



Description of flood defence structures for pilot sites

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SUMMARY

This report describes individual flood defence structures of all pilot sites in **FLOODsite**. For each of the 10 pilot sites the following details are described:

- the flood prone area
- the failures observed in the past
- an overview of all defence structures
- the flood defence structures in detail together with their potential failure modes

The overall purpose of this report is to give an overview of all flood defence structures of the pilot sites. This information will be needed by Theme 1 to concentrate research efforts on the key flood defence structures and their failure modes. Details of available knowledge on failure modes are then described in a different report so that further needs for research within **FLOODsite** can be concluded.

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1 Introduction

FLOODsite is aiming for Integrated Flood Risk Analysis and Management Methodologies. New research efforts in this field will be undertaken to fill gaps in knowledge and to achieve a better understanding of the underlying physics of flood related processes.

Any new knowledge developed in **FLOODsite** will be developed and tested at selected pilot sites in Europe which will help to identify missing elements in research. These pilot sites are

- River Elbe Basin
- River Tisza Basin
- Flash Flood Basins
- the Cévennes-Vivarais Region (France);
- the Adige River (Italy);
- the Besos River and the Barcelona Area (Spain);
- the Ardennes Area (Trans-national);
- River Thames Estuary
- River Scheldt Estuary
- River Ebro Delta Coast
- German Bight Coast)

It can be seen that pilot sites are well distributed over the types of waters like rivers, estuaries and coasts as well as types of floods like plain and flash floods. For each of those sites at least two pilot areas with different properties have been selected to test as many newly developed tools as possible.

The methodologies developed under **FLOODsite** are partly based on a probability based risk analysis. This analysis will require a set of failure modes and related limit state equations for each of the flood defence structures under question. Therefore, the aim of this report is to provide an overview of all structures available within the pilots of **FLOODsite** together with a list of their potential failure modes. The report will not provide the limit state equations which will be used for these failure modes. This will be done in a different report provided by Task 4 of **FLOODsite**.

Each of the chapters of this report will contain the description of one of the pilot sites, starting with some details and maps for the flood prone area. After that, a brief overview of the defence structures and their percentage of the overall flood defence system will be given. The third part of each of the chapters contains a description of failures which have been observed in the past with special consideration of the failed defence structures. Finally, details of the defence structures, a cross section and a list of potential failure modes are given.

2 Pilot Site 'River Elbe Basin'

2.1 Overview of flood prone area

Note: The information provided here was not readable and is requested in a different format.

2.2 Overview of defence structures

The flood defence system of the River Vereinigte Mulde in Saxony consists of earth embankments, to some extent of concrete walls (settlements), polders and artificial flood channels. According to a Saxony wide prioritising of the modernisation and renewal of the built flood risk and defence system after the flood 2002 along the Mulde River a **medium status of priority** had been defined (Table 1; see Saxon State Ministry, 2005).

Table 1 Flood defence structures for the Vereinigte Mulde River in Saxony after the flood 2002.

Parameter	Parameter value(s)
River	Vereinigte Mulde
Community	Bad Düben, Bennewitz, Bockelwitz, Döbeln, Doberschütz, Gleisberg, Grimma, Großbothen, Großweitzschen, Jesewitz, Laußig/Kossa, Leisnig, Löbnitz, Nerchau, Niederstregis, Rosswein, Thallwitz, Trebsen, Wurzen, Zschadraß, Zscheplin, Wellaune
Existing flood defence structure(s)	Earth embankments, dikes
Degree of protection (in % of total defence structures)	HQ5 = 6 % HQ25 = 41% HQ50 = 29% HQ100 = 3%
Estimated damage potential of 100-year flood [T€]	∑ 42141 €
Preferred flood defence measure	new construction: circle dikes renewal: earth dikes, polders, concrete walls,
Protection priority recurrence interval [a]	(>25) ... 50 - 100
Total amount of costs for achieving the protection level [T€]	∑ 45597 €
Status of priority	medium

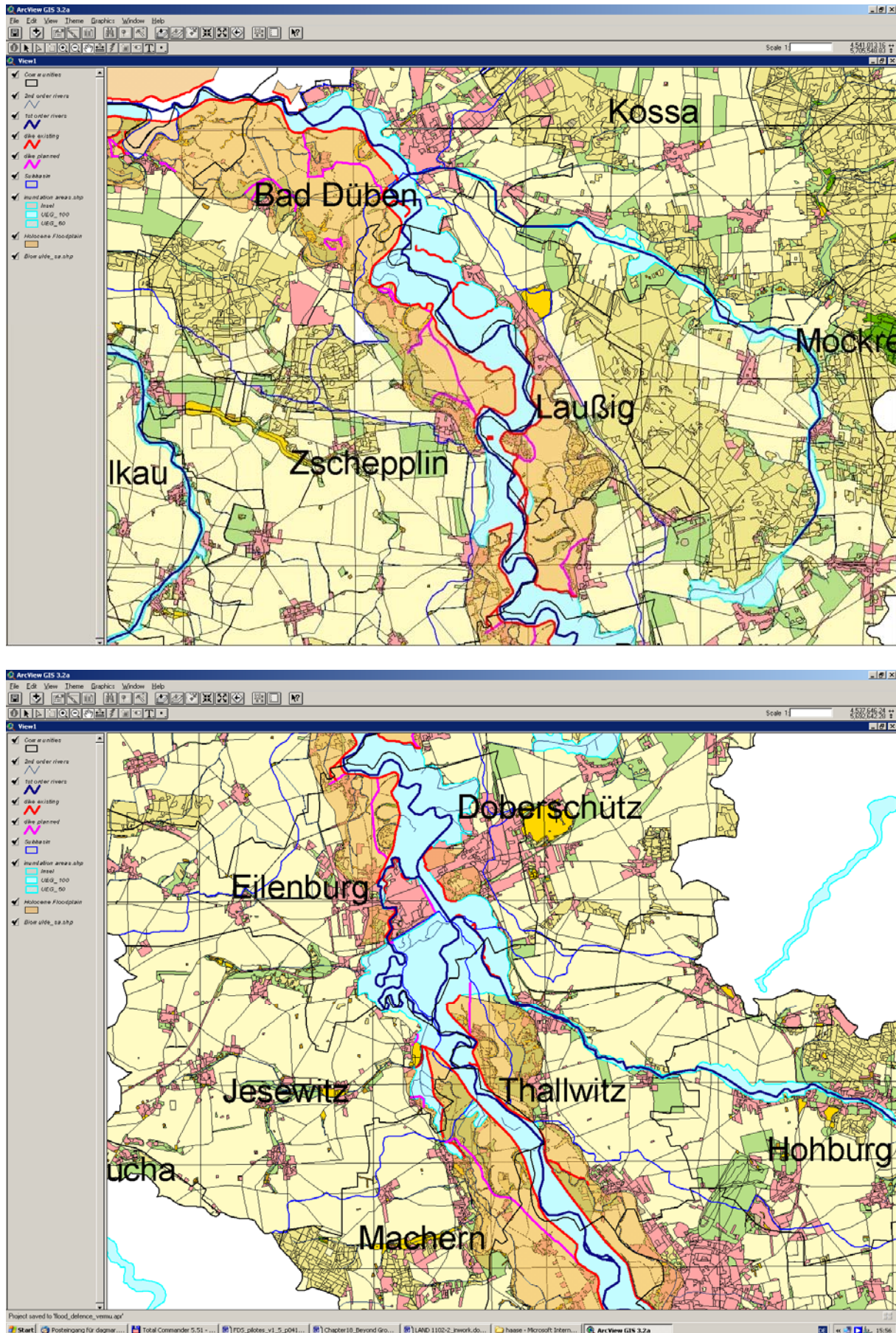


Figure 1 Spatial configuration of the dike system along the River Vereinigte Mulde in Saxony.

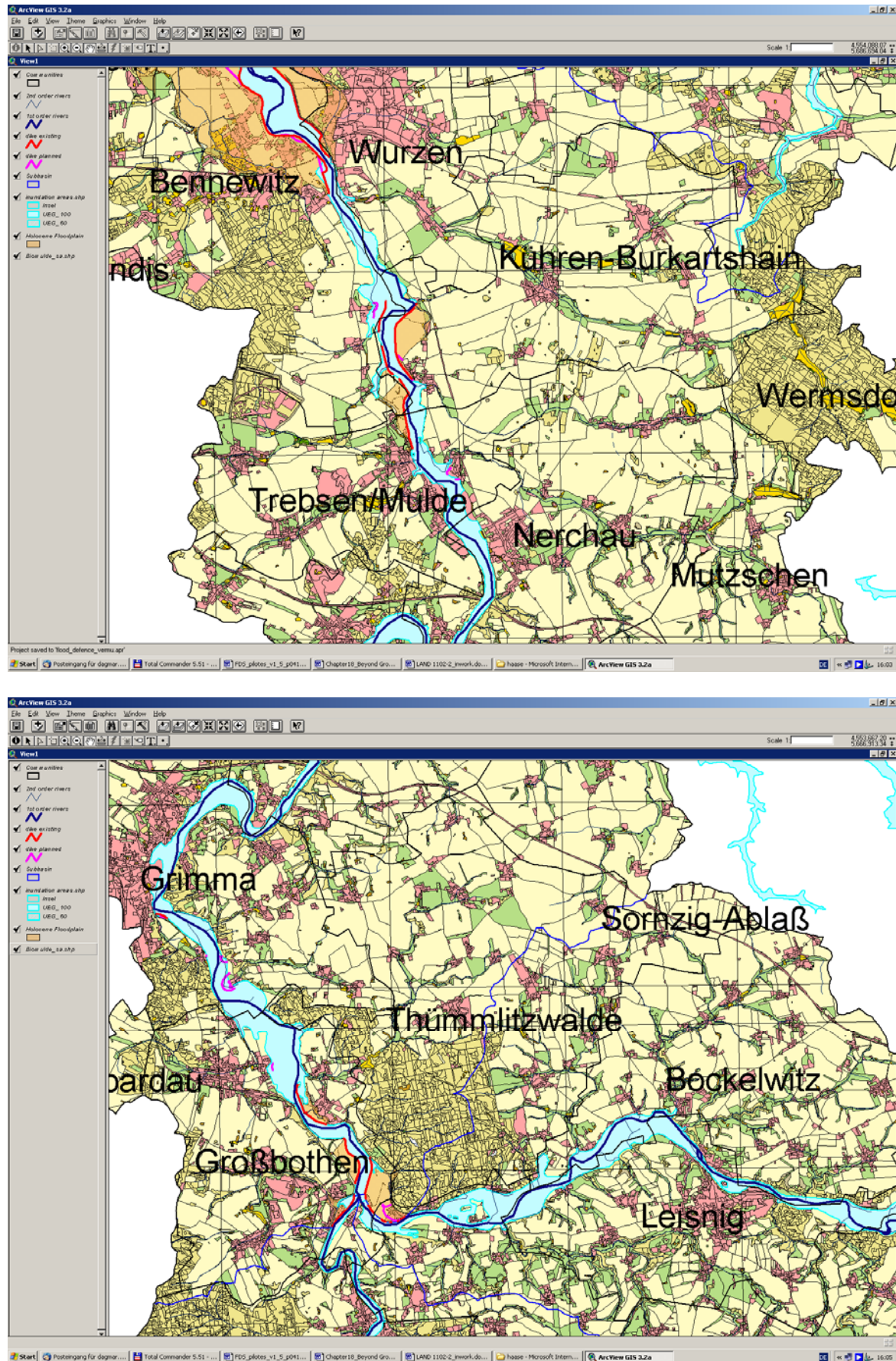


Figure 2 Spatial configuration of the dike system along the River Vereinigte Mulde in Saxony.

2.3 Failure in the past

There was no further information provided other than the four photos below.



Figure 3 Examples for flood defence structures of the River Vereinigte Mulde.

2.4 Details of flood defence structures

No information was provided for this section.

3 Pilot Site 'River Tisza Basin'

The River Tisza is 600 km long in Hungary and there are plenty of interesting sites along it. It is therefore very difficult to define the actual pilot site and provide data for that. The definition of the real pilot site depends very much on the requests of other Tasks within FLOODsite.

3.1 Overview of flood prone area

The pilot site is situated along the right bank of the River Tisza, downstream the city of Szolnok. Extension of the floodplain cells protected by the flood defence section 10.01 covers an area of 125 km². There are 7 communities in the flood cell, namely: Lakitelek, Tizsakécske, Tiszabög, Tiszajenő, Tiszavárkony, Vezseny, Tószeg. Significant agricultural production is going on in the flood cells: vineyards, orchards and arable lands are here. Industrial activities are growing in the Tizsakécske region. Tourism is rapidly growing especially in the larger holiday resorts of Lakitelek-Tőserdő as well as Tizsakécske-Kerekdomb.



Figure 4 Overview of pilot site "River Tisza Basin"

3.2 Overview of defence structures

The flood defence system of the pilot site consists of earth embankments, high ground defence sections and gated sluices crossing the dike. The overall length of the flood defence line of the flood cell is 64.5 km which can be split up to individual sections as follows:

- earth embankment (flood dike) 31.358 km
- high ground defence line 31.043 km
- railway embankment 1.428 km
- road embankment 0.671 km

The pilot site focuses on a shorter section of the defences erected along the right bank of the so called Vezseny band of the River Tisza between Tiszavárkony and Tiszajenő in a length of 15.9 km between 30+000 and 45+900 sections that splits up to 4.1 km earth embankment and 11.8 km high ground defence line.

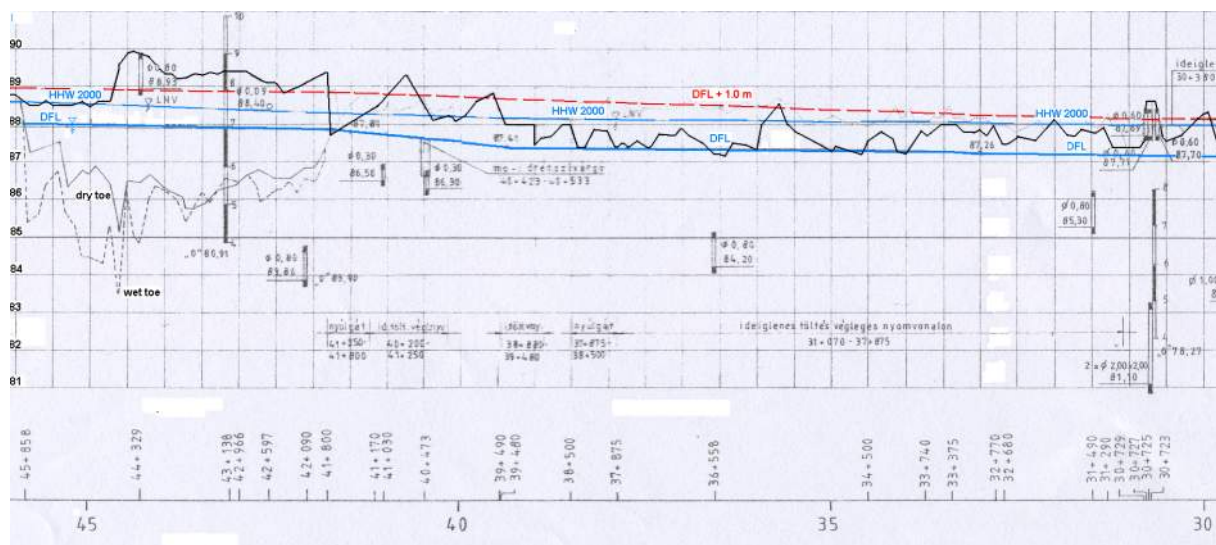


Figure 5 Longitudinal section of the River Tisza within the selected pilot site

In the floodway of the above mentioned primary defence lines there are summer dikes as well. There are all together 6 summer dikes along the 64.5 km flood defence sections, two of which can be found within the pilot site.

The Tiszajenő summer dike is situated between the 32+770 and 39+480 main dike sections, starting at 297,0 fkm ending at 302,2 fkm in a length of 5.85 km, protecting 1018 ha territory. Its licensed height was 2.0 m below the designed (1/100 year) flood, but in fact, it was further erected during earlier floods, therefore the licence was withdrawn in 1997. During the record floods in 1999 and 2000 the summer dike height was reduced to the licensed in several hundred meters' length.

The Vezseny summer dike is situated between the 39+490 and 41+170 main dike sections, starting at 302,2 fkm ending at 314,5 fkm by joining the high ground. Its length is 10.7 km, protecting 750 ha territory. Its licensed height was 2.0 m below the designed (1/100 year) flood, but in fact, it was further erected during earlier floods, therefore the licence was withdrawn in 1998. During the record floods in 1999 and 2000 the summer dike height was reduced to the licensed in several hundred meters' length.

3.3 Failure in the past

There was only one dike failure in the defences of the flood cell at the 12+000 section on March 2, 1879. The registered cause of the failure was improper compaction degree of the dike and the lack of proper maintenance. The above failure spot is out of the pilot site.

3.4 Details of flood defence structures

3.4.1 Earth embankments (for example)

a) Description and cross section

The first flood embankments were built between the high ground sections in the period of 1866-1878. Following the significant floods in 1876, 1919, 1932 the dikes were heightened and reinforced. The dikes are built of silty clay and sandy silt soils founded on similar strata. Depth of covering layer is only 1.2-1.5 m underneath of which relatively permeable layers, silty fine sand and sand soils can be found with sandy clay inclusions.

Height deficiencies along the flood dike relative to the design dike crest (DFL + 1.0 m freeboard) are in the range of 0.3-0.6 m along 44+400 - 47+490.

b) Potential failure modes

Potential failure modes are seepage, cracking, slope slide, wave erosion, overtopping.

3.4.2 High ground defence lines

a) Description and cross sections

The former high ground defence lines represent a special problem recently as the former flood crests have several times been exceeded during the last years' record floods. Originally the almost 65 km defence line developed as a result of interconnections of the high grounds located at Tiszaalpár, Lakitelek, Tiszakécske-Vezseny, Tiszavárkony, Tószeg by earth embankments. The height capacity of these high banks became questionable first during the flood of 1970 when sandbagging in a total length of 16.4 km became necessary in the built-up area of Lakitelek, Tiszakécske, Tiszabög, Tiszavárkony and Tószeg. However, the flood of 1970 was the result of an extraordinary coincidence of extreme floods of all main left hand tributaries of the River Tisza, therefore the extension of flood dikes here was not considered.

In 1999 the flood crest exceeded along the Middle-Tisza the previous records triggering emergency heightening of the former high banks in a total length of 13.9 km. The extreme flood of 2000 reached a flood crest at Szolnok 150 cm over the record of 1970, resulting in the emergency heightening in a total length of 24.2 km within the flood cell housing the pilot study site. Therefore the new development plans contain provisions on the extension of flood embankments along the track of the former temporary heightening, significantly reducing the length of high banks.

b) Potential failure modes

overtopping

3.4.3 Crossing structures – gated sluices

a) Description and cross sections

42+090 – Ø 80 cm gated culvert, reinforced concrete, Réthy-type gate with manually driven rack-and-pinion – drainage of Vezseny;

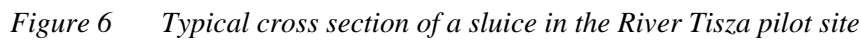


Figure 7 Second typical cross section of a sluice in the River Tisza pilot site

No failures are expected in case of proper maintenance, repair and operation.

