WP 3: Reliability of Urban Flood Defences

D3.1 Guidance on improved performance of urban flood defences

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Summary

FloodProBE work package 3 relates to the question of assessing earthen levees safety, more specifically in urban area. Task 3.1 actions deal with fundamental knowledge about the failure mechanisms or resistance of the dike. Task 3.2 actions deal with rapid, cost-effective investigation techniques. Task 3.3 deals with the question of the assessment methodology itself.

This document reports on Task 1 of work package 3, which deals with improving fundamental knowledge of:

- Mechanisms leading to failure of an earthen work (internal erosion, 3.1.1),
- Structural weaknesses and associated failure modes (structure transitions, 3.1.2),
- Performance of the levee (resistance to external erosion brought by vegetation, 3.1.3),

which are essential for understanding the possible failure modes of the levee.

On internal erosion, the report contains (chapter 2):

- A description of physical processes of erosion, in a way easily understandable by levee managers,
- A description of the different scenarios of failure by internal erosion through four successive phases leading (or not) to a breach, with a matrix representation,
- Information about testing facilities available in Europe and some other countries for measuring erosion parameters: parameters of erosion that can be measured, types of soils that can be tested, etc,
- Results of cross-tests on two pilot sites of the project (Orléans and Humber) and overview of existing data bases for erosion parameters,
- And finally description of key soil parameters reflecting internal erosion susceptibility.

Chapter 3 provides an assessment of the different types of structure transition, the processes that might occur at transitions and recommendations as to how such transitions should be managed, assessed, designed and repaired to limit flood risk. In this chapter, we can find:

- An overview of hydraulic loading and erosion mechanisms that can occur and which will affect failure modes at transitions,
- Some case study examples where transitions have failed and lessons learnt,
- A typology of the transition structures. The more important transition types are then described within structured templates, including details in connection with their management, assessment, design or repair,
- How structure transitions are now, and could and should be incorporated into the performance assessment of flood defences structures,

- A summary of key conclusions from the review and issues that require further study in order to improve knowledge about the transitions and hence help manage flood risk attributable to transitions.

Specific research actions on grass performance are described in Chapter 4 and comprised:

- A review of project initiatives related to the performance of grass;
- Investigation into grass performance data collected at the USDA Stillwater centre over the past 20 years, to identify what aspects might be relevant to European practice;
- Confirmation of existing European and US guidance on grass performance, followed by identification of either (i) Updates to guidance using existing international research findings or (ii) clarification of longer term R&D needs to improve knowledge and performance of embankment grass cover layers.

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1 Introduction

1.1 The FloodProBE Project

FloodProBE is a European research project with the objective of providing cost-effective solutions for flood risk reduction in urban areas. FloodProBE aims to develop technologies, methods and tools for flood risk assessment and for the practical adaptation of new and existing buildings, infrastructure and flood defences leading to a better understanding of vulnerability, flood resilience and defence performance. This research supports implementation of the Floods Directive through the development of more effective flood risk management strategies. The work is being undertaken in close partnership with industry, and is utilising pilot sites across Europe, to help provide practical guidance and cost-effective construction solutions.

The FloodProBE activities have been structured according to the following work packages (WP):

- *WP2* addresses issues related to the vulnerability understanding and assessment of the vulnerability of urban areas or systems.
- *WP3* deals with failure modes and the assessment and identification of weak spots in urban flood defences.
- *WP4* investigates cost-effective construction technologies and concepts for improving the performance of existing and new flood defences and for increasing the flood resilience of urban systems and assets
- *WP5* supports integration of the research and newly developed knowledge into existing decision support models or systems, the production of industry guidance and the interaction and integration of pilot site studies across Europe.

The dissemination and stakeholder-involvement activities are addressed under *WP6* whilst *WP1* comprises all activities related to the management of the consortium.

The work described in this report is part of *WP3*.

1.2 Task 3.1 within the general framework of WP 3

The recent and dramatic floods of the last years in Europe (Windstorm Xynthia in France, February 2010, floods in South of France, 2002 and 2003, historical floods in Central Europe, Summer 2005), United-States (Hurricane Katrina, August 2005) and Asia (Thailand, 2011) have shown the vulnerability of flood defence systems composed of man-made structures (as levees, walls, etc.) and natural structures (as dunes, etc.). The first key point for avoiding these dramatic damages and the high cost of a failure and its consequences lies in the knowledge of the safety level of the protection system. Identifying weak points of the system is the most important but the most difficult issue.

Most of the levees are old structures, built several centuries ago, then rebuilt or repaired (after a breach), modified, heightened several times, with some materials that do not necessarily match the original conception of the structure. The levee foundations are also heterogeneous and in general were not properly treated to improve their water-tightness or other fundamental properties.

Other factors introduce weaknesses in a levee: (i) trees, roots, burrows or termite nests could modify the structure of the levee and reduce its mechanical properties; (ii) particular geological formations and their evolution could also threaten the dike, as it occurred in the city of Orléans, France, where levees have collapsed in karstic areas. In urban context, the levees present many other singularities, such as embedded networks, pipes, human constructions like houses and walls. Due to all these factors, levees have to be considered as heterogeneous structures. Considering the stretch of hundreds of kilometres and the heterogeneity of the levees, both good assessment methods, based on sturdy fundamental knowledge of the failure mechanisms and the strength of the levee components, and rapid, cost-effective and reliable techniques for data acquisition and surveying the defence system are necessary.

FloodProBE work package 3 relates to the question of assessing earthen levees safety, more specifically in urban area. Task 3.1 actions deal with fundamental knowledge about the failure mechanisms or resistance of the dike. Task 3.2 actions deal with rapid, cost-effective investigation techniques. Task 3.3 deals with the question of the assessment methodology itself.

An assessment[1] is a process that has the objective to evaluate the performance of a levee system relating to one of its main functions: to protect against a given natural event and to be stable/safe. A complete assessment should include a diagnosis of the actual or possible causes of failure, in order to remediate or prevent them.

The assessment process can be described, in a very simple way, as the use of one or more methods of treating and combining data in order to obtain an evaluation of the performance of the levee system, according to its main function (protect against flood) and/or its reliability (against the possible failure modes). This can be done in different ways, as there are different assessment methods used in different countries, all based on a combination of data processing, using expert judgment, index based methods, empirical models, physical and/or mathematical models.

Assessment make use of a lot of data. Some are already available at the start of an assessment process, while other ones are needed but unavailable; so specific data gathering has to be made during the assessment process. These data gathering can be done during specific inspections and investigations. And all data has its place in the information system of the levee manager.

Task 1 of work package 3, on which this deliverable reports, deals with improving fundamental knowledge of:

- Mechanisms leading to failure of an earthen work (internal erosion, 3.1.1)
- Structural weaknesses and associated failure modes (structure transitions, 3.1.2),
- Performance of the levee (resistance to external erosion brought by vegetation, 3.1.3)

which are essential for understanding the possible failure modes of the levee.

Task 2 of work package 3 deals with rapid and cost-effective investigation techniques:

[1] ASSESSMENT relates to the global process of evaluating the safety level of the levee; DIAGNOSIS comes in complement when we want to specifically analyze the causes (mechanisms, failure modes) of a problem or of a risk.

- Geophysics, to complement classical geotechnical investigations and tests,
- LiDAR to get a high quantity of topographic information as well as high resolution pictures and videos.

which are essential data to be used during an assessment process.

Task 3 of the work package 3 details the general assessment framework developed during the project, as well as presents different examples of assessment methods and the way they can be improved using the developed framework.

2 Internal erosion (action 3.1.1)

When looking at internal erosion as a whole, a number of key questions may be asked:

1. What are the internal erosion processes?
2. How do we measure erodibility?
3. What (soil) parameters adequately reflect erodibility?
4. What models exist, covering the range of parameters above, for predicting internal erosion?

These questions are being answered through the following aspects developed in this report:

1. Physical processes of erosion are described, in a form that is scientifically consistent but as simple as possible to be understandable by levee managers;
2. Scenarios of failure development by internal erosion are described (how physical processes are linked and may lead to the failure of a levee);
3. After an international enquiry, an inventory and description of existing testing facilities available in Europe and some other countries for measuring erosion parameters is given: parameters of erosion that can be measured, types of soils that can be tested and a list of available databases that contain relevant erosion parameters for internal erosion;
4. From cross-tests performed on levees in two pilot sites, we present the testing programs that could be achieved and their results *(to be included later).*

2.1 General framework of internal erosion

2.1.1 Context, overview and methodology

The present report is a compilation of many results and proposals from national and international programs, completed or in progress. The most significant ones are the French project ERINOH (funded by the French National Research Agency and by IREX, the French Institute for applied research and experimentation in civil engineering), FLOODsite (FP6 program), ILH (International Levees Handbook), the ICOLD bulletin on internal erosion (International Commission On Large Dams) and the Dutch research program SBW (funded by the Ministry of Waterways and Public Works). Before these studies, specific recommendations about internal erosion risk were extremely limited, such as in the Eurocode7: only the use of filters is recommended, without any quantitative criteria or description (or even definition) of the different mechanisms.

The objective here is to unify these recent approaches in the same general framework to present a summary review of internal erosion for levee managers, engineers and technicians working on the safety of hydraulic structures and (urban) flood defences.

Soil erosion is the cause of failure of the majority of dikes and composite flood defence structures whether through internal erosion, wave overtopping or overflow. For structure failure modes in general, it was agreed by the project partners to follow the definitions of the FLOODsite Report T04-06-01 (Allsop et al., 2007) as a starting point. However, for internal erosion, the definitions of the ICOLD European Working Group on Internal Erosion of Embankment Dams will be followed, stating that *internal erosion occurs when soil particles within an embankment dam or its foundation, are carried downstream by seepage flow. Internal erosion can initiate by concentrated leak erosion, backward erosion, suffusion and soil contact erosion*. Note that this definition also holds for dikes and levees.

We can distinguish four successive phases of internal erosion: (a) initiation when one of the phenomena of detachment of particles occurs, (b) continuation when erosion process can be (or not) stopped by filtering, (c) progression when internal erosion comes to a pipe through the structure or increase pore pressure in the downstream part, (d) failure and breach resulting in uncontrolled release of water in the plain.

Ultimately, potential scenarios of failure can be constructed by combination of the basic mechanisms of internal erosion together with others elementary processes (settlement, uplift, sinkhole, clogging,…).

The implementation of the successive phases of internal erosion in such scenarios of failure is presented more in details in section 2.4.

2.1.2 Glossary and definitions

2.1.2.1 Internal erosion

Internal erosion is related to all processes which involve soil particles detachment and transport by seepage flow within the dam or levee, or its foundation. Such processes can ultimately lead to the instability of the hydraulic work.

2.1.2.2 Basic mechanisms of internal erosion

A consensual view in the community of water retaining structures is that four different basic processes can be identified within the general definition of internal erosion (see ICOLD, 2012). These mechanisms are: backward erosion; concentrated leak erosion; suffusion; contact erosion. Their general definitions are the following (a more detailed description will be given in section 3):

- *Backward erosion*: Detachment of soil particles when the seepage exits to an unfiltered surface and leading to retrogressively growing pipes and sand boils;
- *Concentrated leak erosion*: Detachment of soil particles through a pre-existing path in the embankment or foundation;
- *Suffusion*: Selective erosion of the fine particles from the matrix of coarse particles;
- *Contact erosion*: Selective erosion of the fine particles from the contact with a coarser layer.

Note that the term “piping” can be confusing since it is often used to describe either backward erosion, concentrated leak, or internal erosion as a whole. Consequently, it was discarded here as also proposed by ICOLD.

It is also noteworthy that, until now, only have been studied and described the configurations of a completely homogeneous material or composed of homogeneous layers. To account for heterogeneities within a dike, more research is needed, both from a theoretical/conceptual perspective than from experimental one with new testing devices to be built, is necessary. Work is starting on this aspect and should naturally fill the gap of knowledge on the problem of structure transition developed in Action 3.1.2.

2.1.2.3 General conditions for occurrence of internal erosion

Two conditions should be fulfilled for internal erosion to occur.

- Fundamentally, the first condition is that particles can be detached, i.e. that hydraulic shear stresses are larger than resistant contact forces. To reach this hydro-mechanical criterion, water seeping through the flood defence must have sufficient velocity to provide the energy needed to detach particles from the soil structure.
- The second condition is that detached particles can be transported through the soil. Two criteria should be fulfilled. First, a hydro-mechanical criterion: flow is sufficient to carry the eroded particles. Second, a geometric criterion (which is specific to internal erosion): voids exist in the soils within the flood defence that are large enough for detached particles to pass through them. This void is either a pipe inside the soil, as in backward erosion or concentrated leak erosion, or pore space within the grains of a coarse layer, as observed in suffusion and contact erosion.

The nature of the soil in the embankment determines its vulnerability to erosion. Two main classes have to be distinguished:

- Granular non-cohesive soils: erosion resistance is related to particle buoyant weight and friction; hydro-mechanical transport criterion is linked to rolling and sliding resistance of the grains.
- Cohesive soils: erosion resistance is mainly related to attractive contact forces in between soil’s particles; the main transport mode is suspension flow.

Specific properties of cohesion or particle size (related to permeability) may be required for several of the mechanisms of internal erosion to occur as it will be discussed in section 3.

2.1.2.4 Successive phases of internal erosion

After ICOLD, the process of internal erosion of embankment dams or levees and their foundations can be represented by four phases. This applies for all types of internal erosion.

- *Initiation*: first phase of internal erosion, when one of the phenomena of detachment of particles occurs.
- *Continuation*: phase where the relationship of the particle size distribution between the base (core) material and the filter controls whether or not erosion will continue.
- *Progression*: phase of internal erosion, where hydraulic shear stresses within the eroding soil may or may not lead to the erosion process being on-going and in case of backward and

concentrated leak erosion to formation of a pipe. The main issues are whether the pipe will collapse, or whether upstream zones may control the erosion process by flow limitation.

- *Breach*: final phase of internal erosion. It may occur by one of the following five phenomena (listed below in order of their observed frequency of occurrence).

  1. Gross enlargement of the pipe (which may include the development of a sinkhole from the pipe to the crest of the embankment).

  2. Slope instability of the downstream slope.

  3. Static liquefaction (which may include increase of pore pressure and sudden collapse in eroded zone).

  4. Unravelling of the downstream face.

  5. Overtopping (e.g. due to settlement of the crest from suffusion and/or due to the formation of a sinkhole from a pipe in the embankment).

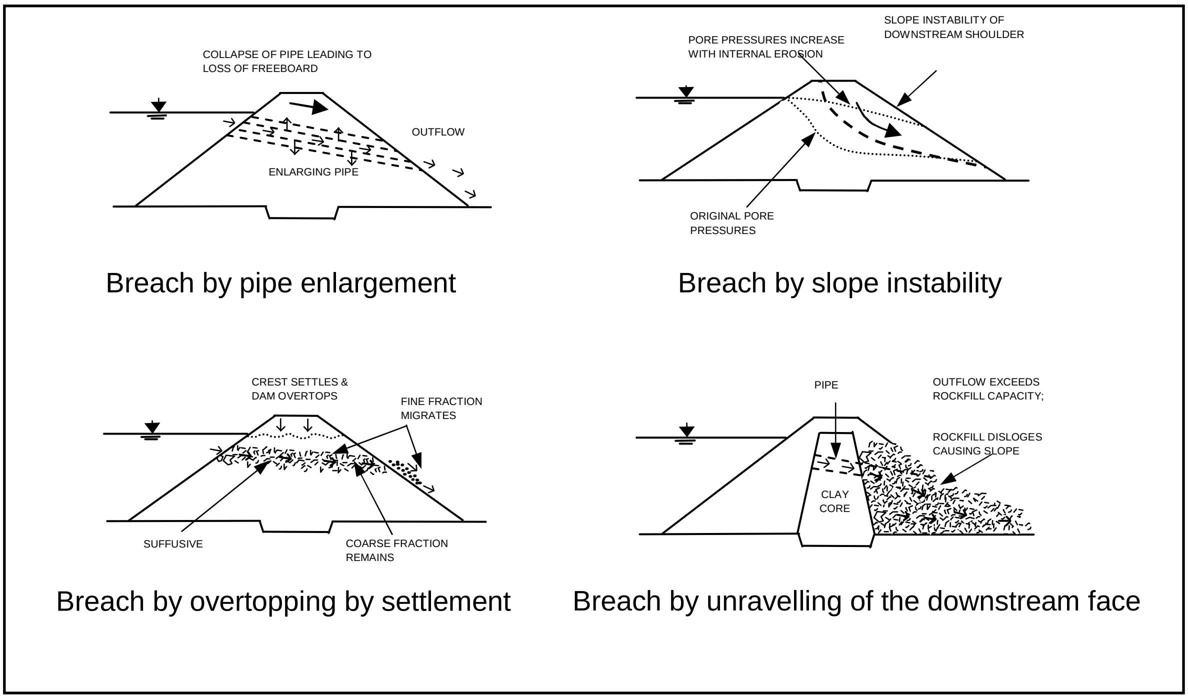


Figure 2.1 Potential failure events (Fell and Fry, 2007)

2.1.2.5 Common field observations of internal erosion

On levees, different types of internal erosion and the actual phase of the process can be hardly distinguished from each other by visual inspection (except for the phase of breaching). The usual field observations of internal erosion are: vegetation or soaked area (related to seepage outflow); fine particles deposit on the downstream slope of the levee; deformation of the levee (settlement, sinkhole); seepage flow with fine particles in suspension (observation during floods); etc.

2.2 Detailed description of the four basic mechanisms

2.2.1 Backward erosion

2.2.1.1 Description

Backward erosion involves the detachment of soil particles when the seepage exits to a free unfiltered surface. For example the ground surface downstream of a soil foundation or the downstream face of a homogeneous embankment or a coarse rock fill zone immediately downstream from the fine grained core.

The seepage flow erodes particles upwards and backwards below the embankment through erosion pipes, sometimes called 'worm-holes', and sand boils form on the surface. In critical circumstances, such as floods, the head difference increases, these pipes may grow progressively from the area with a lower hydraulic head towards the higher head, hence the name 'backward erosion'.

The erosion shortens the seepage path and increases the gradient leading to higher flow velocities causing further backward erosion, increasing the length of the worm-hole, and causing failure when the worm-hole extends backwards to greater than half the width of the embankment base.

One can distinguish two configurations:

- Backward erosion in a sandy layer below an impermeable roof (clay layer, horizontal structure). This configuration involves the development of retrogressively growing pipes in the sand layer below the dike due to ground water flow. Since erosion involves removal of sand, backward erosion is only possible if there is an opening or crack in the impermeable layer. Cracks may develop if the inner impermeable layer is subject to uplifting. This occurs if the hydraulic pressure in the sand layer connected with the river or sea exceeds the weight of the inner impermeable layer. In some cases there is no cohesive cover layer or a ditch cuts through the cover layer, thereby creating a free exit.

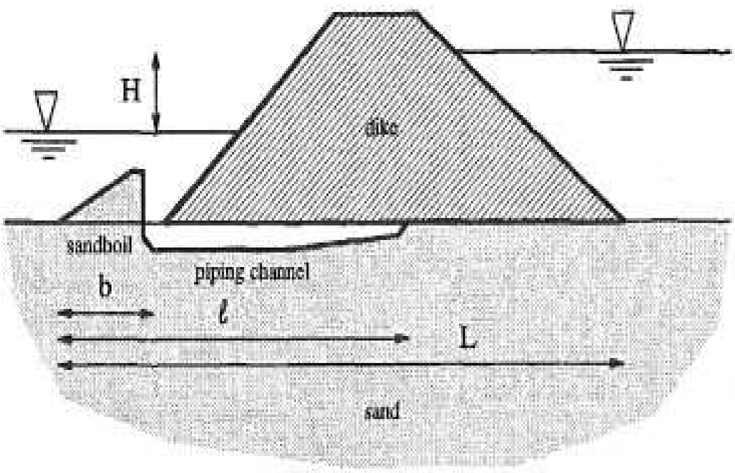


Figure 2.2 Typical example of backward erosion in a sandy layer (Koenders and Sellmeijer, 1992).

- Backward erosion in a cohesive soil. In this configuration, erosion is initiated by a leakage at the exit of a cohesive core to the foundation. The formation of a hole in the core increases erosion rate and hence leads to progressive backward extension of a pipe.

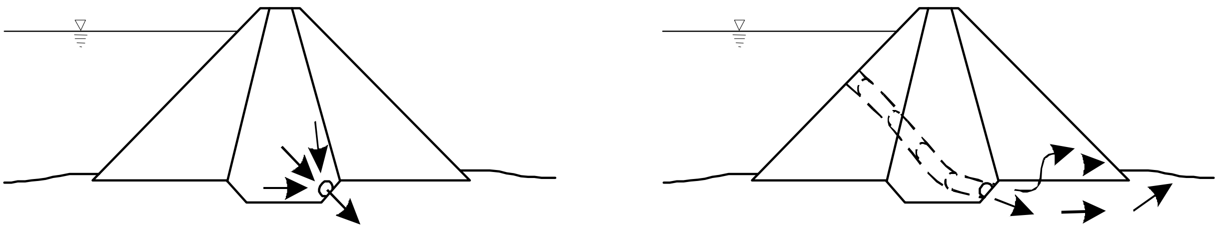


Figure 2.3 Typical example of backward erosion in a cohesive soil (clay core) (Fell & Fry, 2007).

2.2.1.2 Physical/empirical models

Models exist only in the configuration of backward erosion in a sandy layer below an impermeable roof.

- Model of Bligh

Experience with weirs in India between 1910 and 1915 has led to the empirical rule of Bligh, also known as the creep theory, in which percolation factors are established for different soil types, thereby determining a safe head to prevent internal erosion. More engineers followed this empirical approach, such as Griffith and Lane. Lane (1935) argued that vertical length contributes more to safety than horizontal length and adjusted the empirical rule, based on a total of 200 cases in the US.

The rule of Bligh states that failure by backward erosion occurs if:

$$\Delta H - 0.3h_t > \Delta H_c = \frac{L}{C_{Bligh}}$$

In which:

ΔH_c critical hydraulic head over the flood defense for backward erosion according to Bligh [m]

ΔH actual hydraulic head over the flood defense [m]

h_t thickness of top layer [m]

L seepage length (distance between entrance and exit point of aquifer (in horizontal direction) [m]

C_{Bligh} Creep factor of Bligh [-]:

Type of material	*Approximate value for C_{Bligh}*
Fine silty sand	18
Moderate fine sand	15
Course sand	12
Fine gravel	9
Course gravel	4

The presence of a structure, such as a cut-off wall or a weir, causes an extra barrier for the piping path. Lane (1935) concluded that the presence of vertical seepage length causes much more resistance than horizontal seepage length and derived an empirical rule, in which the horizontal and vertical seepage lengths are weighted; in this rule vertical seepage length weighs three times more than horizontal seepage length:

$$\Delta H - 0.3h_t > \Delta H_c = \frac{L/3 + L_v}{C_{lane}}$$

In which:

L_v vertical seepage length [m]

C_{Lane} weighted creep factor of Lane [-]:

Type of material	*Approximate value for C_{Lane}*
Fine micacious sand	14,5-16
Fine quartz sand	12,5-14
Coarse quartz sand	10-12
Shingle	8
Boulders	4

- Model of Sellmeijer

Large research programs were performed in the 1970s and 1980s to create a better understanding of the backward erosion mechanism. The understanding of the process gave rise to the development of a theoretical model, in which the equilibrium of grains in the bed of the pipe is used as criterion for development of the pipe. The critical head can be calculated by combining ground water flow with the flow conditions in the pipe. Curve-fitting resulted in a formula relating sand characteristics to the geometric properties of the sand bed. More recently, the model has been validated with small-, medium- and full-scale experiments (van Beek et al., 2011; Sellmeijer et al., 2011).

According to the model of Sellmeijer (for horizontal retrogressive erosion in a sand layer below a clay dike) backward erosion is possible if (Sellmeijer et al., 2010):

$$\Delta H - 0.3h_t > \Delta H_c = F_{resistance} F_{scale} F_{geometry} L$$

with:

$$F_{resistance} = \frac{\gamma'_p}{\gamma_w}[\eta \tan(\theta)]$$

$$F_{scale} = \frac{d_{70m}}{\sqrt[3]{\kappa L}}\left(\frac{d_{70}}{d_{70m}}\right)$$

$$F_{geometry} = 0.91 \cdot \left(\frac{h_a}{L}\right)^{\frac{0.28}{\left(\frac{h_a}{L}\right)^{2.8}-1}+0.04}$$

In which:

ΔH_c critical hydraulic head over the flood defense for backward erosion according to Sellmeijer [m]

ΔH actual hydraulic head over the flood defense [m]

h_t thickness of top layer [m]

γ'_p unit weight of sand grains under water (about 16,5 kN.m^{-3})

γ_w unit weight of water (10 kN.m^{-3})

θ bedding angle[2] of sand grains [°] (37°)

η White's constant [-] (0,25)

κ intrinsic permeability of aquifer (uppermost sand layer sensitive for retrogressive erosion) [m^2] ($\kappa = k\mu_w/\gamma_W$ with μ_W [kg.m^{-3}.s^{-1}] is viscosity of water and k [m.s^{-1}] is hydraulic permeability)

d_{70} d_{70} of aquifer (on the grading curve, diameter corresponding to 70% of material passing through the d size sieve) [m]

d_{70m} average d_{70} of small scale tests (208 µm) [m]

h_a thickness of aquifer (uppermost sand layer sensitive for retrogressive erosion) [m]

L seepage length (distance between entrance and exit point of aquifer in horizontal direction) [m]

[2] This angle determines how a grain is situated on two other grains and is only related to weight and geometry since the model assumes that the grain rolls over the others without fiction.

