Ministry of Water Resources

Bangladesh Water Development Board

Coastal Embankment Improvement Project, Phase-I (CEIP-I)

Long Term Monitoring, Research and Analysis of Bangladesh Coastal Zone (Sustainable Polders Adapted to Coastal Dynamics)

Component 6: Updating of design parameters and specifications for construction works, and management practices of the polders including development of performance monitoring mechanism

6.1: Review & Update of Design Parameters

June 2022

Ministry of Water Resources

Bangladesh Water Development Board

Coastal Embankment Improvement Project, Phase-I (CEIP-I)

Long Term Monitoring, Research and Analysis of Bangladesh Coastal Zone (Sustainable Polders Adapted to Coastal Dynamics)

Component 6: Updating of design parameters and specifications for construction works, and management practices of the polders including development of performance monitoring mechanism

6.1: Review & Update of Design Parameters

September 2022

Bangladesh Water Development Board
Coastal Embankment Improvement Project, Phase-I (CEIP-I)

Long Term Monitoring, Research and Analysis of Bangladesh Coastal Zone

Office: Flat #3/B, House #4, Road #23/A, Banani, Dhaka 1213, BANGLADESH Phone +880 1307 693299

Memo No: CEIP/LTMRA/1022/201 04 October 2022

Project Management Unit
Coastal Embankment Improvement Project, Phase-I (CEIP-I)
Pani Bhaban, Level-10
72, Green Road, Dhaka-1205

Attn: Mr. Syed Hasan Imam, Project Director

Dear Mr Imam,

Subject: Submission of the Report on Review & Update of Design Parameters (D-6.1)

It is our pleasure to submit herewith Five copies of the report titled “Review & Update of Design Parameters” which is a deliverable of component 6. This report provides a review on the design practice and guidelines followed in designing dike reinforcements in Bangladesh. The review consists of a comparison of the design standard to the international state-of-the-art and providing suggestions for improvement where relevant.

It may be mentioned that the report was send to you online on 30 September 2022.

Thanking you,

Yours sincerely,

Dr Ranjit Galappatti
Team Leader

Copies: Engineer Mr. Fazlur Rashid, Director General, BWDB
Dr. Zia Uddin Baig, ADG (Planning), BWDB
Dr Kim Wium Olesen, Project Manager, DHI
Ms Sonja Pans, Deltares Project Manager
Mr Zahirul Haque Khan, Deputy Team Leader
Mr AKM Bodruddoza, Procurement Specialist
Ms Swarna Kazi, Sr. Disaster Risk Management Specialist, World Bank

Contents

FIGURES

ACRONYMS AND ABBREVIATIONS

BDP2100- Bangladesh Delta Plan 2100

BWDB- Bangladesh Water Development Board

CEIP- Coastal Embankment Improvement Project

CEP- Coastal Embankment Project

CERP-Coastal Embankment Rehabilitation Project

FGD- Focus Group Discussion

GBM- Ganges Brahmaputra Meghna

GIS- Geographical Information System

IPCC- Intergovernmental Panel for Climate Change

IPSWAM- Integrated Planning for Sustainable Water Management

IWM- Institute of Water Modelling

KII- Key Informant Interview

LCC- Life Cycle Costs

LGED- Local Government Engineering Department

LGI- local Government Institute

LRP- Land Reclamation Project

MCA- Multi Criteria Analysis

MoWR- Ministry of Water Resources

PPMM- Participatory Polder Management Model

TRM- Tidal River Management

ToR- Terms of Reference

UP- Union Parishad

WARPO- Water Resources Planning Organization

1 Introduction

1.1 Scope of work

The main objective of the Long-Term Monitoring, Research and Analysis project is to create a framework for polder design, based on understanding of the long term and large scale dynamics of the delta and on sustainable polder concepts. These polders should offer their inhabitants a safe environment to live in and sufficient opportunities for their livelihood. Among other issues, land use (agriculture including cropping pattern and adaptation of new varieties of cultivars that are emerging through research, aquaculture, housing, urbanization, etc), management of drainage congestion through control over ground level, sedimentation balance inside and outside the polder, and salinity in rivers and groundwater, are the key parameters in coming to these concepts, taking climate change into account.

As part of Component 6, a review is made of the design practice and guidelines of designing dike reinforcement in Bangladesh with recommendation for future design criteria Component 6.1.

1.2 Approach

This report provides a review on the design practice and guidelines followed in designing dike reinforcements in Bangladesh. The review focusses on the geotechnical aspects, the strength, of dike design. The review consists of a comparison of the design standard to the international state-of-the-art and providing suggestions for improvement where relevant.

According to our information the current design practice for the design of dikes in Bangladesh is provided by the *Standard design manual, volume I standard design criteria* developed by the Bangladesh Water Development Board, BWDB, hereafter referred to as the design manual.

There are many documents available that describe the state of the art of geotechnical aspects in dike design. One of the most complete set of documents in this field is the International Levee Handbook, hereafter referred to as ILH. The ILH is written by a group of international experts and published by CIRIA in 2013. The ILH is freely downloadable through: The International Levee Handbook resources (ciria.org). This website also contains links to a community of practice and a series of on-line lectures on the different chapters of the ILH. The ILH contains the latest insights on soil behaviour and different failure mechanisms in a ready to use practical framework. As such the ILH is used for the comparison to the design manual.

This report first elaborates on dike failure and the different mechanisms that play a role, in chapter 2. Next, chapter 3 discusses the design manual as explained above. Chapter 4 provides the conclusions and recommendations. Finally, chapter 5 elaborates some typical cross sections for dikes in Bangladesh delta. It should be noted that these are principle sketches. The geotechnical concepts discussed in the sections 2, 3 & 4 haven not been applied in deriving these sketches due to lack of information at this stage.

1.3 Loads & Strength

Design in general consists of a careful evaluation of loads and strength. For dikes loads are imposed by the hydraulic conditions. The strength follows from the sub soil characteristics. This report focusses on the strength of dikes. The relevant hydraulic conditions for the Bangladesh delta are presented in *Storm Surge, Wave, Hydrodynamic Modelling and Design Parameters on Drainage System and Embankment Crest Level, Volume-III: Package-3, Appendix-B: Storm Surge and Monsoon Water Level*, March 2018.(CEIP, 2018).

The principal sketches, presented in section 5, are based on the conditions presented in this report.

2 Dike failure

2.1 Failure paths

A dike fails when the strength, or resistance, of the dike is not sufficient to prevent water flowing into the area which is meant to be protected from flooding. A dangerous way of failing is found when the dike breaches and an uncontrollable amount of water flushes into the area to be protected. Dikes should be designed to prevent breaching. Breaching is rarely due to a single event. Typically, several events need to occur either sequentially or combined before breaching is reached. These events might occur within a single high-water event or develop over a period of time including several high-water events.

A failure path is a schematic projection of the succession of events that lead to failure. The failure path is a single line that starts with an initiating event, for example high river water levels or storm surge, followed by different events and results in failure. A failure tree is a combination of different failure paths. The use of failure paths and failure trees is recommended in several design codes such as the ILH (CIRIA 2013). It helps to distinguish between the different mechanisms and shows the interaction between the different mechanisms. Moreover, by the use of event trees relevant failure paths and key mechanisms can be distinguished for specific cases. The design of dike improvement can then be focussed on the identified key mechanisms.

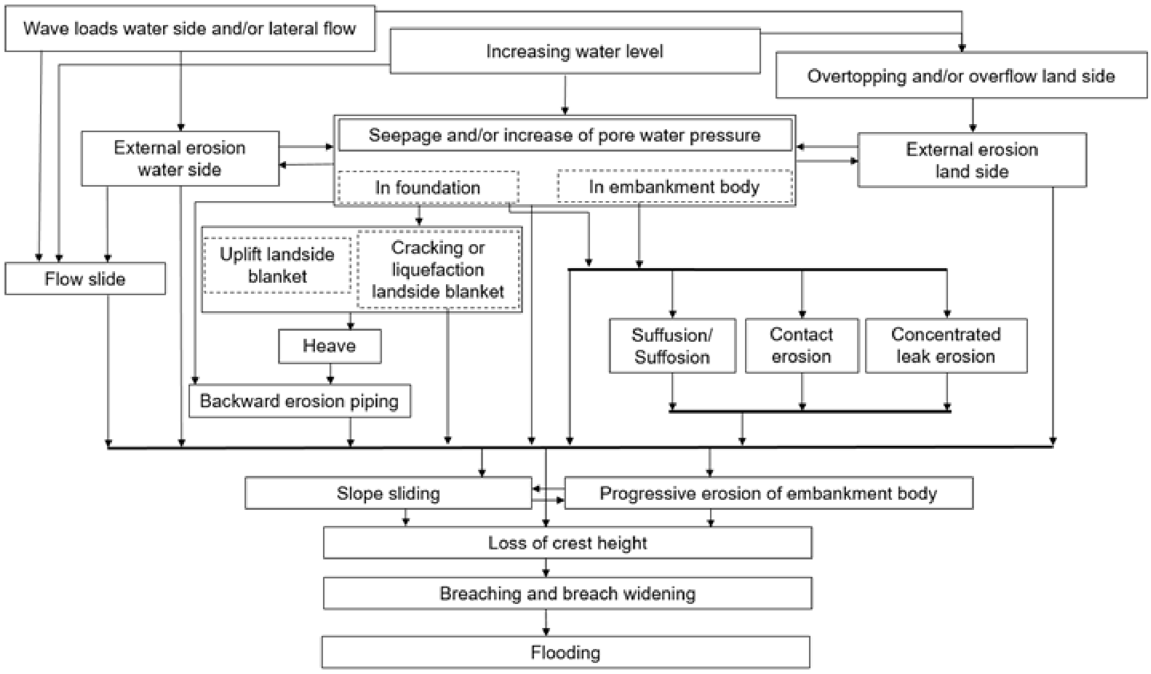


Figure 2-1, Example of a generic event tree for dikes, from: TC201 (2022)

