

A probabilistic design of a dike along the Senegal River



Master thesis

TU Delft
Royal HaskoningDHV
Alexander Henny
October 2012

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Alexander Henny
Master thesis
Faculty of Civil Engineering
Delft University of Technology

In collaboration with Royal HaskoningDHV

October 2012

Graduation Committee:

Prof Drs. Ir. J.K. Vrijling, Delft University of Technology
Dr. Ir. P.H.A.J.M. van Gelder, Delft University of Technology
Dr. Ir. O.A.C. Hoes, Delft University of Technology
Ir. M. R. Tonneijck, Royal HaskoningDHV

Probabilistic Dike Design on the Senegal River

M.Sc. Thesis Alexander Henny

Royal HaskoningDHV – Delft University of Technology

Summary

In the North of Senegal, on the border with Mauritania, the 1800 km long Senegal River (Fleuve Sénégal) is located. In response to great draughts in the nineteen seventies and eighties, two dams were built; the Diama dam in the river Delta near the embouchure and the Manantali dam approximately 1200 km upstream. The Diama dam was built for water level regulation, irrigation and navigation purposes and the Manantali dam was built for generation of electricity. The construction of the Diama dam resulted in the formation of a water reservoir upstream of the dam up to approximately 350 km land inwards (BCEOM, 1999), with a water level of 2.25 m IGN (Institut Géographique National) in the dry period, and in the blocking of salt sea water flowing upstream into the river. These two effects caused the growth of *Typha Australis* in the floodplains of the Senegal River Delta. The negative side effects of this plant are multiple, like the non-accessibility to the river for the local people, the threat to public health by the development of water related diseases, and reduction of food production as irrigation canals are blocked.

Many attempts were done to solve the *Typha* problem, like the chemical and mechanical control, but none appeared successful. The proposed solution to this problem is to transform the floodplains of the river into polders. The aim of this solution is the sustainable control of *Typha*. The goal of this thesis is to determine the economic optimal dike height for the new polder dikes in a probabilistic manner.

A first step towards this goal is to get an adequate understanding of the Senegal River system as a whole, and especially of the river Delta. For this purpose, hand calculations based on hydraulic theory were done and a one dimensional hydrodynamic Sobek model of the river Delta was created. The model is used to get insight in the river's functioning in normal and high discharge situations. The probability of malfunctioning of the Diama dam is not analysed in depth, but is investigated through a sensitivity analysis. The model is also used to investigate the influence of the construction of polders on the water level.

A second step is to define the failure probability for various dike heights. In this study, the failure mechanism of overflowing is investigated. When considering the present value of amount of loss due to flooding, depending on the kind of crop, and the construction costs of the dike, the economic optimal dike height is determined. This is done by using the economic optimization model of Van Dantzig (1956). This model adds the present value of the amount of loss due to flooding to the construction costs. The optimal dike height is located where the sum of these two aspects is minimal. The costs for polder preparation are not taken into account. This study results in an optimal dike height and flooding frequency. In Figure 1, the Senegal River Delta with the proposed polder locations is presented.