In contrary to the model of Bligh, the model of Sellmeijer is semi-empirical and accounts for scale effect (ratio between grain size, seepage length and gradient ∙ H/L). Recent results of backward erosion tests performed in SBW research program have shown that also uniformity, grain shape and relative density have influence on the critical head. However, this influence appears to be limited. For this reason they have not been included in the equations.

2.2.1.3 Parameters to be determined for models and ranges of validity

- Parameters for model of Bligh

Parameter	*Meaning*	*Method of determination*
h_t	thickness of cohesive top layer [m]	Boreholes, CPT, geological maps, geophysical methods
C_{Bligh}	Creep factor of Bligh [-]	Grain size distribution and table in previous section
L	seepage length (distance between entrance and exit point of aquifer (in horizontal direction) [m]	Topography, Cross sections of the levee, Surveying, Remote sensing (Flimap), etc

In the model of Bligh, creep values are given for D_{50} varying from very fine (<105 μm, C_{Bligh}=18) to very coarse (>16 mm, C_{Bligh}=4) sands.

- Parameters for model of Sellmeijer

Parameter	*Meaning*	*Method of determination*
h_t	thickness of cohesive top layer [m]	Boreholes, CPT, geological maps, geophysical methods
κ	intrinsic permeability of aquifer [m^2]	Grain size distribution
d_{70}	d_{70} of aquifer [m]	Grain size distribution
h_a	thickness of aquifer (uppermost sand layer sensitive for retrogressive erosion) [m]	Boreholes, CPT, geological maps, geophysical methods
L	seepage length (distance between entrance and exit point of aquifer (in horizontal direction) [m]	Topography, Cross sections of the levee, Surveying, Remote sensing (Flimap), etc
η (optional)	White's constant [-]	From experience (0.25)
θ (optional)	bedding angle sand grains [°]	From experience (37º)

The model of Sellmeijer is validated for the following ranges of properties: d_{70} in between 150-450µm; $Cu=d_{60}/d_{10}< 2{,}6$; Roundness[3] in between 35 and 70; range of relative density>50%; Silt fraction (<63 µm)<10%.

2.2.2 Concentrated leak erosion

2.2.2.1 Description

Concentrated leak erosion appears in a preferential path such as opening cracks or pre-existing holes. Along this path, water infiltration is sufficient to initiate soil particles detachment from lateral surfaces and transport away inducing enlargement of the path.

In presence of cohesive materials able to "hold a roof", theses openings result in the formation of a continuous tunnel called a "pipe" between the upstream and the downstream side of the embankment or its foundation.

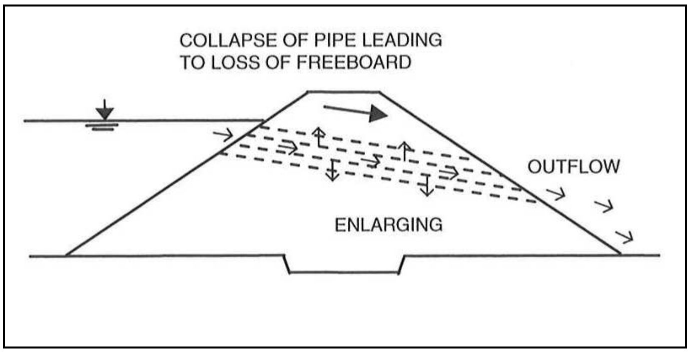


Figure 2.4 Typical example of concentrated leak erosion (Fell and Fry, 2007)

In some circumstances, these openings may be sustained by the presence of structural elements such as spillways, conduits, culverts (see Section 3). But in a large majority of cases, concentrated leak erosion occurs in cohesive soils.

Several causes of pre-existing defects can be identified:

- Cracks can be induced from desiccation and tension at high level in fill due to: Freezing and thawing; Hydraulic fracture in a cohesive clay core; Continuous permeable zone containing coarse and/or poorly compacted materials which form an interconnecting voids system.

- Holes can result from animal burrows or rotten roots.

Concentrated leak erosion could also appear subsequently to pipe formation by backward erosion or contact erosion.

[3] varying between 10 (angular) to 90 (perfectly round) in the Dutch classification.

2.2.2.2 Physical/empirical models

The first model to interpret concentrated leak erosion was proposed by Wan and Fell (2002, 2004a&b) for a specific type of tests, called Hole Erosion Test. This test reproduces concentrated leak erosion in a pre-existing cylindrical pipe. More recently, a model combining hydrodynamic equations for a turbulent pipe flow and tangential erosion law was able to interpret more accurately experimental HET results (Bonelli et al, 2006; Bonelli and Brivois, 2008; Bonelli, 2011).

These models use a local erosion law which is often written in the form of a threshold law:

$$\varepsilon = k_{er}\left(\tau - \tau_c\right)$$

Where:

ε eroded mass rate [kg.m^{-2}.s^{-1}]

τ hydraulic shear stress exerted by the flow at the lateral surface of the hole [Pa]

τ_c critical bed shear stress [Pa]

k_{er} coefficient of erosion [s.m^{-1}]

"Erodibility" of soil is characterized by the two last physical parameters. The critical shear stress is the minimum hydraulic shear stress required to initiate the detachment of soil particles. Below this value, no erosion is observed. The coefficient of erosion reflects the rate of the detachment of soil particles when the stress is maintained constant above the critical shear-stress.

2.2.2.3 Parameters to be determined for models and ranges of validity

Parameter	*Meaning*	*Method of determination*
ΔP	Pressure drop [Pa]	Pressure transducers; water level
Q	Flow rate [m^3.s^{-1}]	Flow meter (if possible)
τ	Hydraulic shear stress [Pa]	Deduced from pressure drop and flow rate or from pressure drop and radius
R	Radius of the pipe [m]	Deduced from flow rate and pressure drop measurements; to be estimated in situ
L	Length of the pipe [m]	Given in HET tests; to be estimated in situ

The Bonelli model is validated for all cohesive soils able to sustain an internal pipe without collapsing. The fitting procedure of the model on pressure temporal evolution determines the erodibility characteristics of the soil, namely k_{er} and τ_c.

2.2.3 *Suffusion*

2.2.3.1 Description

Suffusion involves selective erosion of fine particles from the matrix of coarser particles. The fine particles are removed from the voids or pores between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. Suffusion involves little or no change in volume of the soil mass but an increase in permeability and in seepage velocities. Suffusion is a priori possible in a cohesive soil but this does not seem relevant to hydraulic structures and flood defenses. Only non-cohesive granular soils are considered.

This phenomenon is sometimes referred to as suffosion in the literature. Some authors introduce a slight difference between suffusion and suffosion. This document being provided for non-specialists, we made the choice to use "suffusion" as a generic term for all related phenomena.

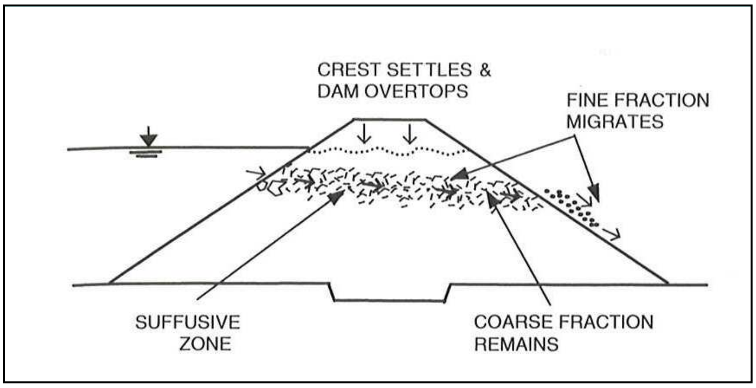


Figure 2.5 Typical example of suffusion (Fell and Fry, 2007)

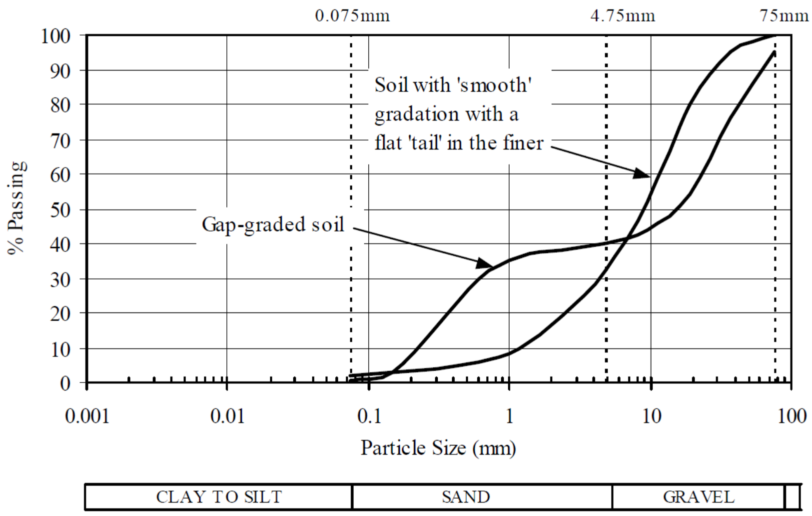


Figure 2.6 Soil gradation types which are responsible to suffusion (Foster and Fell, 1999)

Widely graded and gap graded materials which are vulnerable to suffusion are said to be not internally stable (Kenney & Lau, 1985).

2.2.3.2 Physical/empirical models

Both geometric and hydraulic conditions must be fulfilled for suffusion to occur. Many granulometric criteria exist in the literature. However, one of the most commonly used is the standard proposed by (Kenney & Lau, 1985) which combines grain size distribution and filtration rules. More recently, (Li and Fannin, 2008) have compared this criterion with another one proposed by (Kezdi, 1979) while (Wan and Fell, 2008) have shown that the previous commonly used methods are conservative for silt-sand-gravel or clay-silt-sand-gravel soils.

- Model of Kenney and Lau

It consists for each particle size d to determine the mass percentage *F* of grains smaller than *d* and the mass percentage *H* of grain sizes between *d* and *4d* ($H = F_{4d} - F_d$). Grains smaller than *d* can be detached if there are not enough grains in the interval [*d* to *4d*] to keep them trapped. They proposed that, in the range $0<F<X$, the soil is unstable if the curve *H* versus *F* is located wholly or partially below the line defined by $H=F$. For soils with a coefficient of uniformity $Cu=d_{60}/d_{10}<3$, *X* is chosen equal to 0.3 and $X=0.2$ when $Cu>3$.

- Model of Kezdi

With the same definition for *F* and *H*, Kezdi has simply proposed that a soil is unstable if $H<0.15$. This criterion is more conservative than the previous one for $F<0.15$ and less otherwise.

- Comparative analysis by Li and Fannin

Using some new experiments and many previous ones found in the literature, Li and Fannin have recently proposed to use Kezdi criterion for gap-graded size distribution whereas Kenney and Lau criterion is suited for widely-graded soils.

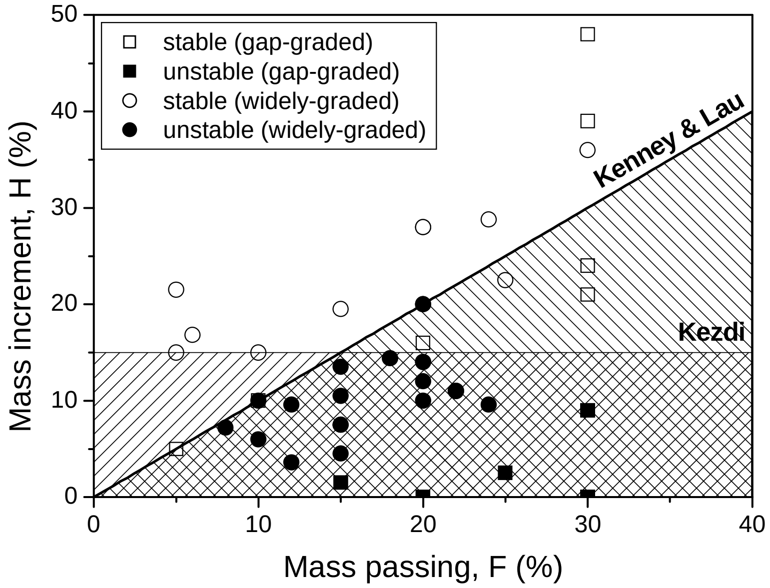


Figure 2.7 Comparison of Kezdi and Kenney & Lau criteria, adapted after Li & al. (2008)

Li (2008) proposed also a hydro-mechanical criterion in terms of threshold hydraulic gradient i_{suf}, validated on experiments on unstable soils, which is simply a fraction of the critical gradient i_c first introduced by Terzaghi:

$$i_{suf} = \chi . i_c = \chi \frac{\gamma'}{\gamma_W}$$

where:

γ_W specific weight of water [N.m^{-3}]

γ' buoyant specific weight of soil [N.m^{-3}]

χ reduction factor [-]

The coefficient χ accounts for the reduction of the vertical effective stress carried by the fine particles as first introduced by Skempton and Brogan (1994). These authors proposed that χ depends only on the geometry and reads:

$\chi = 3.85(d'_{85}/O_{50}) - 0.616$

where:

d'_{85} characteristic diameter d_{85} of the fine fraction of soil [m]

O_{50} effective constriction size of the coarse fraction [m]

2.2.3.3 Parameters to be determined for models and ranges of validity

- Model of Kezdi and model of Kenney & Lau

Parameter	*Meaning*	*Method of determination*
F	Mass percentage of grain sizes < d [-]	From grain size distribution
H	Mass percentage of grain sizes between d and 4d [-]	From grain size distribution
Cu	Coefficient of uniformity [-]	$Cu=d_{60}/d_{10}$

- Model of Li

Parameter	*Meaning*	*Method of determination*
γ'	Buoyant soil specific weight [N.m^{-3}]	By weighting
d'_{85}	d_{85} of the fine fraction of soil [m]	From grain size distribution
O_{50}	Effective constriction size of the coarse fraction [m]	From grain size distribution

2.2.4 Contact erosion

2.2.4.1 Description

Contact erosion is a form of internal erosion which involves erosion of fine particles from the contact with a coarser material layer, caused by flow through the coarser layer. For instance along the contact between silt and gravel sized particles (example: in alluvium in the foundations of dams or levees, or in a filter). Two basic situations may be discerned: one with flow parallel to the interface where the particles from the fine material layer are eroded and transported through the pores of the coarse material as indicated in Figure 2.8; the other with flow transverse to the interface as shown in Figure 2.9. In our knowledge, only one study was devoted to this last situation (Ziems, 1969). In both cases, small particles wash out, leaving the larger particles behind. Eventually, the soil matrix may collapse.

The consequences of contact erosion depend on the respective position of the fine material layer: underlying or overlying. In the first case, the cavities created by fine particles washed out induce settlements in the overlying coarse material. In the second case, cavities could be created by erosion, that remain stable only if the material is cohesive enough. Several scenarios could be observed as summarized in Figure 2.10: a) sinkhole, b) concentrated leak erosion or backward erosion, c) development of a weak zone in the levee and overall instability by sliding, d) clogging of the permeable and increase of pore water pressure (heave, static liquefaction).

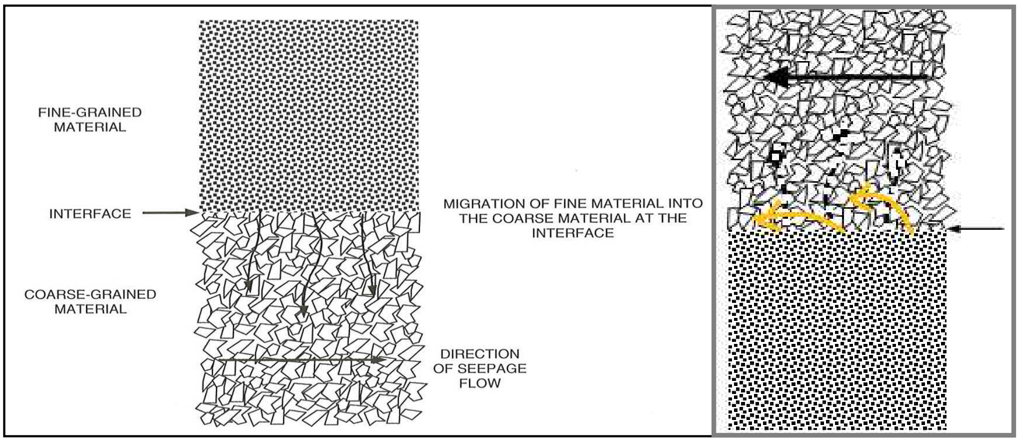


Figure 2.8 Sketch of contact erosion with parallel flow after Fell and Fry (2007).

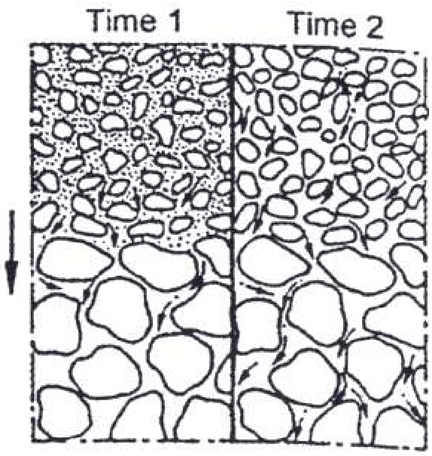


Figure 2.9 Sketch of contact erosion with transverse flow after Ziems (1969).

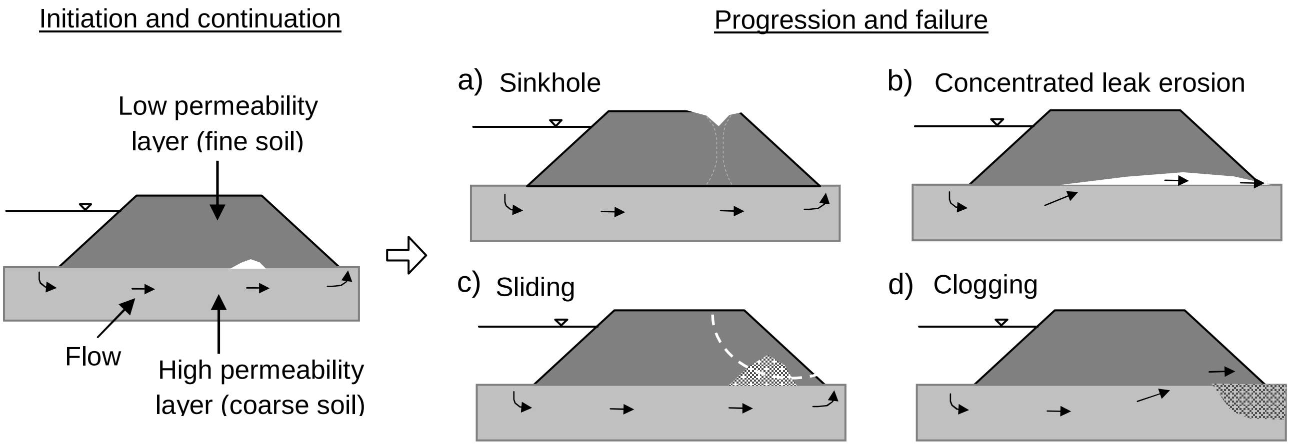


Figure 2.10 Consequences of contact erosion, adapted after Beguin (2011)

2.2.4.2 Physical/empirical models

As in suffusion, both geometric and hydraulic conditions must be fulfilled. But unlike suffusion which concerns a unique material with a broad graded grain size distribution, contact erosion appears at the interface between two different materials having distinct grain size distributions. Consequently, the geometric condition for contact erosion to occur is fulfilled when the classical filter rules[4] are not satisfied and the studies related to contact erosion have mainly focused on hydraulic threshold.

Most of the models proposed for contact erosion are dedicated to the first configuration, i.e. an underlying fine material layer with non-cohesive soils (sand). They result from an adaptation of Shields criterion (Shields, 1936) with an empirical coefficient that accounts for the specific geometry of the coarse layer (Brauns, 1985; Bezuijen et al., 1987). Darcy velocity has been

[4] Since the well known Terzaghi filter criterion ($D15/d85 \leq 4$ or 5) which still governs the main philosophy of filtration in the current design practices, Sherard et al. (1984) among others have later extended this original filter rule; see for instance the ICOLD bulletin (1994) on this topic.

chosen by the majority of the authors as a good indicator of the hydraulic loading. This threshold reads:

$$U_{crit} = \alpha n_D \sqrt{\frac{\gamma'}{\gamma_W} g d_{50}}$$

with:

n_D porosity of the coarse layer [-]

g gravity [m.s^{-2}]

d_{50} median diameter of the sand grading curve [m]

α empirical coefficient [-]

The empirical coefficient α is approximately equal to 0.65 as proposed by Brauns (1985), or depends on the type of fine soil and flow characteristics (Bezuijen et al., 1987).

More recently, the inverse configuration as well as cohesive soils has been studied (Schmitz, 2007; Guidoux et al., 2010; Beguin, 2011). Based on experimental results of contact erosion tests with silts and clays, Guidoux et al. (2010) adapted empirically Brauns expression to take into account the adhesive forces.

Beguin (2011) proposed to use the same threshold erosion law as the one used for concentrated leak erosion model (see section 2.2.2.2). This requires a relation between shear stress and hydraulic gradient (or equivalently Darcy velocity) as the one proposed by Reddi et al. (2000) or Wörman, (1992). Note that for cohesive soils, Beguin (2011) also successfully used the excess shear stress erosion law proposed for concentrated leak erosion.

2.2.4.3 Parameters to be determined for models and ranges of validity

Parameter	*Meaning*	*Method of determination*
n_D	Porosity of the coarse layer [-]	In situ measurement of bulk density and water content
d_{50}	Median diameter of sand [m]	From grain size distribution
γ'	Buoyant soil specific weight [N.m^{-3}]	By weighting

2.2.5 Matrix representation of internal erosion

Two synthetic matrices are proposed to summarize respectively the basic mechanisms of internal erosion and the existing models with the methods for determining the involved parameters.

Table 2.1 Matrix of the basic mechanisms of internal erosion in dams and levees

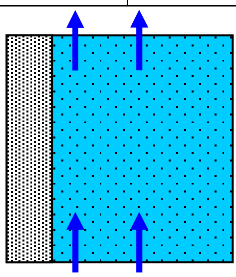
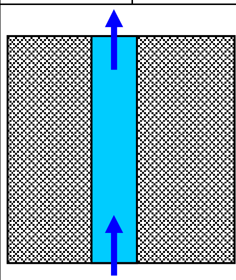
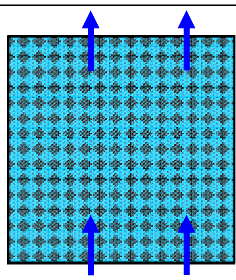
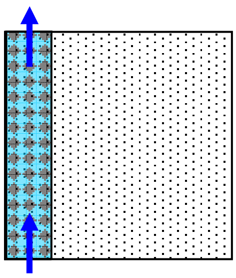
Type of internal erosion		Type of soil	Presence of a boundary layer: Coarse filter	Presence of a boundary layer: Impermeable layer	Location of erosion
Backward erosion		Non-cohesive	No	Roof (parallel to flow)	Downstream along the roof
		Cohesive	No	No	In volume
Concentrated leak erosion		Non-cohesive	No	Roof	Along the roof
		Cohesive	No	No	In volume
Suffusion		Fine enough particles (cohesive or not) for transport through the fixed granular skeleton	Possible	No	In volume
Contact erosion		Fine enough particles cohesive or not) for transport through the coarse layer	No	No	Along the contact zone

Table 2.2 Matrix of models of internal erosion and parameters to be determined

Type of internal erosion	Models	Parameters	Methods of determination	Domains of validity	Key soils' parameters
Backward erosion (in non-cohesive material)	Threshold: $\Delta H_c = F_{res} F_{scale} F_{geo} L$ (Sellmeijer)	κ: aquifer permeability	Permeameter, grain size distribution	d_{70}:150 – 450 µm; d_{60}/d_{10}< 2,6; roundness: 35 – 70; relative density > 50%; Silt fraction (<63 µm) < 10%	Grain size distribution
		d_{70} of aquifer	Grain size distribution		
	Threshold(Bligh) $\Delta H_c = L/C_{Bligh}$; $\Delta H_c = (L/3+L_v)/C_{Lane}$	C_{Bligh}: creep factor C_{Lane}: weighted creep factor	Grain size distribution and empirical table	d_{50}: from very fine (<105 µm) to very coarse (>16 mm)	
Concentrated leak erosion	Excess shear stress erosion law: $\varepsilon = k_{er}(\tau - \tau_c)$ (Bonelli)	Critical stress: τ_c	Hole Erosion Test (by adjustment of the erosion law)	Cohesive soils (sufficient cohesion to hold up drilling a cylindrical hole for testing)	Water content; Compaction Fine content; Chemical activity
		Erosion coefficient: k_{er}			
Suffusion	Self-filtration rules $H>15$ (Kezdi); $H>F$ (Kenney & Lau)	F: mass passing (size d) H: increment of mass passing (size $D=4d$)	Grain size distribution	Non-cohesive soils	Grain size distribution
Contact erosion	Threshold (Beguin) $U_c = Fr_c n_D((s\text{-}1)gd_{50})^{1/2}$ $Fr_c = f(d_{50}, D_{15}, Re_D)$	k and n_D: coarse soil permeability and porosity; d_{50} and D_{15}	Grain size distributions	Non-cohesive soils	Grain size distribution
	Excess shear stress erosion law $\varepsilon = k_{er}(\tau - \tau_c)$ (Beguin)	Similar to concentrated leak erosion	Contact Erosion Test (adjustment)	Cohesive soils	Similar to concentrated leak erosion

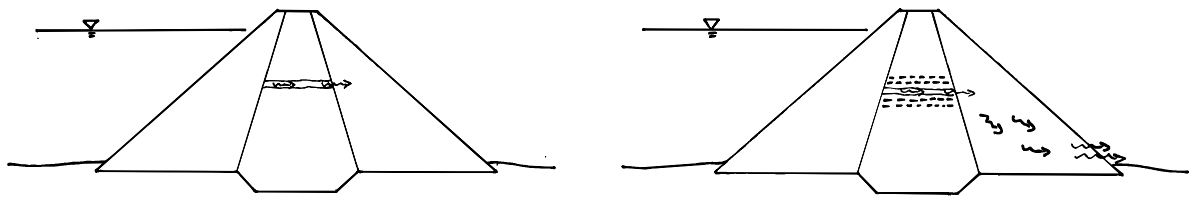
2.3 Scenarios of failure development by internal erosion

2.3.1 General construction of a scenario

As already mentioned in section 2, failure of the flood defence by internal erosion may not necessarily result from a continuous evolution of one of the of four basic erosion processes reported above but rather from a combination of some of these mechanisms together with other processes. The timing of these failure scenarios can be broken down according to the four steps previously proposed: initiation, continuation, progression and breach (or failure).