4 Pilot Site 'Flash Flood Basins'

A flash flood is a flood that follows the causative event in a short period of time and often is characterized by a sudden increase in level and velocity of a flowing water body. The term “flash” reflects a rapid response to the causative event, with rising water levels in the drainage network reaching a crest within minutes to a few hours of the onset of the event, leaving extremely short time for warning. A threshold of approximately 6 hours often is employed to distinguish a flash flood from a slow-rising flood. Thus, flash floods are localized phenomena that occur in watersheds with maximum response times of a few hours—that is, at spatial scales of approximately 10,000 km² or less, depending on the catchment characteristics. Most flash floods occur in streams and small river basins with a drainage area of a few hundred square kilometres or less. Such basins respond rapidly to intense rainfall rates because of steep slopes and impermeable surfaces, saturated soils, or because of human- (i.e., urbanization) or fire-induced alterations to the natural drainage.

The hazard generated by flash floods is related to both stream response (flood) and landscape response (landslide and erosion). The intense erosion and solid transport associated with these extreme events add to the hazard and strongly influence the quality of soils, waters and ecosystems. This was the case, for example, for the events of Brig (1993), Biescas and Versilia in 1996, Fortezza (1998), Soverato (2000), Val Canale (2003) and Weisseritz (2004).

The twofold consequence of the above observations is that i) catchment wide interventions are generally better suited than local measures to mitigate flash flood risk, given the dispersed nature of the flash flood hazard; ii) catchment wide intervention should cope with both the hydraulic risk and the hydrogeologic risk. Owing to these reasons, focus here will be on catchment wide interventions as an example of structural measures which can be advocated to mitigate the flash flood risk. Catchment wide interventions are very often implemented in mountainous regions frequently hit by flash floods; e.g. in north-eastern Italy, where a large fraction of the population is still living in mountainous areas (and it is therefore highly vulnerable to flash flood hazard), catchment wide interventions represent the first option to mitigate the flash flood risk.

Providing general information on the flood defences in the flash flood basins is very difficult, basically for two reasons:

- the flash flood network includes 4 pilot sites with a region-wide approach (rather than a river or a coast segment approach). This is due to the need to monitor flash flood at any point in a region (distributed monitoring) and to provide a distributed forecast. This seems to place difficulties in getting the right size of information, simply because there is lot of data to report.
- the second issue concerns the size of basins: rarely flash flood occur on moderate-to-large rivers; it is more frequent to have flash flooding on relatively small rivers (and basins). This means that the focus is on the multitude of small-to-moderate basins, where there are, correspondently, a lot of small defence infrastructures. This is exemplified in one of the sites (Adige) by the huge number of check dams used to train the small rivers which can experience flash flood and sediment transport.

Therefore, this section describes the main types of catchment wide interventions implemented in the Adige river basin.

4.1 Overview of flood prone area

Due to the aforementioned reasons it is very difficult to provide detailed information on all pilot areas for the flash floods. Describing flash flood sites means to describe whole regions rather than river catchments or confined estuaries. Flash flood regions where **FLOODsite** works are undertaken are:

- i. the Hydrometeorological observatory in the Cévennes – Vivarais region (OHM-CV), France
- ii. the Vizza basin (field campaign organized under **FLOODsite**), Italy

- iii. the headwater basins of the Elbe river
- iv. the region of Catalonia

Some further description of these regions will be provided within Activities dealing with these basins under Task 1 of FLOODsite.

4.2 Overview of defence structures

Structural measures for flash flood regions include different types of works and interventions aimed at either controlling flood or reducing flood peak. The former include flood defences constructed locally along watercourses and their corridors so as to contain the surplus of water, whilst the latter include catchment-wide interventions to reduce or delay runoff from rainstorms.

Flood control measures include mainly water retention basins, river training interventions and enhancement, rehabilitation and restoration of the river corridor, whilst reduction and delay of runoff can be attained by adequate agriculture and forestry management practices, including also related works.

Catchment-wide interventions can be effective to decrease surface runoff and soil erosion and therefore to reduce flood peak. These interventions should consider a number of basic principles related to the main factors influencing runoff and erosion, namely soil, topography, land cover and use and farming practices.

Soil properties such as texture, structure, organic matter content and pH directly affect soil permeability, and therefore infiltration and runoff. Topography greatly influences the energy of the water particles and therefore runoff speed and timing and erosive potential. Protective effects of the vegetable covering against surface runoff and soil erosion vary according to the vegetable species, which form the covering. In fact, the protective function of forest species is greater than that of fruit trees, and similarly, pasture herbaceous species provide a better protection than arable herbaceous species such as, for instance, cereals, corn and soybean.

In cultivated lands, comprising herbaceous and woody crops as well as tree plantations, surface runoff and erosion are greatly affected by farming practices such as tilling, surface laying-out, type of crops, covering duration, management of residues coming from previous cultivation and preservation of soil fertility. Thus, runoff coefficient is higher when ploughing is performed along the maximum gradient, whilst deep ploughing allows greater water retention and reduction of total runoff. Surface laying-out by means of drainage ditches is one of the most effective systems for surface runoff regulation, as is the use of forage crops and their rotation, whilst ensuring a greater soil covering duration. Also, increase of organic matter released in the soil improves the structure, and thus the physical features of the soil with respect to erosion.

In more natural systems, distinct behaviour can be observed between natural turf forming plants, woods and abandoned lands. The former provides stability and resistance to the soil against erosion. In general, pastures provide higher protection against runoff and erosion than arable crops. However, inadequate exploitation of pastures such as an unsuitable livestock number with regard to grazing practices, as well as burning can have negative effects on vegetation retention capacity, thus increasing runoff and erosion. Woods contribute to regulate runoff and prevent soil erosion mainly because of interception from foliage and litter, greater speed of water infiltration and delayed concentration of water masses downhill. Rain interception is higher for evergreen species, while infiltration is higher in forests with no pastures and old plants. Finally, land abandonment generally results in a significant increase in permanent vegetable cover (mainly shrubs and turf), so insuring higher protection against erosion and runoff. Also, abandonment of forest land, especially in humid regions, can often cause a natural increase of forest cover therefore giving greater protection, although high precipitation can cause in some areas exceptional erosion and landslides.

Based on the above principles, specific actions for agricultural and forest area management as well as for water control works in the catchment are presented in Section 4.4.

4.3 Failures in the past

Flash floods have occurred over various regions. Extreme floods have occurred in the Adige and Tagliamento river basins during the period 1999 – 2004 which will be analysed in more detail within FLOODsite (Figure 8).



Figure 8 Pictures taken shortly after an extreme flash flood occurred on 29 August 2003 on the Upper Tagliamento river basin.

4.4 Details of flood defence structures

4.4.1 Catchment-wide interventions

a) Agriculture and forestry actions

Cultivated lands

- Conserve quick hedges and existing agricultural lay out: dry walls, water storage channels, terraces, etc.
- Avoid the shaping of slopes aimed at changing the size of agricultural holdings.
- Carry out periodical servicing of all water channels, especially of water mains, which impound water from the ditches of agricultural holdings.
- Favour permanent vegetation on water mains and ensure a sufficient section for downflow.
- Promote farming practices aimed at increasing organic matter in the soil by manuring, rotation with improving crops and rational management of residues from previous crops.
- Build cross ditches with appropriate spacing, based on soil texture (e.g. higher for sand than for clay), and slope angle.
- Carry out crosswise tillage where possible, and in case tillage along the maximum gradient proves to be necessary, build cross ditches as well.
- Favour pluriannual rotation of crops planted both using the plough and without it, contouring crops and contour strip-cropping.
- Select crops that ensure longer covering, especially in rainy periods.
- Stimulate the cultivation of species with a greater covering action (e.g. broadleaf species and fast-growing species).
- Favour grass growth on the entire surface, or at least on inter-rows, in case of tree plantations.

Turf forming plants

- Avoid conversion into arable lands where slope is high (e.g. greater than 25%).
- Keep quick hedges and existing cultural lay out: dry walls, water impounding channels, terraces, tracks;
- Favour all cultural practices, which aim at increasing organic matter in the soil (manuring).
- Avoid pasture renewal through fires, since this reduces soil organic matter.
- Regulate grazing through the correct assessment of optimum livestock number.
- Ensure a more homogenous distribution of livestock within the grazing area by a balanced spreading of livestock concentration spots (e.g. watering, feeding and standing spots).
- Use the rotation grazing method as much as possible.

Woods

- Promote and favour forest management plans, including also long term transformation changes.
- Favour more evolved and ecologically stable type of woods, e.g. by stimulating the formation of woods with a more complex structure (woods of different age), or with a different composition (by increasing the rate of mixture of the species), or by increasing the biomass (transformation of coppices into high forests).
- Favour the planning of access to woods, taking into account the different management of woods.
- Favour coppice cuttings with minimum development along the maximum gradient.
- Carry out coppice logging operations in non-rainy periods.
- Design adequate skidding tracks/roads for logging.
- Favour new roads useful to wide forest basins, thus servicing many users and agricultural properties.
- Increase the variability of coppice seedling bearers.
- Favour the conversion from coppice to stable, mixed seedling forest where possible.
- Favour mixed woods by thinning.
- Favour uneven aged woods by increasing their structural complexity.
- Favour forestry use in small areas (e.g. less than 5 hectares, and less than 5,000 m² in case of clear cutting).
- Favour the increase of autochthonous forest species (through selection thinning).
- Favour improvement and care of abandoned woods, possibly by conversion to coppices or seedling.

b) Water control works

Landowners and farmers can implement minor, inexpensive, environmentally friendly works to ensure an efficient protection from runoff. In these works, wood material is most frequently used. This must be durable and mechanically resistant. Thus, larch is most widely used in mountain areas, whilst chestnut and Douglas fir are employed in warmer climates. When carrying out this works not only constructive aspects, materials, use and cost must be considered, but also maintenance, impact to the environment and ecological aspects. Main works are discussed in the following paragraphs.

Check dams

Cross works positioned in drainage beds are useful both to retain coarse floating or suspended material and to rectify the gradient of the course. Main types of check dams are:

- **Timber check dams** - They consist of boles fixed across dug ditches by nails and stirrups. They are used in catchment areas and minor waterways, main drains of cross ditches, and those places where mechanisation is difficult and the material is available on the spot.
- **Timber and loose stones check dams** - They are built with chestnut or conifer boles, nails, and small homogeneous boulders for the face and variable-sized stones for the back filling. They are used in mountain stretches of waterways or main drains placed in steep slopes. They result in an insurmountable obstacle for ichtyofauna.

- **Dry check dams** - They consist of homogeneously sized locally found stones, staggered in such a way that the largest diagonal of the dam is parallel to the axis of the torrent. They are used in gently sloping small catchment areas and drains. They might result in a possible obstacle for the ascension of fish.

From these works, timber check dams have the lowest cost.

Drainage ditches

Coating and canalisation works are used especially in case of watercourses running in sloping and easily erodible substratum. Main types are:

- **Timbered ditches** - They consist of ditches with chestnut boles laid down on the bottom and along the walls of the ditch, fastened with fasteners and nails. They are mainly used as coating of small sized ditches and to protect main ditches from erosion. They enable greater draining speed of surface waters.
- **Ditches made out of timber and stones** - They consist of trapezium-shaped ditches with stones at the bottom and chestnut boles both crosswise and lengthwise the oblique ditch walls. They are used as main drains of surface and road ditches and to convey water from landslides. They make the draining speed of water increase. Their cost is slightly higher than that of timbered ditches.

Surface drainage

It includes works aiming at improving surface water draining in slopes, where erosion due to runoff or to intrinsic water disarrangement makes it necessary to reduce the time of permanence of water. Main types are:

- **Drainage by fascines** - This most common structure consists of tied faggots made up of vegetable material (usually cuttings or branchwood of willow-trees), fixed to semicircular ditches dug along the maximum slope gradient. For deeper drainage pebbles should be laid down at the bottom. It is used in slopes prone to water stagnation and surface erosion. It favours growth of autochthonous vegetation.
- **Filtering wedges** - They consist of wedges dug at the base of the slopes, filled with coarse gravels and boulders and willow trees cuttings. They are used to stabilise the basis of slopes and to enhance water draining. Their cost is much higher than that of fascines.

Wattlings

These structures form horizontal alignments on the slope, consisting of flexible pleached twigs fixed to the ground by cuttings of species which can take root. They are useful to reduce surface erosion in eroded slopes and landslides as well as to reclaim small landslides. Their cost is similar to that of fascines above.

Bench-terraces

They consist of horizontal benches with a slight counterslope, filled with earth, branchwood and willow-tree cuttings in a special arrangement. Terraces are covered with the back-filling coming from the upper terrace. These works can strengthen slopes where numerous landslides occur and in soils prone to water stagnation. Their cost is slightly higher than that of wattlings or fascine-based drainage.

Stone retention walls

They represent the most traditional handwork to strengthen small-sized slopes, especially in rural areas where ground was sloping and the material was easily available. It consists of stones making up vertical facing upstream, and an oblique facing downstream. The thickness at the top of the structure is lower than at its base. They can be used whenever it is necessary to strengthen small-sized slopes, or to change the gradient of slopes (terracing). Although always visible, these walls appear as part of the historical landscape.

Revegetation

This measure enhances vegetation development, thus preventing sudden moisture variations, helping also to prevent erosion and shallow landslides. Main methods include water seeding, revegetation through turf, broadcast sowing with chaff and laying of grass coverage. Amongst these, water seeding is the most diffused and the less expensive revegetation method. It consists of spreading a mixture of water, seeds, organic fertiliser, ligands and soil ameliorating substances over a previously prepared seedbed by means of high-pressure sprinkling machines. This technique is used to completely revegetate bare areas including those due to erosion, landslides and excavation.

Road system

There are two main types of small arrangements to canalise water discharge in roads and agricultural/forest paths:

- **Channels** - They are dug along roadsides. The side downstream conveys the water coming from cross gutters. Their surface can be coated with prefabricated plain concrete or with stones if necessary.
- **Cross gutters** - Gutters crossing the roadway, placed at variable distance and following the slope. They can be simple ditches of earth or be coated with boles cut into half and fixed to the gutter. They keep surface water away from the roadway itself.

4.4.2 River training interventions

River training interventions are widely applied for prevention and mitigation of flash floods. Their main aim is to control and optimise the water discharge regime in watercourses by limiting its dynamic energy, therefore managing and controlling the morphological evolution of watercourses. These interventions also have the function of reducing solid transport and the natural processes of bed and bank erosion along the watercourses.

In this section, additional emphasis is placed on the application, where possible, of techniques with a low impact on both the ecological aspects of river habitats and on the landscape. To this end, the use of vegetal species (bioengineering techniques) and rocky material available on the torrent bed should be pursued. This generally contributes to reducing pollutants in the flooding waters whilst creates or preserves natural corridors, which favour the diffusion and preservation of different living species within the riparian ecosystem.

An overview of the most common flood prevention and mitigation structures and interventions along watercourses is given below. The various works have been originally classified according to their transversal and longitudinal position with respect to the watercourse.

In general, combined interventions are more efficient than single ones. It should also be born in mind that these interventions should not cause non-mitigating environmental impacts nor have negative effects in adjacent watercourse reaches.

a) Transversal protection works

Check dams

These are low structures built with erosion-resistant materials (stones, gabions, concrete, logs or other), which slow water flow and increase deposition. Check dams decrease the morphological gradient of the torrent bed. They reduce the water velocity during flood events by increasing the concentration time of the hydrographic basins and reducing the flood peak and solid transport capacity of the water flow. They also help mitigating erosion processes and controlling solid transport, so favouring the prevention of landslides on natural slopes and artificial banks. Check dams are often constructed in succession along the watercourse to provide stabilisation of the bed over long distances. Check dams usually require additional protection structures in the bed or on the banks to provide defence from undermining and breaking.

Check dams may represent a physical barrier for the diffusion of fish along watercourses. To mitigate these effects their height should be low or, alternatively, ramps or lateral corridors should be built. Main types of check dams are discussed below.



Figure 9 Cemented stone or concrete dam

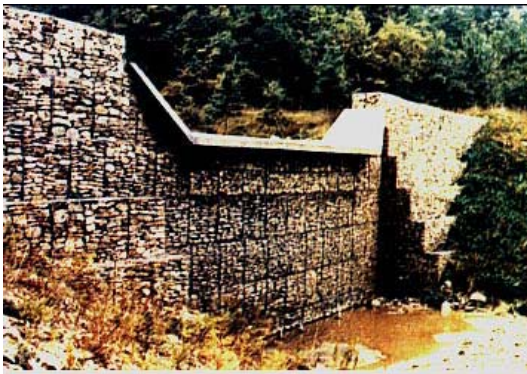


Figure 10 Gabion dams



Figure 11 Wood or rocks and wood dams

Cemented stone or concrete dams

These structures can be constructed along the entire watercourse length because of their adaptability to different morphological and hydrodynamic conditions of the torrent bed. Their strong rigidity enables them to have considerable dimensions. Both the building yard and the dam itself cause however a significant environmental impact.

Gabion dams

They consist of adjacent wire baskets filled with heavy stones that can sometimes be found in the torrent bed. They can reach a height of up to 10 m and can be adapted along the entire hydrographic network. Other types are more flexible, thus adapting better to more unstable watercourse bed sites. The preparation of the building yard causes a significant environmental impact. However, the dam itself permits a rapid re-naturalising by natural plants or cuttings.

Wood or rocks and wood dams

They consist of a well-organised structure made of rocks and wood poles, in general any kind of wood with strong resistance to water and rocks that can be found in the area of construction. Their height does not usually exceed 2 m. For this reason they are usually built in the upper portion of the hydrographic network or along small tributaries with low or intermittent water flow. Partial re-naturalising of this type of dam is possible, thus reducing its environmental impact.



Figure 12 Dry stone/wall dams

Dry stone/wall dams

They consist of a well-organised structure made of big stones, which might be found near the site of the structure. These dams are usually built in the upper reaches of the hydrographic network or along small tributaries where the intermittent water flow with low discharge favours their durability. Their height does not usually exceed 2 m. Their environmental impact is relatively low and has the lowest cost amongst check dams.

Out of these, those not made of cement or concrete are more flexible, thus adapting better to more unstable watercourse bed sites. They also integrate better in the natural environment.

Sills

Sills are structures built across the bed of a stream to prevent scour or head cutting. They are used along river stretches with a medium-low morphological gradient subjected to bank and bed erosion (that may cause instability on natural and artificial embankments or on other existing hydraulic works), where the preservation of torrent bed elevations does not require the construction of check dams. Sills are usually accompanied with bank protection structures upstream to guarantee durability of their anchorage and prevent undermining of the embankment.

Sills cause in general a low environmental impact because of their low height (often under the water surface). Amongst the most common types of sills discussed below, those using natural materials (wood, rocks or gabions) favour vegetation growth, thus permitting a higher integration with the watercourse environment than when using concrete.



Figure 13 Concrete or stone sills

Concrete or stone sills

These have been the most commonly used sills so far because of ease of construction, despite having the highest cost. They can be constructed in all morphological conditions, especially in those of lower watercourse reaches.



Figure 14 Gabion sills

Gabion sills

Sills made of gabions can be applied to many different hydrodynamic conditions. Their limited height allows for a considerable width. Sometimes, building of gabions can be carried out with the rocky materials available along the torrent beds.



Figure 15 Sills made of blocks

Sills made of blocks or blocks anchored to ground or wood and rocks

They are mostly built in the upper mountainous reaches of watercourses or in sites with morphological constraints. In general, any kind of wood with strong resistance to water that can be found in the area of construction can be used, although it is recommended to use such species as larch, chestnut and natural or treated resinous plants. Sills made of gabions or rocks and wood facilitate the hydraulic fitting along torrent beds with strong morphological modifications in relation to the flexibility of their structures.