Figure 2-1, Shows an example of a generic event tree for dikes (TC201, 2022). Starting with a hydraulic load, events like external erosion, overtopping or seepage might occur which might be followed by successive events until flooding occurs and the dike has failed. It should be noted that a failure path represents a specific single path in this event tree.

Events, or a combination of events, that occur over a longer period time allow for maintenance by which the probability of failure of a dike can be strongly reduced. Examples of this type of events is long-term erosion of the fore shore settlement of the dike due to compaction of the sub soil. These events develop gradually, and maintenance will stop, slow down the process or will bring the dike back into its preferred condition. An event tree will help in assessing the actual safety against flooding helps in prioritizing the need for maintenance of the different dike sections.

A more in-depth discussion on the application of failure paths in design and maintenance of dikes is discussed in TC201 (2022).

2.2 Mechanisms and events

The general event tree for dikes, Figure 2-1, shows the different combinations of events, failure paths, that result in flooding. Several of these events are related to the development of different mechanisms. As an illustration, this section describes the most relevant mechanisms for Bangladesh conditions.

Some of the mechanisms include the subsoil. As such the strength of a dike not only depends on the dike body and the subsoil has a considerable impact on the resistance against flooding. The geology of deltaic areas typically shows a layered system in which different clay, sand and sometimes peat layers are deposited on top of each other. The properties of these layers, layer thickness, density, permeability, strength, stiffness, are relevant for safety assessments.

In arbitrary following order the following mechanisms are briefly discussed:

External erosion

Strong currents or wave attack might result in erosion. Due to erosion the cross section of the dike will reduce. Additional events like slope failure or internal erosion by the water flow through the dike might further affect the dike until breaching is reached. A distinction can be made between erosion of the foreshore, as shown by Figure 2-3 and erosion of the outer slope, Figure 2-7.

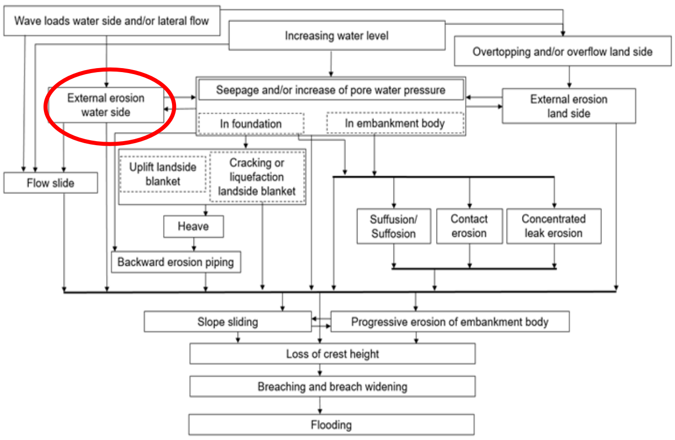


Figure 2-2, Position of external erosion in general event tree

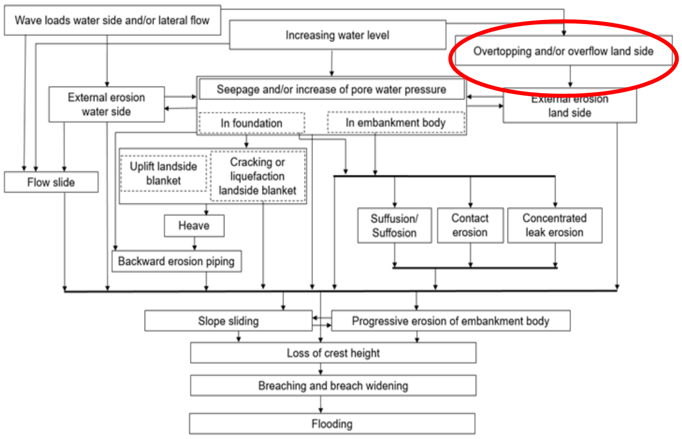
Depending on the length of the fore shore and erodibility of the fore shore, erosion of the fore shore is not necessarily a direct danger to the dike. Erosion of the fore shore develops typically over a period of time which might include several high-water events. Efficient maintenance might reduce the risk flooding when erosion of the fore shore is developing.

Erosion of the outer slope of the dike affects the available cross section of the dike and represents a more direct danger to the dike than erosion of the fore shore.

Figure 2-3, Examples of fore shore erosion in Bangladesh, photo: Mark de Bel, Deltares

Overflow / overtopping

When the dike crest is too low, water might flow over the dike. A continuous overflow is found when the water level rises above the crest, see left photo in Figure 2-5. Overtopping is found when, due to wave action, water flows discontinuous, irregular, over the dike, see right hand photo in Figure 2-5. It should be noted that for overtopping the elevation of the still water level lays below the dike crest and the peak of the wave.



Overflow and overtopping might cause erosion of the inner slope. This erosion might affect the cross section of the dike and breaching might be reached depending on the erodibility of the dike and duration of the overflow / overtopping. More details on overtopping can be found in the overtopping manual, Eurotop (2018)

Figure 2-4, Position of Overflow / overtopping in the general event tree

Figure 2-5, Examples of overflow and overtopping, Left: overflow, photo: Waternet, The Netherlands, Right: overtopping of a quay wall, photo: repository TUDelft

Failure revetment
To prevent erosion a slope protection can be build. Different types of protection are available. Each of these are characterised by different failure mechanisms. Once a slope protection fails erosion might start, which could eventually lead to breaching of the dike. An example of a slope protection is a stone revetment, see Figure 2-7. A stone revetment contains blocks that are carefully placed on the slope to be protected. Due to the pattern in which the stones are placed and the shape of the stones, interlocking of the stone provides the strength of the revetment. When for some reason stones are lifted from the revetment wave action or currents might further deteriorate the revetment. A strongly deteriorated revetment will be prone to erosion by wave action or currents.

The roughness of the slope protection influences wave run up. The revetment can be designed such that it reduces wave run up. A reduced wave run up reduces both, the possibility for erosion of the outer slope and overtopping.

In cases where overflow or overtopping is to be expected during design conditions a protection of the inner slope is needed. Typically, the protection of the inner slope is created by a high-quality grass cover.

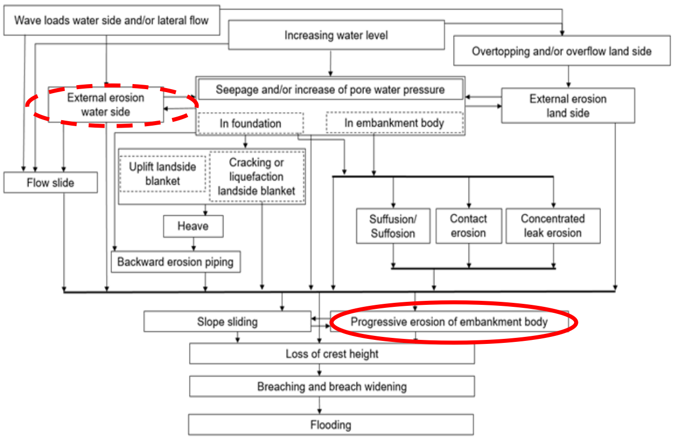


Figure 2-6, Position of revetment failure in general event tree

Revetment failure is not mentioned in the general event tree. It represents an underlying mechanism. In this frame work an underlying mechanism is a mechanism that needs to take place before the mechanisms or events mentioned in the general event tree, Figure 2-2, occurs. Figure 2-6 shows the position of the revetment failure in the general event tree.

Figure 2-7, Example of revetment failure, photo: Mark de Bel, Deltares

Settlement
Compaction of the subsoil due to the weight of the dike results in settlement. As consequence of settlement the original elevation of the ground or crest of the dike is lowered over time, enhancing the mechanisms overflowing and overtopping. Settlement should be accounted for when designing or building new dikes or dike re-enforcements.