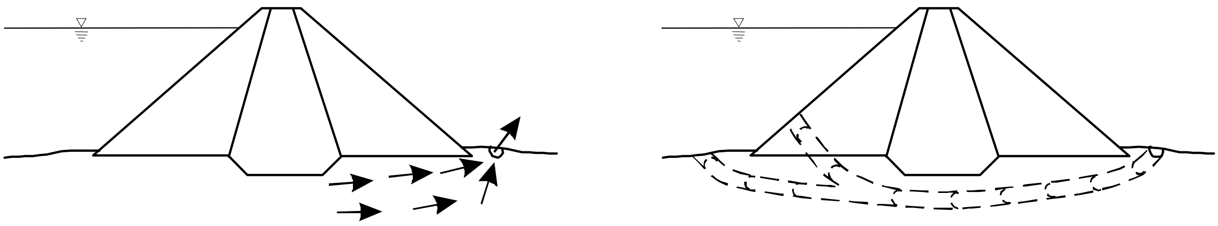
2.3.2 Some illustrative examples

This report is not intended to be an exhaustive record of all failure scenarios that can be envisaged within a dam, a levee or their foundation where internal erosion may occur. Only a few illustrative examples are presented below and a more detailed one is proposed for backward erosion in a sandy layer in Appendix A of the supplementary report on internal erosion. A more complete summary of the main scenarios is provided in section 2.3.3 within table 2.3.



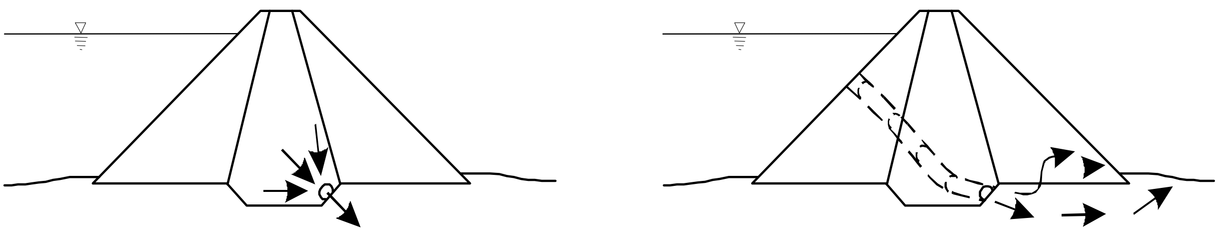
INITIATION →	CONTINUATION →	PROGRESSION →	BREACH
Concentrated leak forms, erosion initiates along walls of crack	Continuation of erosion (if filter rule is not fulfilled)	Enlargement of concentrated leak	Breach mechanism forms

Figure 2.11 Failure resulting from concentrated leak erosion; adapted after Foster and Fell (1999)



INITIATION →	CONTINUATION →	PROGRESSION →	BREACH
Leakage exits from the foundation and backward erosion initiates	Continuation of erosion (if filter rule is not fulfilled)	Backward erosion progresses to form a pipe	Breach mechanism forms

Figure 2.12 Failure resulting from backward erosion in the foundation; adapted after Foster and Fell (1999)



INITIATION →	CONTINUATION →	PROGRESSION →	BREACH
Leakage exits from the core into the foundation and backward erosion initiates as core erodes into the foundation	Continuation of erosion (if filter rule is not fulfilled)	Backward erosion progresses to form a pipe. Eroded soil is transported in the foundation	Breach mechanism forms

Figure 2.13 Failure resulting from backward erosion from the embankment to the foundation; adapted after Foster and Fell (1999)

2.3.3 Matrix representation

A third synthetic matrix is proposed here to summarize the main failure scenarios of embankment failure initiated by internal erosion.

Table 2.3 Matrix of the main scenarios of embankment failure by internal erosion

Type of internal erosion	Initiation	Continuation	Progression	Failure
Backward erosion	Uplift (at toe) Local defect (hole, root) Induced concentrated leakage (suffusion, contact	Beginning of pipe extension (parallel to flow)	Acceleration of pipe extension	Roof collapse Sinkholes
			→ Concentrated leak erosion	
Suffusion	Self-filtering condition not fulfilled	Without filter downstream	Yes	Settlement, Sinkholes
			→ Backward erosion	
			→ Contact erosion	
		With filter downstream	Clogging, pore pressure increase	Diffuse instability (liquefaction)
Contact erosion	Tangential flow erosion	Cavities settlements (locally)	→ Concentrated leak erosion	
			→ Backward (or forward) erosion	
Concentrated leak erosion	Pre-existing opening (settlement, structure transition, layering) Induced opening (contact erosion, settlement, backward erosion)	Beginning of pipe enlargement (normal to flow)	Acceleration of pipe enlargement	Roof collapse Sinkholes

2.4 Some practical considerations for risk assessment of failure by internal erosion

Looking carefully at table 2.3, we notice that one of the basic mechanisms of internal erosion can be distinguished from the three others. This is concentrated leak erosion that is systematically late in the chronological chain of all scenarios. It is therefore the predominant mechanism leading to the final stage of failure. One the contrary, the other mechanisms are mostly involved during initiation of erosion, with much slower kinetics than concentrated leak erosion, and they generally conveyed the place to faster phenomena as the hydraulic structure approaches a situation of potential failure.

Concretely, for risk management purpose, concentrated leak erosion is the mechanism that must be controlled in a priority. For this, we can propose the following simplified formula from the model developed by Bonelli and Benahmed (2011) for earthen water retaining structures. More specifically, these laws are based on the most frequent case where roof collapse is the final failure process and they are derived 1) by simplifying the hydro-mechanical model already presented in paragraph 2.2.2 and 2) by assuming a simple geometric criterion for occurrence of roof collapse, namely. The authors then propose the following expression for the remaining time before failure once a pipe was detected:

$$\Delta t_f \approx \frac{2\gamma_d L}{k_{er}\gamma_w g H}\ln\left(\frac{H}{D_d}\right)$$

Where:

Δt_f remaining time before failure [s]

γ_d unit weight of dry soil (about 26,5 kN.m^{-3})

γ_w unit weight of water (10 kN.m^{-3})

L length of the structure [m]

H height of the structure [m]

g gravity [m.s^{-2}]

k_{er} coefficient of erosion [s.m^{-1}]

D_d diameter of the pipe when first detected [m]

As noted by Bonelli and Benahmed (2011), the latter value is not always easy to know but one can reasonably assumed that it is of the order of 10cm: indeed, a leak in a gap of about one centimeter is unlikely to be detected while that led to tens of centimeters has already generated a flow rate well above the detection threshold.

The authors propose finally an even more simplified relation applicable under the standard conditions of a river dike (Bonelli, 2008):

$$\Delta t_f\left[mn\right] \approx 5\times 10^{I_{er}-2}$$

Where:

I_{er} erosion index defined by $I_{er}=-log(k_{er})$ [-]

As explained by the authors, this expression gives, according to the soil's erosion index, an estimate of the time before failure for a 5 to 10 m high dike, when detecting a concentrated leak during a crisis. If the soil has an erosion index of about 2, the failure by roof collapse will take place in a few minutes while it will take several hours to several days for soils with erosion indices ranging from 3 to 4. In the latter situation, it gives time to intervene, or to observe a flood recession, and the failure is therefore very unlikely.

Note that using these formulas require knowledge of parameters k_{er} which can be obtained from various tests of erosion as detailed just below.

2.5 Determination of erosion parameters

2.5.1 Inventory and description of existing testing facilities

An international inventory of lab equipments and testing devices available for characterizing the vulnerability of soil against erosion phenomenon was lunched through the European Working Group on Internal Erosion, ICOLD community and the partners of ERINOH Project.

This action aims to gather and complete as much as possible a database with the existing devices around the world for measuring erosion parameters, determination of the key parameters influencing the erosion process, classification of erodible soils, etc.

The inventory listing is presented on table 2.4. Some information was collected from the web.

When received, the detailed device datasheets are attached in Appendices B of the supplementary report on internal erosion.

It should be noted here that almost all of these testing devices have been developed in the last decade which illustrates the forthcoming interest of the internal erosion topic in the community of civil and hydraulic engineering.

INVENTORY OF EROSION DEVICES (PAGE 1/2)

Institutions	Country	Type of device	Mechanism studied	Type of soil involved	Results
IRSTEA (Ex Cemagref)	France	- HET - JET	-Concentrated leak erosion - Erosion by impinging Jet	- Cohesive soils	- erodibility parameters *
IFSTTAR	France	- HET - MoJet - New Crumb Test	-Concentrated leak erosion - Erosion by impinging Jet - Slaking, dispersivity	- Cohesive soils	- erodibility parameters *
LTHE	France	- CET	- Contact erosion	- Cohesive and non cohesive soils	- critical velocity
LOMC	France	- Suffusion Test	- Filtration - Suffusion	- Cohesive and non cohesive soils	- erodibility parameters *
GeM	France	- Triaxial suffusion test - Oedometer suffusion test - JET	- Suffusion - Erosion by impinging Jet	- Cohesive and non cohesive soils	- erodibility parameters *
GeM and IFFSTAR	France	- CVET	- Volume erosion		- erodibility parameters *
** CERMES / ENPC	France	- FTest	- Filtration	- Cohesive and non cohesive soils	?
ESTP	France	- EFA	- Tangential erosion	- Cohesive soils	- erodibility parameters *
LTDS Lyon	France	-FTest	- Filtration	- Non cohesive soils	?
GeophyConsult	France	- HET - JET	- Concentrated leak erosion - Erosion by impinging Jet	- Cohesive soils	- erodibility parameters *
** USBR	USA	- HET - JET	- Concentrated leak erosion - Erosion by impinging Jet	- Cohesive soils	- erodibility parameters *
UNSW	Australia	- HET - SET	- Concentrated leak erosion	- Cohesive soils	- erodibility parameters *

INVENTORY OF EROSION DEVICES (PAGE 2/2)

Institutions	Country	Type of device	Mechanism studied	Type of soil involved	Results
** USBR	USA	- HET - JET	- Concentrated leak erosion - Erosion by impinging jet	- Cohesive soils	- erodibility parameters *
UNSW	Australia	- HET - SET	- Concentrated leak erosion	- Cohesive soils	- erodibility parameters *
**Texas A&M University	USA	- EFA	- Tangential erosion	- Cohesive soils	- erodibility parameters *
**USDA ARS	USA	- JET	- Erosion by impinging jet	- Cohesive soils	- erodibility parameters *
University of Granada	Spain	- NEF test	- Filtration	- Non cohesive soils	?
**University of British Columbia	Canada	- Permeameter	- Internal instability (suffusion)	- Cohesive and non cohesive soils?	- Critical hydraulic gradient?

* Critical shear stress τ_c and erosion coefficient k_{er} determined from threshold erosion law $\dot{m} = k_{er}(\tau - \tau_c)$

** Information collected from websites

? Missing information

Table 2.4 List of existing testing facilities on soil internal erosion based on inquiries among ICOLD[1], EWG[1] on internal erosion, ERINOH[1] Project

[1] see section 2.1.1 for meaning of the acronyms

2.5.2 Cross tests (from pilot sites)

Samples from a levee near Orléans (France) have been tested to determine susceptibility to (a) backward erosion by Deltares (Delft, Netherlands) and (b) concentrated leak erosion by IFFSTAR (Paris, France).

- (a): The backward erosion tests were performed to validate the Sellmeijer model for sand with characteristics that fall out of the validation range given in 2.1.3 (which more or less covers the sands occurring in the Netherlands). Compared to the Dutch sands the Loire sand is characterised by a relatively high percentage of fines (<63 μm), although still smaller than 10%, and relatively high values of $Cu=d_{60}/d_{10}$. For the material studied the rule of Sellmeijer seems to underestimate the critical head for backward erosion significantly. Since only two experiments were performed and there is no complete clarity with respect to the large difference between the critical heads found in the two experiments, the conclusions concerning the validity of the Sellmeijer rule are rather speculative. At least the values obtained with the model of Sellmeijer are conservative rather than unsafe. More details are given in the report attached in appendix C1 of the supplementary report on internal erosion.

- (b) : The concentrated leak erosion tests were planned in order to investigate the erodibility of the Levee of Bou soil and to determine the erosion parameters; critical shear stress τ_c and coefficient of erosion k_{er}. Due to the type of soil which is poorly graded sand with very low cohesion, the samples have shown an extremely low critical shear stress (less than 1.25Pa) and collapsed few seconds after the beginning of the test. Because of extremely low resistance to hydraulic shear stress, no further tests were performed. More details are given in the report attached in appendix C2 of the supplementary report on internal erosion.

Concentrated leak erosion tests (HET, Hole Erosion Tests) on soil samples from Humber tidal embankment (UK) have been carried out in the laboratory by Irstea (Aix-en-Provence, France). The purpose of the experimental study was to characterize and to quantify the sensitivity to erosion phenomenon by determining the critical shear stress τ_c and the erosion coefficient k_{er} of Humber tidal embankment soil. The test program includes 13 hole erosion tests on the samples. The erosion tests were performed on reconstituted samples. The soil was compacted in layers inside the testing mould using Proctor hammer at the dry density and moist content measured in our laboratory on undisturbed soil samples. After each HET, the soil sample was recovered and specific identification tests for classification of soil and determination of its basic physical properties were performed: particle size distribution, density (bulk density, dry density, and particle density), moisture content, Atterberg limits. The aim was to find a correlation between the soil properties and the erosion parameters.

Due to the type of soil which is silty clay loam (high percentage of silt and clay) with an important cohesion, almost of the tested samples exhibit high resistance to the erosion process. In fact, despite the application of an important pressure gradient, no or very low erosion was observed. In addition some of the tests have not been successful because of the slipping of the samples inside the cell, due precisely to the applied pressure.
Only two tests lead to the determination of the erosion parameters. However, the values obtained (high critical erosion stress and moderately slow kinetic of the erosion process) confirmed the strength of this soil against erosion. More details of the experiments results are given in the comprehensive report on internal erosion (FloodProBE ref WP3-01-12-05).

2.5.3 Data bases for erosion parameters

Several references with available data that contain relevant erosion parameters for internal erosion are listed in Appendix D1 of the comprehensive report on internal erosion (FloodProBE ref WP3-01-12-05). Non-cohesive soils are studied trough suffusion test and contact erosion test. Note that parameters for backward erosion are not determined from backward erosion tests which were only performed to improve and validate Sellmeijer model (see 2.1.3). For cohesive soils, HET, JET and contact erosion test have been used to determine soil erosion parameters.

For HET tests carried out in *Irstea*, a database is given in Appendix D2 of the comprehensive report on internal erosion (FloodProBE ref WP3-01-12-05).

2.5.4 Key soils parameters

2.5.4.1 Non-cohesive soils

In the case of non-cohesive materials where erosion is the result of local destabilizations of particles at the interface induced by the hydraulic flow, it is obviously the grain size distribution of the material that determines the resistance of soil to erosion. This main influence is also obvious in the different expressions of the erosion threshold in the models of backward erosion (see 2.1.3), suffusion (see 2.3.3) and contact erosion (see 2.4.3) where the main parameters are typical diameter, as d_{70} or d_{50}, permeability and more complex parameters (as H and F in suffusion models) deduced from the grain size distribution.

2.5.4.2 Cohesive soils

Presently, no simple relationship was found between soil characteristics and erodibility parameters determined from direct testing of cohesive soils. However, soil parameters with the most significant influences have been clearly identified.

According to the databases built from HET tests on natural soils from dams in USWA (Wan & Fell), from HET tests on reference and natural soils in Cemagref (Benahmed and Bonelli, 2012) and from JET tests carried out in USBR (Hanson 2001) or by Reggazoni (2011) on natural soils (in situ or in laboratory), the following statements are proposed to account for the key sensitivities of erodibility parameters to some more classical features of soils.

- For a given soil, the more influencing parameters are compaction density, water content, degree of saturation (Hanson 2007, Hanson 2011, Lim 2006, Regazzoni 2011, Benahmed 2012). They acts positively on soil erodibility; i.e. an increase of these parameters leads to an improvement of soil resistance against erosion phenomenon.

- For a given nature of fines (i.e. for a unique chemical activity), fines content in a soil has obviously the more important impact on erosion parameters (Pham, 2008; Benahmed, 2012). The critical shear stress of erosion increases significantly as the percentage of fine content present in the soil increases.

- For different types of clay, chemical activity is absolutely fundamental and a very strong discrepancy is observed from one clay to another, even within the same “class” of clays (kaolins for instance) (Benahmed 2012). Dispersivity of soil has a significant impact on erodibility (Lim 2006, Regazzoni 2011).

It is also noteworthy that the aging effect plays a significant role on soil erodibility parameters with strong differences between intact and reconstituted soil samples (Benahmed 2012). Preliminary erosion experimental results have shown that intact samples are more resistant than reconstituted samples. This is certainly due to the destructuration of soil in case of reconstituted samples. However few attempts have been made till now to investigate this point.

3 Structure transitions

3.1 Problems linked to structure transitions

3.1.1 Introduction

Most levee failures that are not caused by overtopping or overflowing are related to internal erosion, in one form or another (see Chapter 2 of this report). It can be estimated, that more than half of these internal erosion problems are linked to some form of transition in the levee. This has been demonstrated through the analysis of various flood event case studies.

Other problems possibly leading to failure can also be related to transitions, as shall be seen later in this document.

It can also be seen that it is invariably difficult to analyse the causes of a breach after the event, given the typical level of destruction of the area. Furthermore, most reports and analyses about past events tend to categorize the cause of breaches in a limited way, generally through a list of mechanisms, while a more modern and analytical approach to the analysis of levee breaches (International Levee Handbook, section 3.5.1) examines breaches as the consequences of a chain of mechanisms; these chains can be referred to as scenarios. As more than one mechanism is typically at work during a breach, it is possible that the same breach could arise from different "causes": one mechanism can hide another one. Hence, some problems caused by internal erosion or other transition related problems may have been attributed to other causes, and in particular to overflowing (see the River Agly case study - section 3.3.2 of this report).

Whilst we refer to problems arising at transitions, it should be recognised that the specific mechanisms (particularly the different types of internal erosion) can occur on one or other side of the transition, as well as specifically at the transition point.

Hence, the subject of transitions is of particular importance for levee systems. It is even more relevant in urban flood defence systems, as it is more common to find frequent variations in flood defence structures in urban areas and specific structures in or near levees in urban areas that are not directly linked to the levee or flood protection structure.

Structures associated with levees can also cause risk and asset management problems since the owner/manager of a structure can be different from the levee owner/manager. In this case the structure is called an ENCROACHMENT. This situation can cause problems for efficient inspection and maintenance, and hence can restrict effective assessment and management of the levee safety.

Transitions can be studied and described in terms of:

- the failure modes (scenarios and mechanisms) they can be related to,
- if possible, the limit state equations, fragility curves, performance curves or indicators linked to these failure modes,
- the geotechnical problems linked to these transition types or failure modes, in order to be able to propose mitigation and/or remedial measures,

- the possible means of detection of unknown transitions,
- the possible means of detection of a problem occurring,
- the characterization of the structure,
- the characterization of the transition.

Solutions for improving the management of transitions can be proposed in terms of:

- management of the encroachments: organisation (coordination) of the management of the levee AND the structure,
- inspections (pre, during or post flood),
- assessment methods (in some cases this may lead to a need for further research, as we might not have a complete knowledge of the processes involved),
- improvement works (decide between rebuild/remove/act on the soil or act on the structure, propose technical options).

3.1.2 Typical problems linked to transitions

Processes that are usually called "failure modes" for a levee are in fact scenarios. In these, different physical mechanisms, affecting one or more parts (components) of a levee, follow each other, causing degradation or destruction of one or more components, and so the failure of one or more of the elementary functions of the levee. Finally, the levee itself fails, by breaching or by letting uncontrolled water into the protected area. Failure modes are commonly named from the leading or originating mechanism; for example, overtopping, external erosion, sliding etc.

This process is shown schematically in Figure 3.1 below which illustrates failure scenarios as chains of events including mechanisms, deteriorations and degradation/failure of functions.

This shows that the first mechanisms of a failure scenario are initiated by external conditions or actions on the levee. A mechanism can then result in the deterioration or damage of one or more components. Subsequently, component deterioration or damage results in the degradation or failure of one or more functions, associated to the said component(s). Degradation or failure of a function then leads to an unwanted mechanism appearing or being aggravated.

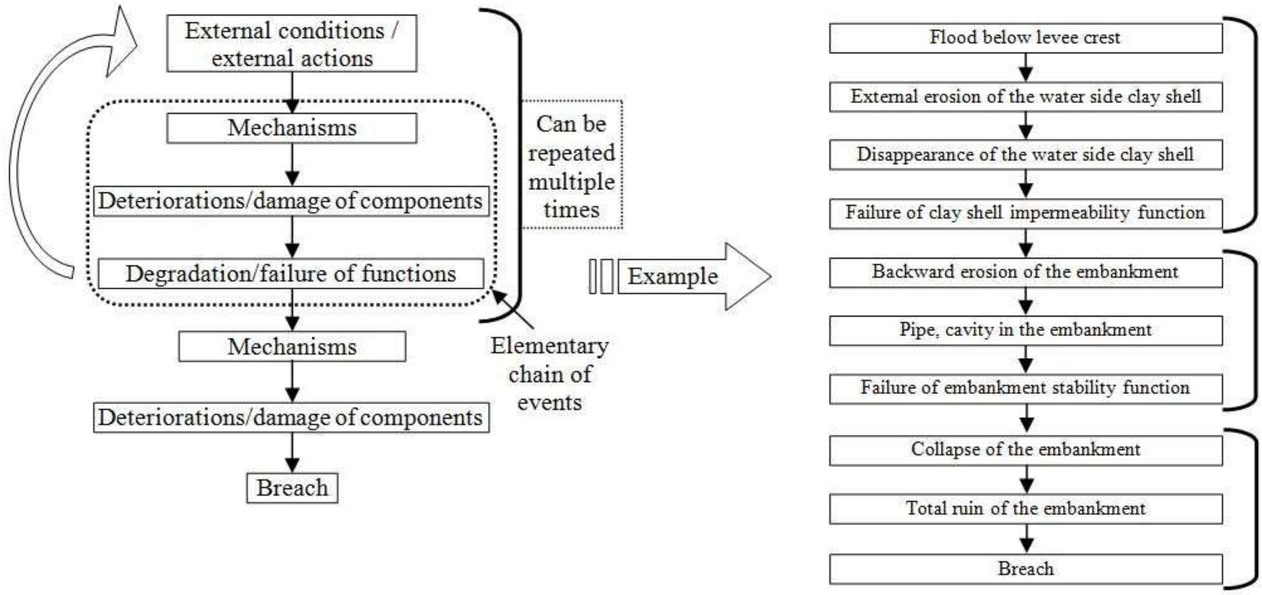


Figure 3.1 Breach scenarios (ILH, 2013)

The main family of mechanisms that may lead to failure seems to be erosion at a transition area (embankment / "hard" structure or embankment / embankment interface). Very often the mechanism will be internal erosion, but also in some cases it can be through external erosion.