Beam dams and screen dams

Their main aim is to retain rock, earth and vegetal material transported along the watercourse during strong alluvial events so as to reduce the downstream discharge effects and prevent the obstruction of narrow hydraulic sections, covered stretches or check dams with fixed outlets that could cause catastrophic flooding. These dams are constructed in alluvial fans, stretches with a steep slope, wooded areas and areas undergoing frequent mass movements (e.g. debris flows and mudflows), although most often in narrow torrent beds at the end of the valley before the alluvial fan or flat area. Most of the transported materials sediment in a retention basin or pool to be built, whose capacity must be based on the upstream watercourse and catchment characteristics.

Beam dams and screen dams must be constructed with a strong structure in concrete, whilst the retention parts for sediments and vegetation may be built with different materials offering sufficient resistance against the impact stress caused by transported materials. Accompanying structures for protection of banks and foundations must also be built. In addition to regular maintenance work of the structure, the material filling the sedimentation basin or pool should be removed after flood events to recover its storage capacity.

These structures produce a significant environmental impact, although they enable fish migration through their openings. The most common types of dams are the following:

- **Screen dams with vertical steel or concrete bars** (Figure 16): They are used mainly for retaining vegetal materials. Screen dams can be built in different reaches of watercourses. These dams offer a high resistance against the transported materials. Alternatively, dams with bars made of wood can be built along small torrents or agricultural or forest channels.
- **Beam dams with central pylon bars** (Figure 17), vertical opening (Figure 18) and horizontal steel bars (Figure 19) - These dams are constructed with concrete and steel structures. Their main objective is to prevent mass transport that could affect urban areas.



Figure 16 Screen dam with vertical bars



Figure 17 Beam dam with vertical pylon bar



Figure 18 Beam dam with vertical opening



Figure 19 Beam dam with horizontal bars

Groynes

Groynes are small dykes reaching from the bank into the river. They are mainly used to protect the riverbank from erosion or other protective structures from undermining by diverting the stream flow and dissipating its energy. Sedimentation between groynes is also favoured, thus creating natural banks that protect the riparian zones. Groynes are also built to recreate natural meandering condition in the watercourse beds. Groynes are usually concentrated along reaches with a medium-low morphological gradient. The number and distance of groynes is a function of their length, the watercourse hydraulic characteristics and the sediment discharge.

Groynes cause a low environmental impact to watercourses because these structures are partially or completely submerged by water. When these structures stand out above the water level for long periods, revegetation by using bioengineering techniques contributes to reduce the environmental impact. The most common types of groynes consist of:



Figure 20 Groyne made of concrete or cemented stones

Groyne made of concrete or cemented stones

They are usually adopted along torrent beds subjected to strong bank erosion and are usually carried out in the lower portion of watercourses. The short transversal dimension of groynes cause limited environmental impact on watercourses. However, they cannot be re-naturalised through bioengineering works. These groynes have the highest cost.



Figure 21 Groyne made of gabions

Groyne made of gabions

Because of their flexibility, groynes made with gabions can be easily applied along torrent beds subjected to strong bank and bed erosion. They cause a limited environmental impact. Gabion groynes have the lowest cost.



Figure 22 Groyne made of prefabricated or natural blocks

Groyne made of prefabricated or natural blocks

Groynes made with non-cemented prefabricated or natural rocks are usually built along the upper reach of watercourses. Natural blocks available inside the torrent bed can be used. They cause a low environmental impact, especially when using natural materials (rocks, woods, plants) by bioengineering techniques, hence allowing for revegetation.

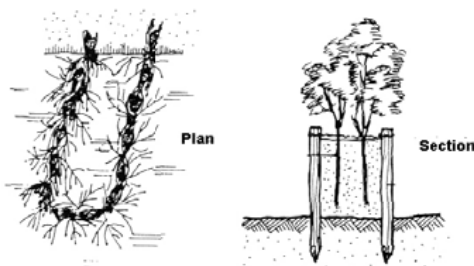


Figure 23 Groyne made of rocks and cuttings or wood and cuttings

Groyne made of rocks and cuttings or wood and cuttings

These groynes can be applied in different morphological and hydraulic conditions in order to solve small erosion-related problems involving the upper reach of watercourses. They can usually be constructed with materials available on site. It is recommended to reinforce them with steel cables and poles to ensure their resistance to water flow. They are easily vegetated and cause limited environmental impact.

Channel lining

Channel lining concerns the protection of watercourse beds or banks against erosion by means of concrete, soil cement, rocks or bituminous or plastic material. Channel-lining works enable high water flow velocity. This ensures sediment load transport, thus preventing episodes of deposition and aggradation in the bed and avoiding erosion of torrent bed and banks. These works are recommended in catchments highly prone to erosion, especially in urban and alluvial fan reaches. They are also applicable to the regularisation and stabilisation of channel bed reaches prone to frequent divagations and altimetrical variations (e.g. torrential streams in semi-arid regions).

Amongst the most common types of channel lining structures, those made of wood, stones and gabions permit to assure a longer durability because of their better flexibility with respect to foundation shifts and settlements.

The realisation of channel lining works must be carried out only in particular conditions where there is need for defence and protection, since their construction produces a remarkable environmental impact on the stream reach, especially for concrete and cemented lining. The use of constructive typologies which make use of natural materials (wood, stones or gabions) reduces or minimises the environmental impact, ensuring both a partial revegetation of these structures and the continuity of interactions between the fluvial habitat and those surrounding it.

The most common techniques and construction materials used for channel lining include:



Channel lining made of concrete, cemented stones

Channel lining requires setting up strong building yard actions in order to reshape the water course and its hydraulic transversal sections. Use of concrete and cemented stones produces a strong environmental impact on the watercourse habitats, and implies the highest cost with respect to other channel lining works.

Figure 24 Channel lining made of concrete, cemented stones



Channel lining made of gabions

In some cases, carrying out of channel lining with gabions have involved long stretches of water courses subjected to erosion processes that produce frequent three-dimensional modification of their course. These interventions encompass strong building yard actions in order to reshape the watercourse and its hydraulic transversal sections. Consequently, they produce a strong environmental impact on the torrent habitats during the construction phase.

Figure 25 Channel lining made of gabions



Channel lining made of wood and stones or wood

Channel lining using rocks or woods are mainly applied where ecological and environmental respect requires using natural materials. This type of lining consents limited building yard actions fitting the intervention to the natural characteristics of the watercourse. Channel lining with rocks or wood, which are generally available locally, permit to re-naturalise the watercourse. Insertion of cuttings among the structure allows for stronger channel lining. Channel lining using wood and stones implies the lowest cost.

Figure 26 Channel lining made of wood and stones or wood

b) Example of functioning of check dams during a flash flood event

Open-check dams are check dams having an opening in their central part, often provided with grids or bars, with the function of regulating the solid discharge. In fact, open-check dams temporarily retain the sediment transport intercepting coarser material while letting finer grain-size sediment pass through.

According to the historical development of these structures, and in connection with their specific purpose, the open-check dams can be divided into two large categories: (1) Beam dams; and (2) slit dams, characterized by a different management of the sediment transport:

- A beam dam (Figure 27) has wide openings and its purpose is mostly filtering sediments and logs; the retention effect is due to a selective mechanical sieving of the bigger particles. Beam dams were therefore originally conceived assuming the material to be retained by a mechanical sieving exerted by the grid. The opening of the dam was designed as large as possible, in order to avoid any contraction of the torrent section and to increase as much as possible the surface interested by the retention. The distance between the grids varies in the literature—from 1.2 to 1.5 times—to 3 times the diameter of the material to be retained. Quite quickly, the material accumulated upstream of the grid obstructs the opening of the check dam, hindering the attempted sieving and filtering effect. The presence of a considerable amount of vegetal material might accelerate the clogging process. Therefore this type of check dam, after a certain time from the beginning of the hydrological event, tends to clog and starts working as a traditional closed-check dam.
- Slit dams (Figure 28) are based on a completely different criterion. This kind of check dam presents one or more narrow vertical openings, going from the dam's base up to the weir. The effect is mostly dosing the sediment transport rate and it is obtained by means of a backwater effect that allows most of the particles to deposit upstream of the dam (hydrodynamic sorting).

Open-check dams are usually designed according to clear-water principles, without considering the effects of sediment transport. In this respect, the slit width b is chosen in such a way that the specific energy upstream of the dam, under uniform flow conditions, H_0 is less than the minimum specific energy in the slit section



Figure 27 Example of open beam check dam



Figure 28 Example of open slit dam

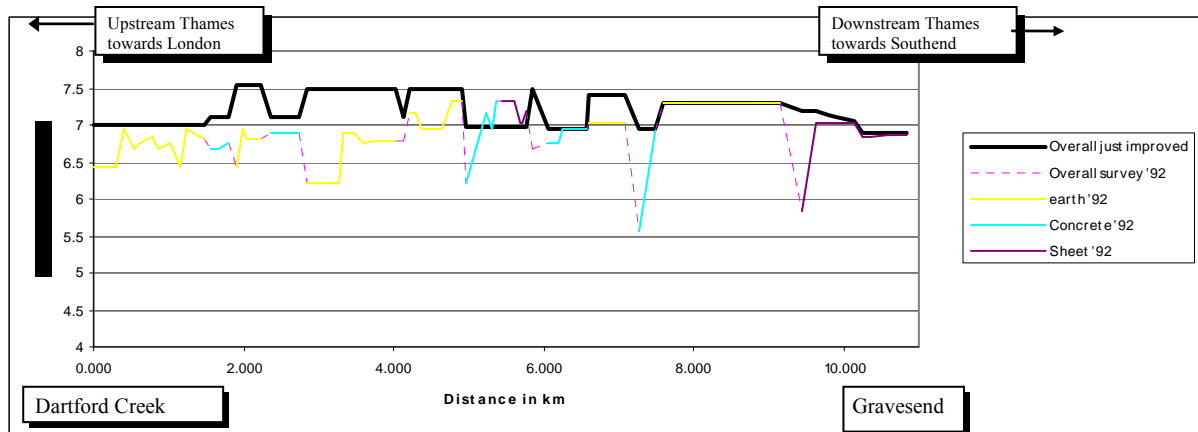


Figure 30 Elevation of the flood defence line between Dartford Creek to Gravesend. The figure includes the elevations per structure type as surveyed in '92 versus those indicated in the as designed / as constructed drawings.

5.3 Failures in the past

Within the pilot site River Thames the following failures have occurred in the past:

1953 – Overtopping of the crest, seepage into fissures and cracks followed by decreasing shear strength and slope instability of earth embankments. Crest levels then corresponded with the current lower crest. Improvements aimed to provide a 1 in 1000 year standard of protection.

1953 – uplifting and piping behind the earth embankments of the impermeable clayey and peaty layers. As part of the improvements, pipes were applied in ditches behind the embankments reaching into the water conductive layers below the impermeable layers. The water can drain into the ditch, thus relieving the hydraulic uplifting pressures underneath the impermeable layers. Presumably filters are applied at the bottom of the pipes to prevent erosion of the material in the water conductive layer.

1970s-1980s – during the construction of the improvements a stretch just downstream of Gravesend failed due to slope instability of earth embankments. The construction works were carried out under strong time limitations. Due to this time constraint, the weight of the new defences was applied too quickly, leading to insufficient drainage of the weak clayey and peaty layers (and therefore insufficient recovery of the strength of the foundational soil).

5.4 Details of flood defence structures

5.4.1 Earth embankments

a) Description and cross section

The earth embankments along the Dartford Creek to Gravesend defence line typically have two crests. In the late '70s and early '80s the Thames Estuary defences were improved. The lower riverward crest is the old pre-improvement defence line, the higher landward crest has been constructed as part of the improvements. The defences are founded on weak clayey and peaty soil layers with a thickness in the order of magnitude 14 to 20 m. Those impermeable layers are in turn founded on a water conductive layer formed by sandy or chalky layers. To avoid the occurrence of deep seated slip circles during and after construction, berms were applied on the inside and outside toes of the defences to provide for sufficient stabilising weight. See Figure 31 for an example cross section and Figure 32 for a recent photograph.

The compressible clayey and peaty soil layers lead to substantial settlements due to the '70s and '80s improvements. Settlements in the order of magnitude of 0.5 to 1 meter took place over about 30 years. In some of the areas, local people use the earth embankments for motor crossing, leading to damage to the crest and grass on the inside slope. Fissuring and cracking caused problems in the 1953 storm. The course of the Thames is subject to change, causing different local hydraulic boundary conditions

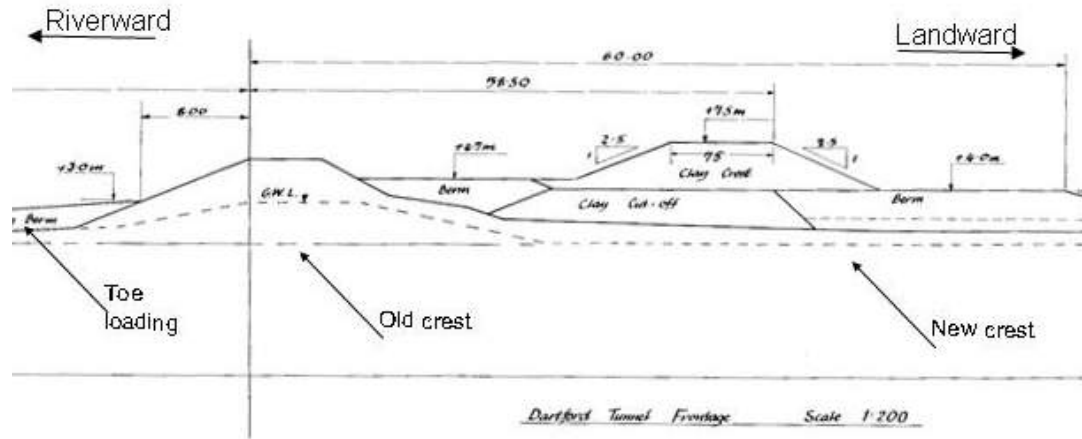


Figure 31 Typical cross section of earth embankments between Dartford Creek to Gravesend at the 'River Thames' pilot site



Figure 32 View along the embankment at the 'River Thames' pilot site

b) Potential failure modes

The potential failure modes of earth embankments are seepages into fissures and cracks. Instability of slope occurred and uplifting and piping behind the embankments have been located. Figure 33 shows a picture from 1955 of a crack in the previous earth embankment.



Figure 33 Example of “crack” in earth embankment crest before the landward improvements were built at the ‘River Thames’ pilot site

5.4.2 Concrete walls

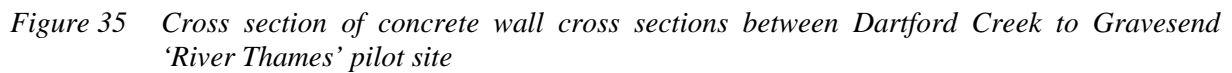
a) Description and cross section

The concrete walls were built as part of the Thames Estuary flood defence improvements in the ‘70s and ‘80s. An example of a concrete wall along the Dartford Creek to Gravesend defence line is given in Figure 34. Sheet piles applied underneath the concrete structure prevent seepage/piping and mobilise the soil between the piles for extra stability. Variations of the concrete structure shown in the figure are:

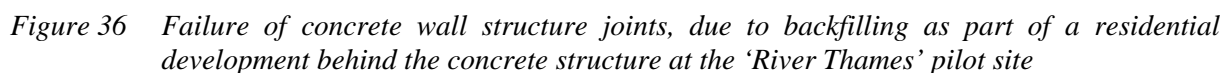
- Application of a single sheet pile sometimes in the form of an anchored sheet pile.
- A mirrored version of the structure is shown in Figure 35, with the vertical wall on the landward side rather than the Thames side.



Figure 34 Concrete wall between Dartford Creek to Gravesend at the ‘River Thames’ pilot site



Problems with concrete wall structures are currently mainly caused by backfilling of the concrete wall as part of residential developments. The concrete wall is not designed for this type of loading, resulting in: cracking / spalling of the concrete (there is no reinforcement to deal with the tensile stress on the opposite side of the wall), uneven settlements and the associated failure of joints. See Figure 36 for an example of joint failure.



5.4.3 Sheet pile walls

a) Description and cross section

Sheet pile walls were built/improved as part of the Thames Estuary flood defence improvements in the '70s and '80s. Figure 37 and Figure 38 show an example of a sheet pile wall applied along the Dartford Creek to Gravesend defence line. In some cases old frontages in the form of for instance masonry walls are still present in the ground behind the current sheet pile walls, the space in between the walls backfilled with concrete. In other cases, the old frontage was used to anchor the sheet pile walls in or the rubble of the old frontage was used as backfill material. At the time of the construction of the defence improvements, parts of the frontage between Dartford Creek and Gravesend were docks. Because of the function as a dock, besides the typical sheet pile wall a large variation of sheet pile wall cross sections and combinations with concrete structures occur. By now, the frontages are not in use as docks anymore. Another type of sheet pile wall occurs without anchors as part of an earth embankment. The sheet pile wall then provides an additional 0.5 to 1.0 m freeboard without demanding extra space associated with a sloped elevation.



Figure 37 Examples of sheet pile walls between Dartford Creek and Gravesend at the 'River Thames' pilot site

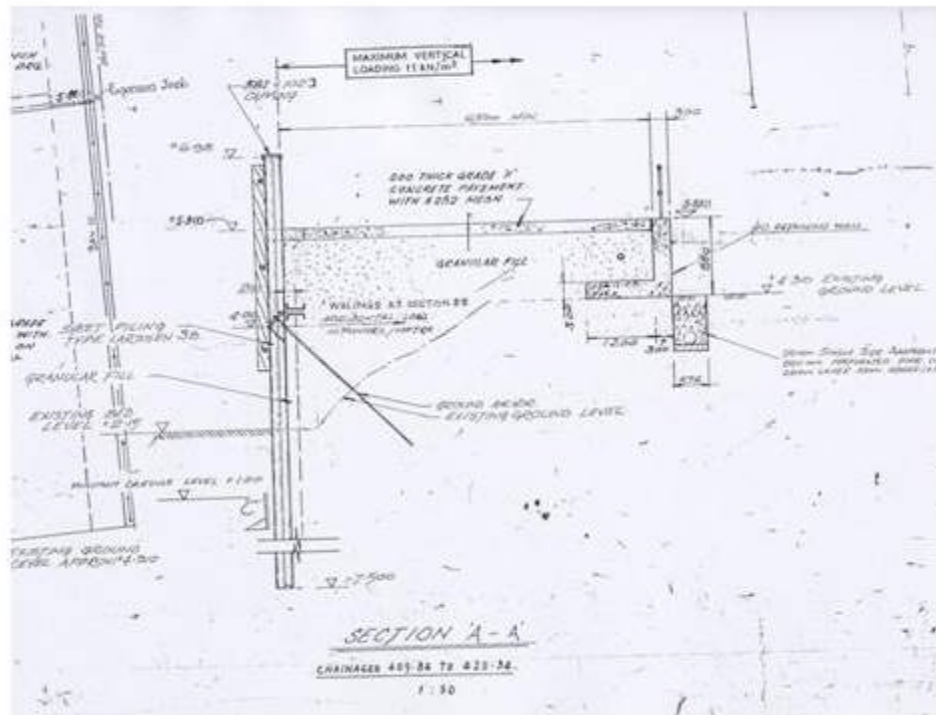


Figure 38 Cross section of sheet pill wall between Dartford Dreek and Gravesend at pilot site 'River Thames'

b) Potential failure modes

The sheet pile walls have not been painted or otherwise significantly maintained during their life span. As a result Accelerated Low Water Corrosion (ALWC) has corroded the surface of the sheet pile walls. Corrosion has also reduced the diameter of the ground anchors. Other problems are caused by residential developments which damage ground anchors. At Greenhithe this has caused one out of four anchors to fail.