The susceptibility for settlement is different for different soil types. Silts and sands are typically stiff and compaction due to loading will be relatively small. For sands and silts the deformations will typically develop during construction or shortly afterwards. For organic soils, like peats or organic clays, compaction might be considerable. For these materials, the compaction develops over a long period of time and might still occur long after construction of the dike or re-enforcement is finished.

Although sands and silts might show little compaction under static loading, loosely packed sands and silts might compact under cyclic loading. The shear forces induced by the cyclic loading might result in a rearrangement of the sand or silt particles, reducing the pore space between the particles. Typical sources for cyclic loading are wave action or earthquakes. In extreme cases, the sand or silt will lose its integrity and liquify under cyclic loading. Liquefaction of sands or silts causes a dramatic reduction in strength of these layers and can have severe consequences for dike stability.

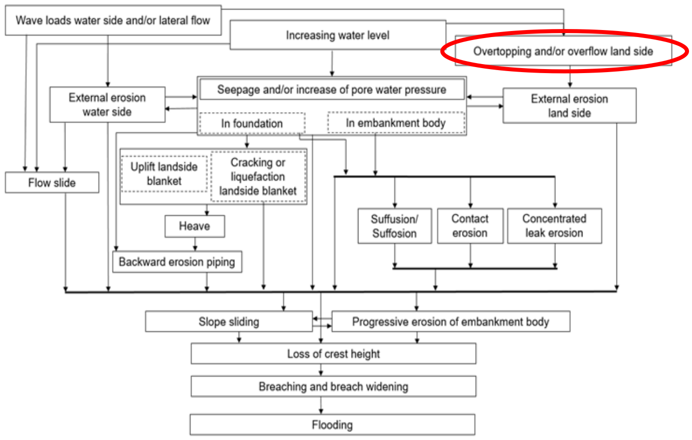


Figure 2-8, Position of settlement in general event tree

Settlement is an underlying mechanism and not directly mentioned in the general event tree. Settlement facilitates overflow and overtopping.

Slope Stability
Failure of the slope strongly affects the cross section of the dike body. After failure of the slope the remaining cross section might be endangered by erosion due to overflow or overtopping, or by internal erosion. These different sources of erosion might result in breaching of the dike body.

Typically, slope stability is evaluated by slip circle analysis, see Figure 2-9. Slip circle stability not only considers the dike body itself, but also the subsoil. Classical slope stability analysis considers equilibrium of a circular or wedge type failure plane. Many classical handbooks provide further background information on slip plane analysis, for example CIRIA (2013), Verruijt (2012).

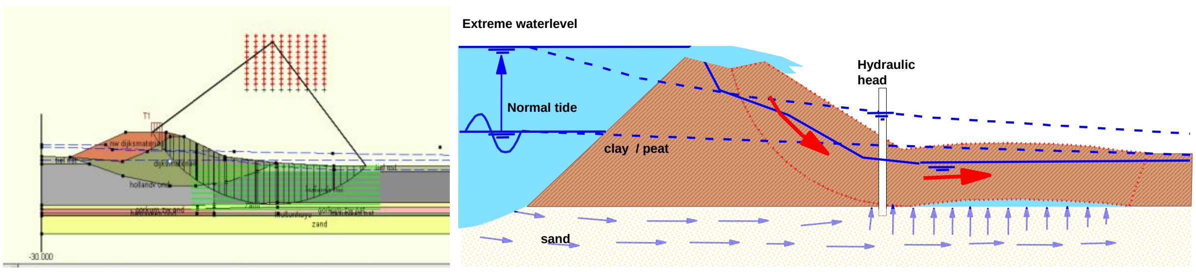


Figure 2-9, *Slope stability, including subsoil strength, left: classical deep slip circle, right: uplift, the influence of hydraulic head in deeper layers. (from left to right Figure a, b, c)*

Figure 2-9 shows the influence of a soft, low strength, clay layer at some depth below the embankment in the Figure 2-9a. Besides strength characteristics, hydrology might also play a role, as shown in the right-hand Figure. If a permeable layer is in hydraulic contact with the sea or river, an extreme water level will induce an increase in hydraulic head in this permeable layer. When the permeable layer is overlain by a low permeable clay layer at the inner toe of the dike, a hydraulic pressure might act on this clay layer. The hydraulic head might reach a level at which the clay layer is uplifted and friction between clay and sand layer is reduced. Due to this uplift the support of the slope is reduced, and slope failure might occur.

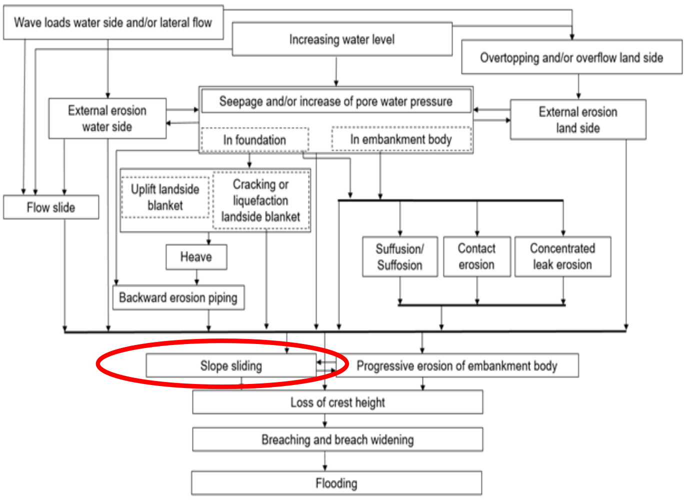


Figure 2-10, Position of slope stability in the general event tree

Internal erosion / piping

During high water conditions water will flow through dike and subsoil. For some conditions this water flow might reach velocities that causes erosion. When erosion occurs in the dike body this process is referred to as internal erosion. Piping refers to erosion in the subsoil. The combination cohesive and non-cohesive allows the creation of hollow spaces underneath the cohesive layers due to erosion in the non-cohesive layers, see Figure 2-11. Collapse of the hollow spaces might reduce in lowering of the crest height followed by overflow of overtopping which might result in breaching.

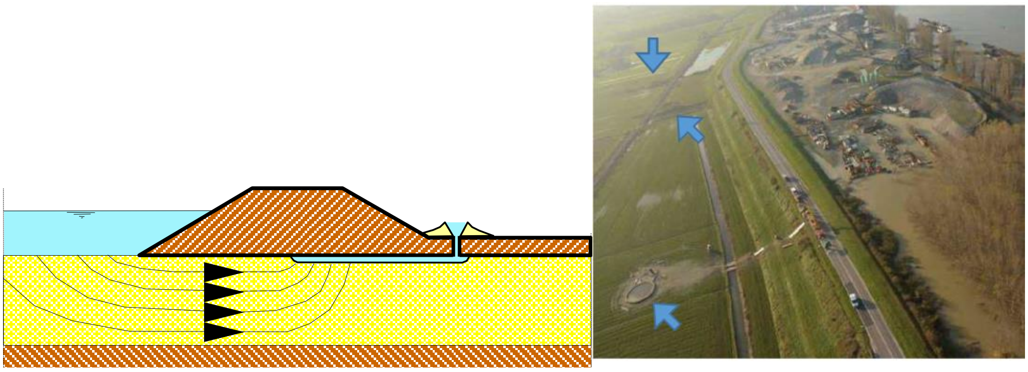


Figure 2-11 *Backward erosion piping, left; sketch of the mechanism, right: High water along the river Po, Italy, exit points indicated by blue arrows, from: Bezuijen (2017) (from left to right Figure a, b)*

Figure 2-11 shows the piping mechanism in Figure 2-11a. Groundwater flow underneath a dike constructed of cohesive material might result in erosion. At the polder site an exit point is found where water runs of along ground level and the eroded material forms a crater. The right-hand Figure shows the conditions along the river Po, Italy. At some distance from the dike several exit points are found, indicated by the blue arrows. This mechanism is also referred to as backward erosion piping in contrast to suffusion and suffosion for which the grain skeleton is not stable and fine particles are washed out, followed by compaction of the sand layer and settlement of the dike.

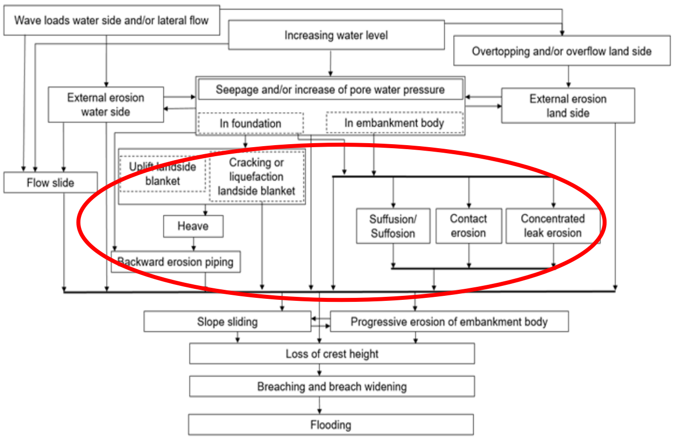


Figure 2-12, Position of internal erosion and piping in general event tree

Internal erosion and piping are found at the heart of the general event tree, see Figure 2-12.