In the case of internal erosion, this is because the contact zone is a preferred seepage area, where erosion can be initiated and develop. This contact between the different zones can be "rough" or "smooth" or even "loose", allowing more or less water flow and so more or less erosion. At this time, the physics of the phenomenon are not known well at the microscopic scale, or characterised well by means of numerical models. The physics of contact erosion probably differ in the case of a transition between a hard structure and an embankment or in the case of a transition between two embankments, for instance at the limit of two stretches of the same levee, with different geotechnical cross sections.

In the case of external erosion occurring at the surface of a transition area, it can be caused either by contact erosion occurring inside, or by an external cause, such as turbulence caused by a difference in roughness coefficient across a transition between two materials. In addition, a simple geometric irregularity can also lead to turbulence and external erosion.

Sometimes the function of the structure or the way it has been built can cause specific erosion problems:

- leakage from pipes (or into pipes),
- poor levee soil conditions around a good condition structure (e.g. sand around a pipe through a levee).

Both of these conditions can initiate or facilitate internal erosion. As pipes are the most common type of "hard" structures found in a levee (see Orleans analysis, in the Loire case study), it is particularly important to consider this specific case. It should be noted that these problems could be avoided or detected through close cooperation between the levee managing organisation, and the pipe managing organisation.

Another type of problem is related to settlement under or near a hard structure, causing a preferred area for flow, which will then result in erosion or increased internal pressures leading to uplift or sliding.

In some cases, the presence of a structure or a transition can also induce sliding (shear), because of additional forces not taken into account in the initial stability analysis.

The failure and collapse of an included structure may lead to either settlement in the levee, a potential overflow, or a loss of cohesion of the levee material in the structure area and hence internal erosion during a flood. Due to the presence of the structure, or because of its failure, mechanical failure (collapse, sliding …) may also happen.

Hence, the typical mechanisms that might occur, alone or as part of a process (or scenario) at structure transitions are (in no particular order of importance):

- external erosion,
- instabilities (sliding, collapsing, settlement)
- contact erosion between soil and hard structure,
- contact erosion between two different soils, or constructed soil interfaces,
- backwards erosion along a decompressed contact area (or along a pipe),
- concentrated leak erosion (generally the final mechanism in an internal erosion scenario, and not the initiating one),

From a geometrical point of view, a transition between a levee and a structure can be seen externally (a line) or internally (as a surface). An included structure that is visible from the outside has both an internal contact surface, and an outside line of limit. An included structure can also be completely hidden, buried within the levee or its foundation.

Erosion may occur both, together or exclusively:

- Internally (for example, internal erosion at the contact surface),
- Externally (for example, external erosion facilitated and starting at the limit).

Erosion may also develop in an embankment behind a protection structure if there are insufficient filtration measures:

- in cracked masonry joints,
- between concrete slabs,

- ➢ underneath asphalt revetments,
- ➢ underneath rip-rap.

In this case the protection structure HIDES erosion instead of avoiding it (Figure 3.2). In a second phase, when the eroded space underneath the protection structure becomes large enough, then the protection structure suddenly fails (through block failure or via whatever units the structure is constructed from).

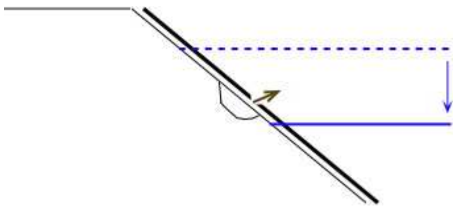


Figure 3.2 Erosion of the embankment hidden by the protection structure

For the situation with a buried structure spanning from the water side to the landward side of the levee (either within the levee or within the foundation), the transition is AROUND and ALONG the structure.

In the case where the transition is between two very different stretches (segments) of a dike (upstream/downstream), the transition is LATERAL to the structure (or levee).

In the case of a transition between layers (surface protection, road, spillway etc.): the transition is BEHIND (or under) the structure.

3.1.3 Further structure of this chapter

This chapter provides an assessment of the different types of structure transition, the processes that might occur at transitions and recommendations as to how such transitions should be managed, assessed, designed and repaired to limit flood risk.

Section 3.2 provides an overview of hydraulic loading and erosion mechanisms that can occur and which will affect failure modes at transitions. Section 3.3 then provides some case study examples where transitions have failed in different situations. Lessons can be learnt from these reviews.

Section 3.4 presents a typology of the transition structures. A systematic approach to the determination of the typology of transitions is provided, taking into account both type of structure and geometry. The more important transition types are then described within structured templates, in Section 3.5, including details in connection with their management, assessment, design or repair.

Section 3.6 reviews how structure transitions are now, and could and should be incorporated into the performance assessment of flood defences structures, and in particular for flood system risk analysis.

Section 3.7 provides a summary of key conclusions from the review and identifies issues that require further study in order to improve knowledge about the transitions and hence help manage flood risk attributable to transitions.

3.2 Hydraulic loading and erosion mechanisms

Whilst a transition structure poses a hazard, the potential flood risk is not realised without some form of hydraulic loading and one or more failure mechanisms. As such, it is important to understand how transitions respond under different hydraulic load conditions and how different erosion mechanisms may develop.

3.2.1 Hydraulic loading: Structure position versus water flow

The following hydraulic parameters are important for determining the possibility and intensity of erosion:

- velocity and direction of the water flow (relative to the transition direction) and including possible sediment transport,
- water level and its dynamic change, including wave characteristics,
- the resulting hydraulic head along the transition zone,

The **hydraulic gradient** and or hydraulic head (Figure 3.3) has to be considered in the case of seepage or flow through the embankment. It is strongly linked to the risk of various forms of internal erosion.

The **uplift-pressure** (or interstitial water pressure) is involved in all mechanisms regarding overall stability. It should be calculated by a hydraulic model in transient or steady conditions, depending upon the duration of the hydraulic head.

The **water velocity** in the river, specifically along the levee face or toe, should be considered when surface erosion is a possible failure mechanism. Associated to water velocity and wave characteristics, local **turbulence** is often the initiating cause of erosion at an interface between two different revetments on the water side of the embankment.

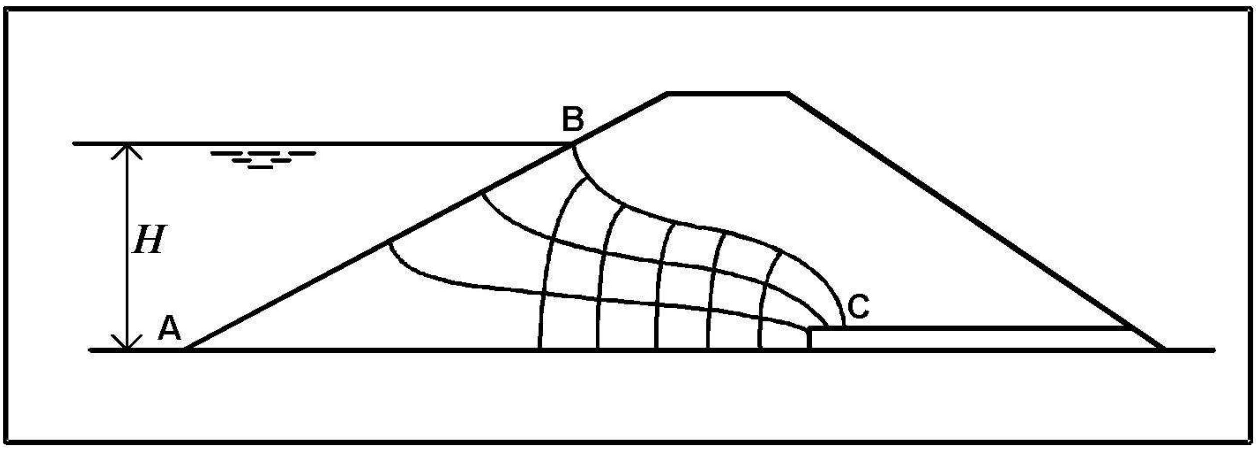


Figure 3.3 Illustration of internal flow and hydraulic gradient (H) through a levee

3.2.2 Erosion mechanisms

Erosion in relation to transition structures may be divided broadly into surface erosion and internal erosion.

Typical surface erosion processes related to transition structures can include:

- Surface erosion and gullying as a result of surface water runoff being focussed at the interface between a structure and a levee,
- Surface erosion at the transition due to local flow turbulence caused by structure geometry or roughness variations.

There are many forms and permutations of the above processes in conjunction with different types of transition structure and hydraulic load.

Internal erosion processes are, by their nature, hidden until the erosion reaches such an extent that partial or complete collapse of the levee occurs. Indicators of internal erosion, such as seepage flow, may be visible before collapse, but this is not always the case.

Research under FloodProBE (see Comprehensive report on internal erosion - FloodProBE ref WP3-01-12-05) has helped to advance knowledge and understanding in this area. This starts with a clear definition of internal erosion and the various forms that can develop, as described in Chapter 2.

Internal erosion is related to all processes which involve the detachment of soil particles and transport by seepage flow within the dam or levee, or its foundation. A consensual view in the water community is that four different basic processes can be identified within the general definition of internal erosion. These mechanisms are: backward erosion; concentrated leak erosion; suffusion; and contact erosion. The general definitions of these processes are:

- *Backward erosion*: Detachment of soil particles when the seepage exits to an unfiltered surface and leading to retrogressively growing pipes and sand boils;
- *Concentrated leak erosion*: Detachment of soil particles through a pre-existing path in the embankment or foundation;
- *Suffusion*: Selective erosion of the fine particles from the matrix of coarse particles;
- *Contact erosion*: Selective erosion of the fine particles from the contact with a coarser layer.

A more detailed explanation of these processes, including diagrams, was given in Chapter 2 of this report.

3.3 Case study examples

The significance of transitions as points of weakness within flood defence systems can be clearly demonstrated by reviewing a number of historic flood events, and looking more closely at the locations and mechanisms of failure where breaches or defence failures occurred.

Case study examples have been drawn from the following flood events:

France:

- Rhône River, 1993-94
- Xynthia Storm, 2010
- Loire River, 19th Century
- Agly River, 1999

USA:

- Hurricane Katrina (New Orleans)

Thailand:

- Lower Chao Phraya River Basin (Inc Bangkok), 2011

In addition, a review of typical flood defence transitions has been undertaken using aerial photos:

UK:

- Humber Estuary (2011)

Details of the French and Thai case studies are provided in the following sections; the other case studies may be found in the more detailed FloodProBE report on transitions (FloodProBE Report WP03-01-12-10).

3.3.1 Case studies from France

3.3.1.1 Rhône (France), 1993-94

In France, during the two winter floods of 1993-94 the southern Rhone River flooded the Camargue delta. However, no overtopping or overflowing occurred; internal erosion has undoubtedly been the cause of most of (even all of) the breaches and of the major damages to the levees. An initial analysis was produced at the time of the events relying on local witnesses [Bonnefoy, R. and Royet, P. (1994)] which attributed most of these failures (13 of 16 in total) to animal burrows and some of them (3 of the total of 16) to pipes, with a possible involvement of tree roots. Since this original work, it has been suggested that the responsibility of animals could have been over stated and may have hidden other more complex mechanisms, all related in some way to forms of internal erosion.

A lot of pipes existed in these old levees, most of them for individual uses such as the irrigation and drainage of agricultural land. A lot of them were poorly maintained or, in some cases even forgotten. The levees were typically built, raised and reinforced over the years and consequently included many transition zones between different types of soils.

Lesson learned from this case study: in old levees there can be many crossing pipes, some of them even forgotten and not maintained, which can be the cause of breaches during floods.

3.3.1.2 Xynthia (France), 2010

A lot of damage occurred to French coastal levees during the Xynthia storm, mainly in the two departments of Charente Maritime and Vendée. A lot of this damage was caused by overtopping or possibly even by overflowing, due to the unusually high level of loading on the levees; the direction of the waves was also unusual so that some locations which had overtopped during other events were not during this one and vice versa.

Nonetheless, many damages were related to transitions of the "levee" with its protection revetment. The mechanisms and scenarios involved were:

- undermining of the lower part of the sea-side revetment, and subsequently eroding internal parts of the levee (sand, earth);
- varying pressure under the revetment (caused by defects on joints and high pressures caused by the wave impact) either on the sea side or on the crest, leading to a revetment failure, then erosion of the levee body;
- failure of the protection revetment itself then formation of cavities under the revetment, and finally collapse of the revetment.

These mechanisms involving transition between the levee, its foundation and the protection revetment had been previously identified and described (BRLi, 2006) and regularly happen during any storm loading these sea levees. In the same report, another mechanism involving the revetment is described but involves a first step of erosion on the land side by overflowing/overtopping before failure of the revetment.

Lessons learned from this case study: in coastal levees, the protection revetment is essential for the levee safety, and must be properly maintained. The transitions between the revetment and the rest of the levee and its foundation must be designed and constructed in such a way as to avoid erosion of both the levee and its foundation.

3.3.1.3 Loire river (France), XIXth century major floods and recent history

(Written with the contribution of Jean Maurin, head of the Studies and Works on the Loire Department in DREAL Centre)

In the city of Orleans, the levee system (a pilot site of the FloodProBE project), is 52 kms long and represents about 10% of the length of levees managed on the Loire river by the French State. A recent survey identified 50 linear crossing structures (pipes or cables, including cables in sheaths), 2 non linear crossing structures (a pumping station and a gate) and 35 non linear non crossing included structures (houses, utility buildings). This gives an indication of the extent of the problem of transitions for the levee management organization; an average of 1.7 transition structures per km in this case.

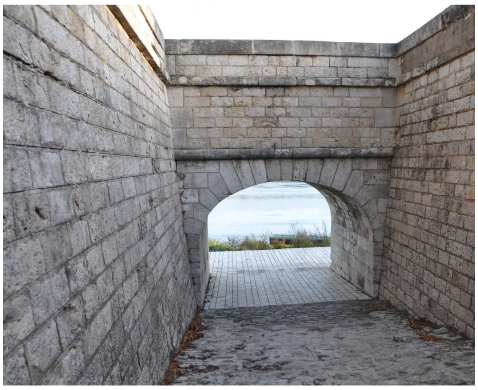


Figure 3.4 Jargeau's gate

However, in recent history there has only been one levee failure attributable to a transition. This was in Montlouis in 2003 during a flood with a 10 years return period.

There were historic floods of the XIXth century when 3 floods occurred in 1846, 1856 and 1866, all having a water level with a return period of around 100 years. It is alleged that crossing structures caused no breach or damage, as at this time none was present in the levees. A majority (66%) of the problems (breaches or major damages) were caused by overflowing, but a huge number of the remaining problems (15.4% of the total, so 45% of the remaining) were attributed to the presence of "banquettes" on the top of the levee (see Figure 3.5 and

Figure 3.6). These "banquettes" are small (50cm-1m) embankment structures either at the limit of the crest and the river side slope or at the limit of the crest and the land side slope or at both of these limits. The banquettes can be either made of earth or rocks, with earthen banquettes being sometimes covered with armour stone.

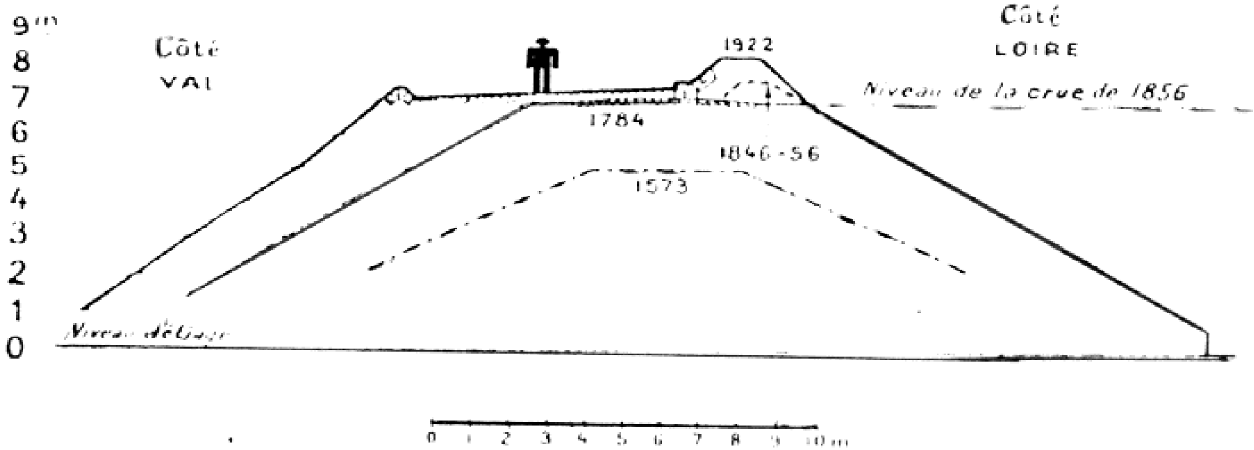


Figure 3.5 A typical cross section of the historic reinforcement of Loire levees

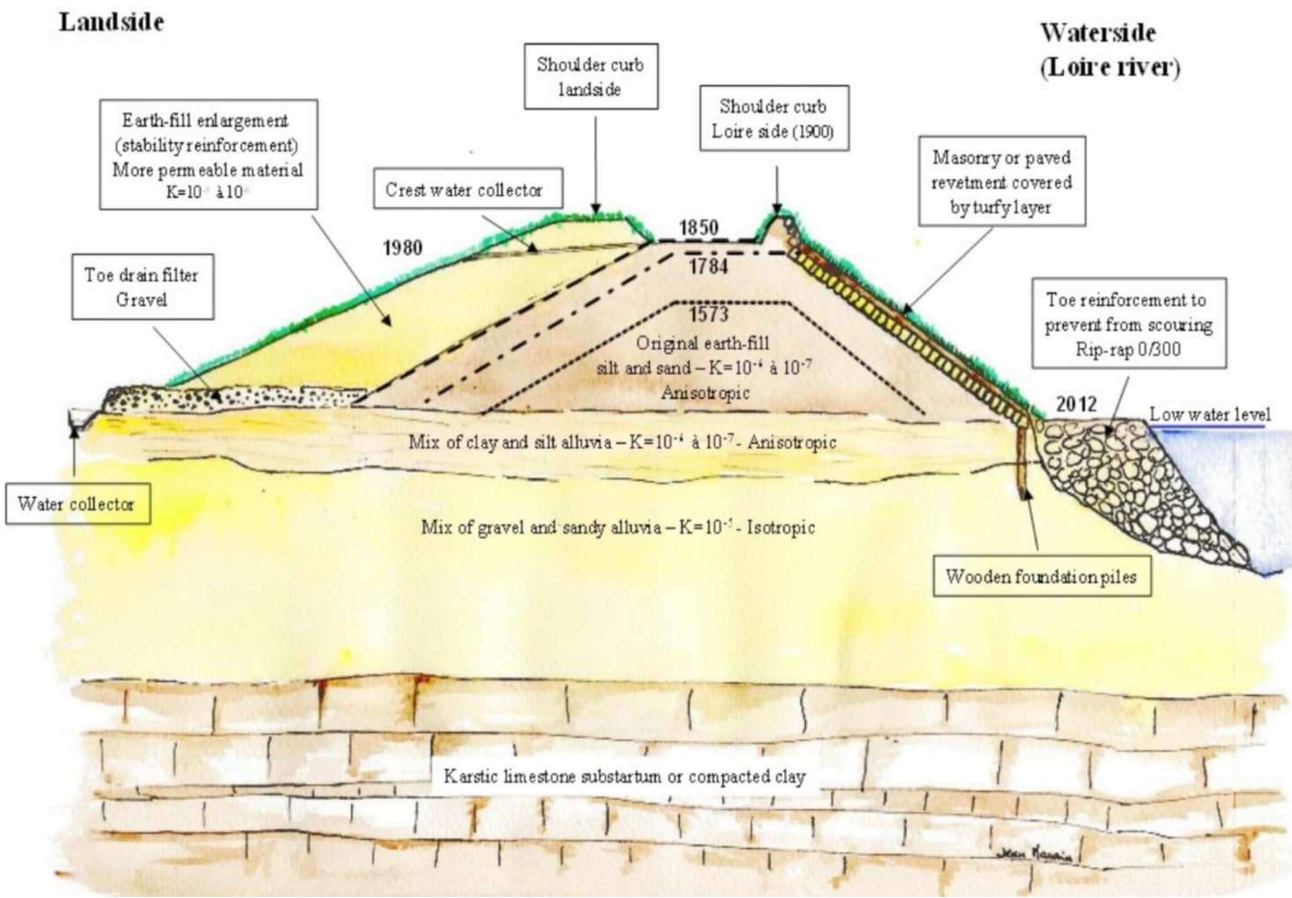


Figure 3.6 A typical Loire levee cross section in the Orléans area

The problems in relation to the banquettes were caused by internal erosion at the limit of the levee and the banquette (i.e. contact erosion between two soil layers), erosion inside the banquette (i.e. suffusion), or by drainage structures crossing the banquettes (i.e. contact erosion between a soil layer and a hard structure). Banquettes also had a detrimental effect in the situation of overflowing, as they allowed an increase in hydraulic head without offering the same erosion resistance as a full width levee section.

The Loire River offers one of the only documented cases of a failure caused by an included structure. This was during the XIXth century and caused by a house in La Chapelle sur Loire.

Lessons learned from this case study: there can be many pipe crossings in old levees that are weak points, especially as they have been installed through existing levees. Other types of transition can also exist and cause problems, particularly crest level raising structures and embedded buildings.

3.3.2 Agly river (France), November 1999

This case study was edited from Appendix 4 of Surveillance, maintenance and diagnosis of flood protection dikes. A practical handbook for owners and operators (Mériaux, P. and Royet, P., (2007).

A flood, with a peak discharge of about 2000m^3/s, occurred along the Agly River, in the South of France, in November 1999. The river has levees on both sides of the main river channel along a length of 13.2kms. Overflowing occurred along many kilometres of levees, and caused major damage to 550m of levee sections. One breach (only…) occurred, just in front of the sewage treatment plant; at the time this was attributed to overflowing. However, given the presence of the plant discharge pipe it is highly likely that the location of the breach was facilitated by this pipe and that more than one single mechanism caused the breach.

Lessons learned from this case study: the presence of included structures can facilitate breaches, including via mechanisms not necessarily based on the transition itself, such as through overflowing.

3.3.3 Case studies from Thailand

3.3.3.1 Flooding in the Lower Chao Phraya River Basin (Bangkok)

The following case study has been edited directly from a report organised by ENW, entitled "Post-flood field investigation in the Lower Chao Phraya River Basin (23-27th January 2012). Findings of the Thai – Dutch reconnaissance team. Some additional content sourced via the web has been added to enhance the case study.

Large parts of central Thailand have experienced severe flooding during the second half of the year 2011. The economic and societal damage is enormous: more than 800 fatalities and more than US $ 45 billion (Sources: Wikipedia, Worldbank), making it one of the most costly disasters at a global scale ever.

The flooding in Thailand has been characterized by a number of failures of dykes and structures around the large industrial estate areas, the Chao Phraya river dykes and adjacent irrigation canal dykes. An investigation was organized to investigate the dyke failures and performance of various systems. There are three primary areas of interest:

1. Dykes around industrial estates near Bangkok;
2. The system in the Lower Chao Phraya river Basin (north of Bangkok);
3. King's dyke, i.e. the ring dyke for the protection of Bangkok.

In addition, some historical sites at Ayutthaya (north of Bangkok) were visited

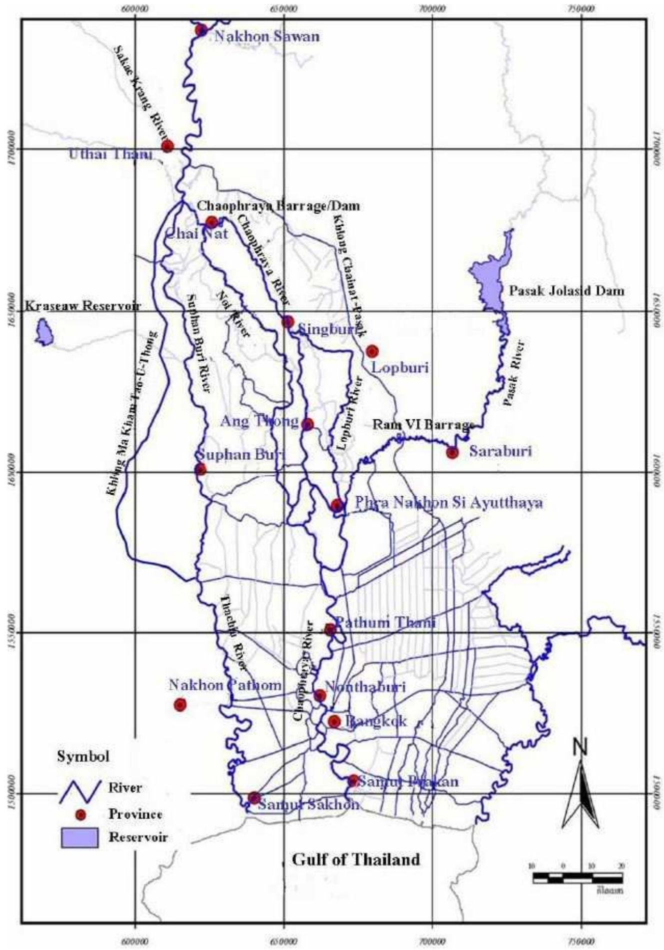


Figure 3.7 River network of the Lower Chao Phraya River basin (UNESCAP)

The Lower Chao Phraya river basin is a complex network of rivers, canals and streams (Figure 3.7). The maximum discharge capacity in the lower part of the basin ranges from 2750 m3/s to 3500 m3/s. During the 2011 floods Thailand was struck by one of the most devastating natural disasters in Thailand's modern history. Sixty five out of seventy seven Thai provinces were subject to flooding along the Mekong and the Chao Phraya river basins. The flooding persisted in some areas for months and resulted in more than 800 deaths and 13.6 million people being directly affected (Figure 3.8).