5.4.4 Floodgates

a) Description and cross section

As the frontages between Dartford Creek and Gravesend partly had a function as a dock during the '70s and '80s improvements, sufficient passages from the floodplain to the Thames had to be ensured. For this reason, a large number of floodgates were built into the flood defence line (Figure 39). By now, some of these floodgates are permanently in a closed position and some were replaced by a fixed defence line as part of commercial or residential developments. Others are part of a telemetry system and need to be closed to prevent flooding. The width of the opening varies from smaller gates, for instance 5 m, to larger gates of e.g. 20 m.



Figure 39 Example of a floodgate for pilot site 'River Thames'

b) Potential failure modes

Underneath the sill of the floodgates seepage sheet pile screens are applied. Besides problems that might occur in closing the gates, another problem might be the seepage through the gates.

6 Pilot Site 'River Scheldt Estuary'

6.1 Overview of flood prone area

The trans-national Scheldt Estuary extends from the upper reaches near Gent in Belgium to the lower reaches and mouth at Flushing in The Netherlands (Figure 40). The Dutch part is called "Western Scheldt" and there the estuary is a meandering multiple channel system, with intertidal islands and intertidal areas at the inner side of channel bends. The Belgian part is called "Sea Scheldt", and this part of the estuary is a single meandering channel, with intertidal areas at the inner part of bends, but without intertidal islands. The upper parts of the intertidal areas along the shores of the estuary host fauna and flora-rich salt marshes. The lower intertidal areas are important feeding grounds for birds and resting areas for the increasing population of seals. This estuary is of strategic importance as a major shipping artery, hosting the harbour of Antwerp, as well as providing an access route to the harbour of Rotterdam via the Rhine-Scheldt canal.

As large parts of the Netherlands are at or below sea level, the country has a long standing history in flood control and management. The country is divided in so-called 'dyke ring areas': areas enclosed by dykes, dunes and/or higher grounds for which a specific flood safety standard has been attributed (see Figure 41). With regard to the Dutch part of the Scheldt study site, 5 dyke ring areas are relevant:

- No. 29 (Walcheren)
- No. 30 (Westelijk Zuid-Beveland)
- No. 31 (Oostelijk Zuid-Beveland)
- No. 32 (Zeeuws Vlaanderen)
- No. 33 (Kreekrakpolder)

For No. 32 the higher ground boundary is located in Belgium. The other dyke ring areas are entirely located in the Netherlands. For each of these dyke ring areas a safety standard of 1:4,000 has been given.

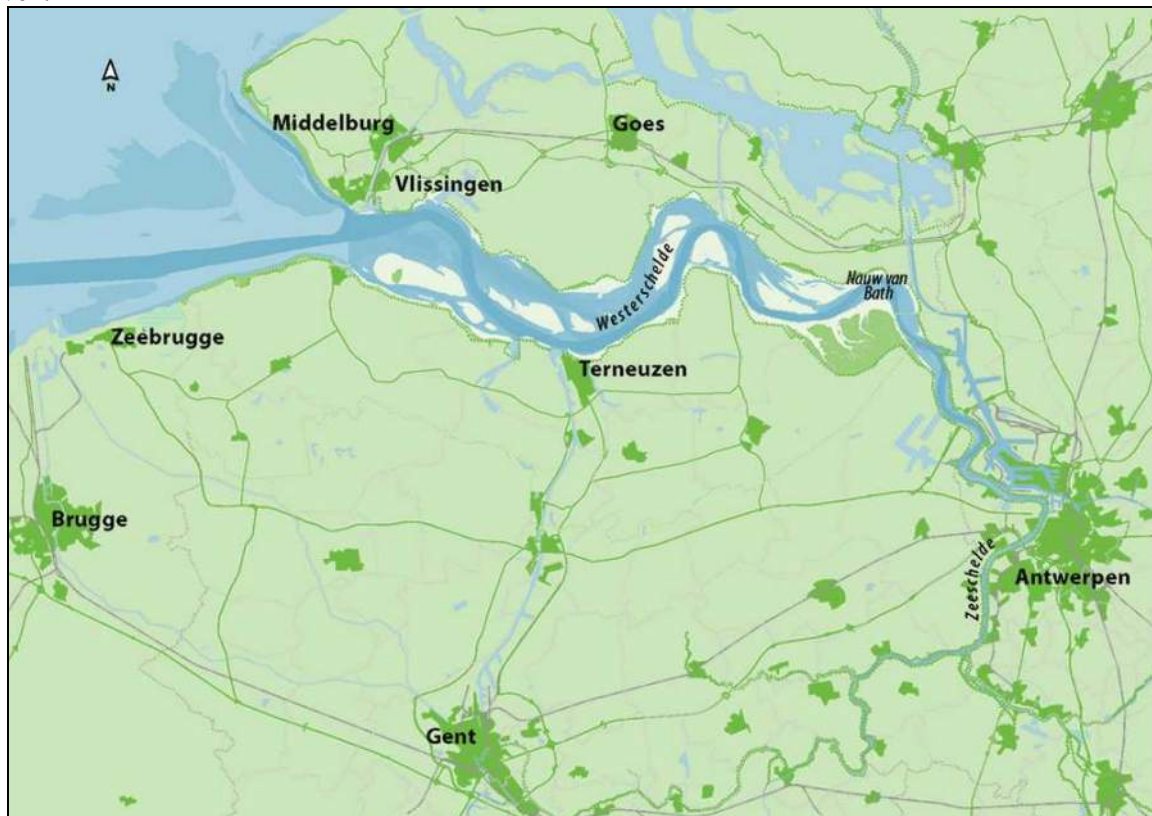


Figure 40 Overview of the pilot area Scheldt

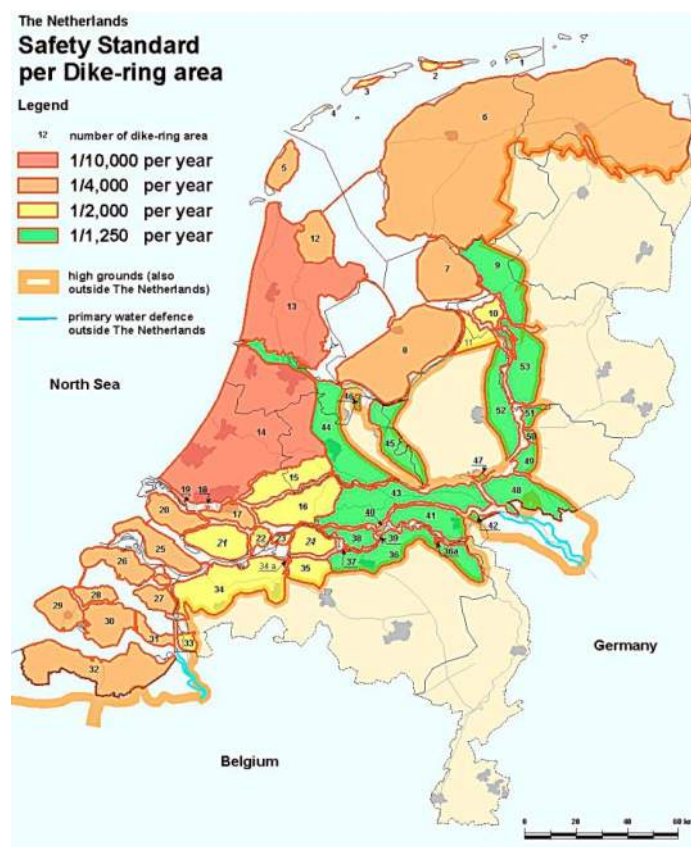


Figure 41 Dike ring areas of the Netherlands

6.2 Overview of defence structures

Figure 42 shows the flood defence structures on the Westernscheldt (Dutch part of the study area).

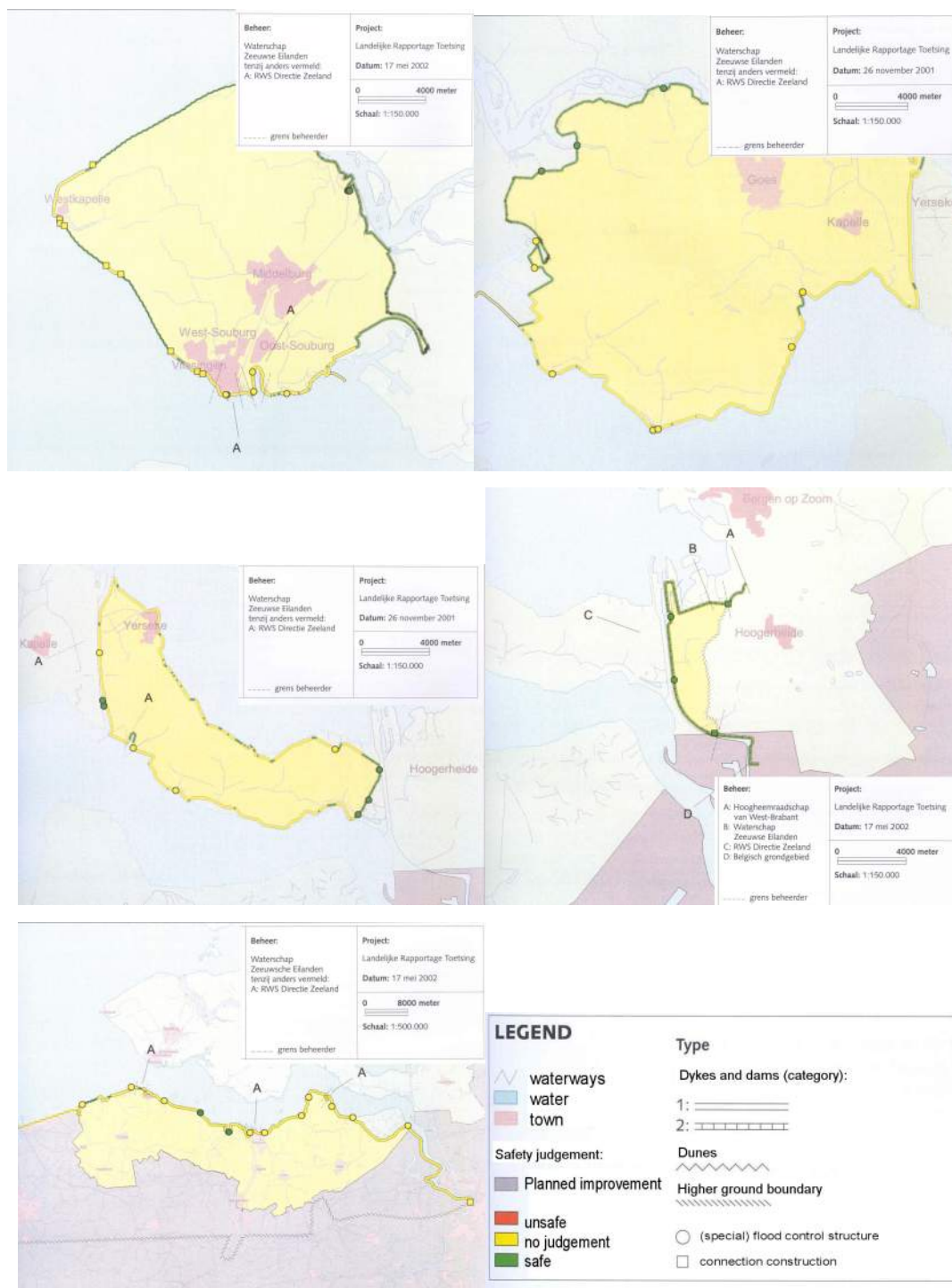


Figure 42 Flood defence structures along the Westernschelt

Dyke ring area	length of dykes	length of dunes	number of flood control constructions
No. 29 (Walcheren)	45.7	23.3	6
No. 30 (Westelijk Zuid-Beveland)	88.6	0	13
No. 31 (Oostelijk Zuid-Beveland)	53.4	0	7
No. 32 (Zeeuws Vlaanderen)	79.4	6.8	12
No. 33 (Kreekrakpolder)	20.6	0	2

There are basically 4 categories of civil engineering constructions discerned in Holland:

Type 1: constructions that fulfil a flood control function completely independently

Type 2: constructions that fulfil a flood control function in combination with a soil construction

Type 3: constructions that have to fulfil flood control after another construction has failed

Type 4: constructions without a flood control function, but that can damage the flood control after failure.

Within the study area numerous constructions are present, such as sluices, floodgates, pumps and seawalls.

6.3 Failure in the past

In 1953 a violent storm breached the dykes in many places in Zeeland and in Flanders and caused a disastrous flooding. The number of casualties was more than 1800. Approximately 72,000 inhabitants from the south-western disaster area had to be evacuated for a long period of time. More than 47,000 head of cattle and 140,000 poultry drowned. Three thousand houses and 300 farms were destroyed and more than 40,000 houses and 3000 farms were damaged. Almost 200,000 hectares of land were inundated. The total material damage is estimated to have been 1 ½ billion Dutch guilders. (eq. 3.1 billion € taken into account the inflation rate over 50 years).



Figure 43. Overview of inundated areas after the 1953 storm surge

In total around 100 breaches were counted after the storm. The lower and less well-maintained dikes on the south side of the polders were the first to go. The first dikes to burst were at Kruiningen, Kortgene and Oude Tonge. At Stavenisse, the force of the waves gouged out an 1800 m breach. And even in Noord-Brabant, near Willemstad, Heijningen and Fijnaart, the dikes could not withstand the

storm. It was the same story in the Hoekse Waard of Zuid-Holland at 's-Gravendeel, Strijen and Numansdorp.

Most of Schouwen-Duiveland was flooded. Only the dunes at the head of Schouwen and a few polders near Zonnemaire stayed dry. Except for the land on the lee side of the dunes and a few polders near Melissant and Dirksland, the whole of Goeree-Overflakkee was covered with water. Houses collapsed and were swept away by the current. The rising waters even caused the total destruction of some small villages. The hamlets of Schuring near Numansdorp and Capelle near Ouwerkerk were obliterated by the water. Not a house was left standing. Other places were more fortunate. At Colijnsplaat, where men had been trying for some time to keep the flashboards from breaking, a barge suddenly broke loose from its moorings, ended up in front of the cut and worked as a breakwater, sparing the town.

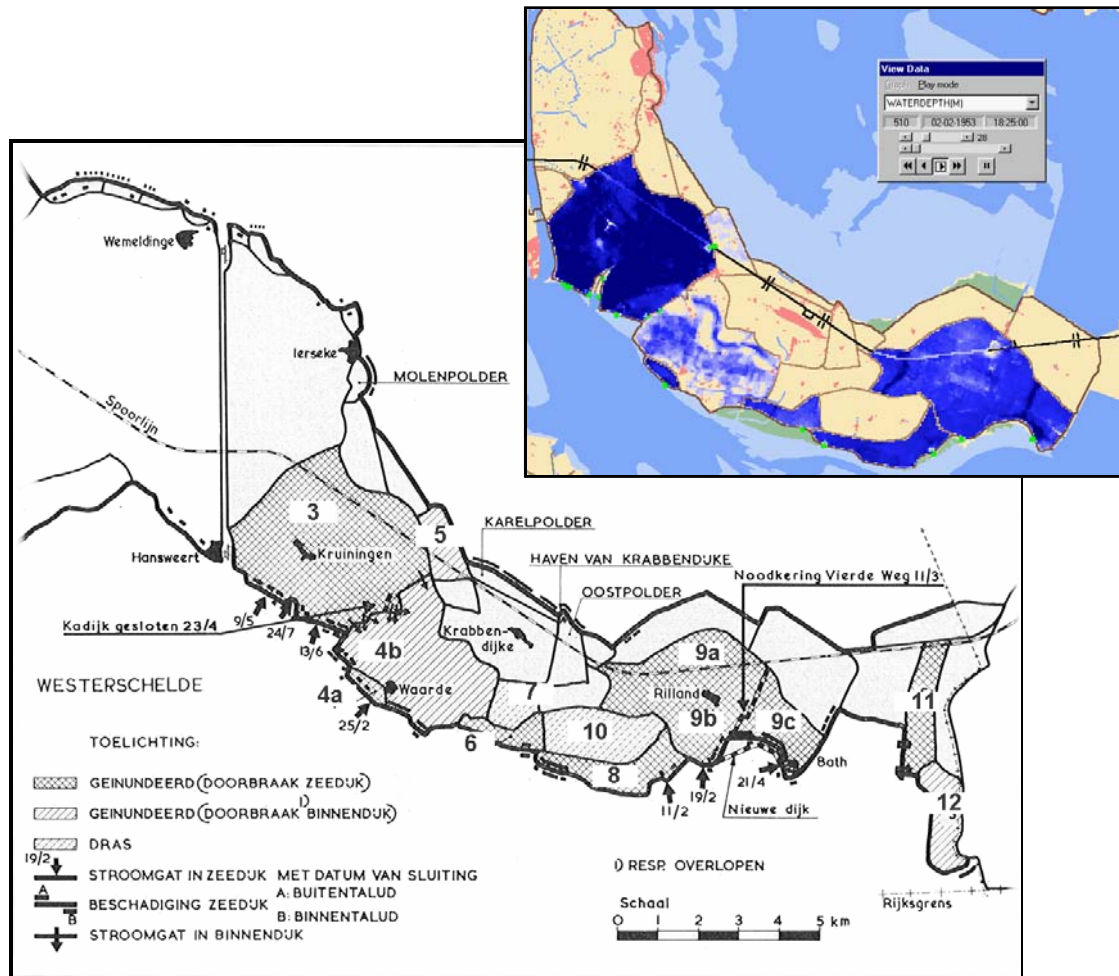


Figure 44 Eastern Zuid-Beveland showing the locations of dyke failures during 1953 flood. Inset: Delft1D2D flood model result.

Most of the flood defence structures (dykes) that failed were just too low. The overtopping of the water caused first of all damage to the inner slope of the dyke. Sliding and erosion of the inner slope subsequently led to a complete breach in the dyke.



Figure 45 Dike breach during 1953 disaster

Around 20 % of the total area of Zuid Beveland has been inundated, i.e. 7500 ha., see Rijkswaterstaat (1961). Nearly all breaches occurred along the southern seadike (bordering the Westernscheldt) (see Figure 44).

The total area that was inundated in Zeeuws Vlaanderen was around 2880 ha and was least hit compared to other parts of the Delta. Figure 46 shows the locations of dyke breaches.

With the mathematical model Delft1D2D the 1953 flood event has been simulated, using the same breaches as actually occurred and the observed water levels, see Asselman (1999).

Another important, but much smaller flood occurred in Flanders (Belgium) in 1976. This time the affected area was the land along the tidal branches of the Sea Scheldt, in Belgium. The city of Antwerp and many other villages were flooded.