3 Content design manual

Chapter 7 of the design manual, BWDB (1995) discusses the design criteria for embankments. In 14 pages it discusses the geotechnical aspects of water retaining embankments. Chapter 7 distinguishes first between full flood protection embankments, submersible embankments and sea dikes, with the main difference being the hydraulic loading on the embankments. After a discussion on the alignment of the embankment, the design manual discusses the required embankment height, followed by internal and external stability of the embankment. Regarding stability the following aspects are discussed:

- The countryside slope should be stable under outflowing seepage through the dike
- The river / seaside slope should be stable after a rapid river level drawdown
- The phreatic line should be well within the downstream face
- Sufficient stability of the embankment fill
- Slope protection against wave action and rainfall

Regarding seepage through the dike the design manual discusses the position of the phreatic line and the derivation of the maximum hydraulic gradient, i_{max}, relates the i_{max} to the slope angle, α and checks if the i_{max} is sufficiently smaller than the buoyant weight of the soil. This is a first approximation to check on internal erosion due to seepage through the dike body. This procedure is only related to the geometry of the dike body and does not account for different construction materials. It is to be expected that grain size distribution of the dike body plays a role as well as differences between different materials within the cross section of a dike body. A more detailed discussion on the internal stability is provided in the ILH.

Regarding slope stability, the design manual prescribes a slip circle analysis following the Swedish method or Bishop's Method. The design manual further gives an explanation how to obtain the critical slip circle manually. The key in applying these clip circle methods is to find the most critical slip plane. Low budget software or even freeware, like D-Stability (D-Stability - Deltares) is available to rapidly assess slope stability using slip circle methods more accurately than can be achieved manually. The different software packages also allow to use more advanced slip plane methods, like the Spencer method or the Morgenstern – Price method. Slip planes in these advanced methods have more degrees of freedom and as such can find slip planes that deviate from the shapes prescribed in the less advanced models, like circular slip planes in Bishop's method. It should be noted that these advanced methods use the same input parameters as Bishop's method or Swedish method and as such can result in an improvement of the stability assessment at low costs.

The design manual prescribes a required Safety Factor, $SF_{req} = 1.5$. For cross sections resulting in a lower calculated SF, the geometry should be changed by changing the inner slope or stability berm to increase the calculated SF until SF > 1.5. The design manual does not prescribe how to obtain the strength parameters and if best guess parameters or cautious estimate parameters or otherwise should be used. This procedure does not acknowledge geotechnical uncertainty. Due to natural heterogeneity, measurement inaccuracies etc., the geotechnical parameters include uncertainty. Not including this uncertainty in the stability analysis might result in a wrong impression of the probability of failure. This is explained in Lacasse et al. (2019) and repeated in Figure 3-1.

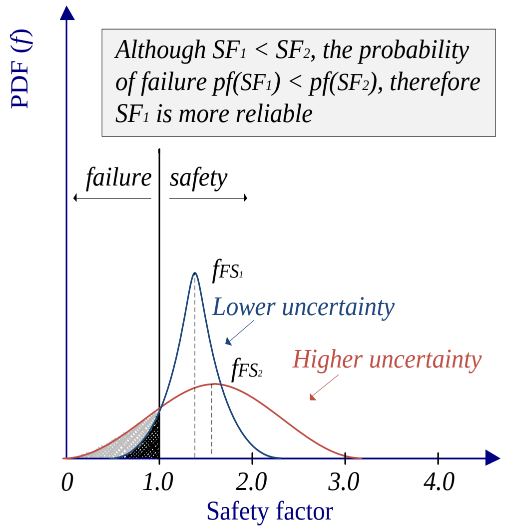


Figure 3-1 *Safety factor and failure probability of a slope (from: Lacasse et al, 2019)*

Figure 3-1 shows two cases for which safety factors are calculated. One case has a high uncertainty, red line in Figure 3-1 and another case with a lower uncertainty, blue line in Figure 3-1. Calculations resulting in SF < 1.0 represent slope instability and the portion of the probability density function, PDF for which SF < 0, represents the probability of failure. The case with the highest uncertainty, in this example, also results in the highest mean value for the calculated SF. Only looking to the mean values of the calculated SF results in a wrong conclusion on safety. Since, in this example, the case with the highest uncertainty not only results in the highest calculated mean value of SF, but also in the largest probability of failure. Due to the large uncertainty, the case with the highest calculated mean value for SF, does not represent the safest situation.

There are several ways to include geotechnical uncertainty in the analysis. For practical applications it is referred to the ILH. It should be noted, that including geotechnical uncertainty is rewarded by possibility to optimize the design by obtaining more information on uncertain parameters.

It is remarkable that the design manual prescribes one required safety factor, SF_{req}, while making a deviation in different types of water retaining embankments, like full flood protection embankments, submersible embankments and sea dikes. It could be argued that the consequences of failure of a submersible dike will be different from failure of a sea dike. As such, an optimisation in design could be found by applying different SF_{req} for the different types.

In section 7.10 the design manual describes the procedure to calculate settlement induced by the weight of the embankment. The procedure, described in the design manual, does not account for the change in stiffness when reaching the yield stress, nor does it include creep. The stress increments at larger depth, considering stress distribution in depth, is prescribed by a design chart, based on uniform soil conditions and uniform geometry.

It should be noted that several low-budget or free software is available to calculate stress increments and corresponding settlement for complex geometries. These more advanced techniques are especially beneficiary for dike reinforcements, where an initial, small, dike body is improved to a larger construction and geometry is far from uniform.

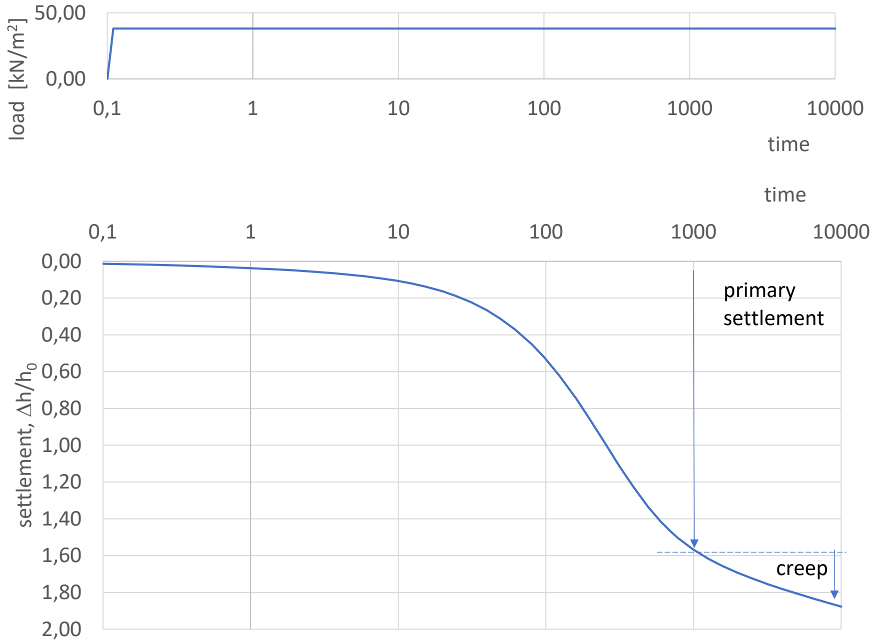


Figure 3-2 *Typical time settlement curve*

Figure 3-2 presents a typical time settlement curve induced by sub soil loading. Typically, settlement is subdivided in different components of which primary settlement and creep are the most relevant. Primary settlement follows from a re-arrangement of the soil particles and the outflux of pore fluid, while creep reflects the long-term viscous behaviour of soil. Sub soil compressibility strongly depends on the soil type. For sands and silts creep might be small and neglecting creep for these materials will have little to no influence on the design.

For clays and organic soils, like peat, creep can have a considerable impact on the settlement observed in the field. The typical time – settlement plot, given by Figure 3-2, shows settlement until 10,000 days after application of the load. For clays creep might contribute 20 to 30% of the total settlement reached at 10,000 days, while for peats this might be 40 to 50%.

Although the design manual prescribes removal of peat from the subsoil before constructing the dike, organic and non-organic clays also show creep behaviour. Moreover, removal of peat layers from the subsoil might not always be possible.

