to lower over the height needed to fill the volume of the pipe. From this time onwards the embankment’s failure mechanism transfers to that of a surface erosion process.

2.5 Grass failure

A headcut or surface erosion process initiates once the grass cover is considered to have failed. The elapse time before the assumed, instantaneous, failure of the grass cover depends on the resistance of the grass cover under the highest flow conditions. The highest flow velocities are met along the landside slope where the depth approaches the normal depth. The flow velocity corresponding to the normal depth follows from Equation 2.28 [1]

$$u_n = \frac{d_n^{\frac{2}{3}} i_{bi}^{\frac{1}{2}}}{n} \tag{2.32}$$

Assuming a wide shallow breach flow the normal depth follows from Equation 2.12. Mohamed found the following relationships for the critical flow velocity for the failure of a grass cover of quality 1,2 or 3 by curve fitting the experimental results given in the CIRIA guide 116, TN71 by Whitehead [6]

$$u_{g1} = 2.385 - 0.0167 \cdot \ln\left(T + \frac{5.333}{T}\right)$$
$$u_{g2} = 2161 - 0.131 \cdot \ln\left(T + \frac{3.551}{T}\right) \tag{2.33}$$
$$u_{g3} = 1.889 - 0.236 \cdot \ln\left(T + \frac{2.767}{T}\right)$$

u_g is the velocity at which the grass cover fails in m/s at time T in hours and the subscript *1,2,or 3* indicates the grass quality.

3 AREBA

For the purpose of testing the proposed method, the equations were coded up in Matlab leading to the AREBA model. The resulting code is not sensitive for model instabilities and therefore the time stepping of the model is solely determined by the rate at which; the water level in the reservoir drops, the water level in the flood area rises, or the embankment erodes. Since relatively large time steps could be used, AREBA is able to give a hydrograph prediction, irrespective of the chosen failure mode in approximately 0.2s. The input required by AREBA consists of:

- Type of failure mode and quality of the grass protection: The choice of protection affects the time at which the embankment starts to erode and could therefore have a large effect on the breach hydrograph shape in case of a short duration of high water levels upstream of the breach.
- Upstream and downstream boundary conditions consisting of a reservoir volume with an in and outflow and a flood area size. In its standard form AREBA assumes a box shaped reservoir and flood area. However for testing purposes of the model, stage area curves has been used as input with a relatively small effect on the run time of the model.
- Embankment geometry including crest height, crest width and slope gradients A correct input of the embankment geometry becomes more important in case of less erodible soils. In case of reasonable erodible, non -cohesive soils, the embankment is assumed to fail due to surface erosion whereby the hydrograph shape is often determined by the rate of downward crest erosion. In less erodible soils, the rate of retreat of the landside slope has a larger effect on hydrograph shape and the model becomes more sensitive for a correct input of the embankment geometry.
- Embankment material properties including erosion parameters of the soil. The erosion rate calculated by AREBA is linearly dependent or higher order dependent on the value for the soil erodibility or Manning coefficient. Besides this the choice of failure mode also affects the shape of the hydrograph. Often, accurate data on the soil parameters is not available. In this case the tables in Appendix 1 could be used to obtain an estimate of Manning coefficient and erodibility.

Besides the hydrograph shape and development of the breach width and depth, AREBA also provides values for the upstream and downstream water level. Every time step, AREBA updates the water level in the reservoir and water level in the flood area by assuming an instantaneous spread of the discharged water into the flood area or instantaneous overall decrease in water level in the reservoir. The error made by assuming an instantaneous overall effect is small compared to the errors originating from other assumptions due to the relatively large time scale of the breach event.

4 Conclusions

A new methodology has been developed for the rapid simulation of breach formation for surface erosion failures of trapezoidal shaped homogeneous embankments. The new methodology incorporates existing methodologies for modelling headcut failures and pipe failures. The application of all three methodologies lead to the development of AREBA (A Rapid Embankment Breach Assessment) which predicts the flood volume, hydrograph shape, and breach sizes for overflow failures and piping failures of flood embankments. The short time needed per model run makes AREBA suitable for incorporation in system risk models.

5 Literature

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Appendix 1

The tables below provide some information on rules of thumb for estimating of the Manning coefficient and soil Erodibility based on

Channel conditions		**Values ($s/m^{1/3}$)**	
Material involved	Earth Rock Cut Fine Gravel Coarse Gravel	n0	0.020 0.025 0.024 0.028
Degree of Irregularity	Smooth Minor Moderate Severe	n1	0.000 0.005 0.010 0.020
Variations of channel cross section	Gradual Alternating occ Alternating freq	n2	0.000 0.005 0.010-0.015
Relative effects of obstructions	Negligible Minor Appreciable Severe	n3	0.000 0.010-0.015 0.020-0.030 0.040-0.060
Vegetation	Low Medium High Very High	n4	0.005-0.010 0.010-0.025 0.025-0.050 0.050-0.100
n follows from:n= n0+n1+n2+n3+ n4.			

Table 1: Indicative Manning coefficients

The Erodibility can be estimated with

Qualitative Description of Values for K_d (Hanson, 2007)	
K (m^3/N-s)	**Description**
> $1.77e^{-5}$	Extremely erodible
$1.77\ e^{-6} – 1.77\ e^{-5}$	Very erodible
$1.77e^{-7} – 1.77\ e^{-6}$	Moderately erodible
$1.77e^{-8} – 1.77e^{-7}$	Moderately resistant
$1.77e^{-9} – 1.77e^{-8}$	Very resistant
< $1.77e^{-9}$	Extremely resistant

Table 2: Indicative erodibility coefficient based on a global soil description

Estimating K (m^3/N-s) as a function of % clay and compaction				
% clay	**Well compacted**		**Poorly compacted**	
	Optimum	**Dry**	**Optimum**	**Dry**
>25	$1.77e^{-7}$	$1.77e^{-6}$	$1.77e^{-6}$	$1.77e^{-5}$
10-25	$8.85e^{-7}$	$8.85e^{-6}$	$8.85e^{-6}$	$3.54e^{-5}$
5-10	$3.54e^{-6}$	$1.77e^{-5}$	$1.77e^{-5}$	$8.85e^{-5}$
0-5	$1.77e^{-5}$	$3.54e^{-5}$	$3.54e^{-5}$	$1.77e^{-4}$

Table 3: Indicative erodibility coefficient based on soil composition and compaction