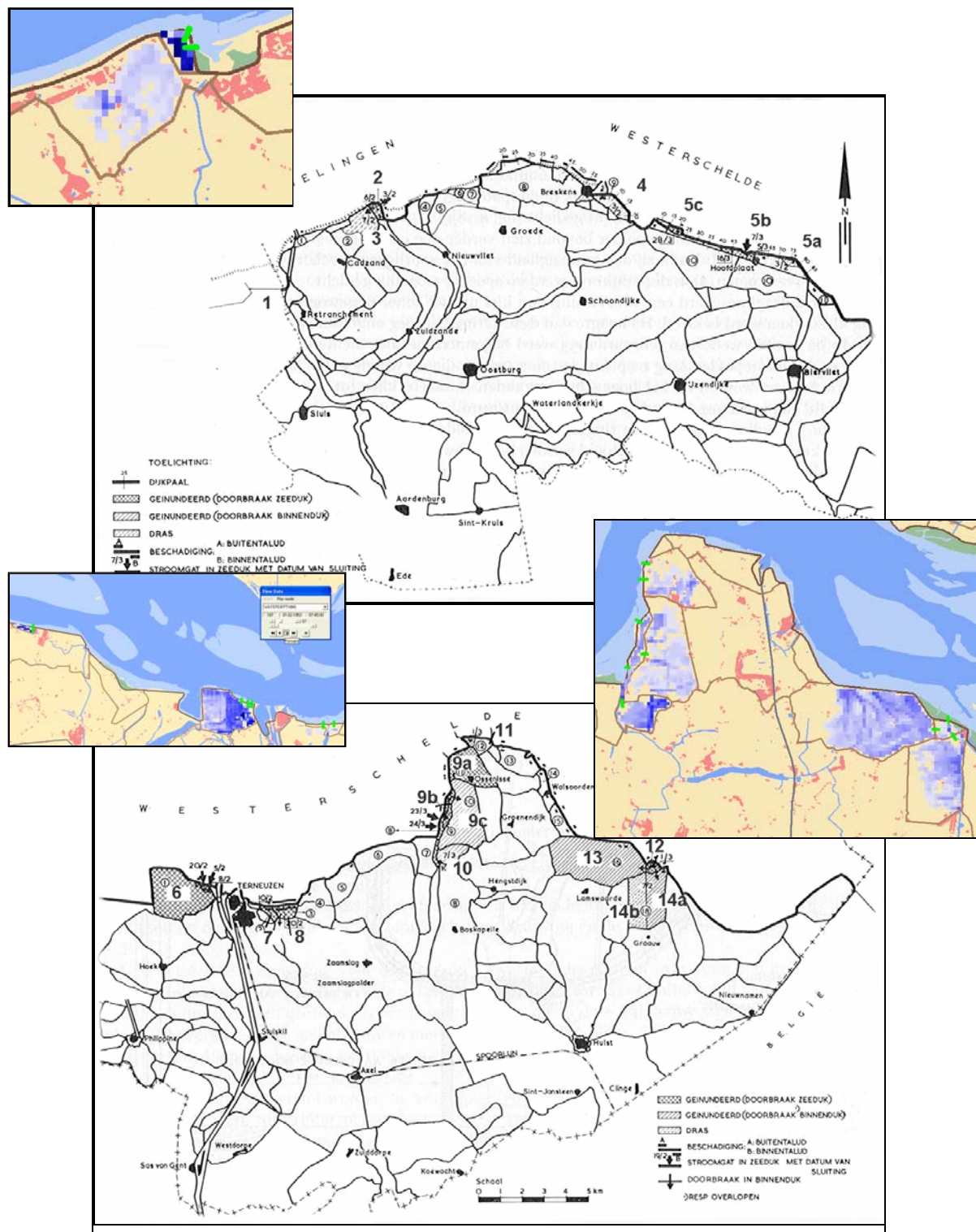


Figure 46 Western part (top) and eastern part (bottom) of Zeeuws Vlaanderen showing the locations of dyke failures during 1953 flood. Insets: Delft1D2D flood model result

6.4 Details of flood defence structures

6.4.1 Seadikes

The section on seadikes is taken from TAW (1999).

a) Description and cross section

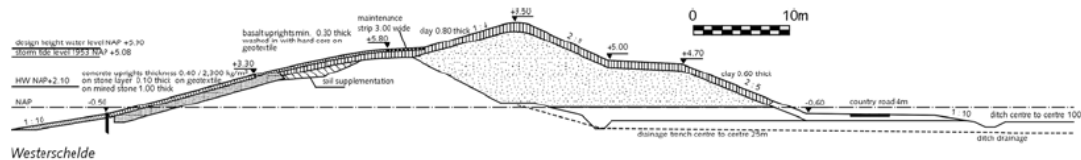


Figure 47 Cross section of a sea dike along the Westerschelde

Westerschelde

- design height water level NAP +5.90m
- storm tide level 1953 NAP + 5.08
- concrete uprights thickness 0.40 / 2300 kgm³
- on stone layer 0.10m on geotextile
- on mined stone 1.00m thick
- basalt columns min. 0.30 thick
- washed in with broken stone on geotextile
- soil fill-in maintenance strip 3.00 wide
- clay 0.80 and 0.60m thick
- country road 4m
- drainage trench centre to centre 25m
- ditch drainage centre to centre 100m

b) Potential failure modes

The height of a dike is of primary importance in determining the quantity of overtopping water. The height is guaranteed by the quality of the dike. The quality is determined by the relationship between strength and normative loads. Insufficient strength can lead to the occurrence of the following failure mechanisms (also see figure below):

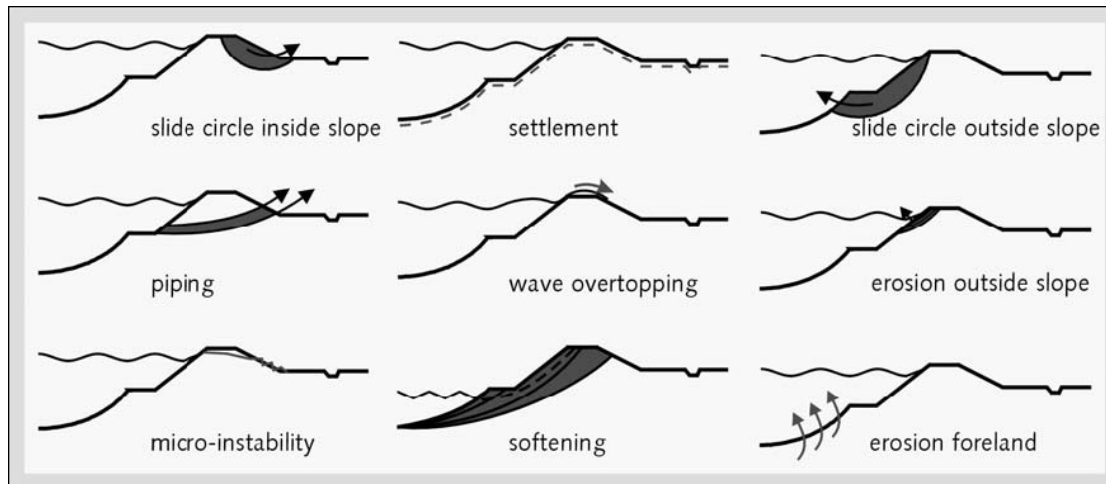


Figure 48 Failure mechanisms of a sea dike

- Vertical and horizontal deformation and tectonic subsidence
 - vertical deformations occur as a result of settlement of the subsoil and setting of the raised material;
 - horizontal deformations occur at thick and weak clay and peat packages in, under and alongside the dike, and can lead to loads on structures in and in the vicinity of the defence, such as conduits and building foundations;
 - tectonic subsidence occurs in water extraction or mineral mining.
- Inadequate macro-stability, including horizontal shearing of the total dike body
Macro-stability is the stability in relation to shearing by an earth body or large parts of it along straight or curved sliding planes.
- Loss of stability as a result of erosion of the outside slope.
- Loss of stability as a result of erosion of the foreland.
- Inadequate micro-stability
Micro-stability is the stability of earth layers of limited thickness at the surface of a slope under the influence of a groundwater flow. Micro-stability is caused by a high water table in the dike.
- Stability in case of overtopping
Overtopping can cause water to infiltrate the inside slope. As a result the top layer of the inside slope is saturated and can shear.
- Erosion of crown and inner slope in case of overtopping
If there is a large quantity of overtopping water, erosion of the inner slope can occur as a result of water flowing along or off the inner slope. Infiltration due to overtopping can lead to shearing of the inside slope.
- Loss of stability due to sand boils (piping)
Piping can be described as a concentrated outflow of groundwater on the inside at high outside water levels, where the velocity of the outflowing water is such that soil particles are carried along and cavities and tunnels originate due to receding erosion which threatens stability.
- Loss of stability as a result of loss of consistency due to settlement flow at the foreland or due to softening of the dike body.
Loss of consistency due to settlement flow is a mechanism in which a water-saturated mass of sand is subjected to a great displacement ('flows') as a result of softening. Softening of sand with a loose packing is the result of an increase in shear strength, where, owing to a rearrangement of the grain structure (decrease in volume) an increase of the water and air pressure occurs in the pores to such a degree that the contact pressure between the individual grains decreases to a significant degree and the sand mass behaves like a heavy fluid. This plays an important role along parts of the Westerschelde.

- Loss of stability of hard revetments, toe structures and bank protection inside the dike by wave forces, internal water pressure etc
- Damage to the dike revetment and the crown as a result of a collision, floating objects and ice.

6.4.2 Dunes

Not relevant for Westernscheldt.

7 Pilot Site 'River Ebro Delta Coast'

The river Ebro Delta Coast has no flood defence structures but is only characterised by an open sea coast. Therefore, the extensive run-up on beaches and associated flooding of the delta is the only failure mode which needs to be considered.

In this section there is however a brief overview of the pilot site and a short report on failure which have been observed in the past.

7.1 Overview of flood prone area

The Ebro delta is located on the Spanish Mediterranean coast about 200 km southward of Barcelona. It has an approximate subaerial surface of 320 km² and a coastline length of about 50 km including the inner coast in the two main lagoons (Figure 49). As many other deltas, it is an ecologically rich environment with about 311 different species of birds (60% of all the species in Europe) and about 515 different plant species (Espanya, 1997). It includes a Natural Park of 7,802 ha giving administrative protection to the areas of highest environmental value, including habitats like freshwater, brackish and saline lagoons, salt marshes and coastal and small dune sandy areas.



Figure 49 The Ebro delta

At the same time, it is actively exploited by means of agriculture, mainly for rice production (about 66% of the total subaerial surface is devoted to rice production and between 10% and 15% to other crops, Museu del Montsia, 1997) and provides support for a significant percentage of the fishing and aquaculture activities in Catalonia. The population is about 50.000 inhabitants, including people living in the delta itself and people with a direct economic dependence on it.

7.2 Ebro delta coastal behaviour

7.2.1 Introduction

When studying morphological changes, a broad spectrum of frequencies in time and space can be considered, with scales ranging from the high frequencies (*e.g.* shoreface bed-form dynamics) to the very low ones (*e.g.* offshore bar system dynamics). However, when the emphasis is on relatively large scale changes, this theoretical spectrum can be narrowed, so that only some of the principal components may be analysed to reproduce or to study the major part of the large scale morphological changes. This applies to deltaic coastal dynamics, where the most “immediate” contributors to coastal morphodynamics and vulnerability can be “fixed” to three main spatial and temporal scales: large/long-term, medium and episodic ones (Jiménez, 1996, Jiménez *et al.*, 1997a). An analysis of the geomorphic vulnerability of the delta at these three scales can be found in Sánchez-Arcilla *et al.* (1998).

Usually, *long-term coastal processes* are associated with changes at a temporal scale of decades and a spatial scale in which the complete deltaic coast is considered. The main changes at this scale are those in the deltaic overall shape and sediment budget, which are characterised by the corresponding net surface and volume changes. The main “driving” or “forcing” agents contributing at this scale have been identified as: river sand supply, cross-shore sediment exchanges at the shoreface, relative sea-level rise (RSLR) induced changes, aeolian transport over the dune fields (or barrier) and overwash transport.

Medium-term processes are associated with changes at a temporal scale of several years and a spatial scale of several km. In this scale cyclic (*e.g.* seasonal) changes are filtered out in such a way that only the net evolutive trend is retained. Most of the observed changes at this scale have been related with the net longshore sediment transport processes and correspond to a coastal reshaping in which eroding stretches are feeding accreting ones (Jiménez and Sánchez-Arcilla, 1993). Although this scale is shorter than the previous one, it has a residual morphological effect visible or detectable at the long-term scale.

Finally, *episodic events* are associated with hydrodynamic processes with a long return period, unknown periodicity and a spatial scale defined by the length of the coastal response. The contribution at this scale, although not present in every climatic cycle, whenever existing is important enough to contribute significantly, in a matter of several days, to the medium-term and, even, to long-term processes, with an eroded volume equivalent to what would happen in a few years without episodic events. The main “driving” agent for these events is the presence of very energetic sea states, generally characterised by the coexistence of storm surges and storm waves, which produced an associated coastal response of “extreme” erosion of vulnerable stretches.

7.2.2 Coastal reshaping

In essence, the time frame to study morphological coastal processes governing the Ebro delta present evolution is that during which present conditions were achieved. Since the main boundary condition for the Ebro delta coast development is the building of dams in the river course, the initial time for this analysis will be that corresponding to the Ebro delta configuration near and from the date of dam building. Otherwise, if previous configurations would be included in the analysis, different results could be obtained.

During the last two centuries, the evolution of the Ebro delta has been characterised by a tendency towards an exponential decrease of the expansion rate of the delta plain. After a period of large expansion (1749-1915), followed a period in which the expansion is attenuated reaching a “hypothetical” equilibrium surface.

The main dam complexes in Ebro river course were built in the beginning of the 1960’s and on the basis of the delta plain surface evolution, the time frame for this analysis was selected from 1957 to present. Due to this, solid discharges of the Ebro river have been decreasing during the last decades and, as a consequence of this, the delta has become more influenced by wave action in such a way that it has been subjected to an intense reshaping process (Jiménez and Sánchez-Arcilla, 1993; Jiménez *et al.*, 1997b). Figure 50 shows the erosion and accretion areas along the delta during the period from 1957 to 2000, where it can be see that the erosion with the largest retreat is Cap Tortosa at the Buda Island area, whereas the apex of both two spits have been experiencing a significant accretion. It can be also observed that the advance of the Northern spit towards the continent is tending to close the bay and, in the case, the process will finish a new lagoon will be created.

This sediment supplies cut-off has led to a decrease in the relative land elevation due to the inexistence of river floods able to distribute sediments along the deltaic plain and, on the other hand, the last processed observations suggest, however, that the overall sub-aerial surface is nearly steady with the coastline being reshaped as it was before mentioned. Further details in coastal dynamics and shoreline evolution in the Ebro delta can be found in Jiménez and Sánchez-Arcilla (1993), Jiménez *et al.* (1997b) and Guillén and Palanques (1997) among others.

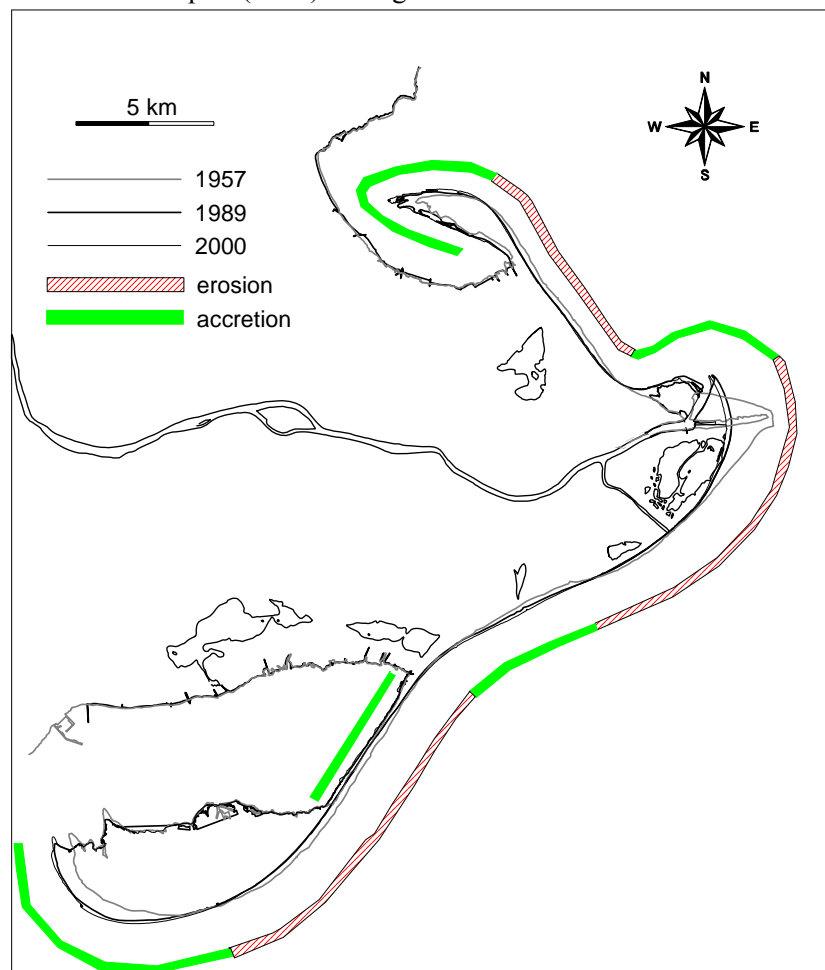


Figure 50 Decadal-scale coastline evolution in the Ebro delta (adapted from Jiménez and Sánchez-Arcilla, 1993).

The deltaic coastline reshaping is mainly driven by the longshore sediment transport gradient along the coast. The transport pattern is controlled by dominant Eastern waves that determines the existence of net transport rates growing from the central lobe of the delta towards both two spits, where they decrease down to zero and they behave as sinks for sediment eroded from the outer coast. Figure 51 shows the net longshore sediment transport pattern along the Ebro delta coast at the medium-term scale (averaged over several years).

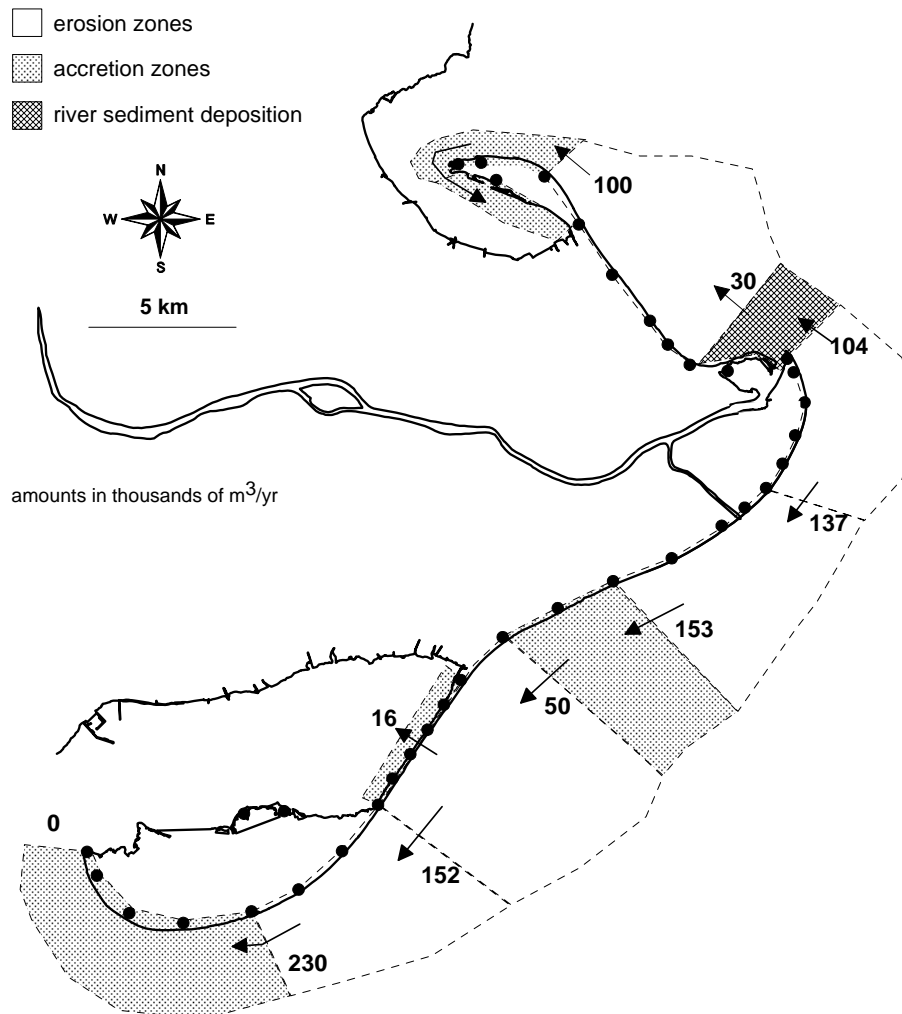


Figure 51 Medium-term longshore sediment transport rates along the Ebro delta (adapted from Jiménez and Sánchez-Arcilla, 1993).