The design manual briefly prescribes the slope protection. Section 7.11 describes grass as a protection method, without specifying the criterion for which a grass cover will provide sufficient resistance against erosion. Chapter 10 discusses placed revetments and rip-rap as a measure to protect slopes against erosion.

A field study of observed damage and failure of dikes shows that erosion by currents, wave attack, overflow or overtopping forms the main failure mechanism for dikes in Bangladesh at this moment. Slope protection is, therefore, an important aspect in dike design. The latest insights in slope protection are found in the International Levee Handbook, ILH or the manual on wave overtopping 2018, www.overtopping-maunal.com

Chapter 7 of the design manual finalizes with some thoughts on submersible dikes and closure dams.

4 Comparison & Recommendations

4.1 Comparison design manual to state of the art

Designing in general contains an evaluation of loads and resistance. For dikes, the loading is formed by hydraulic conditions, like extreme river or sea level, wave action and pore pressure changes in the dike or subsoil. Resistance mainly follows from the geotechnical aspects, like strength and stiffness of dike body and subsoil. A well-balanced design includes a good understanding of both loads and resistance, or hydraulic and geotechnical aspects.

This report provides a brief review of the geotechnical aspects in the design of water retaining embankments, dikes, in Bangladesh. The BWDB design manual includes a chapter 7 which discusses the geotechnical aspects. This chapter 7 does not include the latest developments in geotechnical engineering and at some points uses hand calculations, where reliable low budget and even free software is available for more advanced analysis. As an example of a more detailed design guideline, it is referred to the international levee handbook, ILH.

The safety factors prescribed for the different failure mechanisms do not seem to include geotechnical uncertainty and therefore might not result in a correct view on the level of safety reached in the design. Moreover, the same safety factor is used for the different dike-types, like full flood protection embankments, submersible embankments, and sea dikes. A deviation in factor of safety for the different consequences of failure, for example between submersible dikes and sea dikes or related to the expected financial loss or number of casualties would lead to a better-founded safety philosophy. Accounting for geotechnical uncertainty would provide a better connection between strength of loads, where loading conditions are related to a probability of occurrence and tolerated probability of failure.

The BWDB design manual seems to pay little attention to sub soil characterisation and parameter assessment. This seems to comply to the failure mechanisms typically found for dikes in Bangladesh. An inventory showed that erosion, due to current, wave attack, overflow or overtopping forms the most relevant failure mechanism. However, if dikes are improved such that they can without these erosion processes under design loading conditions, only a true improvement in safety is found when these dikes have sufficient resistance against other failure mechanisms. As such, state-of-the-art knowledge on the different failure mechanisms should be available and applied in dike design.

Besides design, maintenance and operation are important aspects in the actual safety against flooding. The BWDB design manual only covers design. Additional information on maintenance, inspection and emergency management can be found in the International Levee Handbook.

4.2 Additional remarks

The BWDB design manual provides limited information on geotechnical aspects of dike design. The information presented in the design manual is mainly focussed on the dike body itself and not subsoil conditions. This complied to the damage and failure observed in the field during field visits by Mark de Bel. The observed damage is mainly caused by erosion due to strong currents, wave attack, overtopping or overflow. Failure mechanisms that include the subsoil are not reported.

It should be noted that if the dikes are improved and are no longer susceptible for erosion, failure mechanisms that include the subsoil might gain relevance. A real improvement in safety is found when the improved dikes also have sufficient resistance against the failure mechanisms that include sub soil behaviour.

Besides designing new dikes or dike reinforcements, maintenance is important in achieving and maintaining safety against flooding. The design guideline does not seem to pay attention on maintenance aspects. Proper maintenance starts with regular inspection to timely detect deterioration of the dike and planning of repairments based on the observations. Special attention should be given to inspection during extreme loading conditions and rapid response when damage is found. The international Levee Handbook, ILH, discusses the different aspects of maintenance with chapters on Operation & Maintenance, Levee inspection and Emergency management and operations.

To assess the resistance against slope instability and piping or to be able to produce a reliable settlement prediction information of the sub soil is needed. This information can be obtained at different levels.

A first level is the identification of the different layers present in the sub surface. This information can be obtained by borings and CPT(u) data. The data can be combined to geotechnical profiles, as shown by Figure 4-1. It should be noted that gathering all the information to construct a detailed geotechnical profile is an effort in which the required data might be gathered over several years. This implies proper archiving of the available data and a regular update of the profiles when new data becomes available.

Geophysical techniques can be of added value to rapidly detect the sub soil layering for large dike stretches. Examples of geophysical techniques are ground penetrating radar, GPR, electrical resistivity tomography, ERT, or ground conductivity measurements, EM. Also shear wave velocity measurements present a reliable method for detecting sub soil layering, ASTM (2014[a,b]). Recent advances resulted in the multichannel analysis of surface waves, MASW, technique. This technique has been proven to be a very practical and cost-effective method for soft soils (Long & Donohue, 2010), (L'Heureux & Long, 2017)

A second level is characterising the soil by the indices, like density, water content, sieve curves, plasticity indices etc. These relatively simple to determine parameters provide a first indication of soil behaviour. The indices help to further improve the geotechnical profile, distinguish between the different relevant mechanisms and assessment of stiffness and strength parameters using correlations.

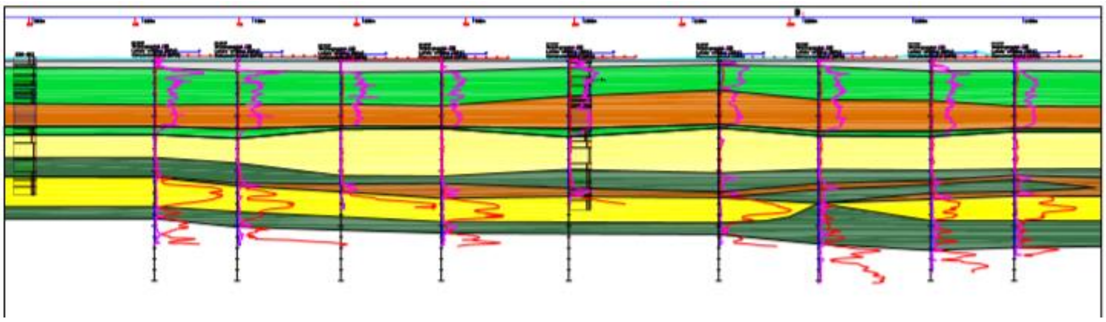


Figure 4-1 *Example of Geotechnical profile, different colours indicated the different soil layers, the pint and red lines represent the CPTu data on which the profile is based.*

A third level includes advanced laboratory testing, which results in stiffness and strength parameters that can be used in evaluating the different failure mechanisms. It might not be feasible for all dike reinforcement projects to conduct advanced laboratory testing. However, it could be applied for cases where the consequences of flooding are the largest. Moreover, advanced laboratory testing can be used to derive or improve the correlations using the indices as determined in the second level. To assure a minimum quality of subsoil information and to allow regional use of locally derived information, for example by building correlations, which have a wider application then the sample location, the design manual should preferably provide guidelines on laboratory testing.

5 Principal sketches

5.1 Introduction to the principle sketches

Along the coast of Bangladesh and more inland along the river systems that form the Bangladesh delta several polders have been created in the past. To improve the safety against flooding of the polders an evaluation of the water retaining systems has been made and several measures to improve safety against flooding are elaborated. One of these measures is improving the dikes around the polders. This memo describes a series of principle sketches of improved dike cross sections. Based on these sketches a first impression of the consequences of dike improvement in this area can be elaborated.

The differences between the different elaborated cross sections follow from differences in loading conditions and bathymetry of the foreshore. It should be noted that subsoil conditions can have a strong influence on the required geometry of a dike. For example, low sub soil strength might result in low bearing capacity and therefore stability issues. Internal erosion processes, like piping, might weaken sub soil strength and as such induce failure during a high-water event. At the moment of writing this report, little to no sub soil related information is available. However, a visual inspection of the protection system of one of the polders shows that besides the height of the dikes and levees, foreshore erosion due to wave action or river scour represents the main failure mechanism. Stability issues and / or internal erosion, either through the dike body or in the subsoil was not observed. It should be noted that these mechanisms could become relevant when upgrading the existing structures such that it can withstand higher water levels.

A proper dike reinforcement design should deal with all relevant failure mechanisms. Due to the absence of subsoil information, the resistance against the different failure mechanisms is not elaborated here. The actual design of dike improvement should be checked following the comments on geotechnical design of dikes given in the previous chapters. The principle sketches, discussed in this section, cannot be seen as a design of the required dike reinforcement. Instead, the principle sketches are meant to provide an educated impression what the dimensions of the dike reinforcement could be. Since raising a dike also requires widening the base, to secure stability issues, conflicts with land ownership and the presence of buildings on or the present dike might emerge. The principle sketches are meant to acquire a first approach in the footprint of the improved dike system and will be the input for a budget estimate required to realize the improved safety against flooding.