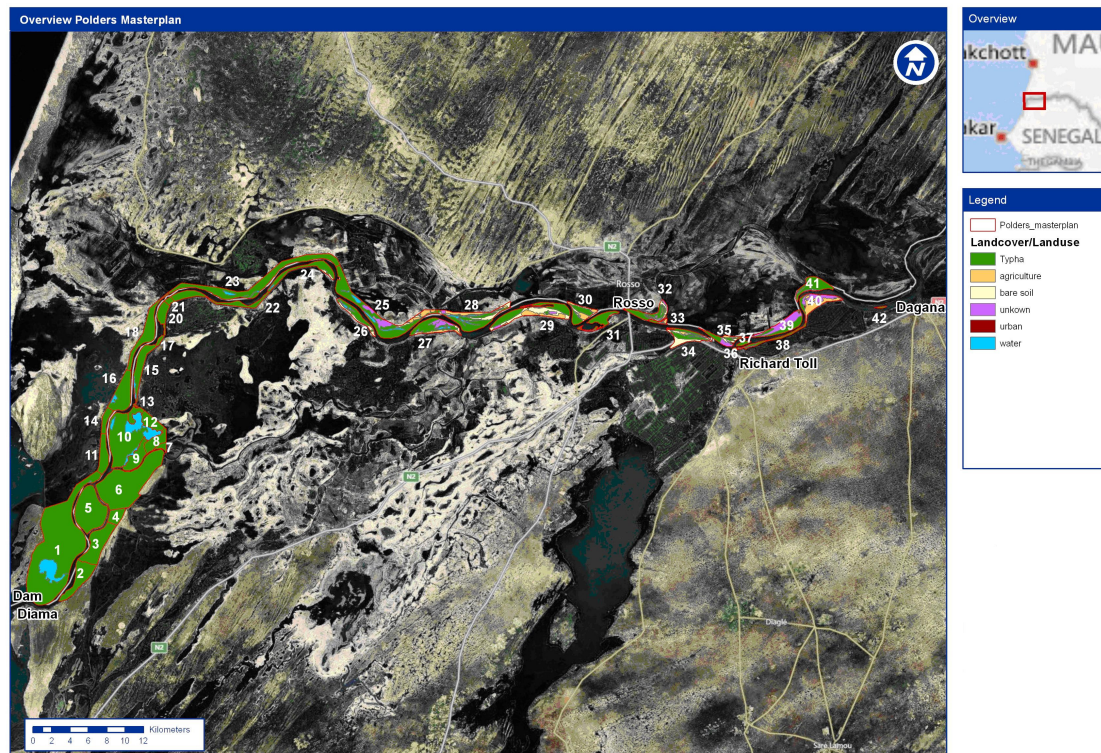


Figure 1: The Senegal River Delta with the locations of the proposed polders by Royal HaskoningDHV.

The analysis of the economic optimal dike height shows that this optimum corresponds to a flooding frequency ranging from 1/8 years in case the dike height is 2.7 m IGN to 1/20 years in case the dike height is 3.5 m IGN considering different kinds of crop and the amount of polders built. These frequencies are relatively high as they are related to a limited amount of expected loss due to flooding. The polders will be used for agricultural purposes and therefore the loss due to flooding is limited and smaller than for instance the loss for an urban area behind the dikes.

Preface

This master thesis is the final part of the Master of Science degree in Hydraulic Engineering at the Faculty of Civil Engineering and Geosciences at Delft University of Technology, the Netherlands. The study was carried out at the engineering company Royal HaskoningDHV. The project focuses on the probabilistic design of polder dikes along the Senegal River. At the time this master thesis was written, the project was still in a start-up phase for Royal HaskoningDHV.

I would like to thank all those people who have made it possible for me to complete this project. My special gratitude goes to Prof. dr. Ir. J.K. Vrijling, dr. Ir. P.H.A.J.M. van Gelder, dr. Ir. O.A.C. Hoes and Ir. M.R. Tonneijck for their supervision during this project. Additionally, I would like to thank S. Zweers Msc. for his input provided and for his guidance during the project.

Moreover, I would like to thank Royal HaskoningDHV for facilitating a place to work, in the Netherlands as well as in Senegal.

Delft, October 2012

A.F. Henny

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PART 1: The Project

1 Introduction

The Senegal River is one of the biggest rivers on the African continent. Its Delta is situated on the border of Senegal and Mauritania. The local river basin management organization “Organisation pour la mise en valeur du fleuve Sénégal” (OMVS) is engaged to control the river since its establishment in 1972. One of the big projects the OMVS completed is the construction of two dams, the Manantali dam 1200 km upstream of the river embouchure, and the Diama dam (near the town of Diama) near the river's embouchure in the Delta. At the same time the Diama dam was built, winter dikes have been built from Rosso, a place 100 km upstream for the embouchure, to Diama in order to prevent the adjacent land of being inundated. These dikes are located at a varying distance from the river's main channel. Due to the construction of the Diama dam, a fresh water reservoir was created and the salt water from the sea could not penetrate into the Delta anymore. Because of these two phenomena, the aquatic weed *Typha Australis* started growing in the river's floodplains which nowadays causes multiple problems like the reduction of access to the river for villagers and the clogging of irrigation canals.