Figure 3.8 (i) Flood evacuation; (ii) Historical sites flooded; (iii) Temporary flood defences; (iv) Widespread flooding

Failure Mechanisms

An extract from the report introductory summary states the following:

"...Several large breaches occurred in the canal dykes in the Lower Chao Phraya river basin mainly due to overflow and consequent erosion of the dyke body that consisted of clay. Most breaches occurred at weak spots in the system (lower parts of the dyke, connections with structures and at obstructions). Three hydraulic structures were visited that all failed at the connection between the structure and the earthen dyke. This illustrates the importance of a robust design of these transitions..."

Some examples of the breaches arising from transition failures include:

Layers within the Dyke

There was one occasion of a breach in a dyke near Bang Chrom Sri floodgate, where the local RID officers that breaching may have been caused by seepage through the dyke. At the time of the visit some flow of water and sand through the dyke was observed. One of the reasons could be that a road had been constructed on the old dyke. Sand layers that were used as part of the road foundation thereby became part of the dyke body, thus creating a potential weak spot for seepage.

Hydraulic Structures in the Lower Chao Phraya Basin

All of the structures that were visited failed at the connection of the structure and the earthen dyke. This indicated how vulnerable these connections are. It was noted that similar observations were made in New Orleans after hurricane Katrina. Since the structures were mainly designed for irrigation purposes with small water heads, provisions to prevent seepage / piping during floods seem to be limited, especially on the connection with the adjoining dykes or dam. Therefore scouring and breaching could occur besides the structures. This is illustrated in Figure 3.9 (Klong Ta Nueng floodgate) and Figure 3.10 (Pra Ngam floodgate), which show a photo of the situation and a sketch of the failure.

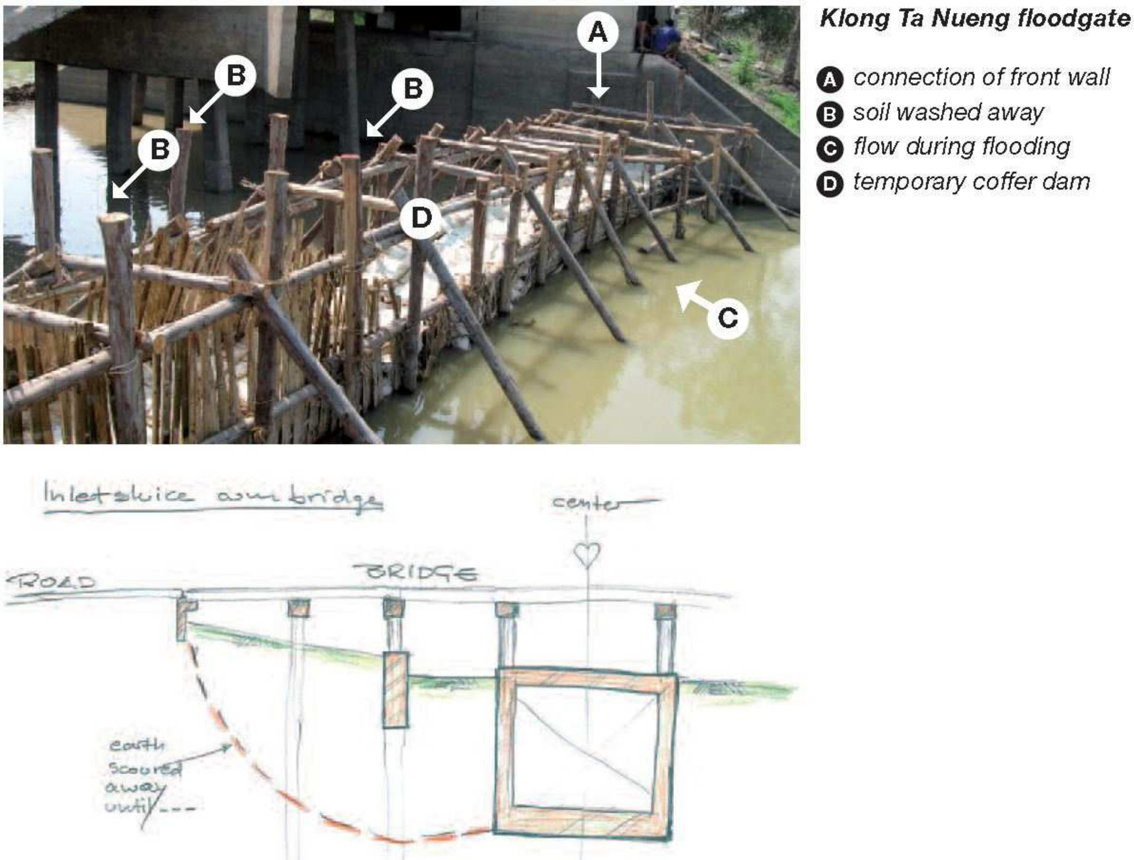


Figure 3.9 Failure of the Klong Ta Nueng floodgate

Pra Ngam floodgate

- **A** *sand fill at the original breach*
- **B** *regular flow*
- **C** *flow during flooding*

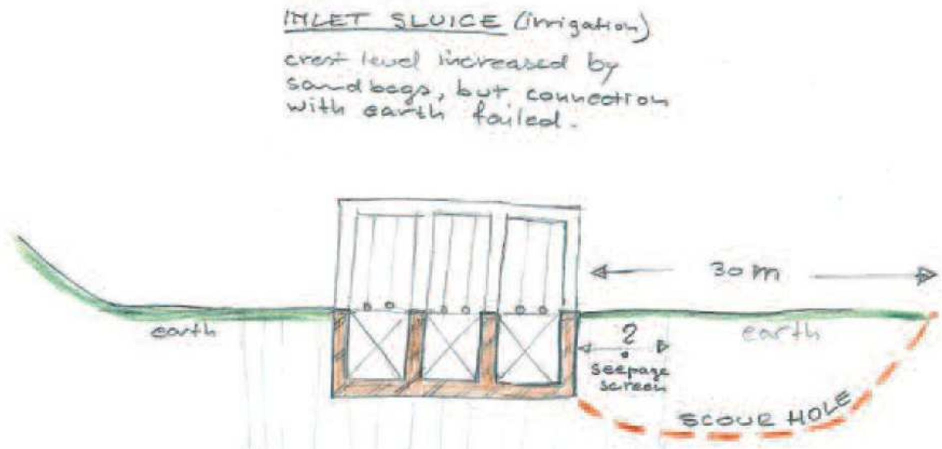


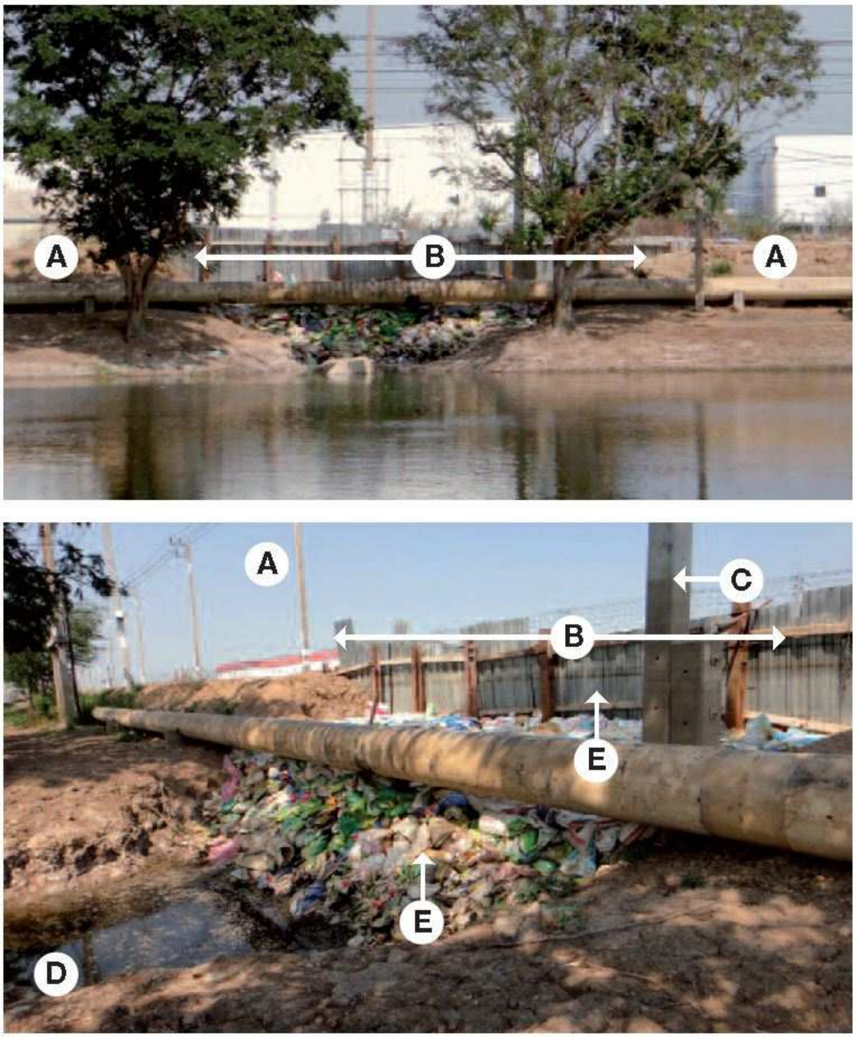
Figure 3.10 Failure of the Pra Ngam floodgate

<u>Rojana Industrial Estate</u>

The Rojana industrial estate which was developed 23 years ago (in 1989). It is located north of Bangkok near the city of Ayutthaya. It is protected by about 70km of dykes and the estate in fact consists of multiple “dyke rings”. The area is protected by dykes with a height of about 4.5m.

Two breaches were visited. The first breach was caused by overtopping and it had a width of about 20m. The canal levee had been raised as flood fighting measure by 0.5m with a backhoe. Still, the final flood levels exceeded the crest level by far (0.8 ~ 1.0m). The pipe along the slope and the columns on the protected area side may be speculated to have contributed to the erosion failure. The failure had occurred after no more than 6 hours after the levee started being overtopped. (See Figure 3.11).

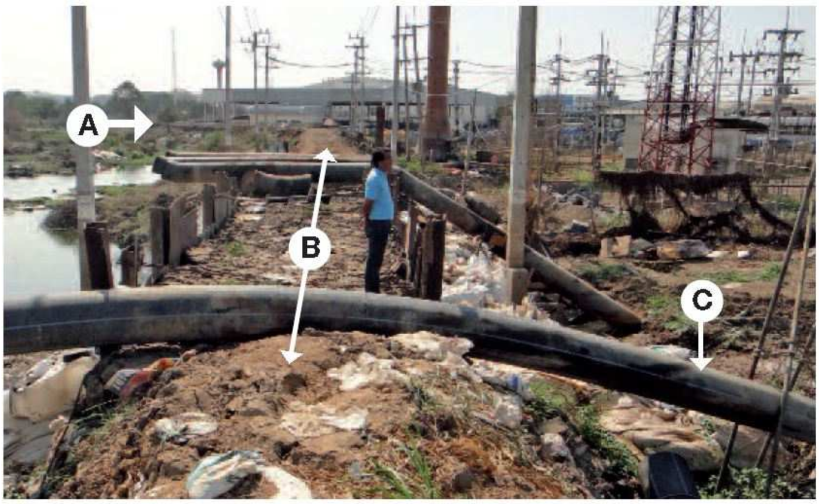
The second breach that was visited occurred at a pipe in the dyke, which was not protected by seepage screens. At this location the failure started with seepage along a pipe through the levee (no seepage screen) and failure progression accelerated when it was also overtopped. According to eye witnesses it took about 2 hours to form the 30 wide breach. The levee was rather new and built after floods in 1999. (See Figure 3.12).



A *earthen dyke*
B *breach and emergency repair*

A *canal side*
B *breach width ~20m*
C *water mark*
D *protected area*
E *temporary repairs*

Figure 3.11 Breach from overtopping; the pipe along the crest may have contributed to the failure



A *overflow direction*
B *breach width ~30m*
C *relocated pipe (through levee without seepage screen before)*

Figure 3.12 Breach through seepage initiated along a pipe passing through the dyke

Lessons Learnt from this case study:

The report concludes with a series of key points. Amongst the lessons learnt was the following statement regarding transitions:

Transitions between hydraulic structures and (earthen) dykes again proved to be weak spots, but these transitions receive limited attention in current methods for design and safety assessment in the Netherlands and other countries. Design guidance is needed to ensure the safety of these connections. As part of the safety assessment or periodic inspections a number of principles, checks and simple design rules should be developed to be able to assess the safety of these connections. As a general rule, transitions should be designed to be more reliable than the adjacent “standard” elements (e.g. dike reaches).

3.4 Structures and transitions typology

This section provides an overview of the different types of transition structures culminating in a flow chart to assist in identifying different types of transition and problems and solutions associated with these transitions (Figure 3.13).

3.4.1 Typology

The following provides a list of different structures and transition types.

Buried structures:

- pipes (metal, plastic, concrete, masonry, …),
- cables.

Part buried, but visible structures:

- culverts,
- gates,
- houses,
- stairs,
- manholes (probably associated with a buried structure or network).

External structures:

- surface protection,
- roads.

Change in type of levee

- contact between the levee and a flood wall (lateral / vertical),
- contact between different levee stretches,
- contact between the levee and natural high ground.

3.4.2 Main types of transition and related potential problems

The type of transition influences the nature of the problems it may cause.

The main types of transitions are described according to a flowchart presented in Figure 3.13.

A transition can be either:

- between two levee segments (including between a levee segment and natural ground and between a levee segment and a flood wall). In these cases the difference between the different segments can be in terms of outside geometry, of protection revetment (slope or crest, road, etc.), and in terms of internal cross section.

- between a levee segment and its own revetment,
- between a levee and a flood wall above the levee,
- between a levee and a linear structure,
- between a levee and a non-linear structure.

The following flowchart (Figure 3.13) provides a tool to identify the type of transition, and the potential problems that could be encountered with each type of transition, as well as the principles for solutions.

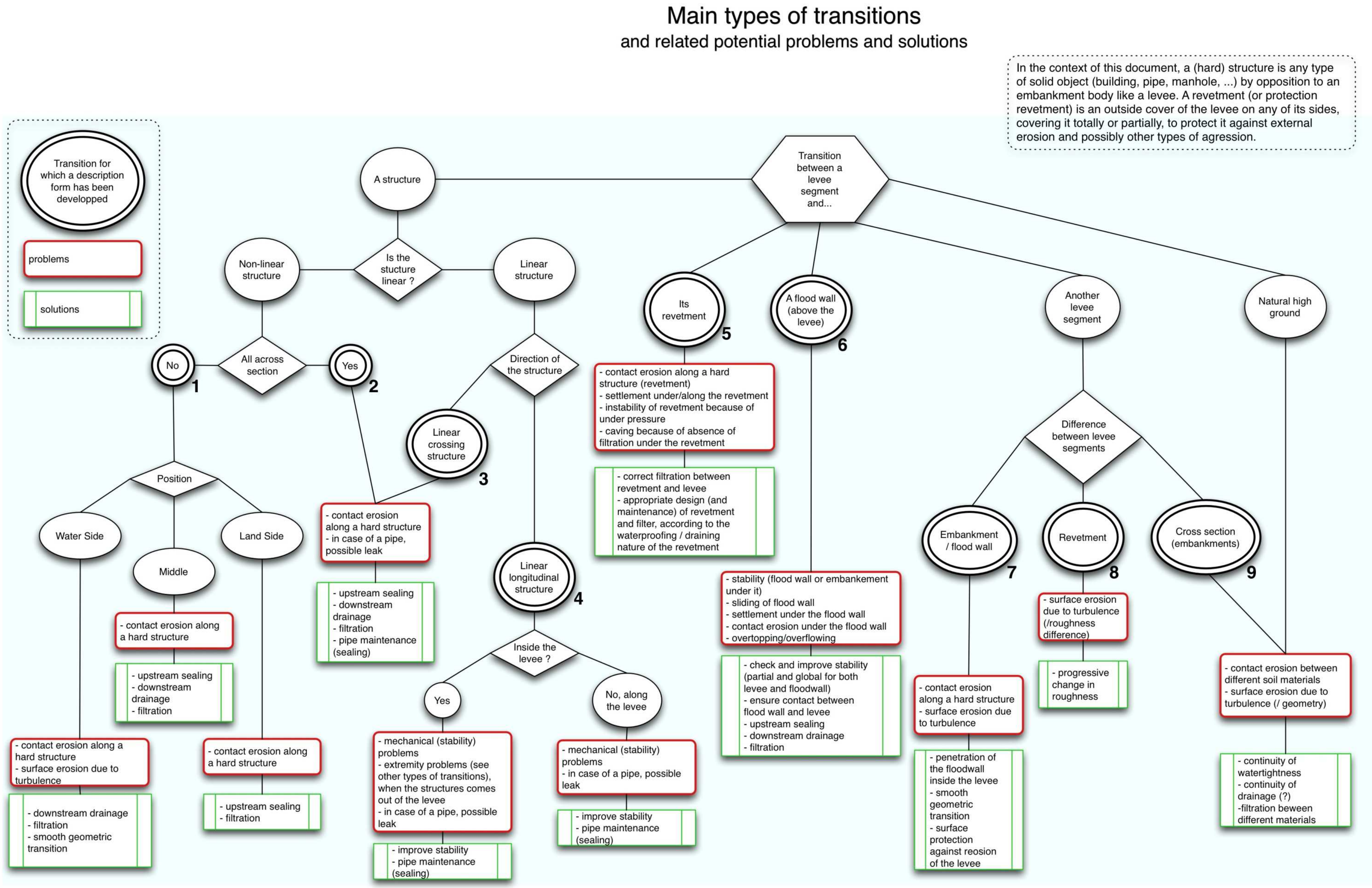


Figure 3.13 Flow chart to identify types of transition and potential associated problems

3.5 Description forms for structure transitions

This chapter provides a detailed description of the different types of transitions found with levees. A standard form is used for each description. Each form includes a description of:

- the transition type,
- the potential failure modes and mechanisms linked to this type of transition,
- the way(s) to detect this type of transition,
- the way(s) to detect the typical problems associated with the transition,
- best practice to avoid problems in terms of design, maintenance and/or management,
- issues and questions associated with the transition
- case studies and examples

Some specific transition cases may relate to more than one form.

Some generic actions can be considered for all types of transitions, mainly in terms of management, in order to facilitate their assessments and maintenance.

A database of all transitions should be filled and maintained for each transition type, with a specific data collecting form for each transition type. Each specific transition should be referenced, described in this database, as well as inspected both regularly and after and if possible during every loading event (flood or storm).

Specific initialisation of the database filling should be undertaken by the manager, including searching through records and archives, questionnaires to networks (water, sewage, power, communication, etc.) managers and using of specific detection methods.

Structures maintained by other managing organisations (encroachments) should be submitted to authorization and approval of the technical details by the levee-managing organisation in case of building or modification.

An example of one template is given here; more detailed information and other templates are provided in the more detailed FloodProBE report on transitions (FloodProBE Report WP03-01-12-10).

3.5.1 Transition 3: Linear crossing structure

Description:

Linear crossing structures can be found: across the levee, under the levee (through the foundation) or partially in the levee and its foundation. Sometimes they cross the levee only partially (for instance from the crest to the bottom of one of the slopes).

This type of structure poses specific management issues, as they are often managed by organizations that are different from the levee management organization.

Illustration:

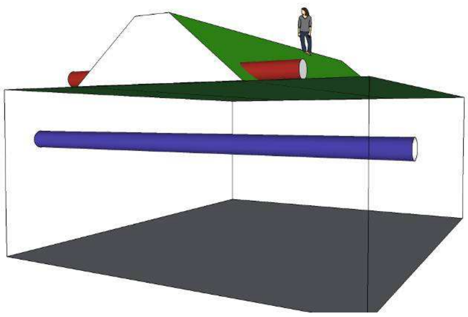


Figure 3.14 Illustration of linear crossing structures, across the levee and under its foundation

Photos :

Figure 3.15 A pipe crossing through and across the water side of a levee (Rhone River)

Figure 3.16 A gate control at the end of a culvert passing through a levee (River Loire)

Main failure modes associated with this type of transition:

- seepage problems at the interface between levee and structure arising from differential settlement,
- internal erosion (with a range of different sub-types) initiated along the structure, caused by seepage
- internal erosion along part of the structure caused by water going into or from a pipe,

Detection of the transition:

- records, archives (not necessary those of the levee manager)
- visual inspections
- aerial photos
- geophysics (refer to FloodProBE D3.2 - ref WP3-01-12-20)

Detection of problems:

- visual inspections (normal conditions, but mainly during or post flood)
- camera inspection from inside any pipes
- topographic monitoring

Problem indicators:

- leakage or evidence of internal erosion (moisture, fine soil particles)
- irregular geometry, signs of subsidence

Best practice to avoid problems at this type of transition:

Design:

- Design measures to prevent internal erosion along the structure levee interface
    - after placing pipe/cable in the trench, fill the trench with vibrated concrete
- place pipe/cables at the upper part of the levee (by means of a siphon, if relevant – Figure 3.20)

Repair / retrofit:

- Ideally, removal, but if not an option then (i) identification of the failure mode induced by the linear structure and subsequently measures to prevent that failure mode (which could relate to structural stability, internal erosion, surface erosion etc)
- Complete excavation and installation of internal erosion protection measures (filters, lengthening of the transition internal line (i.e. reduction of the hydraulic gradient))

- No specific drainage on the land side of the structure if the rest of the levee has no specific drainage, in order not to create a particular location with a higher hydraulic gradient.

Management:

- Agreement between the levee manager and the network manager before starting any work on the crossing structure

Case study example(s)

The following picture was taken during the 1993-94 floods of the Rhône River in Camargue.

Figure 3.17 Levee breach initiated along a pipe crossing (Rhône River). Photo SNRS.

The following set of pictures shows a breach which was probably caused by erosion around some telecommunication cables: in the picture on the right, it can be seen that the cables were put in a trench filled with coarse and non cohesive material which would act as a preferential seepage path. The trench can also be seen on the upper left picture, on the left of the set of cables (just under the house).

Figure set 3.18 Breach initiated along crossing cables (Mosson River).

The following set of pictures shows a pipe which is crossing under the base of a levee with no specific treatment of the contact surface visible, and hence the strong possibility of contact erosion

Figure set 3.19 A pipe crossing at the base of a levee (Dresden)

The next picture presents a solution (adopted as general design by SYMADREM, Rhone River levee management organisation, for retrofitting older pressure pipes), as it lengthens the contact area, and removes most of it at the point of highest gradient.

Figure 3.20 A crossing pipe with " siphon shape" (Rhone River), to minimize the risk of internal erosion along the pipe. Picture of the land side of the levee.

3.6 Including structure transitions within flood risk assessments

Assessment of the performance of a system of flood defences, rather than of an individual flood defence structure, is often required as part of a flood risk assessment. The way in which system performance is assessed varies between countries and according to specific assessment; national policy and cultural approaches have a significant impact on the approaches taken. However, analysis of system performance typically requires the assessment of each component of the system in a methodical manner. Historically, this has evolved by summing the analysis of each of the individual structures or structure lengths. In many countries and organisations, little attention has been paid to the formal inclusion and assessment of transition points.

Within England & Wales, and The Netherlands, probabilistic methods are used to analyse system flood risk. Fundamentally, the approach considers how the flood defence structure performs over a range of load conditions and what might happen when failure occurs. The system adopted in England & Wales links potential inundation and impacts arising to the performance of each specific defence structure or length for each load condition considered (Sayers et al., 2004). Hence by summing all of the performance conditions considered, overall flood risk may be calculated, and contributions to this flood risk may also be attributed to specific flood defence structures and lengths. This provides valuable information for prioritisation of asset management activities helping to ensure the most effective use of invariably limited resources.

However, whilst such an approach offers an effective framework for risk analysis and management, the accuracy of the results depends upon the degree to which the overall system response has been simulated. At present, the contribution of transitions to overall system risk is not considered. Analysis of historic events such as at New Orleans (US Army Corps Engineers (USACE), 2007) and in the south of France, for example, on the River Rhone (Bonnefoy and Royet, 1994) suggest that some types of transition provide a focal point for failure mechanisms and hence pose a greater risk than might otherwise have been recognised. Inclusion of transitions within system risk analyses should therefore be undertaken in order to improve the accuracy of risk prediction.