5.2 Construction material

At this early stage of designing the dike reinforcement, the construction material is assumed to be produced by locally dredged material. At this moment no specifications of the locally dredged material are available and the applicability of this material for dike construction is unknown. When using dredged material for dike construction, the following issues are relevant:

Liquidity;
When the material is too wet, the material volume will considerable shrink upon drying. This volume reduction will lead to considerable cracking when the initial water content is above the liquid limit of the material. It is obvious that considerable cracks in the dike body should be avoided. The liquidity index, I_l, and consistency index, I_c, is a measure for the liquidity of the material:

$$I_l = \frac{w - PL}{I_p}, \quad I_c = \frac{LL - w}{I_p}$$

In which:

I_l = liquidity index
I_c = consistency index
w = water content
PL = plastic limit
I_p = plasticity index, $I_p = LL - PL$
LL = liquid limit

It should be noted that the liquidity index is the inverse of the consistency index, I_c. Typically, after dredging, the water content will be beyond the liquid limit and $I_l > 1$. In that case the material should allow to dry until the water content is dropped sufficiently and $I_l < 0.25$, $I_c > 0.75$, before it is used as a construction material. For the core of the dike $I_c > 0.6$ can be used. Drying clay means that the dredged material should be placed in ridges of 0.5 – 1.0 m high and is allowed to dry for several weeks or months before used as construction material.

Ability to densify
To improve material properties like stiffness and strength, resistance against erosion and reduce development of cracks, the material should be densified when building the dike. For optimum densification, the material should not be too wet, see text above. For clayey material, the densification should be done by using a static load.

Typically, silts and sands in Bangladesh contain mica. Mica is recognised as the shiny thin plates that are present between the sand grains. The presence of the flaky mica particles between the spherical sand particles will cause bridging and other complex behaviour at microscopic level. As such micaceous sands are known for their problematic densification abilities. Before the material is used for construction material the properties of the material and the ability for densification should be tested.

Erodibility
Sand and silty material will easily erode when in contact with water. When the construction material contains a high silt or sand fraction erosion problems are to be expected. Regarding shore protection, a revetment or other protective constructions will be built. However, the material should contain at least enough resistance against erosion due to rainfall. Consequently, the construction material should contain a sufficiently high clay fraction.

The consequences of these issues above are that the dredged material might not be used directly in the same cross section. Instead mining locations should be found where the right material can be found and / or processing of the material is required before it is applied for construction. Construction alternatives in which the dike body is constructed of an erodible material, sand, protected by a clay cover will not be considered. Since the main failure mechanism, observed in the field, is erosion due to waves and river scour, high resistance against erosion is favoured for the entire construction.

5.3 Principle sketches

In total 5 principle sketches are derived for the different loading conditions that can be found in the Bangladesh delta. The loading conditions will be discussed in the description of the principle sketches below. The principle sketches are developed on the following rules:

- The daily mean water table is taken at PWD 0 m.
- The ground level fluctuates between PWD + 1.0 m and PWD + 2.0 m. The dimensions in the principal sketches are elaborated for both values.
- The loading contains a combination of storm surge and wave action. The raised water level due to the storm surge are referred in the principle sketches as Design Water Level. The design wave action is represented by a significant wave height at deeper water, H_s.
- The relevant storm surge levels and significant wave heights are taken from the CEIP technical report on *Storm Surge, Wave, Hydrodynamic Modelling and Design Parameters on Drainage*

System and Embankment Crest Level, Volume-III: Package-3, Appendix-B: Storm Surge and Monsoon Water Level, March 2018, (CEIP, 2018)

- The crest width is taken as 3 m in all sketches. In case of severe wave action, a wider crest might be beneficial in improving the probability of failure due to erosion by wave action.
- The inner slope in all sketches are taken at 1(V):3(H). This is an engineering estimate to create a slope which is sufficiently stable for typical construction materials. For low quality construction material and large construction height for example the sea defence Figure 5-6 and Figure 5-7, the slope might be built shallower.
- The slope angle of the outer slope is selected in combination with dike height and expected wave action during design conditions. Wave overtopping should be kept to a minimum unless the inner slope is protected to withstand the expected amount of wave overtopping. The expected amount of wave overtopping during design conditions is estimated following the Overtopping Manual (Eurotop, 2018) and the computer program PCOverslag.
- According to the overtopping manual an unprotected inner slope should not be loaded with overtopping discharge more than 0.1 l/s/m. A high-quality grass cover can withstand an overtopping discharge of 5 l/s/m. The freeboard presented for the sketches below is based on an overtopping discharge of 1 l/s/m. Consequently, it is assumed that the inner slope is protected by at least a low-quality grass cover.
- In cases where no severe wave action is expected during design conditions, the outer slope angle is selected as 1(V):3(H).
- An additional freeboard of 0.5 m is added. This freeboard accounts for settlement due to added weight, setting of the dike body, general land subsidence and inaccuracies in construction. Due to lack of sub soil information a settlement prediction cannot be made, and 0.5 m is a rough estimate. When organic soils are present in the sub soil settlement, larger than 0.5 m, is to be expected. Regarding settlement it should be noted that:
  - Dikes are already present, so the sub soil is already pre-loaded and not full cross section, as presented below in the principle sketches can be considered as a new loading.
  - Soft soil is expected more inland, where due to reduced wave conditions dike heights are also reduced. The larger dikes are required at the coast where the sub soil contains more stiff sands and silts.
- Due to the additional freeboard the overtopping discharge during design conditions at the start of the life cycle of dike will be smaller than 1 l/s/m/, for which the overtopping freeboard is determined. During its lifetime, the dike will settle, and the expected overtopping discharge will increase to 1 l/s/m.

The different sketches are discussed below:

<u>Sketch 1, Narrow channel with foreland</u>
Specifications:

- Some polders are separated by relatively narrow channels.
- During extreme events the water level might rise considerably.
- Wave action and currents are absent or insignificant.
- Daily water levels are within the channel; Under daily conditions the dikes remain dry
- An example of this situation is the channel between polder P45 and P44, with design conditions given for points 39 and 50 in the appendix B of the technical report *Storm Surge, Wave, Hydrodynamic Modelling and Design Parameters on Drainage System and Embankment Crest Level*

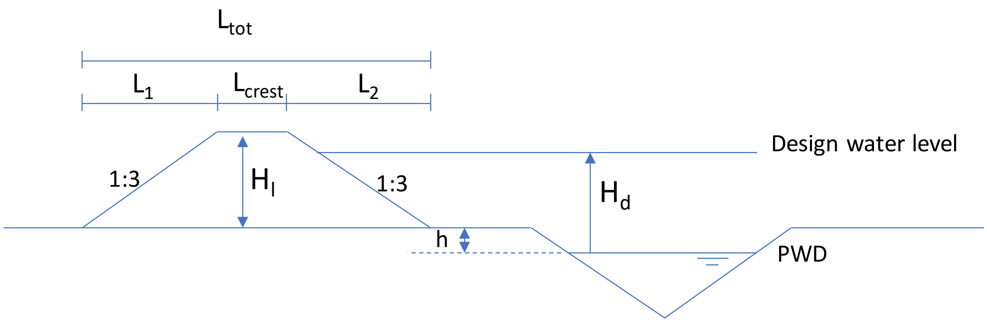


Figure 5-1, Sketch case 1, narrow channel with foreland

parameter	Symbol	unit	set 1	set 2
ground level relative to mean sea level	h	[m]	1.0	2.0
design water level relative to mean sea level	H_d	[m]	5.5	5.5
crest level relative to ground level	H_l	[m]	5.0	4.0
length polder side slope	L_1	[m]	15	12
length river side slope	L_2	[m]	15	12
crest width	L_c	[m]	3.0	3.0
total length dike body	L_{tot}	[m]	33.0	27.0

Due to the absence of wave action and strong currents erosion will be limited. A good grass cover or similar protection will suffice to withstand erosion due to rainfall and occurrence of design water level.

<u>Sketch 2, Narrow channel without foreland</u>

Specifications:

- Basically equal to sketch 1,
- During extreme events the water level might rise considerably.
- Wave action and currents are absent or insignificant.
- No foreland presence; dike directly retains water also during daily conditions.