Various solutions have been tried out to solve the problem of the presence of the aquatic weed pest (*Typha*) in the Senegal River Delta, like physical control (mechanical cutting or cutting by hand), the chemical control (spraying chemicals over the *Typha* fields), but none of these were successful. Therefore, a new kind of solution is attempted: transforming the floodplains of the Senegal River Delta, where the *Typha* is situated, into polders. This will take out the *Typha* from the floodplains, and it will also create agricultural land for the local people. The first “pilot” polders will be constructed in the area of Richard Toll and Rosso, at about 100 km upstream of Diama.

In 2009, the Dutch water board “Rivierenland” approached the engineering consulting company Royal HaskoningDHV with the request to assist the OMVS in realizing this project. Since 2004 this water board is related to the OMVS and acts as an intermediary between the OMVS and Dutch knowledge institutes and companies. Royal HaskoningDHV has fielded a project formulation visit in March 2010 and proposed the before described concept for the creation of agricultural land by building polder systems.

The realization of polders in the floodplains of the Senegal River Delta between Richard Toll and Rosso entails the construction of dikes, within the existing hydraulic system. This thesis is a study in greater depth of the Senegal River system and of the height of the polder dikes to build. It is an addition to the project Royal HaskoningDHV is currently carrying out. The goal of the thesis is to determine the optimal polder dike height in a probabilistic manner, taking eventual consequences of flooding into account.

One of the major difficulties to overcome in this project is the lack of information and data, as the project as a whole is in a start-up phase with the project contract only being signed in December 2011. A description of the hydraulic river system is not readily available. The data used in this thesis are those of the last 20 years.

Chapter two will describe the project in more detail and will elucidate the problems caused by the presence of *Typha Australis* in the floodplains of the Senegal River. Chapter three will give a more profound explanation of the thesis problem definition and the thesis goal. Chapter four will describe the Senegal River system as a whole and chapter five will address the hydraulic functioning of the Delta. Chapter six addresses the failure mechanisms for dikes and their failure probability and chapter seven gives a description of the consequences of flooding. Chapter eight defines the optimal dike height and in chapter nine a sensitivity analysis will be done on the assumed discharge and the dam adjustment. Also, the malfunctioning of the Diama dam will be investigated. Chapter 10 gives an overview of the various conclusions and recommendations of the project.

2 Project analysis

In this chapter the project will be described in more detail. The problem caused by the presence of Typha will be defined and the solution to this problem is given. Also, alternative solutions and disadvantages of the proposed solution are elucidated.

2.1 The Senegal River and its importance

The Senegal River basin is drained by the 1800 kilometre-long Senegal River, the second longest river of West Africa and its main tributaries, the Bafing, Bakoye and Falémé Rivers, see Figure 2-1. It is a rain fed river flowing from its origin in the highlands of Guinea in sub-Sahara Africa through the arid Sahel zone of Mali, Senegal and Mauritania into the Atlantic Ocean. The Senegal River Delta, at the border of the Sahel and the Sahara, is a source of biologic diversity. Wetlands provide an enormous socio-economic potential for the livelihood of the riparian communities. In response to the droughts and food crisis of the nineteen seventies two dams were constructed in the river under the auspices of the OMVS: the Manantali dam and the Diama dam. Due to the construction of these dams, navigation over the river is possible and irrigation got a lift so that agriculture became widely practised. In the entire basin 3.5 million people are living and using the river at a daily basis as a source for irrigation, drinking water and other economic purposes like fishing.