As an example of coastal response, Figure 52 shows a detail of the magnitude of coastal erosion rates in the most erosive stretch along the deltaic coast, the Illa de Buda at the central lobe. As it can be seen, although erosion rates have decreased with respect to the former period, still they are large, reaching a maximum value of about 20 m/yr. However, this retreat does not mean that the beach will disappear and, in fact, a barrier beach fronting the inner lagoon during the last decades has been observed, which seems to maintain an equilibrium width of about 160 m. Thus, the coastal retreat is accompanied by the rebuilding of a barrier beach by overwash processes in such a way that the active beach maintains constant although progressively moves landwards (see Valdemoro *et al.*, 2005).

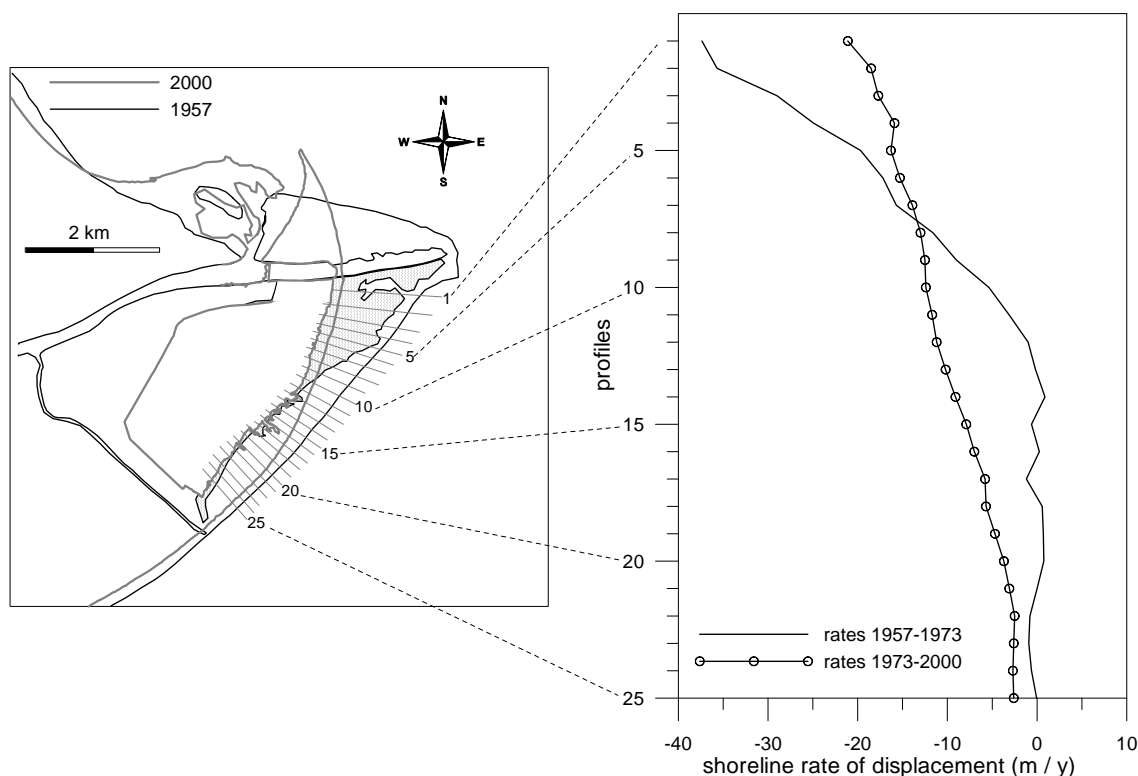


Figure 52 Decadal-scale shoreline rates of displacement along Illa de Buda (adapted from Valdemoro *et al.*, 2005).

7.2.3 Inundation potential

In addition of the coastal processes described above -which essentially drive changes in the deltaic surface-, processes acting at the long-term scale will also affect to the vertical dimension, i.e. the relative elevation of the deltaic body with respect to the sea level. This has very important implications for a low-lying coastal environment, especially in a scenario of rising sea level because the only way to avoid direct inundation by the sea is the vertical accretion of the deltaic plain.

Although natural –pristine- deltas (without any human interference) can cope with RSLR -i.e. the deltaic plain is able to vertically accrete and the coast progrades due to the availability of sediment supply-, deltas where human action has led to a decrease of riverine sediment supplies cannot. In these modified environments, RSLR will increase the probability of flooding of low-lying areas, specially those directly connected to the sea or where a “passive” coastal fringe exists. In the Ebro delta, “passive” stretches are the inner coasts in the two main lagoons where no energetic driving agents are acting nor there is any significant sand stock (and the available one has a large percentage in very fine sediment).

To give an idea of the potential vulnerability of the Ebro delta to flooding, Figure 53 shows the zoning of the Ebro delta plain in terms of vulnerable areas to an inundation level of 0.5 m. In any case, it has to be considered that the used topographic measurements for inundation levels were taken from Ebro delta plain data presented in Ibañez *et al.* (1997). These measurements although not fulfilling the standard requirements of accuracy, qualitatively reproduce the overall deltaic plain relief.¹

The areas potentially able to be inundated for such level were estimated taking into account the role of hinterland structures -such as levees, dikes and roads- in preventing flooding in impounded areas. Additionally, areas not directly connected to the sea but with a high predicted probability to

¹ There is a very recent accurate DEM for the Ebro delta obtained by the ICC which is being acquired for the project.

experience breaching events (see next section) are also considered as potentially vulnerable to direct inundation.

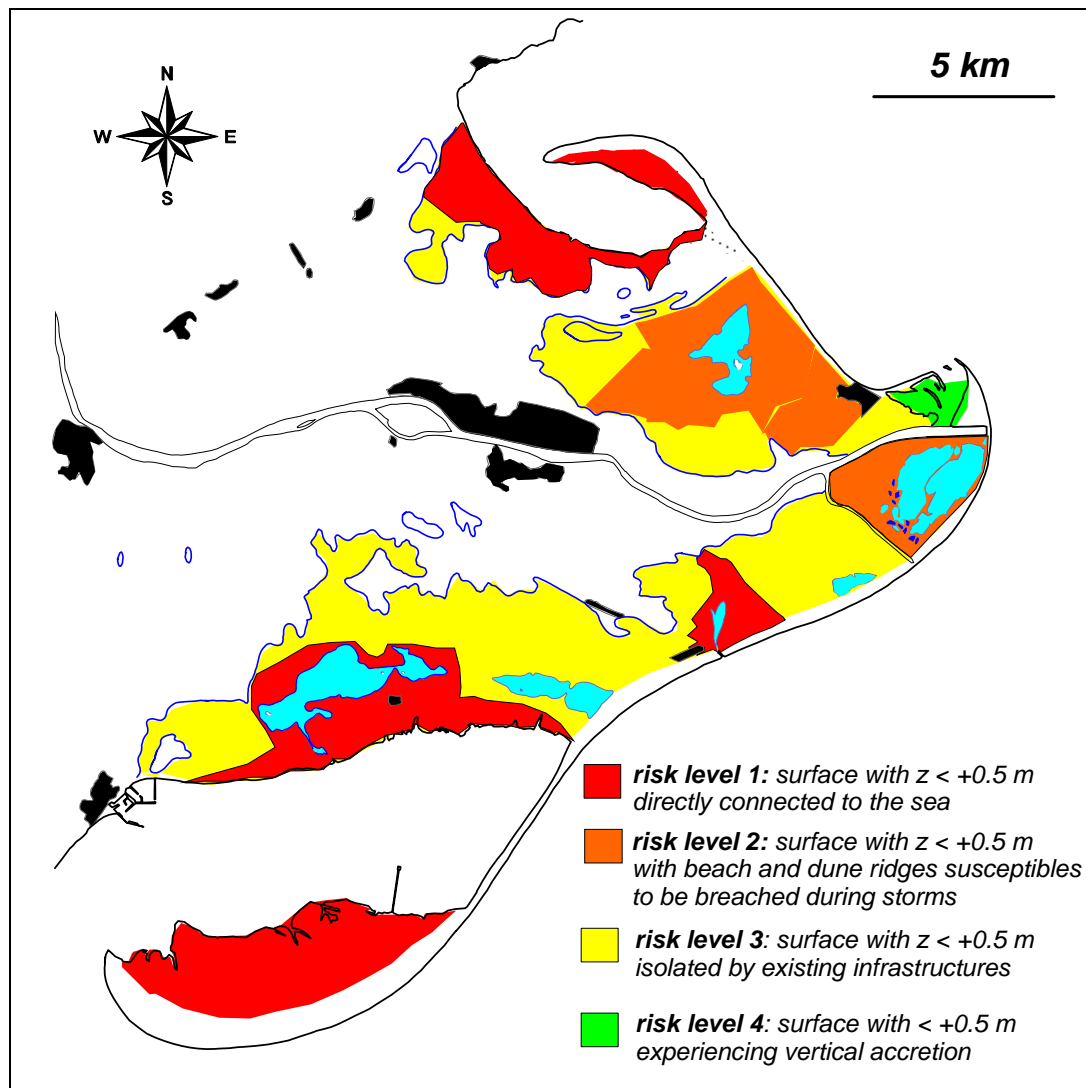


Figure 53 Vulnerable areas to an inundation level of 0.5 m in the Ebro delta (adapted from Sánchez-Arcilla *et al.*, 1998).

7.3 Failures in the past

7.3.1 Overview

Storm impacts on the Ebro delta coast usually occur under the coexistence of surged water levels due to the passage of low pressure systems off the Ebro delta coast and eastern wave storms (Sánchez-Arcilla and Jiménez, 1994; Jimenez *et al.* 1997b). Under these conditions, waves are able to act on the usually non-exposed part of the coastal fringe and to produce large erosion in a very short-time period. Although the entire deltaic coast is subjected to the action of such events, the more vulnerable stretches are those with a narrow emerged beach and fronted by a “low-crested” bar or bar system. Figure 54 shows these sensitive coastal stretches which have been identified taking into account the magnitude of the induced coastal response and/or the magnitude of affectation of uses and resources. These vulnerable areas are: (i) Illa de Buda at the central lobe; (ii) Trabucador beach at the Southern hemidelta and (iii) Marquesa beach at the Northern hemidelta.

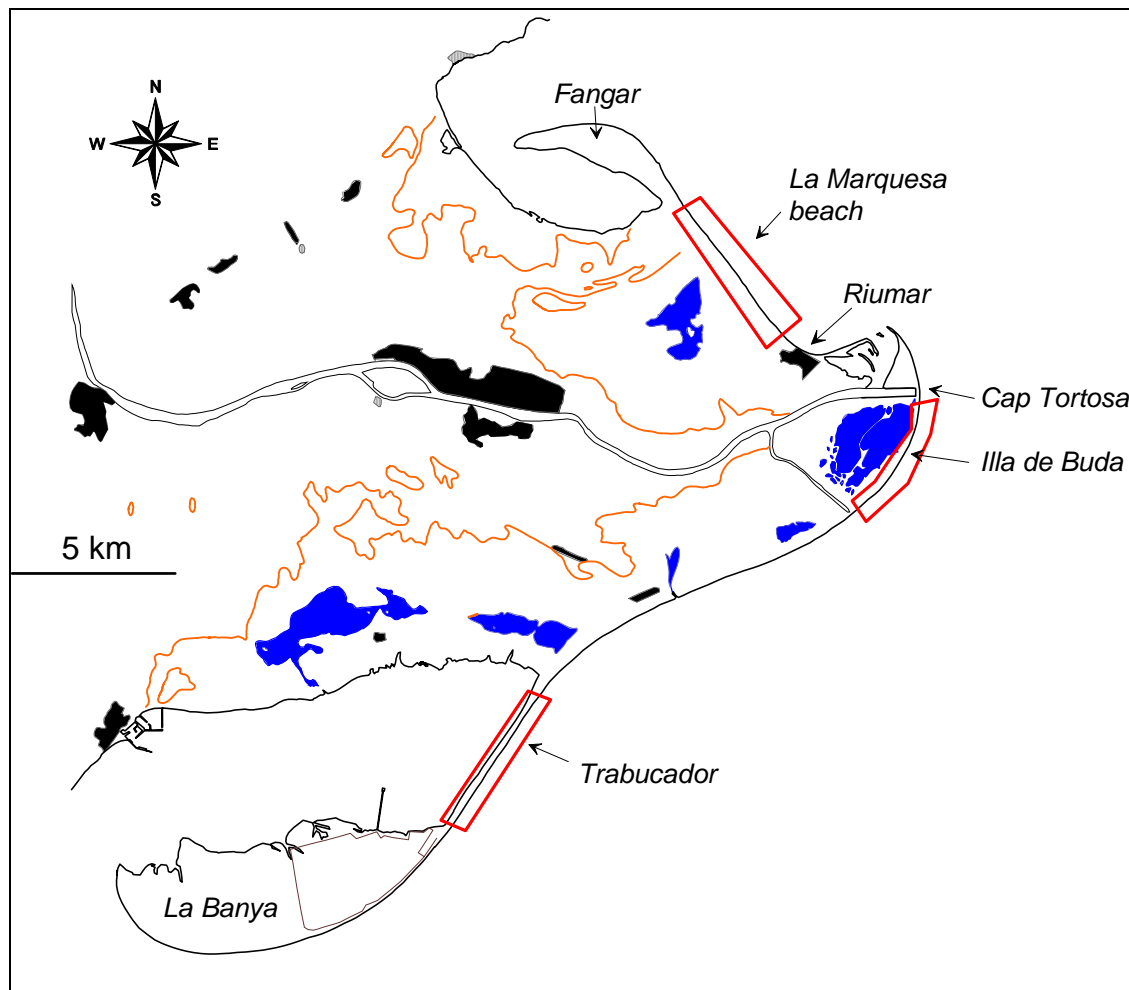


Figure 54 Vulnerable areas to storm impacts in the Ebro delta

Recently, the Autonomous Government of Catalonia (Generalitat de Catalunya) has reported the most hazardous storms impacting the Ebro delta during the last decade. It has to be stressed that this identification has not made use of any measurement of the storm properties but they have been identified taking into consideration the induced “problems” in the Ebro delta coast. Here “problems” can be translated as situations such as (i) affectation of agriculture lands by inundation (local owners being the receptor of the damage), (ii) affectation of natural values due to storm impacts – wave exposure or inundation - (Natural Park being the receptor of the damage), (iii) impulsive coastal erosion of very large magnitude.

These storms together the main induced effects along the Ebro delta coast are shown in Table 3.1, where it can be seen that in most of the cases, the “identified” problematic stretches are the same, *i.e.* the ones shown in Figure 54. It has to be taken into account that these systematically affected areas are coastal stretches subjected to erosive processes in such a way that they show the largest retreat rates along the deltaic coast. Moreover, in all the cases, these areas are characterised by the presence of a low-lying profile fronting the deltaic plain, in such a way that the beach could be easily “overwashed” during these events (see representative profiles of each area in Figure 55)

In addition to this, it is also evident from Table 2 that the identified “harmful” storms seem to be clustered. Thus, most of them have verified in the last four years (from 2001 to 2004). Although this period seems to be a relatively stormy period (Figure 56), this clustering in time may also respond to the cumulative nature of the process. Thus, as more frequent the erosive processes are, the most fragile the coastal stretch will be. Moreover, if during a certain period, the frequency of storms is relative

high, it could be possible that natural recovery processes should have not time enough to be effective and then, storms should impact on already eroded/affected areas.

Table 2 Main storm impacts in the Ebro delta coast inducing problems in the hinterland or in the coast according to local land manager (adapted from Generalitat de Catalunya, 2004).

Storm	Effects
October 1990	Trabucador breaching. Breaches in Illa de Buda.
October 1997	Beach dune erosion in La Marquesa beach. Fangar spit inundation. Breaches in Illa de Buda. Trabucador breaching. Beach erosion in La Banya spit. Affectation of dikes in the salt mine.
November 2001	Strong erosion along almost the entire delta coast. Fangar spit inundation. Massive overwash deposits in some spots in La Marquesa. Breaches in Illa de Buda. Opening of the former North mouth (at Cap Tortosa). Trabucador breaching.
April 2002	Erosion along La Marquesa beach. Breaching of sand dike in La Marquesa.
May 2002	Trabucador breaching. Breaching of sand dike in La Marquesa.
February 2003	Trabucador breaching.
October 2003	Trabucador breaching. Hinterland inundation in the back of La Marquesa beach.
January 2004	Erosion of sand dike along La Marquesa beach. Trabucador breaching.
March- April 2004	Trabucador breaching. Erosion of sand dike along La Marquesa beach and inundation of the hinterland.