Since considerable work has been undertaken over the past decade to develop system risk models it is important to identify a way in which transitions may be included within the established calculation framework. Two fundamental approaches have recently been identified during research being carried out by HR Wallingford as part of a programme to improve the representation of defence fragility for the UK Environment Agency:

1. Direct identification and analysis of transitions as specific 'units' within the system risk model
2. Adjustment of the performance assessment of the defence lengths associated with the structure transition

Each of these two approaches has implications for the analysis work required. Choosing Option 1 - direct identification and analysis of transition structures within the overall analysis framework - requires adaptation of the framework for analysis, along with the development of limit state equations representing the failure modes envisaged for each transition type. Descriptions of the transitions, failure modes and limit state analysis may be summarised through extension of the earlier failure modes work under the EU FLOODsite project (www.floodsite.net) (Allsop et al., 2007). At the time of writing it was not clear whether such analyses would be feasible for the range of transitions and failure modes likely to be typical of flood defence structures.

The second approach offers an easier and quicker solution for analysis, although maybe not as rigorous a solution in the longer term. The approach is to adjust the performance assessment of the defence length(s) associated with the transition to reflect the increased risk of failure. This may be undertaken through integrating analysis of the transition failure process into creation of the fragility curve for the defence length, or by adjusting the 'base' defence length fragility curve using expert judgement. The latter offers the simplest means of including the effects of transitions within the existing analysis framework.

In the longer term is might be foreseen that transitions within and between defence lengths are identified and analysed uniquely (ie. Not recorded and represented as part of an adjacent defence structure). This would be more consistent with the steps required to record, inspect and maintain transitions as part of a range of flood defence structures. Under this approach, individual transition performance analysis (i.e. Option 1) would be a better way to analyse the risks posed.

3.7 Conclusion and needs for further research

3.7.1 Working with transitions – problems and solutions

Whilst flood defence asset managers, who routinely inspect and manage defences, are typically aware of the practical risks posed by transitions between structures, these risks are not yet routinely included within system flood risk analyses. A series of flood events such as in New Orleans (USA), Bangkok (Thailand) or on The Rhône (France) have highlighted that such transitions can create weak points within flood defence systems. Hence transitions within flood defence structures need to be routinely identified and managed as part of the overall flood risk management programme.

Research under the FloodProBE project has provided a framework for the definition of transition types. Consideration of what constitutes a transition shows that there are far more conditions where transitions occur than might initially be considered. Structures buried within, built on top of and through levees all create transitions, as do structures that partially encroach into levees, and sometimes into the crest, from either the land or water sides. The FloodProBE work provides definitions and a flow chart for classifying these transitions.

Along with the classification of transitions, the research also provides a series of transition descriptions - one for each type of transition. A standard format has been applied for each description within which the transition is described, along with diagrams and photos. Typical problems associated with the transition type are identified, along with indicators of those problems that might be seen during a visual inspection of the levee. Subsequently, steps towards potential solutions for both the design and post construction stages are provided as guidance for the asset manager. At this stage, the solutions presented are generally generic rather than detailed. It should be recognised, though, that the best solution remains to avoid creating transitions wherever possible.

Rec.1: Transitions invariably create a risk point within a flood defence structure, hence wherever possible the creation of transitions should be avoided. Where avoidance is not possible, then the failure mechanisms involved and hence the risks associated with the transition should be assessed, and the design for building or repair should be tailored accordingly.

3.7.2 Taking transitions into account

3.7.2.1 For asset management

The goal in reviewing and developing the information on transitions was to provide guidance on how to identify and manage the risks posed. From a practical perspective, it also became clear that identifying where transitions existed was also a problem. For the situations where transitions arise from interfaces between structures buried within or even under the levee, records do not always exist and a visual inspection does not always show any signs of the transition. In this situation a review of historic records and asset manager’s field experience is probably the only initial method that asset managers could employ in order to develop a long list of transition structures for assessment.

Rec.2: In order to manage transitions it is necessary to identify them and their associated problems. Hence transitions should be recorded as specific items within an asset management database. In this way they will be formally recorded, inspected and managed.

Rec.3: If transitions are formally recorded, inspected and managed, then guidance for visual / routine inspection (as is currently provided for levee and flood defence structure inspection) is required.

3.7.2.2 For flood risk analyses

Once transition structures have been identified they may be assessed for any problems. The summary information provided tries to identify problem indicators that may be observed in the field, along with potential failure modes and subsequently solutions. However, it is clear that the degree of knowledge around some of the processes that can occur is limited and the subject of ongoing research.

Rec. 4: Since the risks posed by transition structures affects the overall performance of a system of flood defences, these risks should be included within any overall analysis of performance.

Historically, the analysis and inclusion of risks generated from transitions has not been undertaken – at least within the UK and Dutch frameworks for flood risk analysis. Inclusion of transitions within a modelling framework for risk assessment then poses a number of challenges. For a rigorous assessment it is necessary to include transitions as point or individual structures, against which performance data has to be attributed. This requires adaptation of the analysis framework to incorporate such structures plus sufficient knowledge of the potential failure mechanisms as to allow performance curves (fragility curves) to be produced for each transition structure under a range of load conditions. It is clear that current the understanding and characterisation of some of the processes is not at a sufficient stage to provide a reliable numerical representation of the processes (e.g. the various different forms of internal erosion, as reported in Chapter 2). However, in the absence of numerical models of the failure process, judgement may be used to develop initial estimates of performance based upon field experience.

A simpler approach for the initial inclusion of transitions within flood risk models is to adapt the performance curves of the adjacent flood defence structure(s) to better represent the overall risk posed by the defence length(s) and the transition. Again, this could be achieved numerically or by inclusion of judgement based performance curves.

Rec. 5: The contribution of transitions to overall flood risk should be assessed (within system risk modelling) either through (i) direct identification and analysis of transitions as specific ‘units’ within the system risk model or (ii) adjustment of the performance assessment of the defence lengths associated with the structure transition.

3.7.3 Gaps in knowledge

Whilst the analysis of transitions is an important issue which would help to improve the accuracy of system risk models, and assist day to day asset and hence flood risk management, there are clearly a number of development tasks and gaps in knowledge that need to be addressed to help in this process.

The various work actions under FloodProBE WP3 all contribute towards a better understanding of levee performance and management, hence are interlinked. Some actions provide specific knowledge which supports the identification and analysis of transitions.

FloodProBE task 3.2 addresses the use of geophysics and remote sensing data. The work on geophysics has provided a clear understanding of the different methods and what they can be used for in the assessment of standard levees, but has not gone further than identifying the fact that geophysics techniques are disrupted by some transitions (e.g. reinforcement in hard structures, as shown by the Humber surveys). Hence a further stage of research is now required to build from the transitions typology and to show how the different geophysical techniques may be used (or not) to locate the extent and condition of transitions.

Rec. 6: *R*esearch is required to build from the transitions typology and to show how the different geophysical techniques (FloodProBE Task 3.2) may be used (or not) to locate the extent and condition of specific transition structures.

FloodProBE Task 3.2 also addresses the use of remote sensing data (LIDAR). This has been shown to be excellent for the detailed assessment of the surface features of a levee, and hence for an initial survey of potential transition structures. The Orleans pilot study demonstrates this very well. The Humber transitions case study also shows how use of a tool such as Google Maps can allow a similar process to be undertaken with free publicly available data, but with less precision than from a specified LIDAR survey.

Research under FloodProbe Task 3.3 shows how combining and analysing data of different types and sources can allow you to improve your analysis of levee performance. These concepts also apply to the identification of, and performance assessment of transitions.

Internal erosion is often a contributory factor to failure modes associated with transition structures; hence a clear understanding of these processes, along with methods for performance assessment is essential. In particular, this relates to the processes that can occur at a soil / structure interface and how to improve resistance to erosion in this area (and hence providing the physical basis for generation of performance curves that underpin any numerical flood risk assessment). The following questions are typically asked:

- What are the physics involved in (internal) contact erosion for both ‘soil/soil’ and ‘soil/structure’ interfaces? In particular, if no empty space exists around a pipe or structure crossing, does internal erosion have a greater chance to develop at the transition than in the body of the adjacent soil?
- How do you characterise the ‘soil/soil’ and ‘soil/structure’ interface?
- How do you characterise resistance to erosion at the ‘soil/soil’ and ‘soil/structure’ interface?

- How do you improve resistance to erosion at the ‘soil/soil’ and ‘soil/structure’ interface?
- What is the cause for initiation of contact erosion at the ‘soil/soil’ and ‘soil/structure’ interface? The driving force is hydraulic head, but how can we characterize the resistance?

Rec. 7: The various mechanisms of internal erosion are often a contributory factor to failure modes associated with structure transitions; hence a clear understanding of these processes, along with methods for performance assessment are needed.

The research here has demonstrated the importance of structure transitions and provided a framework for identification. Methods for risk quantification to allow routine inclusion within flood risk analysis and management procedures are required. Hence, for each transition type, representation of the potential failure process is required. This will then allow integration of the risk contribution from transitions into the existing frameworks for analysis – either as an adjustment to the performance of the adjacent structures or as a unique risk contribution from the transition itself.

Rec. 8: Develop a (limit state or other) representation of failure modes for each of the transitions defined within the framework. From these representations, develop fragility curve representations for each transition and / or rules for adaptation of existing defence structure limit state equations and fragility curves.

Rec. 9: Include structure transitions into system flood risk analysis models and methods

3.7.4 Improving the design, construction and management process

Having established the significant role that transitions can play on overall flood risk, we need to ensure that the significance of transitions is understood by everyone within the design and construction chain, as well as by flood risk managers. Ultimately this should lead to a reduction in flood risk through improved design and management of transitions.

There are three key aspects to undertaking this process:

1. Raising awareness and providing guidance on the issues associated with transitions, along with potential options for remedial measures or redesign.

2. Provision of specific methods for the performance analysis of each specific type of transition

3. More detailed guidance on potential solutions for problem transitions (moving from the current generic guidance to transition specific solutions)

In order for a levee manager to decide upon the most appropriate measures for dealing with a transition it is necessary to assess the risk between rebuilding and retrofitting a solution. Whilst the direct costs of such work can be easily estimated, the different risks resulting from differing levels of resulting performance require fragility curves (performance curves based upon limit state analyses, expert judgement or a combination) to be produced for each transition, with sufficient detail to differentiate between the transition condition and potential failure modes arising from different design variations and retrofit solutions.

Rec. 10: Building from Recommendations 8 & 9 above, development of more detailed representations of failure modes in support of the risk based assessment of different design and retrofit solutions. An initial step would be to define more detailed, different options for design and retrofit of each transition type.

4 The performance of vegetation on flood embankments

For this project the term ‘vegetation’ is taken to refer to grass cover on flood embankments. The effect of trees and woody vegetation is not considered within this study.

4.1 Aims and objectives

The grass surface cover on a flood embankment protects against soil erosion and can either prevent breach or delay the onset of breach. Assessing the performance of grass in this context is therefore an important aspect of the overall performance assessment (and hence flood risk assessment) for flood embankments.

The significance of grass cover performance is increased if “acceptable overtopping / overflow” is permitted as part of flood risk management practice. Under such conditions, the estimated performance of the flood embankment will include and depend upon the performance of the grass cover. The effects of climate change appear to be leading towards more extreme conditions for both hydraulic loading (magnitude of flood event) and climatic conditions (prolonged wet and dry periods). These changes pose an increasing pressure upon the performance of grass on flood embankments. Not only do the hydraulic load conditions increase, but the environment pressures affecting the quality and stability of the grass are also changing.

The broad aim of this research action is the development of extended or revised guidance based upon a review of international research results and existing grass performance data from the last 25 years.

4.1.1 Scope of work

Specific research actions on grass performance comprised:

- A review of project initiatives related to the performance of grass;
- Investigation into grass performance data collected at the USDA Stillwater centre over the past 20 years, to identify what aspects might be relevant to European practice;
- Confirmation of existing European and US guidance on grass performance, followed by identification of either (i) Updates to guidance using existing international research findings or (ii) clarification of longer term R&D needs to improve knowledge and performance of embankment grass cover layers.

4.2 Summary of the Literature Review

The following section provides a summary of the literature review that was undertaken. Full details of the key initiatives and publications identified are given within FloodProBE Science Report WP3-01-10-06.

4.2.1 Introductory remarks

Grass is considered to have good performance if it prevents erosion of the underlying soil and ultimately any damage to the flood defence. Determining the loading(s) up to which grass remains in place, offering protection to the structure, is one of the primary aims of the designer and asset manager.

The hydraulic performance of grass on flood embankments can be assessed by the loading(s) to which the grass and embankment are subjected. These loadings fall mainly into the following categories: overtopping, overflow and rainfall runoff.

In general the hydraulic performance of grass can be considered in terms of:

1. its erosion resistance, by means of a “maximum permissible velocity” of the flow or the “effective shear stress” that a grass lined structure can withstand;
2. its resistance to the flow, usually by means of a coefficient of frictional resistance such as Manning’s n.

Both concepts are useful and complementary: the erosion resistance indicates if the grass cover can protect the underlying soil and the resistance (or friction factor) allows the calculation of the flow depth/velocity/rate. For the case of flood embankments, determining the erosion resistance is the primary aim, whereas the resistance to the flow becomes more relevant to the design of grassed channels because it allows the determination of the conveyance of a channel. However, the knowledge of the resistance of grass is also important in the context of flood embankments as it is required in certain methodologies for the determination of the effective shear stress, for example, and allows the estimation of how much flow can reach areas behind the defenses and how fast it can reach them.

It has been found that much of the literature on the hydraulic performance of grass relates to resistance and derives from work on grassed channels rather than grassed embankments. Despite some differences in terms of water velocity and depth, this research is likely to be relevant for the assessment of grass resistance in other situations such as in flood embankments. In particular, there are synergies between shallow grassed channels used intermittently for flow discharge and the flow overtopping flood embankments. On this basis, some of the sources summarised relate to channels.

As a general indication, Coppin & Richards (1990) have suggested permissible velocities (which depend on the soil, the density of grass cover and the longitudinal gradient of the channel/slope) in the range 1.2m/s and 2.1m/s for good grass cover. It has been found that the duration of the flow is a relevant parameter in the erosion of channels that are intermittently subjected to flow, which is the case of flood embankments. Even without reinforcement, grassed surfaces can withstand considerably high velocities for short durations: almost 4m/s for 1 hour or 3m/s for 2 hours. Long term stability is generally achieved if flow velocities remain below 1m/s. (Hewlett et al, 1987).

The relatively recent development of the wave overtopping simulator has encouraged testing of real flood embankments, in-situ, by the controlled release of water down the embankment slope (van der Meer, 2006, van der Meer et al., 2009). This is helping to clarify how the in-situ performance of grass varies with a range of factors relating to the design and state of the flood embankment.

4.2.2 Literature review summary

The performance of grass during flooding has been studied by a number of organisations in Europe and worldwide. In order to build upon the knowledge gained from these studies, a literature review was undertaken. The main objectives of this review were to:

- Identify major work that was or is being undertaken to assess vegetation performance during flooding.
- Undertake critical analysis of the work identified above.
- Gain greater understanding of the performance of grass during flooding.
- Recognise gaps in knowledge.
- Propose a way forward (to improve performance guidance for use in flood risk management).

The main source of information for the literature review was through Internet web based searches. However, knowledge and links from the wider FloodProBE team also contributed. A detailed list of information sources / searches is provided in Appendix 1 of Report WP3-01-10-06.

The literature review was structured according to two geographic regions:

1. Region 1: Europe
2. Region 2: The rest of the World)

Figure 4.1 provides a graphical summary of the initiatives found within each of these regions, listed chronologically. Whilst there are numerous studies into the effects of vegetation on flow (not listed here), there are considerably fewer detailing the performance of grass under overflow conditions.

4.2.3 Conclusions from the literature review

The literature review was structured according to (i) grass performance under (steady) overflow conditions and (ii) grass performance under (wave) overtopping conditions. Findings were grouped according to (i) initiatives in Europe and (ii) initiatives elsewhere.

The current state of the art relating to wave overtopping performance is reported in a separate chapter of the science report. This is because of ongoing research in this field which is advancing the current state of knowledge. A brief summary is provided in Section 4.24 below.

The review of initiatives and guidance relating to overflow conditions identified a range of projects that appeared (initially) to provide guidance or advance the state of knowledge. However, many of these initiatives do not provide specific guidance or data on grass performance, instead either offering subjective guidance on management, or focusing upon grass resistance to flow – this being part of the problem to solve for assessing overall performance, but also an aspect of interest when trying to analyse the flow of water within a grass or vegetation lined channel.

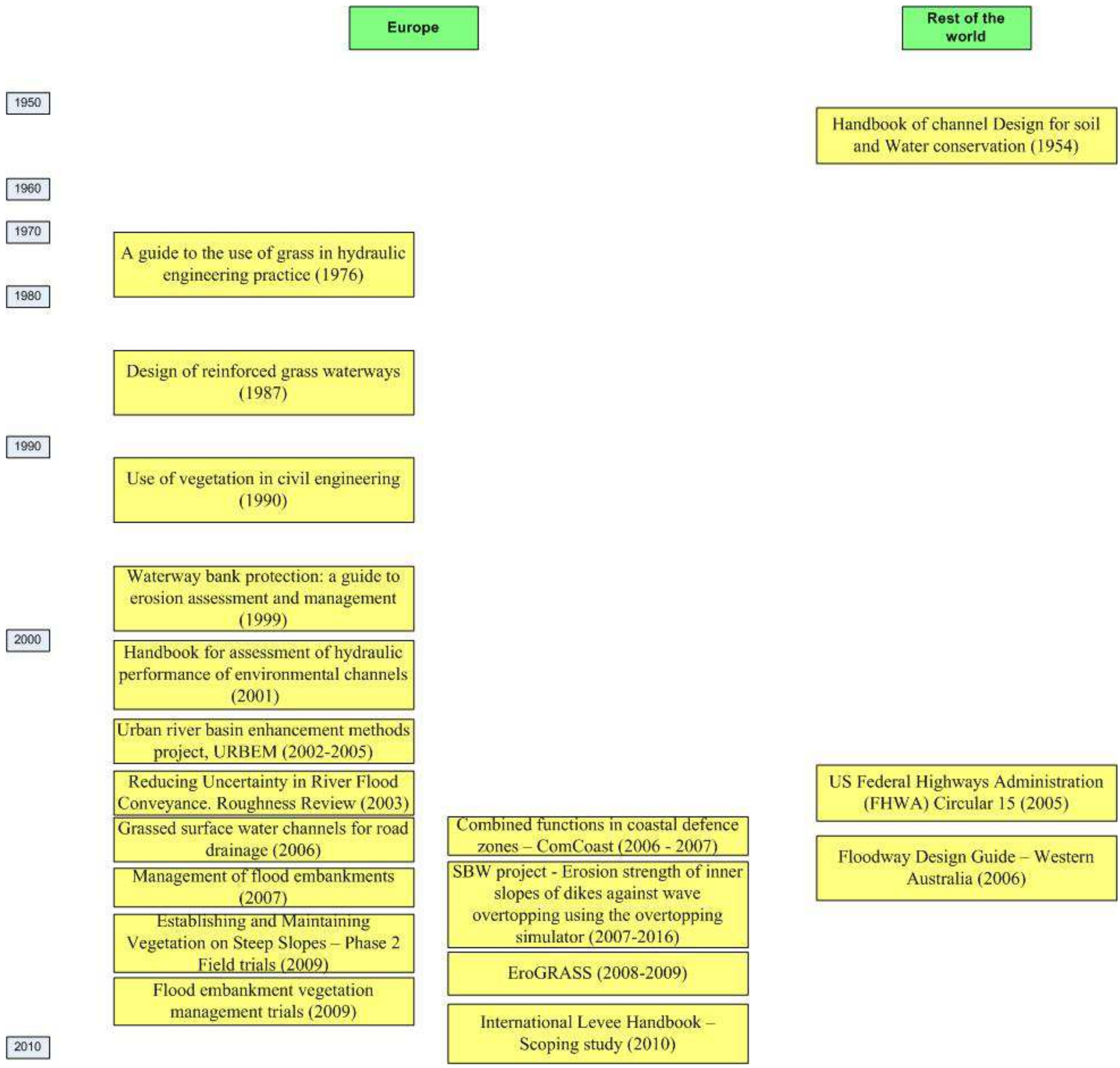


Figure 4.1 Major initiatives and literature identified

Of all the initiatives identified, only three offered specific methods for predicting the performance of grass cover on an embankment. These were:

Method 1: A guide to the use of grass in hydraulic engineering practice (Whitehead et al., 1976). (CIRIA Technical Note 71).

Method 2: Design of reinforced grass waterways (Hewlett et al., 1987) (CIRIA Report 116)

Method 3: Stability Design of Grass-Lined Open Channels – Agriculture Handbook 667 (Temple et al., 1987)

The CIRIA Report 116 guide built on the data used for the earlier CIRIA Technical Note 71 report, including consultation with USDA and use of some USDA data alongside the UK data. Common to both CIRIA reports, the guidance offered simple design curves based upon grass condition (good, average, poor) and an acceptable velocity – duration for overflow.

The USDA approach analyses the effective stress at the soil surface and takes into consideration the soil erodibility, via use of the soil plasticity index.

Having identified three sources of guidance for assessing grass performance, authors of both the CIRIA and the USDA work were contacted to see whether access to the original research and data was possible. In both cases the response was that the original work had been undertaken around 30 years ago and access to the original notes and data was no longer feasible.

As an alternative to reviewing the original data and analyses, it was decided to investigate and compare the performance of the three methods to see whether there were obvious differences or deficiencies (See Section 4.3 below).

4.2.4 State of the art for grass performance under wave overtopping

Within the Dutch SBW program (Strength and Loads on Water defenses) research was carried out on the performance of grass cover under wave overtopping loads and wave impact loads. The research program started in 2007 and ended in 2011. The research program was commissioned by the Rijkswaterstaat to Deltares and was carried out with Van der Meer Consluting, Infram and Alterra as partners of Deltares. In a slightly adjusted framework the research on the performance of grass covers will continue until 2016. In the coming period research will specifically be aimed at the influence of objects in the grass cover, transitions from grass to hard revetments and objects and grass erosion in the wave run-up zone.

The research about vegetation performance under hydraulic loading has led to a report which is expected to be finalized in the first half of 2012. The official quality control procedure, mandatory for most technical documents for safety assessments and design of water defenses, has not yet been completed for the presented models. Hence, this summary gives the current State of Art. The observations at large-scale wave overtopping tests do however show the strengths and weaknesses of grass cover on water defenses.

The Dutch safety assessment of grass cover addresses different failure mechanisms for different zones in a dike cross section. The SBW research on grass erosion was aimed at grass cover in the wave impact zone and in the wave over topping zone. The erosion mechanisms are illustrated in Figure 4.2.

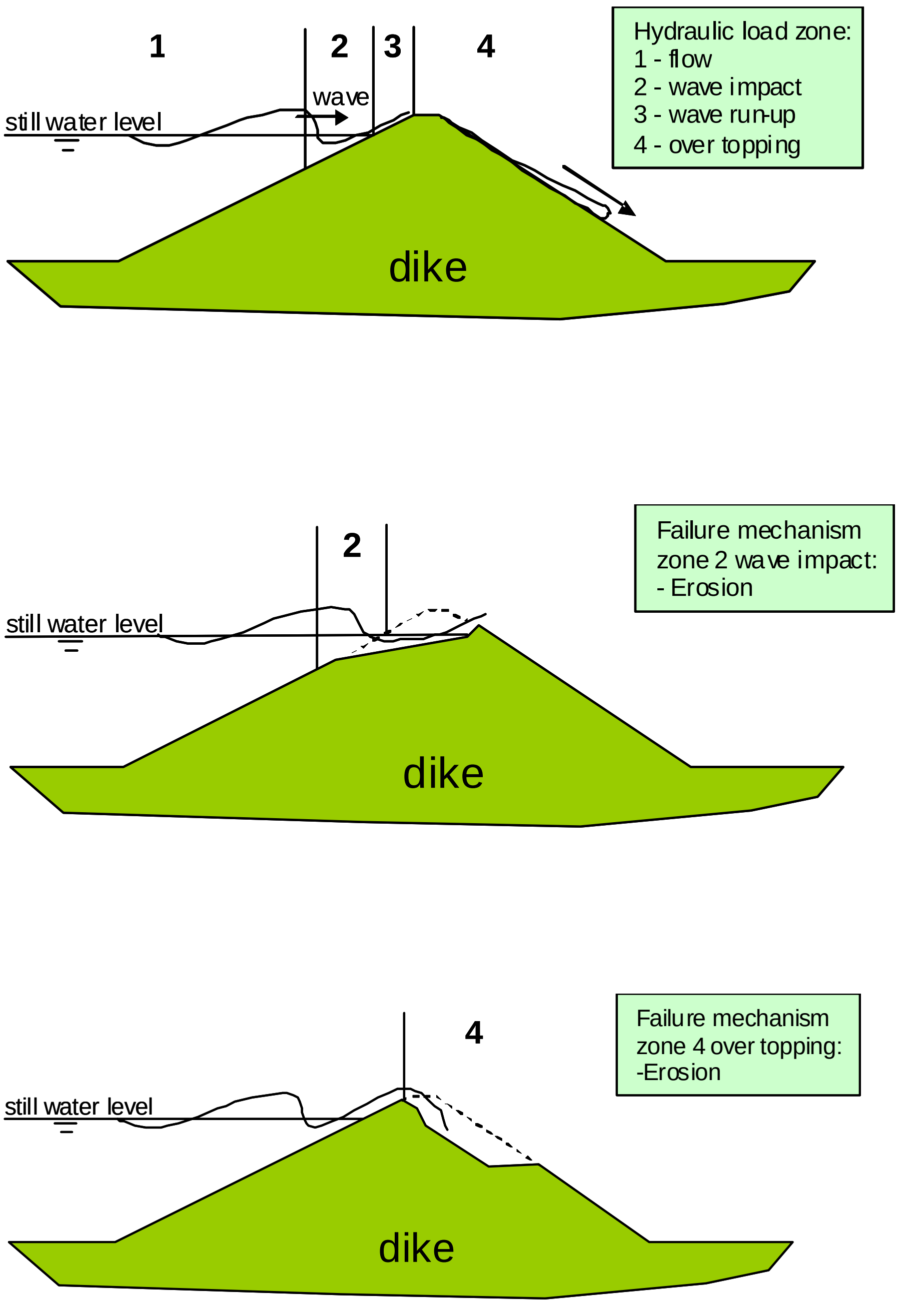


Figure 4.2 Hydraulic load zones (1 to 4) and failure mechanisms addressed in the SBW research programme

A cross section showing grass cover is given in Figure 4.3. In the following sections, the performance of the lower layer is not taken into account. Failure of the grass cover is defined to be failure of the top layer, which means breach of the top layer by erosion of the cover layer as a whole. The top layer is considered to be about 0.2 m thick.