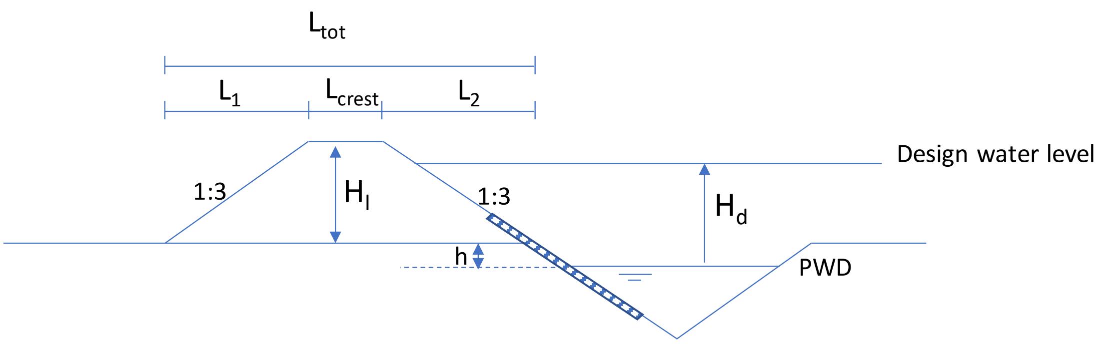


Figure 5-2, Sketch 2, Narrow channel without foreland

parameter	Symbol	unit	set 1	set 2
ground level relative to mean sea level	h	[m]	1.0	2.0
design water level relative to mean sea level	H_d	[m]	5.5	5.5
crest level relative to ground level	H_l	[m]	5.0	4.0
length polder side slope	L_1	[m]	15	12
length river side slope	L_2	[m]	15	12
crest width	L_c	[m]	3.0	3.0
total length dike body	L_{tot}	[m]	33.0	27.0

Due to the absence of wave action and strong currents erosion will be limited. Above the water table, a good grass cover or similar protection will suffice to withstand erosion due to rainfall and occurrence of design water level. Around the water level is stone revetment is required.

<u>Sketch 3, along the riverbank</u>
Specifications:

- Along the riverbank
- Foreland typically above daily water levels
- No scour erosion. (sketch 4 includes the conditions with scour)
- if dense mangrove forest is present wave action at the dike will be negligible. Therefore, two options: with and without wave action

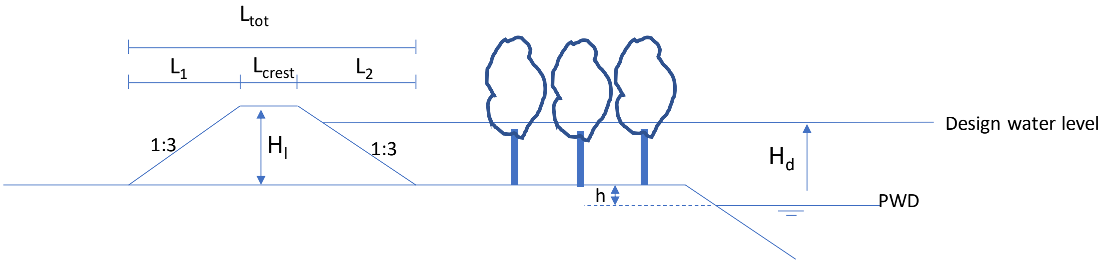


Figure 5-3, Sketch case 3a, river bank with foreland, no wave action

parameter	Symbol	unit	set 1	set 2
ground level relative to mean sea level	h	[m]	1.0	2.0
design water level relative to mean sea level	H_d	[m]	2.75	2.75
crest level relative to ground level	H_l	[m]	2.25	1.25
length polder side slope	L_1	[m]	6.75	3.75
length river side slope	L_2	[m]	6.75	3.75
crest width	L_c	[m]	3.0	3.0
total length dike body	L_{tot}	[m]	16.5	10.5

When the mangrove is absent, waves will reach the dike and additional height might be required to minimise the amount of overtopping. Large waves will break when reaching the foreland, reducing wave impact on the dike. The level of impact reduction depends on the length and elevation of the foreland. The table below provides the dimensions for two different slopes in combination with different elevations of the foreland and a foreland length of 50 m.

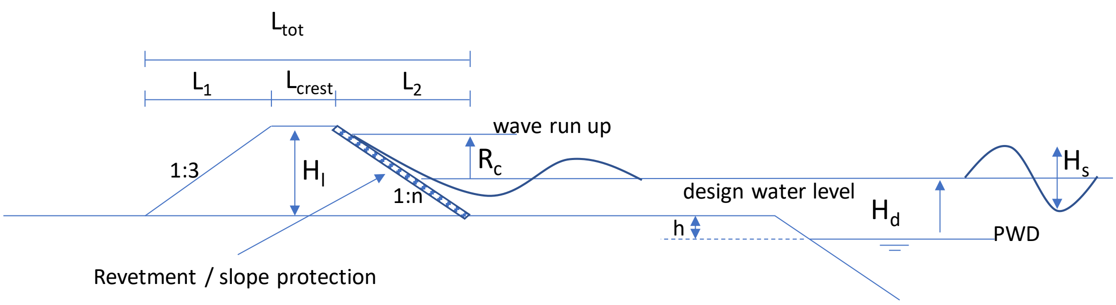


Figure 5-4, Sketch case 3b, Riverbank with foreland and wave action

parameter	Symbol	unit	set 1	set 2	set 3	set 4
ground level relative to mean sea level	h	[m]	1.0	2.0	1.0	2.0

steepness outer slope	n	[-]	3	3	4	4
design water level relative to mean sea level	H_d	[m]	2.75	2.75	2.75	2.75
significant wave height	H_s	[m]	1.5	1.5	1.5	1.5
Freeboard, to reduce overtopping to q_c =1 l/s/m	R_c	[m]	2.25	0.81	1.65	0.81
crest level relative to ground level	H_l	[m]	4.5	2.0	3.9	2.0
overtopping discharge at design conditions	q_c	[l/s/m]	0.25	0.07	0.16	0.07
length polder side slope	L_1	[m]	13.5	6.0	11.7	6.0
length river side slope	L_2	[m]	13.5	6.0	15.6	8.0
crest width	L_c	[m]	3.0	3.0	3.0	3.0
total length dike body	L_{tot}	[m]	30	15	30.3	17

<u>Sketch 4, Riverbank with scour</u>

Specifications:

- Along the riverbank
- Foreland is under attack from river scour
- In the sketch the presence of the foreland is neglected

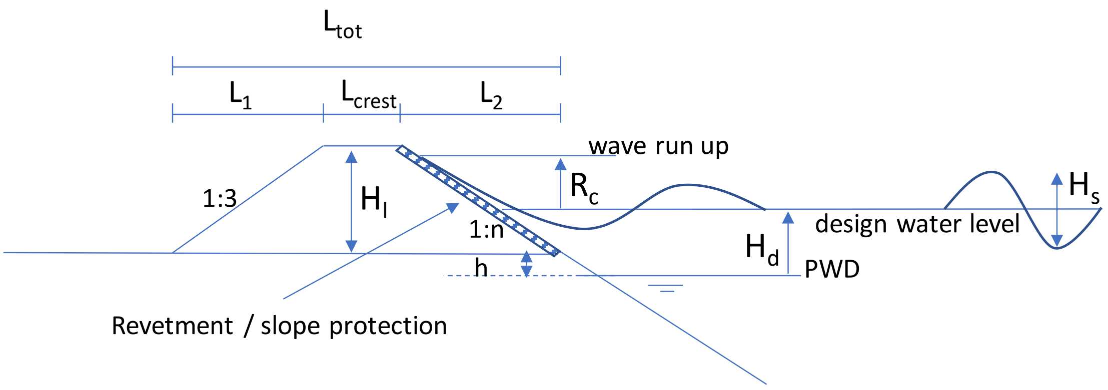


Figure 5-5, Sketch case 4, Riverbank, no foreland with wave action

parameter	Symbol	unit	set 1	set 2	set 3	set 4
ground level relative to mean sea level	h	[m]	1.0	2.0	1.0	2.0
steepness outer slope	n	[-]	3	3	4	4
design water level relative to mean sea level	H_d	[m]	2.75	2.75	2.75	2.75
significant wave height	H_s	[m]	1.5	1.5	1.5	1.5
Freeboard, to reduce overtopping to q_c =1 l/s/m	R_c	[m]	3.25	3.25	2.35	2.35
crest level relative to ground level	H_l	[m]	5.5	4.5	4.6	3.6
overtopping discharge at design conditions	q_c	[l/s/m]	0.32	0.32	0.25	0.25
length polder side slope	L_1	[m]	16.5	13.5	13.8	10.8
length river side slope	L_2	[m]	16.5	13.5	18.4	14.4
crest width	L_c	[m]	3.0	3.0	3.0	3.0
total length dike body	L_{tot}	[m]	36.0	30.0	35.2	28.2

Sketch 5, Sea defence

Specifications:

- Direct wave attack from the sea
- Sandy coast
- Due to large wave loads and storm surge, two solutions have been elaborated;
    - Protected slope
    - Protected slope with spilling berm