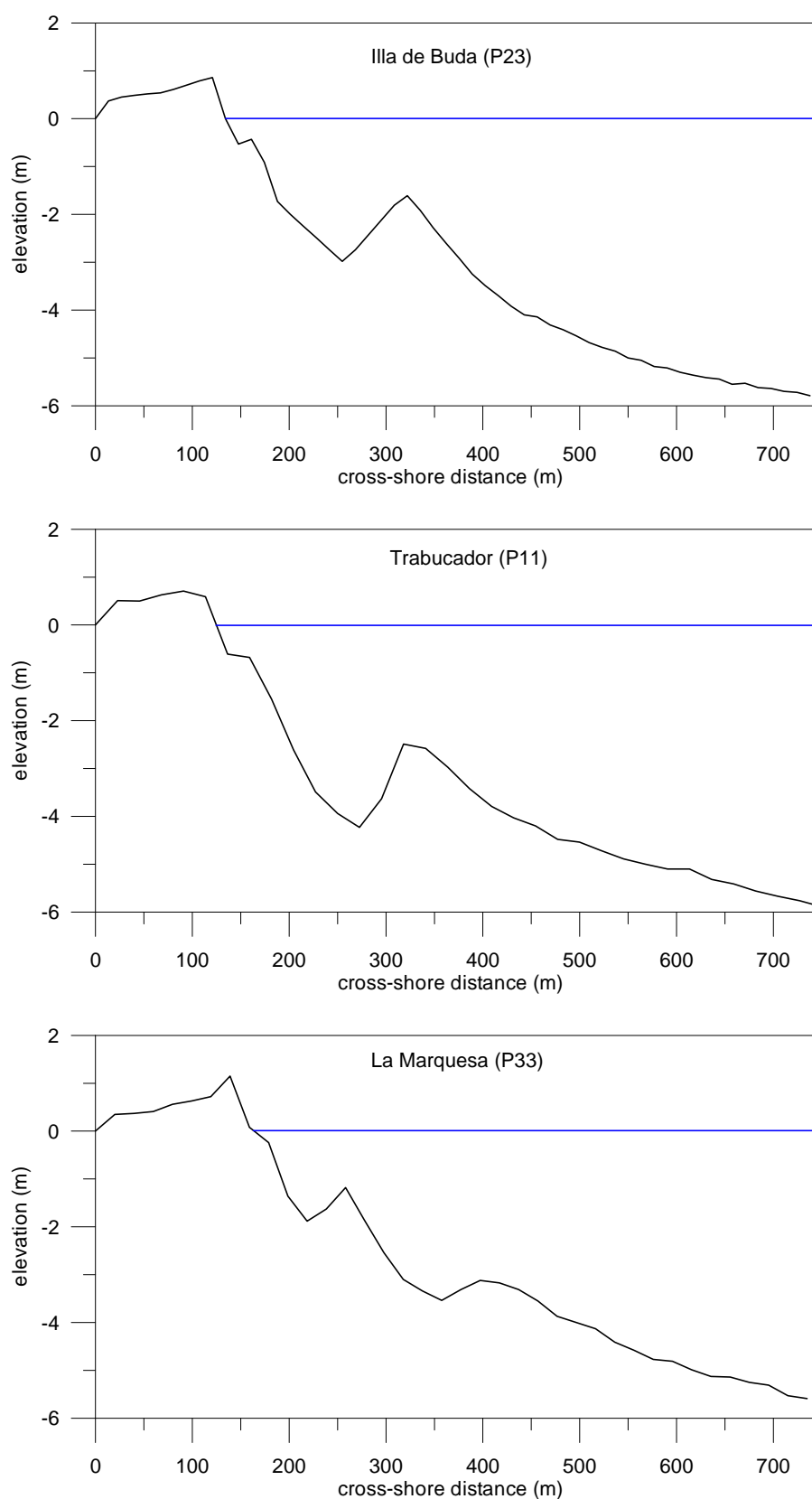


Figure 55 Representative beach profiles for each of the identified sensitive coastal stretches

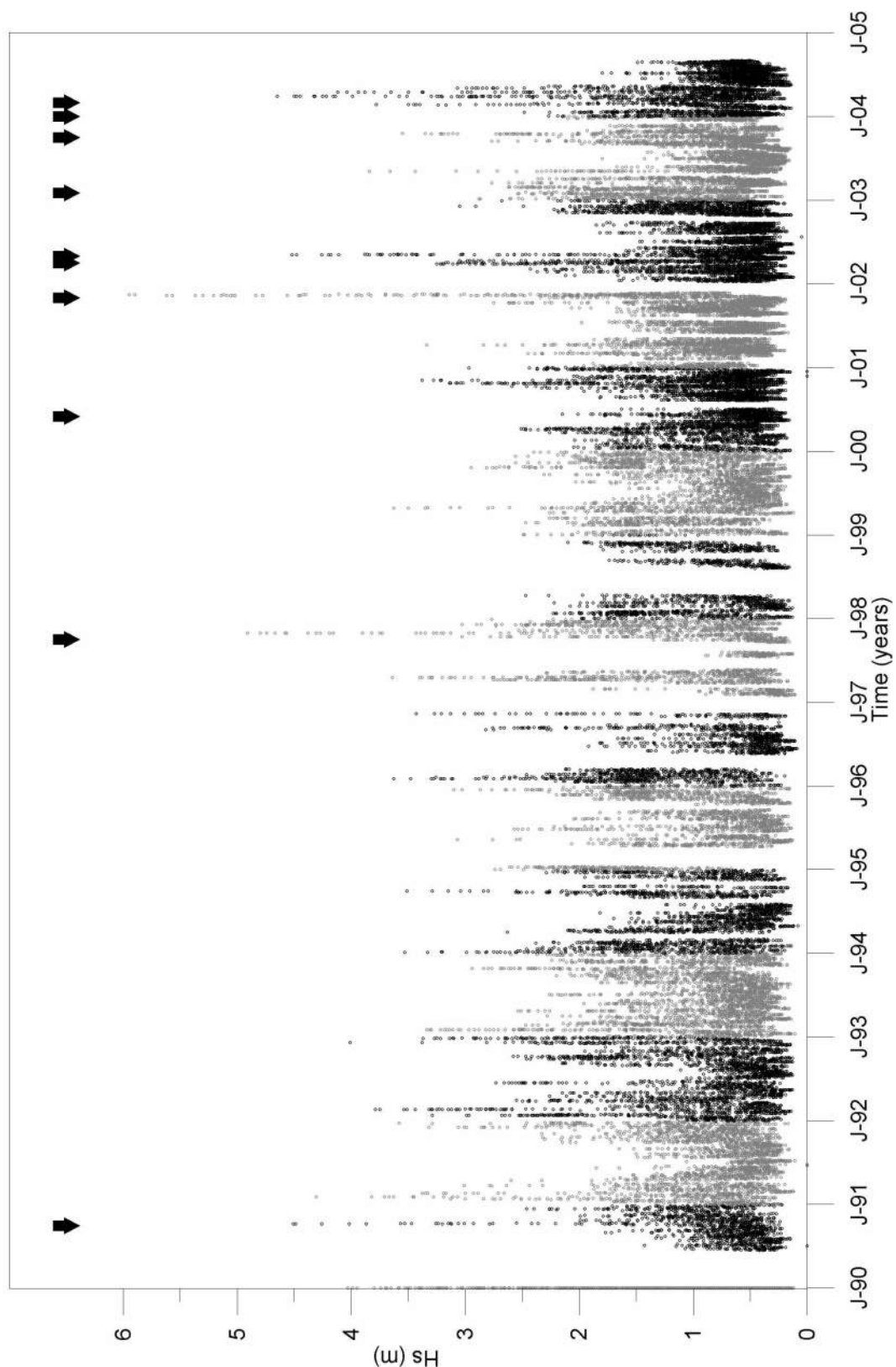


Figure 56 Significant wave height time series recorded off the Ebro delta showing storms mentioned in Table 2 (adapted from Valdemoro, 2005)

In what follows, some examples of the consequences of selected storm impacts in each of the most affected stretches are given (see full description of storm impacts in Table 2).

7.3.2 Impacts in the Illa de Buda

Figure 57 shows two examples of the coastal response in Illa de Buda to the action of extreme events. The first one corresponds to the beach morphology along the Buda Island after the action of the storm of October 1990 where some breaches can be observed. This storm was energetic enough to induce “massive” coastal erosion along the entire delta, including the breaching of the Trabucador barrier (see Sánchez-Arcilla and Jiménez, 1994 for further details). Hydrodynamic conditions during the storm can be summarised by a significant wave height of 4.5 m at the peak of the storm together with a meteorological tide of about 0.45 m (see details in next section). These conditions together with a beach profile fronting the wetland without any protecting dune (the berm height measured before the storm was only 0.90 m above the mean sea level, see Figure 55) determined the beach to be significantly overwashed. The estimated wave run-up at the peak of the storm will thus vary between a value of about 1.1 m -using the expression proposed by Stockdon *et al.* (in review)- and 1.8 m -using the expression of Mase (1989)-. In any case, if the storm surge at the peak of the storm is added, a total water level rise between 1.55 m and 2.8 m should be obtained, which is clearly exceeding the berm height and fulfilling the conditions to produce massive overwash of the beach and, probably also the conditions to produce breaching (see *e.g.* Kraus and Wamsley, 2003).

These conditions created a connection between the lagoon and the open sea and, as a consequence, a continuous influx of sea water verified during the event. Moreover, after the pass of the storm the beach remained breached during several weeks and the connection between the lagoon and the sea was maintained. This resulted in a seawater inflow to the lagoon larger than expected because the breached conditions exceeded the duration of the storm event. Finally, the breach was artificially closed and the barrier morphology returned to the “normal” stage, *i.e.* the barrier equilibrium width.

The second situation corresponds to the action of a storm in October 1997 where a massive overtopping of the beach is clearly observed. Hydrodynamic conditions during the storm were a significant wave height of 4.9 m at the peak of the storm with a surge between 0.2 – 0.3 m (see details in next section). The estimated wave run-up for these conditions varies between a value of about 1.4 m to 3.0 m using the expressions of Stockdon *et al.* (in review) and Mase (1989) respectively. These values were significantly higher than those for the storm of October 1990 –excluding the storm surge– and, as it can be observed from these numbers and directly from figure 3.4, this storm determined the area to be flooded.

In this area, consequences of storm impacts are mainly related to affectation of the natural system, since the hinterland is occupied by a lagoon of natural interest that belongs to the Natural Park (Figure 58). This lagoon, Els Calaixos, receives freshwater inputs from adjacent rice fields mainly during the period from May to October, whereas saltwater inputs mainly occur during winter when coastal storms are more frequent (see *e.g.* Valdemoro *et al.*, 2005).

In this sense, the increase of the influence of saltwater inflows (of any origin as *e.g.* increase in storminess) should induce an increase in the coverage of salt-tolerant species, specially in the Eastern end of the lagoon. Due to this, and regardless of the ideal state of the lagoon in terms of ecosystem quality, it is clear that the dominance of one species over the other ones is strongly controlled by the balance between the two water inputs and, of course, storm impacts control one of them and, in consequence, the stability of the Els Calaixos wetland will largely depend on the frequency of occurrence and intensity of these storm events.

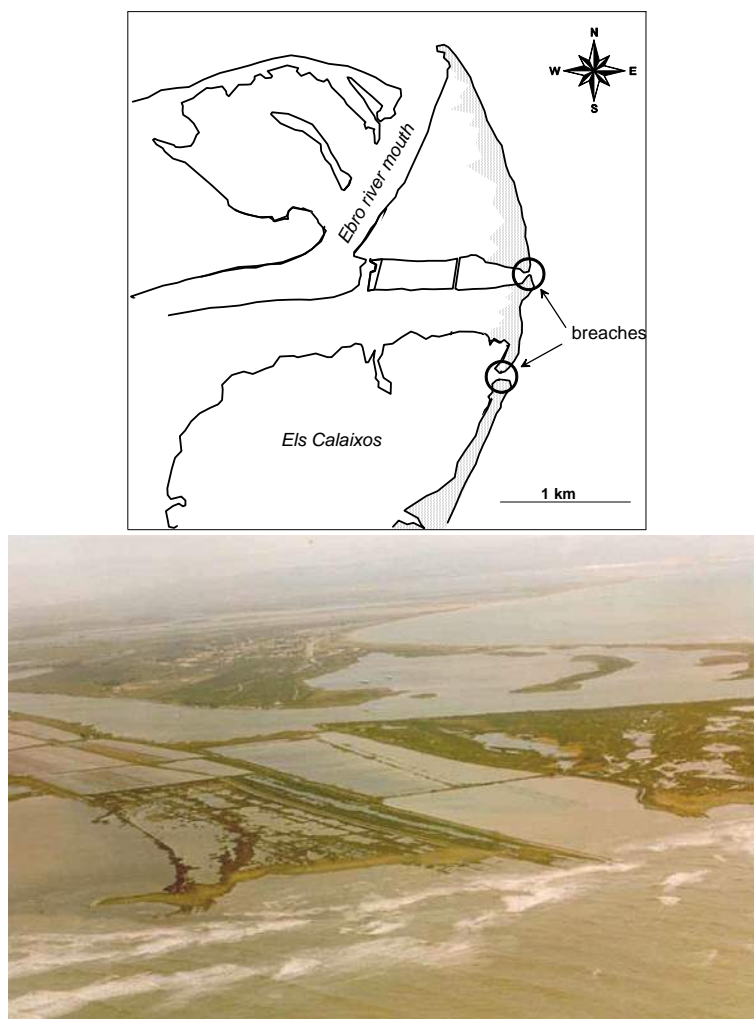


Figure 57 Examples of the impact of storms in the Illa de Buda. (Top) Breaches after the impact of the October'90 storm. (Bottom) Inundation of the beach during the October'97 storm



Figure 58 Aerial photograph of the Illa de Buda

7.3.3 *Impacts in the Trabucador beach*

Figure 3.6 shows the response of the Trabucador beach to the impact of two extreme storms. The first one corresponds to the barrier morphology after the action of the storm of October 1990 and the second one to the impact of the storm of November 2001. In both cases, the barrier was breached with large amounts of sediment being transported towards the inner bay during and just after the storm.

These two storms were typical “llevants” where very high Eastern waves coexist with surged water level due to the presence of a low-atmospheric pressure centre in the Catalan Sea and high E winds blowing towards the coast. Under both two events the barrier was significantly breached and, the combination of hydrodynamic conditions (wave and water level, see section 4) as well as the low barrier profile (Figure 3.3) determined to be under the inundation regime according to the hazard scale of Sallenger (2000) for the impact of storms on barrier islands.

In both cases, the barrier remained breached during a relatively long time (at the human scale) until the owners of the salt mine at the La Banya spit decided that they needed to recover the physical link of the spit with the main body of the delta. This was due to the fact that the production of salt is distributed by trucks and, of course, they require the barrier to be emerged to properly do it. This means that the existence of this business in the southern spit makes this natural process (rapid barrier breaching and slow recovery) a problem (at least from their perspective because it affects the business). This artificial recovery can be seen in Figure 59. A detailed description of the breaching during the October 1990 storm can be seen in Sánchez-Arcilla and Jiménez (1994).

In addition to this effect, another possible influence of these events is the affectation of the physico-chemical properties of the water in the Alfacs bay. The existence of a connection between the sea and the lagoon through the Trabucador will change the properties of the water in the lagoon and, depending on the type of change this could affect the production of bivalves in the lagoon (the Alfacs Bay is intensively used for aquaculture –mainly for production of mussels-). Thus, after the impact of several storms in Spring of 2004 was related to the reduction in a 15% of the production in the area (Generalitat de Catalunya, 2004).

Finally, another obvious influence of the impact of these events will be its contribution to the long-term barrier behaviour. Thus, these events will induce massive landwards transport of sediment over the barrier, which will contribute to the barrier landward rollover. In morphological terms, this implies the erosion of the outer coast and the accretion of the inner one. During the last decades, this process has permitted the barrier migration without a significant change in barrier width (Figure 60, Jiménez and Sánchez-Arcilla, 1993; 2004) although, during that period no significant breaching was reported.



Figure 59 Examples of the impact of storms in the Trabucador beach. (Top) Breaching after the impact of the October 1990 storm. (Bottom) Breaching after the November 2001 storm.

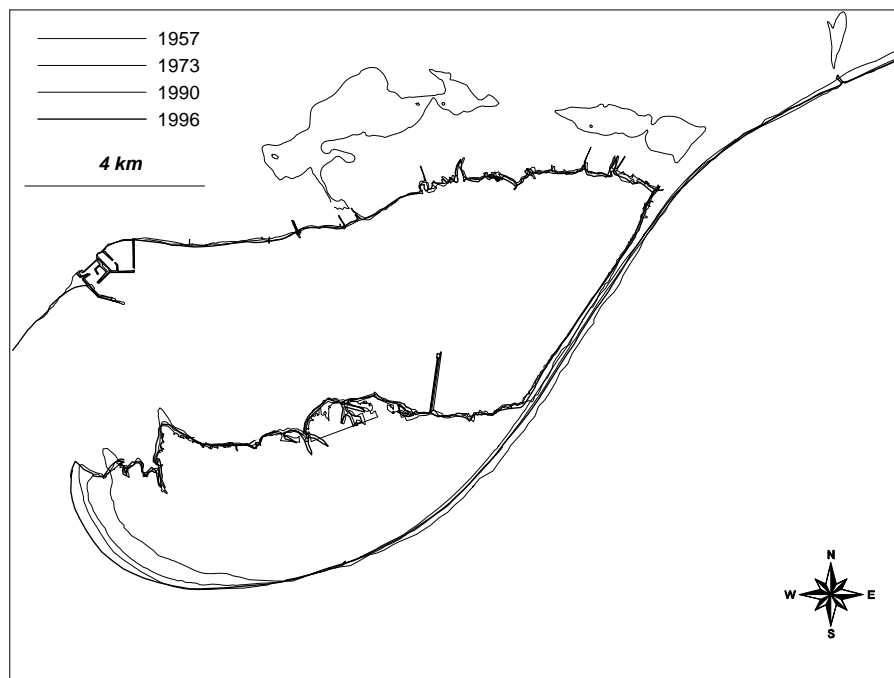


Figure 60 Long-term evolution of the Trabucador-La Banya spit complex (Jiménez and Sánchez-Arcilla, 2004).

7.3.4 Impacts in the Marquesa beach

Figure 61 shows an example of the impact of the storm of October 1997 in the Marquesa beach. Wave conditions during the storm induced significant shoreline erosion as well as the breaching of the existing small dune row along the beach in some parts. As a result of this, the seawater entered into some areas in the hinterland affecting agricultural lands (Figure 61).

The effects of the storm of November 2001 (the most extreme storm recorded by the wave buoy at Cap Tortosa from 1990 until present, see conditions in section 4) in the area are shown in Figures Figure 62 and Figure 63. As it has been already mentioned, this storm significantly eroded the entire coastline of the delta, being this area one of the most affected. Figure 3.9 shows an example of the induced coastal retreat along the area which is clearly visible in the relative position of existing buildings close to the shoreline as the seawall protecting the “Los Vascos” restaurant (this seawall was built by the owners in 1997 to protect their property from the effects of the systematically shoreline erosion suffered by this area).

Another effect of the storm of November 2001 was the overtopping of the beach and the overwash of the rice fields close to the shoreline. This massive overwash affected the rice pads where large volumes of beach sand were deposited (Figure 3.10) and also to some roads and infrastructures close to the shoreline.

In this area, extreme storm impacts have mainly an economic influence since the hinterland is occupied by productive lands (rice fields). As an example, a preliminary estimation of the Department of Agriculture after the storm impact evaluated in about 400 ha of rice fields the affected surface in the Northern hemidelta (Generalitat de Catalunya, 2004), which is the hinterland of the area here generically denominated as La Marquesa beach.

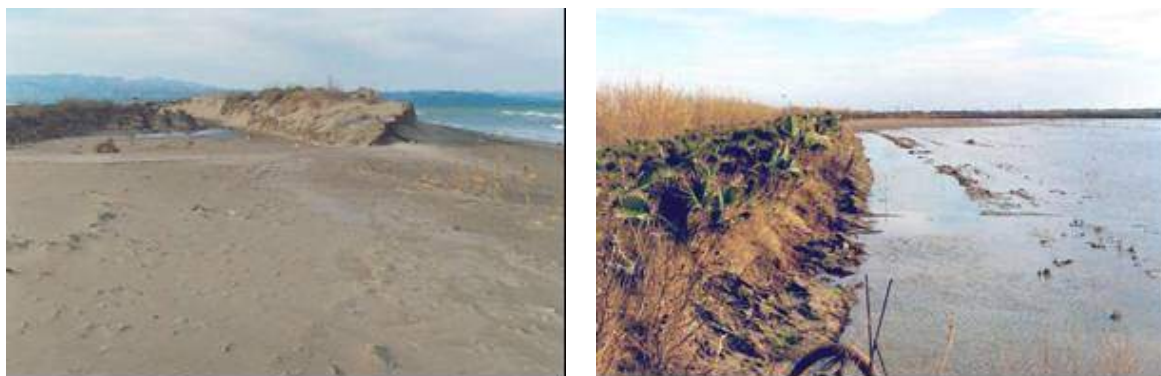


Figure 61 Breaching of the dune row in the Marquesa beach and inundation of the hinterland after the storm of October 1997



Figure 62 Large coastline retreat along the Marquesa beach after the storm of November 2001



Figure 63 Massive overwash deposits along the Marquesa beach after the storm of November 2001

8 Pilot Site 'German Bight Coast'

8.1 Overview of flood prone area

St. Peter-Ording is a large community at the Schleswig-Holstein North Sea coast with the character of a tourist seaside resort. The community is located on the west (=exposed) coast of Eiderstedt peninsula (Figure 64). The size of the study area is approximately 6000 ha; from these about 4000 ha are considered to be flood-prone with the respective height distribution (NN = Ordinance Datum = regional Mean Water Level).