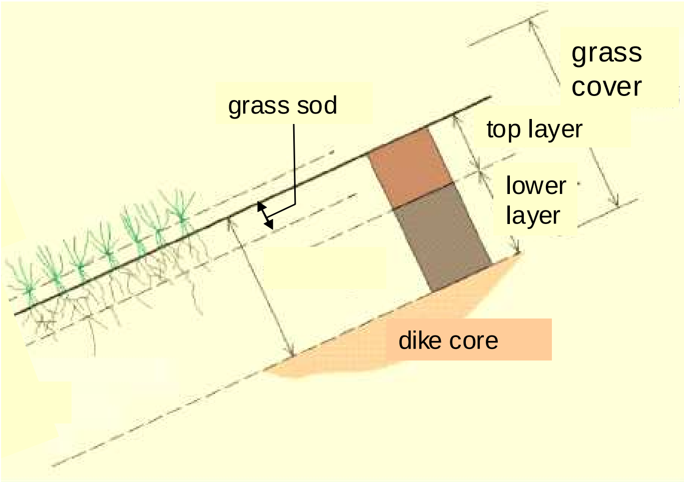


Figure 4.3 Schematic representation of the grass cover, grass sod and top layer

4.2.4.1 Erosion of grass cover within the wave impact zone – failure model

The failure model suggested for erosion in the wave impact zone compares the wave impact load time t_i (hour) with the wave impact resisting time t_r (hour) for different wave height H_s (m) as given in Figure 4.4. The grass sod is sufficiently strong if $t_r > t_i$. The model does allow some minor damage to occur to the sod.

Model limitations concerning the slope angle are 1H:2.5V (or less steep) for $H_s \geq 0.5$ m and 1V:1.5H (or less steep) for $H_s < 0.5$ m. For a slope angle more gentle than 1V:4H the resisting time t_r will increase, however, the model has no prediction capability on how much t_r will increase.

Note that a fragmented sod is not covered by the relationship shown in Figure 6-9. The strength of a fragmented sod is such that it cannot be relied upon to have enough strength to withstand any wave impacts. The strength provided by the soil beneath the sod layer is immediately relied upon.

No research within the SBW framework was aimed at erosion in the wave run up zone yet (zone 3 in Figure 4.2), however, if the grass sod present in the wave impact zone (zone 2 in Figure 4.2) is sufficient, the grass sod in the run up zone will also be sufficient. Pressure gradients in the grass sod and subsoil, causing erosion, are significantly larger in the wave impact zone then in the wave run-up zone. A grass cover will fail in the wave impact zone before it fails in the wave run-up zone, if the grass cover is of equal quality in both zones.

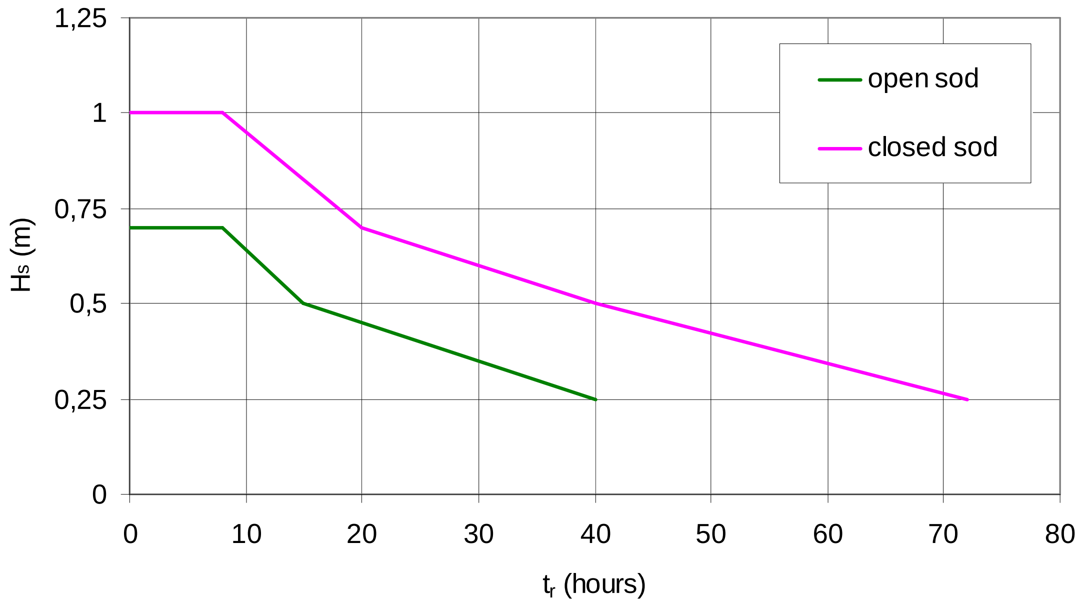


Figure 4.4 Wave impact resisting time t_r (hour) for different wave height H_s (m) and sod quality (open or closed).

4.2.4.2 Erosion of grass cover in the wave overtopping zone – failure model

The suggested failure model for erosion on the crest and landward slope due to wave-overtopping is based on observations at more than twenty wave over-topping tests.

A fragmented sod does not have any strength that can be relied upon. If there is any significant wave overtopping to be expected (c.q. more than 0,1 l/s per m) a fragmented sod is not recommended..

For a closed sod, regardless of the lower layer quality, or an open sod in combination with a clay layer more than 0,4 m thick on top of a sand core (0,4 m including top and lower layer thickness Figure 4), at least 1 l/s per m will not lead to failure by erosion.

For higher overtopping discharges, extra care is needed where the clay layer is less than 0,4 m on top of a sand core (0,4 m including top and lower layer thickness Figure 4.3). A sand core in combination with a thin clay layer is susceptible to internal erosion and subsequent undermining of the clay layer even at low overtopping intensity. A concentrated exit of water through a mole, mouse or rabbit burrow can easily result in sand transport from under the clay layer and lead to undermining.

If the risk of undermining can be excluded, and if a closed sod is present, the cumulative overload model is suggested (Van der Meer et al. 2010). The model is based on the observation of damage occurrence and damage progress during more than twenty wave-overtopping tests.

The wave overtopping tests show that there is not one critical overtopping volume during a storm, but that the larger volumes during a storm contribute either to the initiation of damage or to the extension of damage. Small overtopping volumes, generating depth averaged maximum flow velocities below the critical velocity, do not contribute at all. The transition between contributing overtopping volumes and volumes, which do not, is determined by a critical volume, or a critical velocity which depends on the grass quality.

The observations have led to the following cumulative overload model:

$$\sum_{n=1}^{n=Nov}\left(U_n^2 - U_c^2\right) < C$$ (Note in case of U_n<U_c the result is discarded)

Where,

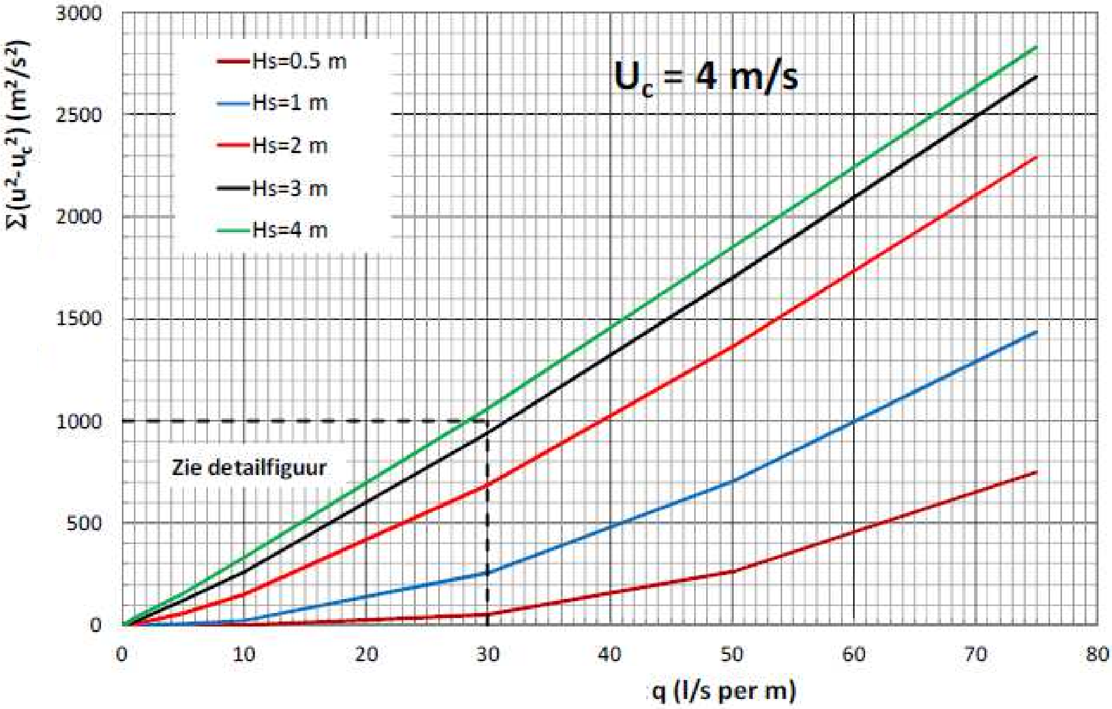
- N_{ov} number of overtopping waves.
- U (m/s) is the maximum depth averaged flow velocity from an overtopping wave, for cases where U>Uc.
- U_c (m/s) critical maximum depth averaged flow velocity depending on the top layer strength.
- C (m^2/s^2) is a critical value where:

  C=500 (m^2/s^2) resembles a situation where initial damage occurs. A large scatter in the initial damage value is however observed.

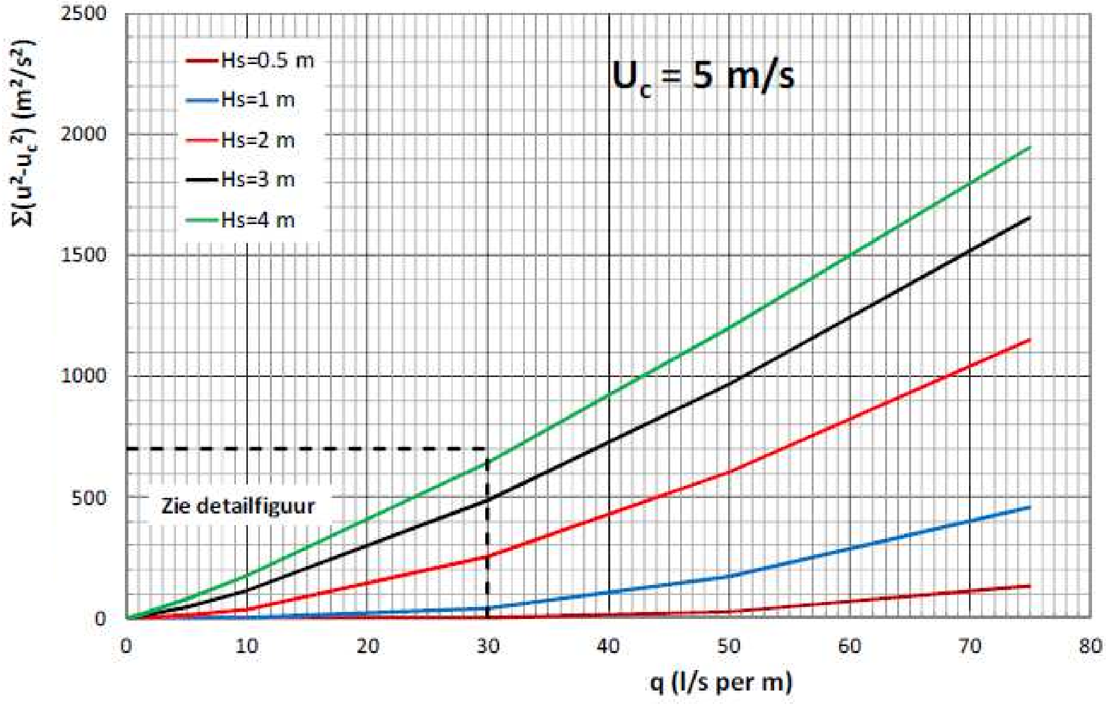
  C=1000 (m^2/s^2) multiple spots with initial damage (not yet failure of the top layer)

  C=3500 (m^2/s^2) failure of the top layer.

The cumulative overload depends mainly on U_c, the storm duration, and the combination of the average overtopping discharge and the wave height H_s. From the wave overtopping tests, critical velocities were back calculated and showed a range from U_c = 4 m/s (critical volume 500 l/m) up to 6,3 m/s (critical volume 2000 l/m), excluding tests with fragmented grass sods. The cumulative overload can be compressed in the graphs given below. The graph gives (on the vertical axis) the cumulative overload for a 1-hour storm condition (Figure 4.5).



U_c = 4 m/s
Hs=0.5 m
Hs=1 m
Hs=2 m
Hs=3 m
Hs=4 m
Zie detailfiguur
$\Sigma(u^2-u_c^2)$ (m^2/s^2)
q (l/s per m)
0
500
1000
1500
2000
2500
3000
0
10
20
30
40
50
60
70
80



U_c = 5 m/s
Hs=0.5 m
Hs=1 m
Hs=2 m
Hs=3 m
Hs=4 m
Zie detailfiguur
$\Sigma(u^2-u_c^2)$ (m^2/s^2)
q (l/s per m)
0
500
1000
1500
2000
2500
0
10
20
30
40
50
60
70
80

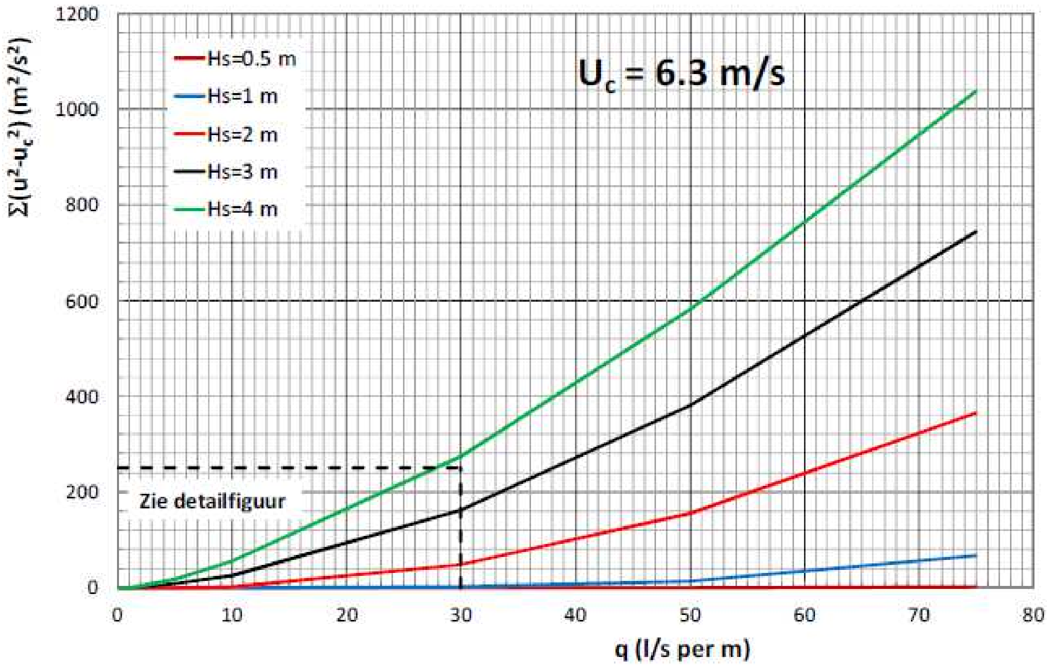


Figure 4.5 Cumulative overload (m^2/s^2) as a function of U_c (m/s), H_s (m) and q (l/s per m) for a 1 hour storm condition

The research within the SBW framework has not yet led to a reliable relation between Uc and field parameters. Based on the test results a value of U_c = 4 m/s and C= 1000 m^2/s^2 is advised for closed grass sods, and excluding cases too far beyond the range of wave overtopping tests, the most important being the slope angle of 1V:2,3H. For a closed sod it is likely that the critical velocity will be larger than 4 m/s, however, the research to predict U_c is still work in progress and the results of this section are preliminary and not yet in the Dutch guidance.

4.3 Data analysis / model testing

Following the identification of these methods, the next step was to evaluate their capability in assessing grass performance. Therefore, a number of test cases were selected for this purpose. In the following sections a description of the test cases, modelling undertaken and analysis of the modelling results is given.

4.3.1 Test cases

Identification of test case data was not an easy task. Available data on embankment grass failure due to overflow is rare and often incomplete. The USDA data (Hanson et al 2005) was the first source for information and WP3 partners (i.e. Cemagref and Deltares) also provided data for testing. However, data from Cemagref and Deltares was eventually found to be of insufficient detail to allow for test modelling. As such, only two test cases were analysed using the USDA data.

Test case 1

The data of this test case comes from large-scale overtopping tests have been conducted at USDA – ARS to provide information relevant to the erosion processes of cohesive embankment breach failures. This test case was a 2.23m high and 7.32m long embankment which was constructed from silty sand soil with a grass cover. Figure 4.6 and Figure 4.7, and Table 4.1 and Table 4.2 provide description of the data for the embankment geometry and soil conditions.

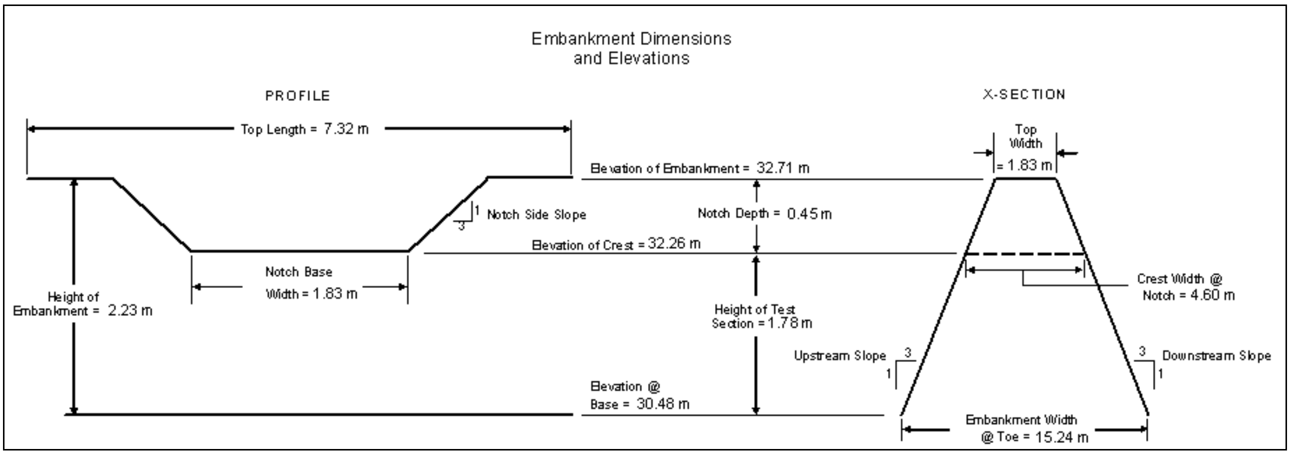


Figure 4.6 Geometry for Test Case 1

Figure 4.7 Test case 1 embankment

Table 4.1 Embankment and test section dimensions

Embankment Dimensions		Test Section Dimensions	
Height of Embankment	2.23 m	Height of Test Section	1.78 m
Elevation of Embankment	32.71 m	Notch Base width	1.83 m
Top Length	7.32 m	Notch Side Slopes	3/1 (H/V)
Top Width	1.83 m	Notch Depth	0.45 m
Upstream Slope	3/1 (H/V)	Crest Width @ Notch	4.6 m
Downstream Slope	3/1 (H/V)	Elevation of Crest @ Test Section	32.26 m
Elevation @ Base	30.48 m		
Embankment Width @ Toe	15.24 m		

Table 4.2 Embankment soil properties

Gradation		Construction	
% Clay < 0.002 mm	5	Compaction Effort (kN.m/m^3)	191
% Silt > 0.002 mm	25	Loose Lift Thickness	0.15 m
% Sand > 0.105 mm	70	Compacted lift thickness	0.12 m
Plasticity Index	Non-plastic	**Sieve Analysis**	% Finer
Soil Classification (USCS)	SM	0.002 mm	5
Grain Density (g/cm^3)	2.67	0.005 mm	7
Unconfined Compressive Strength (kN/m^2)	20.30	# 200 (0.075 mm)	30
Average Dry Density (g/cm^3)	1.71	# 140 (0.106 mm)	39
Average Water Content @ construction %	8.9	# 60 (0.250 mm)	71
Average Total Density (g/cm^3)	1.87	# 40 (0.425 mm)	93
Erodibility Coefficient kd (cm^3 /N.s)	10.2	# 20 (0.850 mm)	99
Critical stress・c (kN/m^2)	0.00	# 10 (2.00 mm)	100

Test case 2

The data of this test case also comes from the large-scale overtopping tests have been conducted at USDA – ARS to provide information relevant to the erosion processes of cohesive embankment breach failures. This test case was a 2.23 m high and 7.32 m long embankment which was constructed from clay-loam soil with grass cover. Figure 4.8 and Figure 4.9 and Table 4-3 and Table 4-4 provide description data for the embankment geometry and soil conditions.

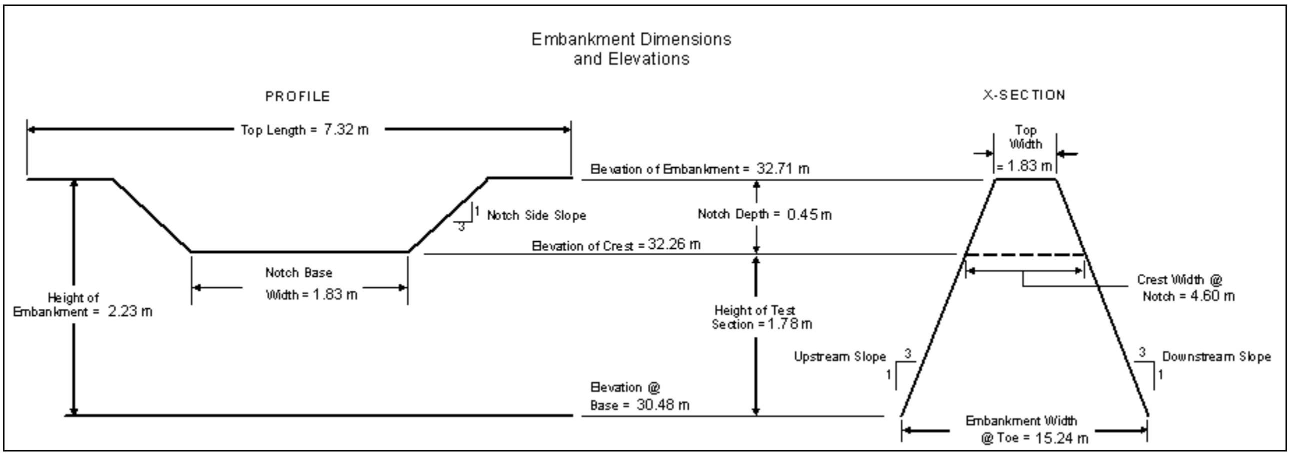


Figure 4.8 Geometry for Test Case 2

Figure 4.9 Test Case 2 embankment

Table 4.3 Embankment and test section dimensions

Embankment Dimensions		Test Section Dimensions	
Height of Embankment	2.23 m	Height of Test Section	1.78 m
Elevation of Embankment	32.71 m	Notch Base width	1.83 m
Top Length	7.32 m	Notch Side Slopes	3/1 (H/V)
Top Width	1.83 m	Notch Depth	0.45 m
Upstream Slope	3/1 (H/V)	Crest Width @ Notch	4.60 m
Downstream Slope	3/1 (H/V)	Elevation of Crest @ Test Section	32.26 m
Elevation @ Base	30.48 m		
Embankment Width @ Toe	15.24 m		

Table 4.4 Embankment soil properties

Gradation		Construction	
% Clay < 0.002 mm	26	Compaction Effort (kN.m/m^3)	191
% Silt > 0.002 mm	49	Loose Lift Thickness	0.15 m
% Sand > 0.105 mm	25	Compacted lift thickness	0.12 m
Plasticity Index	17	**Sieve Analysis**	% Finer
Soil Classification (USCS)	CL	0.002 mm	26
Grain Density (g/cm^3)	2.67	0.005 mm	32
Unconfined Compressive Strength (kN/m^2)	67.92	# 200 (0.075 mm)	75
Average Dry Density (g/cm^3)	1.65	# 10 (2.00 mm)	100
Average Water Content @ construction %	16.4		
Average Total Density (g/cm^3)	1.92		
Erodibility Coefficient kd (cm^3 /N.s)	0.04		
Critical stress τc (kN/m^2)	0.01		

4.3.2 Evaluation methodology

The main objective of the methods evaluation is to show how their performance compares over a range of conditions. This allows the understanding of how the underlying assumptions of each method (e.g. considering or not considering soil type, effect of grass type and quality, etc.) could affect the results. This, in turn, helps to conclude what method(s) should be used or updated for the use in Europe.