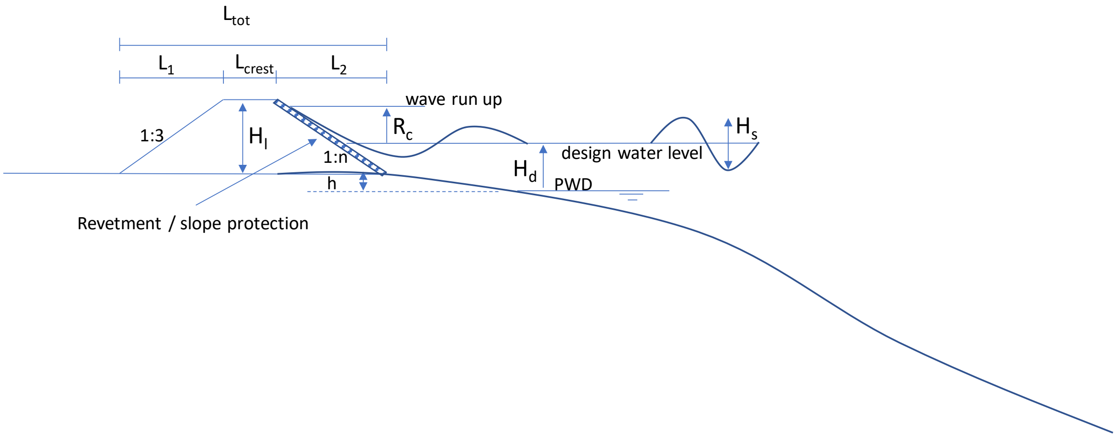


Figure 5-6, Sketch case 5, Sea defence, no berm

parameter	Symbol	unit	set 1	set 2	set 3	set 4
ground level relative to mean sea level	h	[m]	1.0	2.0	1.0	2.0
steepness outer slope	n	[-]	5	5	7	7
design water level relative to mean sea level	H_d	[m]	4.0	4.0	4.0	4.0
significant wave height	H_s	[m]	4.0	4.0	4.0	4.0
Freeboard, to reduce overtopping to q_c =1 l/s/m	R_c	[m]	6.10	6.1	4.25	4.25
crest level relative to ground level	H_l	[m]	9.5	8.5	7.75	6.75
overtopping discharge at design conditions	q_c	[l/s/m]	0.6	0.6	0.4	0.4
length polder side slope	L_1	[m]	28.5	25.5	23.25	20.25
length sea side slope	L_2	[m]	47.5	42.5	54.25	47.25
crest width	L_c	[m]	3.0	3.0	3.0	3.0
total length dike body	L_{tot}	[m]	79	71	80.5	70.5

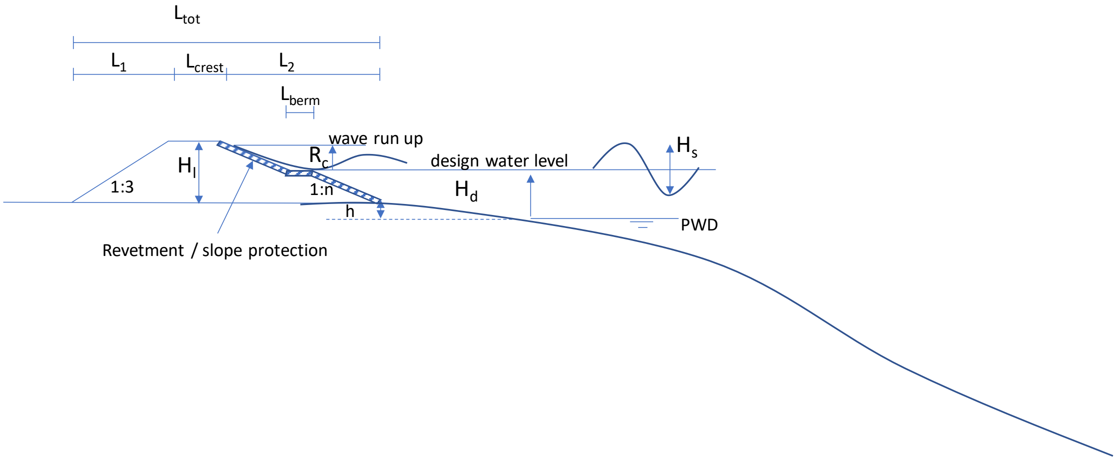


Figure 5-7, Sketch case 5, Sea defence, with berm

parameter	Symbol	unit	set 1	set 2
ground level relative to mean sea level	h	[m]	1.0	2.0
steepness outer slope (below and above berm)	n	[-]	5	5
design water level relative to mean sea level	H_d	[m]	4.0	4.0
significant wave height	H_s	[m]	4.0	4.0
Freeboard, to reduce overtopping to q_c =1 l/s/m	R_c	[m]	4.7	4.7
crest level relative to ground level	H_l	[m]	8.2	7.2
overtopping discharge at design conditions	q_c	[l/s/m]	0.45	0.45
berm length (outer slope berm)	L_{berm}	[m]	10	10
length polder side slope	L_1	[m]	24.6	21.6
length seaside slope	L_2	[m]	51.0	46
crest width	L_c	[m]	3.0	3.0
total length dike body	L_{tot}	[m]	78.6	70.6

Case 5 is also elaborated for more severe loading conditions, storm surge of 7 m and significant wave height, at deep water, of 5 m. The dimensions are elaborated according to the sketches given by Figure 5-6 and Figure 5-7.

parameter	Symbol	unit	set 1	set 2
ground level relative to mean sea level	h	[m]	1.0	2.0
steepness outer slope	n	[-]	7	7
design water level relative to mean sea level	H_d	[m]	7.0	7.0
significant wave height	H_s	[m]	5.0	5.0
Freeboard, to reduce overtopping to q_c =1 l/s/m	R_c	[m]	5.5	5.5
crest level relative to ground level	H_l	[m]	12	11
overtopping discharge at design conditions	q_c	[l/s/m]	0.5	0.5
length polder side slope	L_1	[m]	36.0	33.0
length seaside slope	L_2	[m]	84.0	77.0
crest width	L_c	[m]	3.0	3.0

total length dike body	L_{tot}	[m]	123	113

parameter	Symbol	unit	set 1	set 2
ground level relative to mean sea level	h	[m]	1.0	2.0
steepness outer slope (below and above berm)	n	[-]	7	7
design water level relative to mean sea level	H_d	[m]	7.0	7.0
significant wave height	H_s	[m]	5.0	5.0
Freeboard, to reduce overtopping to q_c =1 l/s/m	R_c	[m]	4.7	4.7
crest level relative to ground level	H_l	[m]	11.2	10.2
overtopping discharge at design conditions	q_c	[l/s/m]	0.48	0.48
berm length (outer slope berm)	L_{berm}	[m]	10	10
length polder side slope	L_1	[m]	33.6	30.6
length sea side slope	L_2	[m]	88.4	81.4
crest width	L_c	[m]	3.0	3.0
total length dike body	L_{tot}	[m]	125	115

6 References

ASTM. (2014[a]). "Standard test methods for crosshole seismic testing."ASTM D4428/D4428M-14, West Conshohocken, PA.

ASTM. (2014[b]). "Standard test methods for downhole seismic testing." ASTM D7400-14, West Conshohocken, PA

CEIP (2018) *Storm Surge, Wave, Hydrodynamic Modelling and Design Parameters on Drainage System and Embankment Crest Level, Volume-III: Package-3, Appendix-B: Storm Surge and Monsoon Water Level*, March 2018

CIRIA (2013) International Levee Handbook, CIRIA document nr C731, London, downloadable through [The International Levee Handbook resources (ciria.org)] (last visited 15 sept 2022)

DWDB (1995) standard design manual, volume I: standard design criteria, Bangladesh Water Development Board, BWDB, standard design manual committee

Escarameia M. (1995) River and channel revetments, Thomas Telfort, London

Eurotop (2018) Manual on wave overtopping of sea defences and related structures, an overtopping manual largely based on European research, but for worlswide application, second edition Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B., www.overtopping-manual.com.

Lacasse, S., Nadim, F., Liu, Z.Q., Eidsvig, U.K., Le, T.M.H., Lin, C.G. (2019) Risk assessment and dams – Recent developments and applications, Proceedings of the XVII ECSMGE-2019, Geotechnical Engineering foundation of the future, ISBN 978-9935-9436-1-3

L'Heureux, J.B., Long M. (2017) Relationship between shear-wave velocity and geotechnical parameters for Norwegian clays *Journal of geotechnical and geoenvironmental engineering* 143 (6):04017013 DOI: 10.1061/(ASCE)GT.1943-5606.0001645

Long, M., Donohue, S., and O'Connor, P. (2008). Rapid, cost effective and accurate determination of in situ stiffness using MASW at Bothkennar. Ground Eng., 43–46.

TC201 (2022) Failure paths for levees, International Society for Soil Mechanics and Geotechnical Engineering, ISSMGE, technical committee 201, dikes and levees, doi.org/10.53243/R0006

Verruijt A. (2012) Soil Mechanics, revision 2012, Technical University Delft, [SoilMechBook2012.pdf (verruijt.net)]