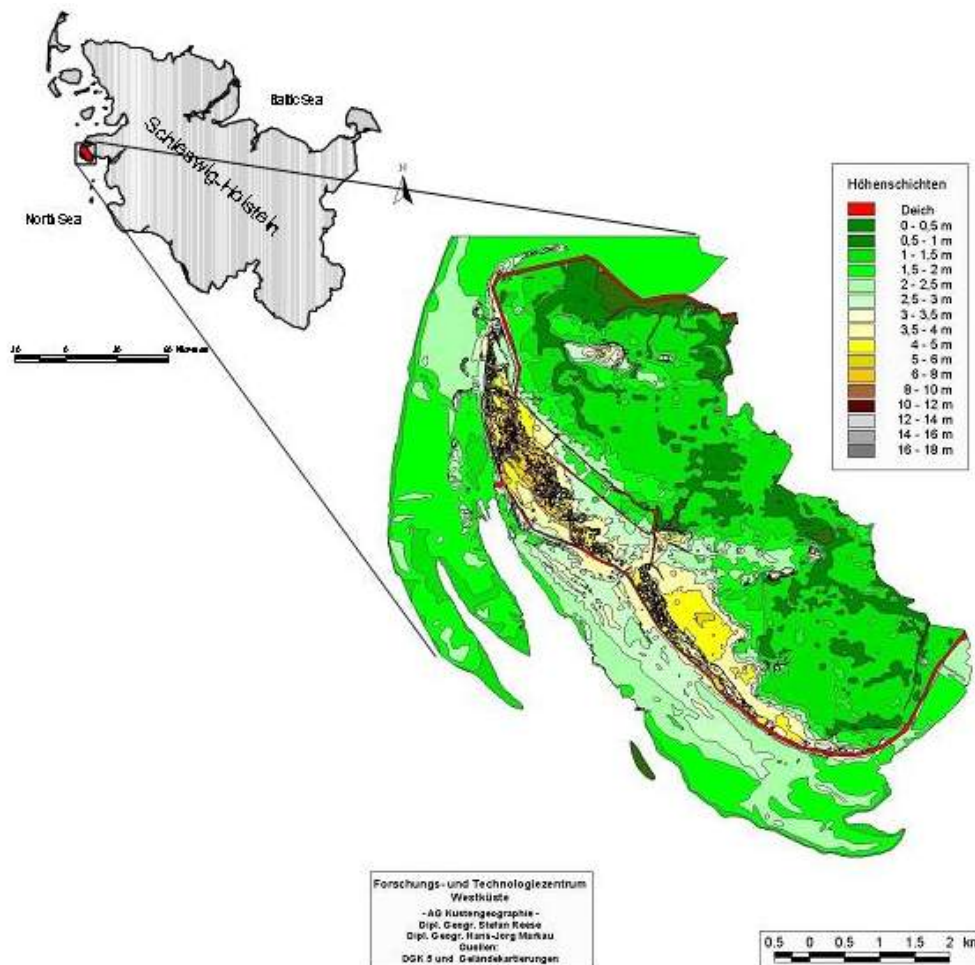


Figure 64 Map of pilot site 'German Bight' (red line illustrates the coastal dike)

The territory of the community amounts to 2800 ha with about 6300 inhabitants. In this area the irregular topography with intermittent small hills and dunes makes it difficult to draw flood-distance boundaries. Presently, flood protection is provided by a major dike (12.5 km long, about 8.0 m high) as well as dune structures 800 m, about 10 m and up to 18.0 m high), surrounding the community on three sides over a length of more than 15 km.

8.2 Overview of defence structures

a) Description and cross section

The defence structure of the pilot site German Bight is a complex coastal defence system (Figure 65). It is divided into a foreland (Figure 71), a major dike line (Figure 67) and a second dike line. The major dike line is 12.5 km long. Furthermore there is a 2 km so called overtopping dike (Figure 70). This type of dikes is designed to withstand wave overtopping and wave overflow. It is therefore considerably lower than standard dikes and is protected by a very solid cover layer such as asphalt. The height of the dike line is not constant (Figure 66). It depends of the kind of defence structure.

In this part of Schleswig-Holstein a second dike line behind the primary sea defences can be found. It is mainly resulting from former dikes which either have been rebuilt or abandoned. However, the second dike line is not complete and parts of this defence line are missing. Therefore, it needs to be investigated on how much protection this defence line is still able to provide for the flood prone area.

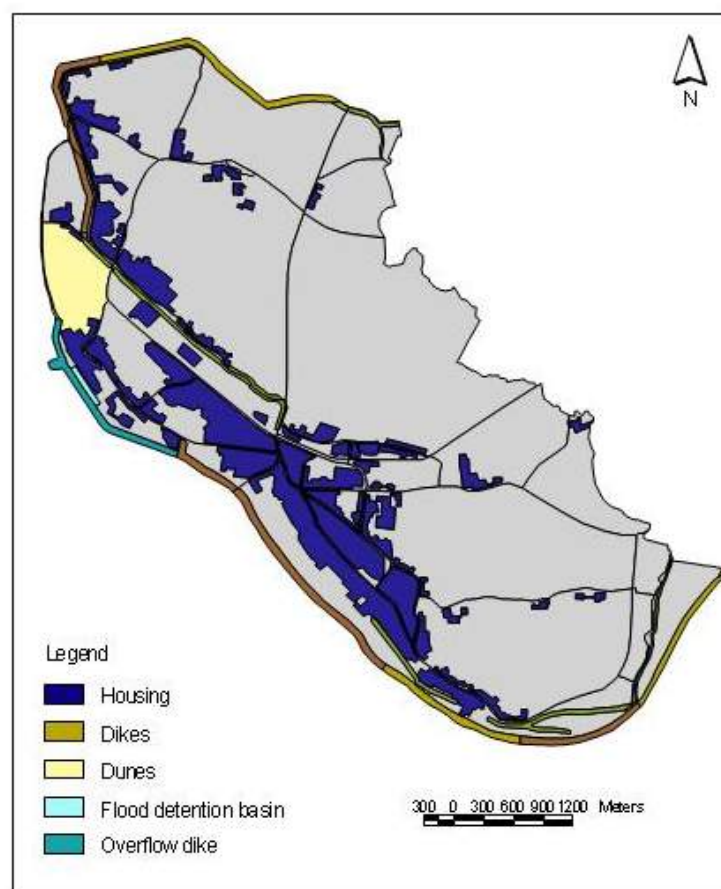


Figure 65 Infrastructure of the 'German Bight' pilot site

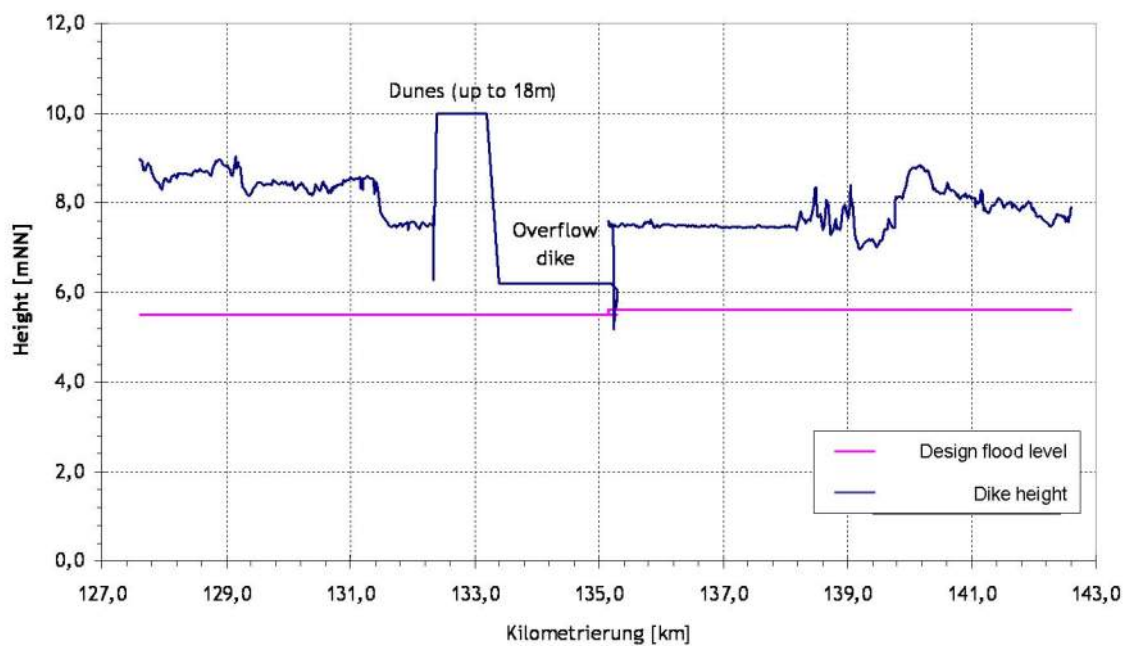


Figure 66 Height of costal defence structures at pilot site 'German Bight' (after MLR (2001))

8.3 Failures in the past

The major threats from the sea result from storm surges which may occur several times a year. Big storm surges have however occurred in 1962 and 1976 only, where during the latter the highest storm surge water levels were recorded with a water level up to 5.6 m (which is about the range of design water level as indicated in Figure 66). Three other storm surges in 1962, 1981, and 1999 have come over the 5.0 m margin with an increase of storminess over the last decades.

These water levels suggest that there was quite some wave overtopping over the overtopping dike but no major failures to the hinterland have been observed.

8.4 Details of flood defence structures

Table 3 gives an overview of coastal defence structures used in St. Peter-Ording. The various sections are defined by their number and a name. Additionally, the start and the end of the section, the resulting length of the section and its height is given. A more detailed discussion of the various coastal defences follows in the subsequent sections of this report.

Table 3 Coastal protection in St. Peter-Ording, taken from MLR (2001)

Section – No.	Name	Start (Kkm)	End (Kkm)	Length (km)	Crest height 2000 (NN+ m)
36.01	Ording Nord (to Nackhörn)	127.603	130.440	2.837	8.6
36.02	Ording Süd (to St. Peter)	130.440	132.352	1.912	8.0
37.01	St. Peter Nord Düne	132.352	133.195	0.843	max. 16.0
37.02	St. Peter Süd Overtopping dike	133.195	135.156	1.961	6.5
38.01	Boehl Nord (Asphalt dike)	135.156	138.176	3.020	7.5
38.02	Boehl Mitte (to Süderhöft)	138.176	139.771	1.595	7.6
38.03	Boehl Süd (to Ehstenkoog)	139.771	141.070	1.299	8.3
39.00	Ehstenkoog	141.070	142.605	1.535	7.8

8.4.1 Main dike line

a) Description and cross section

The 2.8 km long section between Tümlauerkoog and Nackhörn has been reinforced in 1994 - 1996. The height of this part is 8.6 m above MSL (Figure 66). The 1.9 km long dike from Nackhörn to St. Peter is 8.0 m high. Another part of the major dike line is a of 3.0 km long dike with a crest height of 7.5 m. The following 4.5 km long section between Süderhöft and Ehstenkoog has a minimum height of 7.6 m above MSL.

Details of the cross sections of these defence elements still have to be asked for from the local authorities and the state ministries.



Figure 67 Main dike 'Ording Nord' at the North of the 'German Bight' pilot site

b) Potential failure modes

Potential failure modes of a standard sea dike in Germany have been described by Kortenhaus (2003). In this study 25 failure modes for a sea dike have been described and limit state functions have been derived for all of them. Figure 68 gives an overview of these failure modes and where they occur.

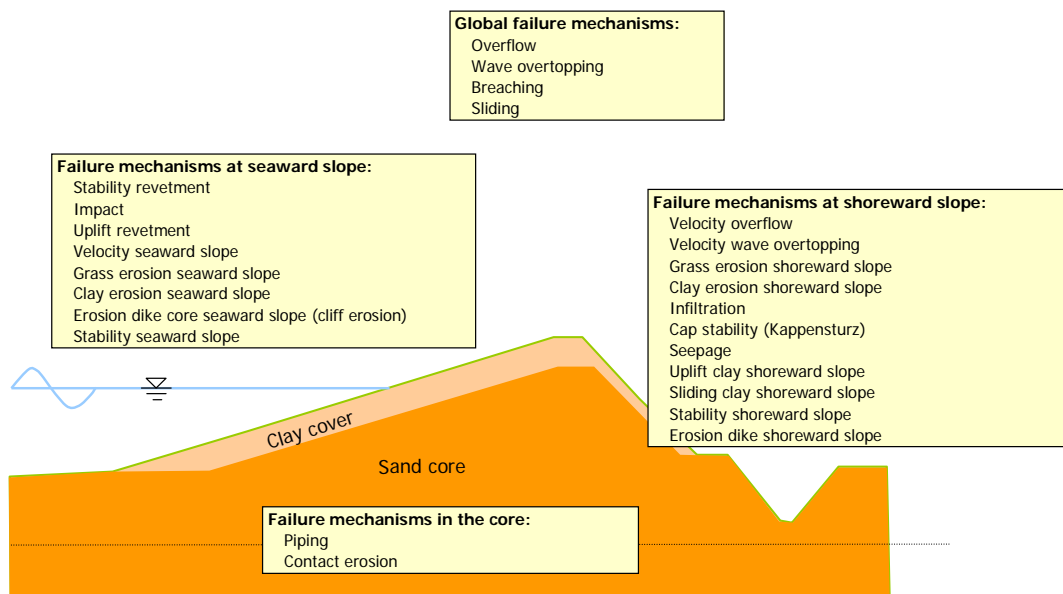


Figure 68 Key failure modes for sea dikes after Kortenhaus (2003)



Figure 69 Section 38.3 'Boehl Süd'

8.4.2 Natural dune belt

a) Description and cross section

The 800 m long natural dune belt is up to 18 m high. To cross the dunes very often staircases and pathways are provided. Figure 69 shows a view from the end of the dune section towards the main dike, photos of the dune belt and cross sections still have to be provided.

b) Potential failure modes

Failure modes of a dune are very difficult to describe in the case of the German Bight since the cross sections which can be found here are very different over the total length of the dunes in the defence line. The key failure mode which has to be described is the backward erosion of dunes when attacked by sea waves. A limit state equation for this failure mode may be derived from Larson et al. (2004) or Judge et al. (2003).

8.4.3 Overtopping dike

a) Description and cross section

An overtopping dike is designed to withstand wave overtopping and wave overflow by a non-erosive top cover such as asphalt. It is therefore considerably lower than standard dikes. The overtopping dike at St. Peter-Ording is the lowest element of the defence line. The height of this dike is 6.5 m above MSL. The dike has an asphalt cover as shown in Figure 70. Behind the overtopping dike there is a retention reservoir located.



Figure 70 Overtopping dike at pilot site 'German Bight'

b) Potential failure modes

Potential failure modes for an overtopping dike are principally the same than for the main dike. Some failure modes must however be adapted to the different cover of the dike. Due to the reduced height of the dike wave overtopping will be one of the key failure modes playing the most important role for this type of sea dikes.

8.4.4 Foreland

a) Description and cross section

Different types of foreland exist at the 'German Bight' pilot site as can be seen from Figure 71. The foreland in front of the main dike and the dunes is usually flat but may have some sandy elevations as well. In the case of St. Peter-Ording it can be very wide (up to several hundreds of meter, cross sections still have to be provided) and may have different vegetation on top.



Figure 71 Different kinds of foreland in front of the coastal defence line at pilot site 'German Bight'

Forelands as shown in Figure 71 will cause bigger waves to break early before they reach the main defence line so that the magnitude of waves reaching the dike is kept smaller than without forelands. The area of the foreland is usually maintained carefully and access to it is only possible in designated areas. It needs to be noted that forelands are only used in combination with one of the other defence structures such as main dike, dune or overtopping dike. It will never be a stand-alone solution as a flood defence structure. This combination of structures has to be considered in the description of the loading of the defence line, see Wang & Grüne (1997), Mai & Von Lieberman (1999) or Von Lieberman & Mai (2002) as an example.

b) Potential failure modes

Potential failure modes might comprise the erosion of the grass cover of the foreland so that the sand underneath is washed away. This erosion might occur either by the actions of waves or extensive use of the foreland (animals, pedestrians, cyclists, etc.).

8.4.5 Floodgate

a) Description and cross section

There is only one major floodgate as part of the hinterland in St. Peter-Ording. It closes one of the major road connections from the Northwest to the Southeast of the peninsula (Figure 72) and therefore is no part of the main defence line. The floodgate splits the flooded hinterland in two parts and is therefore very relevant for any scenario approach to where a failure of the defence line has already occurred. A cross section of the gate and dimensions still has to be received from the state ministry.

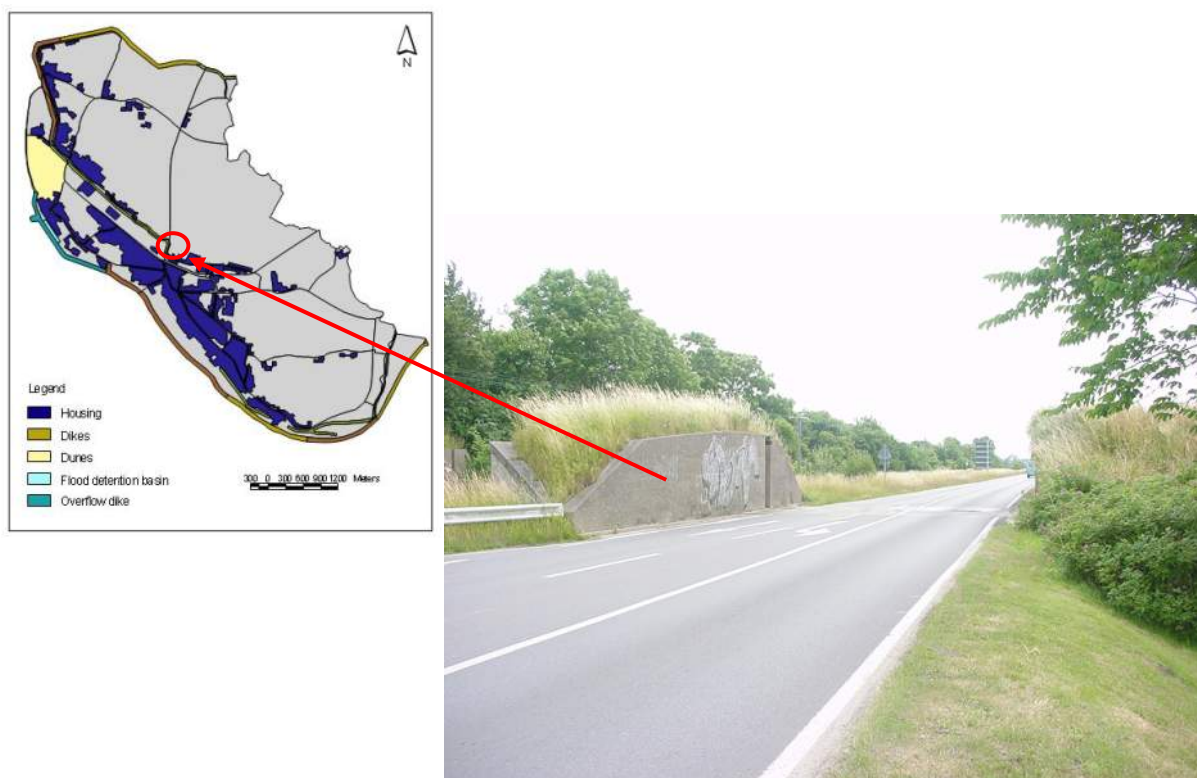


Figure 72 Floodgate in the coastal defence line at pilot site 'German Bight'

b) Potential failure modes

One of the key failure modes will certainly be that the gate cannot be closed in due time before the flood occurs. This may be either due to human or organisational errors (communication problem, availability of staff, misunderstanding, etc.) or technical problems (closure elements broken or

missing, maintenance problems, etc.). Furthermore potential failure modes might comprise the stability of the floodgate, seepage underneath it, or any geotechnical failure of the subsoil underneath.

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Note: References are not sorted in alphabetical order.

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