To achieve this, a number of parameters have been chosen for the purpose of comparison. These parameters have been selected as they could significantly impact the grass performance during an overflow event. A list of the selected parameters with a brief definition is given below:

1. Maximum overflow head* which is the maximum depth of water applied on the grass during an overflow event.
2. Downstream (dry) slope which is the vertical to horizontal distance ratio.
3. Critical shear stress which is defined as threshold stress below which no soil erosion occurs.
4. Soil erodibility which represents erosion rate of the soil.
5. Soil plasticity index which is defined as the difference in moisture content between the soil liquid limit and the soil plastic limit.
6. Vegetation type which is linked to the friction of different vegetation types
7. Vegetation quality which describes the uniformity, density and length of the grass.

Since Methods 1 and 2 are velocity based whereas the third method is shear stress based (which means they use different criteria to predict the grass cover failure), it was necessary to find an approach that ensures that the performance comparison is done correctly. Therefore, the HR BREACH and the WINDAM models have been used to simulate the failure of the grass cover for Methods 1 and 2 and Method 3 respectively. Doing this avoids calculating the intermediate values whilst ensuring each test case conditions are identical - to the maximum possible extent - in both models. Details of the HR Breach and WINDAM model versions that have been used to undertake the comparison are given in FloodProBE Science Report WP3-01-10-06. Figure 4.10 and Figure 4.11 show the graphical user interface (GUI) of the HR Breach and the WINDAM models respectively.

* Changing the overtopping head was achieved by lowering and raising the breach initial invert level.

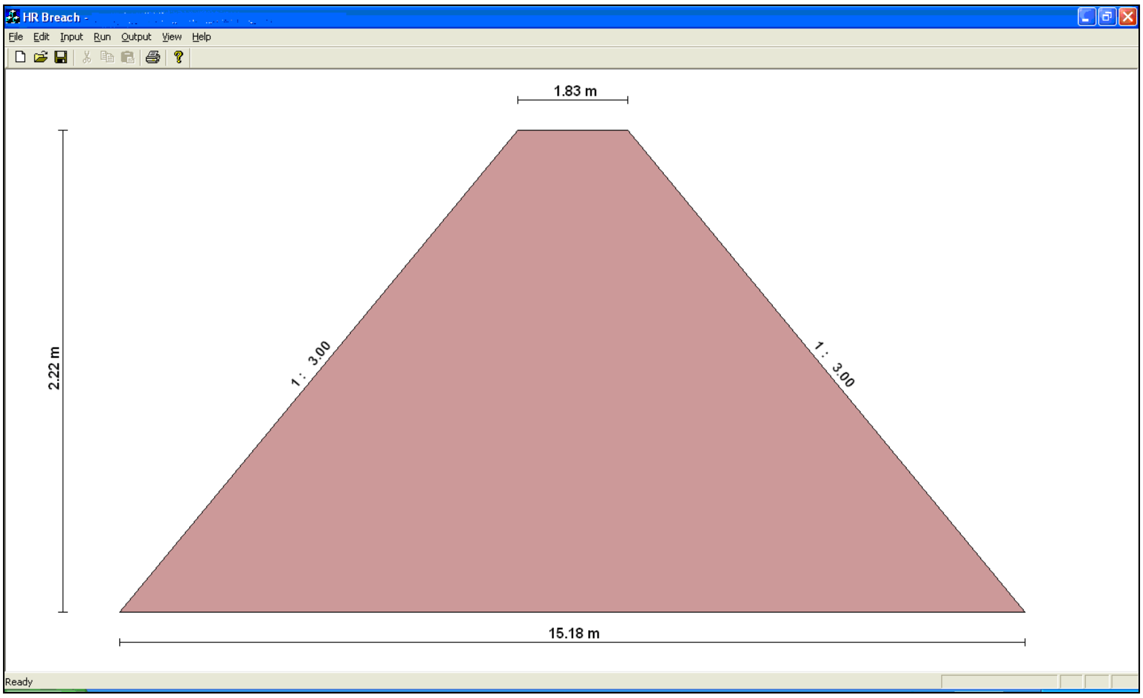


Figure 4.10 GUI of the HR Breach model

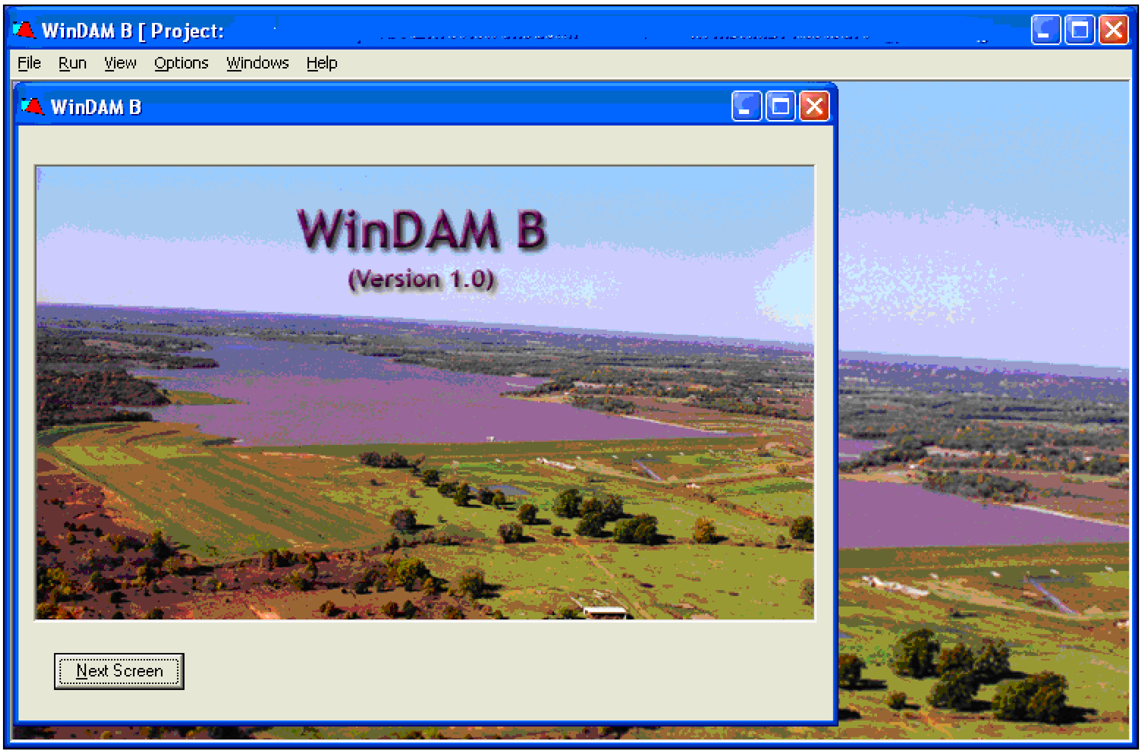


Figure 4.11 GUI of the WINDAM model

4.3.3 Evaluation results

In this section, the evaluation results of the test cases are presented.

Test case 1

Table 4.5 shows the parameter variations that were used for Test Case 1 whilst Figure 4.12 shows the results for each parameter.

Table 4.5 Parameter variation for Test case 1

Parameter	Method 1 (TN71)			Method 2 (CIRIA 116)			Method 3 (Hbk 667)		
	Base	Var. 1	Var. 2	Base	Var. 1	Var. 2	Base	Var. 1	Var. 2
Breach initial invert level (m)	32.25	31.95	32.55	32.25	31.95	32.55	32.25	31.95	32.55
Downstream (dry) slope	1:3	1:2	1:4	1:3	1:2	1:4	1:3	1:2	1:4
Critical shear stress (kN/m^2)	0.00	0.02	0.04	0.00	0.02	0.04	0.00	0.02	0.04
Soil erodibility (cm^3 /N.s)	10.2	5.1	20.4	10.2	5.1	20.4	10.2	5.1	20.4
Soil plasticity index	0	10	20	0	10	20	0	10	20
Vegetation type							Bermuda	Buffalo	Mixture / alfalfa
Vegetation quality*	Poor	Medium	Good	Poor	Medium	Good	3	2	1

The results show that for this test case some parameters do not have any impact on the grass performance in all methods. These parameters are the soil erodibility and critical shear stress. This is understandable since It was found that those parameters are not included in the grass performance assessment formulation in all of the methods. The plasticity index variations showed a significant impact for Method 3 but not for Methods 1 and 2. This can also be explained as the plasticity index is only included in formulation of method 3. Other parameters such as the overtopping head, the downstream slope and the vegetation quality shows various degrees of impact with the vegetation quality showing the highest impact. Vegetation type was not varied for Methods 1 and 2 as they do not inherently have this in their formulation. But, this was varied for Method 3 and surprisingly did not have any effect on the failure time of the grass. This was looked at and it could be because the grass cover had a maintenance code of 3 (i.e. poor) with short length and low density. Therefore, vegetation type features did have a significant impact on the results.

* Vegetation quality is expressed as poor, medium and good in methods 1 and 2. This is equivalent to maintenance code 3,2 and 1, respectively, in method 3.

Generally, the 3 methods followed one trend for all variations which is:

1. Method 1 is the most conservative one (in the sense that it predicts the most rapid failure of the grass cover)

2. Method 3 is the most optimistic one (in the sense that it predicts the slowest failure of the grass cover)

3. Method 2 (TN71) typically lies between Method 1 and 3.

It should be also noted that for the base run, all methods predicted a slower failure than the actual failure time.

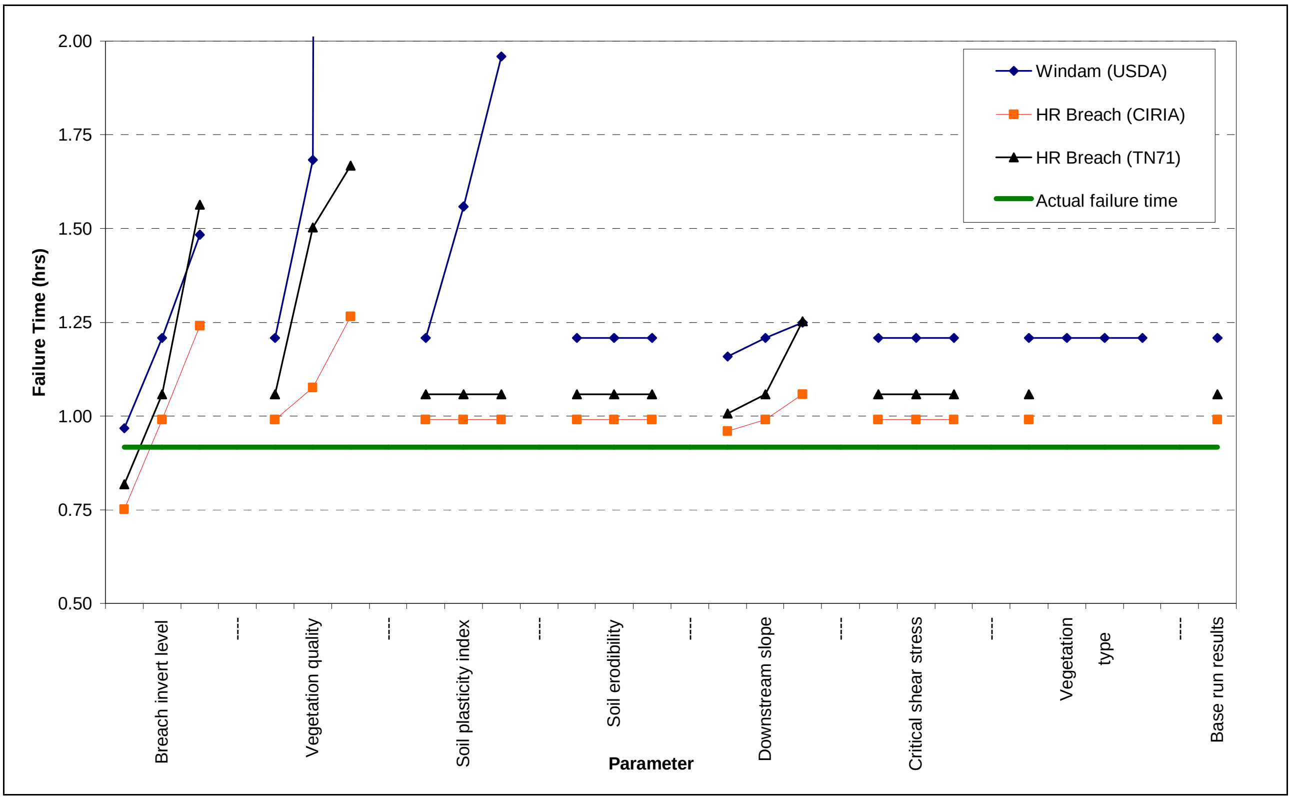


Figure 4.12 Comparison of results for Test Case 1

Test case 2

Table 4.6 shows the parameter variations that were used for Test Case 2 whilst Figure 4.13 shows the results for each parameter.

Table 4.6 Parameter variation for Test case 2

Parameter	Method 1 (TN71)			Method 2 (CIRIA 116)			Method 3 (Hbk 667)		
	Base	Var. 1	Var. 2		Base	Var. 1	Var. 2		Base
Breach initial invert level (m)	32.25	31.95	32.55	Breach initial invert level (m)	32.25	31.95	32.55	Breach initial invert level (m)	32.25
Downstream (dry) slope	1:3	1:2	1:4	Downstream (dry) slope	1:3	1:2	1:4	Downstream (dry) slope	1:3
Critical shear stress (kN/m^2)	0.01	0.02	0.04	Critical shear stress (kN/m^2)	0.01	0.02	0.04	Critical shear stress (kN/m^2)	0.01
Soil erodibility (cm^3 /N.s)	0.04	0.02	0.08	Soil erodibility (cm^3 /N.s)	0.04	0.02	0.08	Soil erodibility (cm^3 /N.s)	0.04
Soil plasticity index	17	0	10	Soil plasticity index	17	0	10	Soil plasticity index	17
Vegetation type	NA			NA			Bermuda	Vegetation type	NA
Vegetation quality	Poor	Medium	Good	Vegetation quality	Poor	Medium	Good	Vegetation quality	Poor

Test Case 2 results show very similar behaviour to Test Case 1. Soil Erodibility and critical shear stress did not have any impact on the failure time results. The plasticity index variations showed also a significant impact for method 3 but not for methods 1 and 2 and the overtopping head, the downstream slope and the vegetation quality showed various degrees of impact with the vegetation quality showing the highest impact. Vegetation type behaviour was also identical to Test Case 1 and this also because of the low quality of the grass cover.

For this test case, the 3 methods followed the same trend of Test Case 1 which is:

1. Method 1 is the most conservative one (in the sense that it predicts the most rapid failure of the grass cover)

2. Method 3 is the most optimistic one (in the sense that it predicts the slowest failure of the grass cover)

3. Method 2 (TN71) typically lies between Method 1 and 3.

The most notable difference between the results of this test case and Test Case 1 is in the comparison of the base run failure time with the actual failure time. In this test case, Methods 1 and 2 predicted faster failure while Method 3 was still slower.

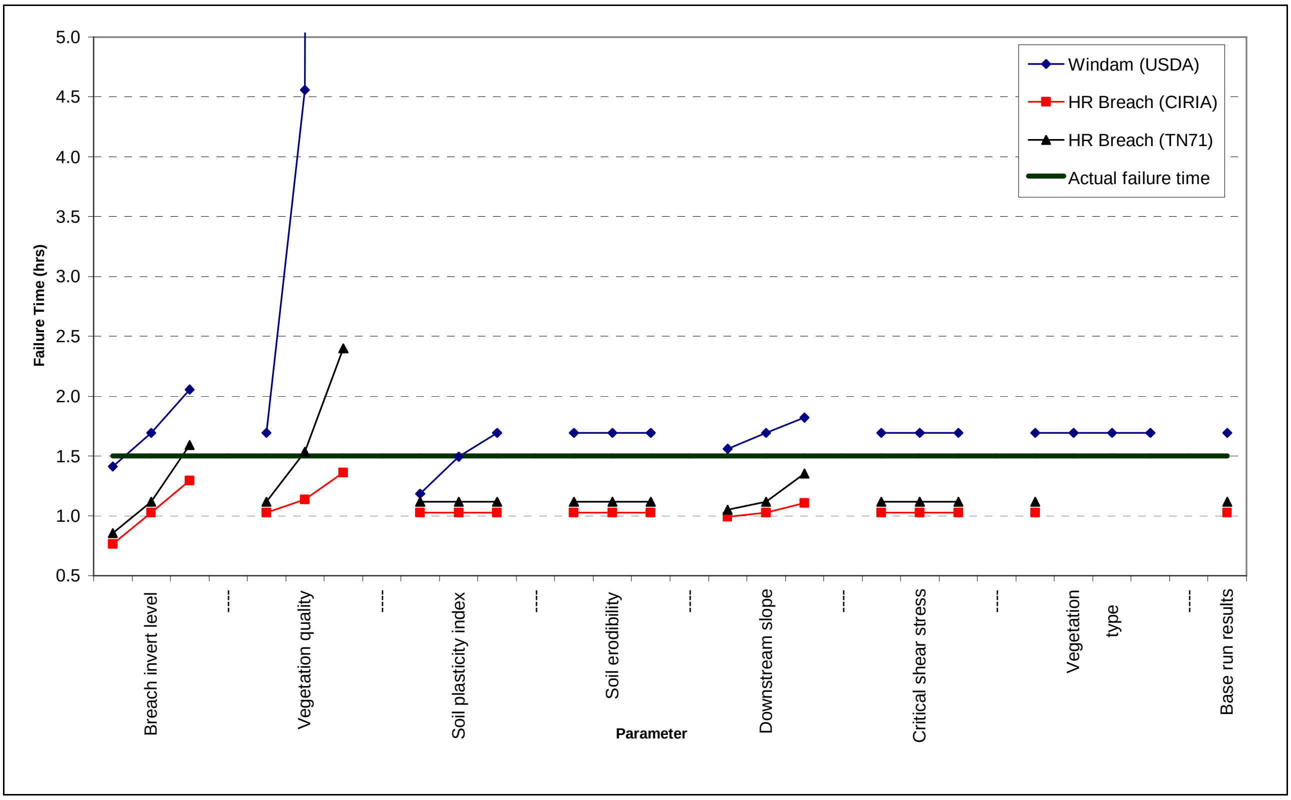


Figure 4.13 Comparison of results for Test Case 2

4.4 Conclusions and recommendations

The review of current guidance, and the data upon which this guidance is based, highlighted some noteworthy issues:

1. Practical and detailed quantitative guidance on performance of grass under overflow conditions is quite hard to find. There appear to be many research initiatives, but little generic guidance with actual methods provided for the design or performance analysis of grass cover. To the contrary there seems to be plenty of general guidance on the maintenance and use of vegetation.

2. A lot of research initiatives seem to focus on the hydraulic resistance of grass / vegetation to flow, rather than performance of the grass in protecting soil from erosion.

3. Based upon the literature review undertaken, the main sources of guidance on erosion protection from grass cover seem to be limited to:

    a. A guide to the use of grass in hydraulic engineering practice (Whitehead, 1976) – CIRIA Technical Note 71.
    b. Design of reinforced grass waterways (Hewlett et al, 1987. CIRIA Report 116).
    c. Stability Design of Grass-Lined Open Channels – Agriculture Handbook 667 (Temple at al, 1987).

It should be noted that the CIRIA 116 method builds from the CIRIA Technical Note 71 data. It also incorporates USDA data, hence all three methods are related to some degree.

4. The extent to which US grass performance data is valid in Europe remains unclear. Where grass performance is related to root density, soil strength etc. it would seem ‘transferable’, but research looking at this issue does not appear to have been undertaken. The CIRIA 116 design guidance does incorporate US data within the analysis.

The method comparison highlighted some interesting issues:

1. The CIRIA 116 design curves consistently predicted quicker grass failure times than the CIRIA Technical Note 71 data. This is consistent with the inclusion of a factor of safety into the CIRIA 116 performance curves (Morris et al, 2010). Users of the CIRIA 116 curves should note that a factor of safety has been included since the relevance of this differs if the curves are used for design or performance / reliability assessment (with design leading to a safer design, whilst with performance assessment, leading to a pessimistic assessment of behaviour).

2. The USDA approach incorporates the plasticity index, which reflects to a degree, the soil erodibility (Morris, In Prep). This shows a significant variation in performance, as soil erodibility reduces, whereas the CIRIA methods show no variation, because soil parameters are not considered.

   It would seem logical that the physical process of grass erosion, with the removal of roots from the soil, would also relate to the resistance of the underlying soil to erosion. The trend in breach analysis is towards the use of soil erodibility in order to improve representation of the embankment performance. Similarly, with the likely trend being towards design or planning for acceptable over flow during increasingly extreme flood events, adoption of a method that includes representation of soil erodibility would seem sensible.

4.4.1 Remaining gaps in knowledge

The research undertaken has confirmed that existing guidance is based upon quite limited data sets, some of which originates from the USA and most of which relates to grass lined channels rather than overflow on flood embankments.

Three prediction methods were identified during the study, two of which are directly related, being methods from sequential studies. The third is also linked, having provided some of the data used within the other two studies. This reinforces concerns that all of the guidance found within this review originates ultimately from the same sources, albeit with some added data or data variations between the different publications.

One original aim of this work was to review and possibly analyse data from the earlier research in order to see whether more could be learned without the need for fresh field testing. Unfortunately, since all of the base data originates from the 1980’s or earlier, it was found that access to the data was not possible; original reports, paperwork etc. detailing the research are no longer available. Consequently, the analysis reported here focussed on comparing the different methods to see whether they were significantly different and whether there were any obvious limitations.

The two main conclusions were that:

i. The CIRIA 116 report contained design curves, most likely with an embedded factor of safety (in comparison to the earlier TN71 report and curves). As such, using the earlier TN71 data gives a more precise prediction of performance, which is more appropriate for use in the performance analysis of flood embankments.

ii. The USDA approach includes a measure of the underlying soil performance, via inclusion of the soil plasticity index. This allows for greater variation in grass performance, as compared to the UK CIRIA methods, since the soil parameter adds another variable that can be ‘controlled’.

Hence, whilst improvement in the reliability of performance assessment can be made by using the USDA instead of CIRIA methods, there still remain a number of ‘gaps in knowledge’ relating to grass performance. These include:

1. The appropriateness of using test data from grasses in the USA for application within Europe.
2. The performance of grass during the initial hour or so of overflow – this period is missing from the CIRIA 116 / TN71 work.
3. Much of the data used relates to flow in steep grass lined channels, as might be used for embankment dam spillways. The applicability to small earth flood embankments is not completely clear.
4. The effect of variation in grass type in Europe is unclear. In addition, as climate change effects start to change the rainfall and soil moisture content, variations in grass type may also occur.
5. Analysis of any link between grass type and performance in conjunction with underlying soil type and performance is also required. For example, where the underlying soils are highly erosion resistant, do they prevent growth and maintenance of an effective grass layer? Conversely weak erodible soils promote stronger grass growth? Which combination of soil and grass cover offers the best solution for flood embankment protection and performance?

It was noted that in recent years there have been a number of initiatives looking at (i) the management of grass (i.e. cutting frequency), (ii) soil erodibility and (iii) wave overtopping on grassed embankments. The missing piece to this jigsaw of research is quality data relating to the performance of a range of grass types, on a range of soil types under steady overflowing conditions. Such data would permit most of the gaps in knowledge listed above to be answered and for more reliable performance guidance to be given.

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