Figure 2-1: The Senegal River Basin and the project area. (Source: Royal HaskoningDHV, OMVS; Source image of Africa: <http://images.wikia.com/althistory/images/6/64/Africa-large-BW.png>)

2.2 The OMVS and other actors

As the Senegal River Delta is located on the border of Senegal and Mauritania and because of the OMVS being an organisation linked to these two countries, which will be described in short firstly. Secondly, the OMVS itself and other actors will be addressed.

2.2.1 The country Senegal

Senegal, officially the Republic of Senegal, is a country in Western Africa. It is externally bounded by the Atlantic Ocean to the West, Mauritania to the North, Mali to the East, and Guinea and Guinea-Bissau to the South. The country covers a land area of almost $197,000 \text{ km}^2$, and has a population of about 13 million people. Dakar, the capital city of Senegal, is located at the westernmost tip of the country on the Cap-Vert peninsula. During the 17th and 18th centuries, numerous trading posts, belonging to various colonial empires, were established along the coast. The town of St. Louis was the capital of French West Africa (Afrique Occidentale Française, or AOF) until 1902. After that, it was moved to Dakar. Dakar later became its capital in 1960 at the time of independence from France. Senegal is a republic with a presidency; the

president is elected every five years as of 2001, previously being seven years, by adult votes. The current president is Macky Sall, elected in March 2012 ([www.wikipedia.org\(A\)](http://www.wikipedia.org(A))).

2.2.2 *The country Mauritania*

Mauritania, officially the Islamic Republic of Mauritania, is also a country in West Africa. The country covers a land area of about $1,030,700 \text{ km}^2$, and has a population of about 3 million people. The capital and largest city is Nouakchott, located on the Atlantic coast. Imperial France gradually absorbed the territories of present-day Mauritania from the Senegal River area and upwards, starting in the late 19th century. French rule brought legal prohibitions against slavery, and an end to inter-clan warfare. During the colonial period 90% of the population remained nomadic, but many sedentary peoples, whose ancestors were expelled centuries earlier, began to trickle back into Mauritania. The country gained independence in 1960 from the French ([www.wikipedia.org\(B\)](http://www.wikipedia.org(B))).

2.2.3 *Organisation pour la Mise en Valeur du fleuve Sénégal (OMVS)*

The OMVS is a river basin management organisation in North West Africa. Its member states are all the neighbouring countries of the Senegal River: Guinea, Mauritania, Mali and Senegal and its headquarters is located in Dakar, Senegal.

The Organisation is the result of a long history of attempts to manage and to exploit the resources of the Senegal River and its Valley. The history of the organisation goes back to colonial times:

- 1934: creation of the Mission d'Etudes et d'Aménagement du fleuve Sénégal (MEAF)
- 1938: creation of the Mission d'Aménagement du fleuve Sénégal (MAS)
- 1963: creation of a Comité Inter-états which regroups the four member states of today
- 1968: creation of the Organisation des États Riverains du fleuve Sénégal
- 1972: creation of the OMVS as known nowadays

The OMVS is placed under the guardianship of the 'Conférence des Chefs d'État et de Gouvernement' (conference of the presidents and the government) and has five organs:

- The ministry council: controlling organ
- The 'Haut-Commissariat': executive organ
- The Société de Gestion de l'Energie de Manantali (SOGEM, the Manantali dam management organization)
- The Société de Gestion et d'Exploitation du Barrage de Diama (SOGED, the Diama dam management organization)
- The Société de Gestion et d'Exploitation de la Navigation (SOGENAV, organization responsible for the navigation on the river)

The five main objectives of the organisation are:

- Realising alimentary self-sufficiency for the populations of the basin
- Reducing the national economies' vulnerability of the OMVS member states to climate-related risks and external factors
- Accelerate the economic development of the OMVS member states
- Preserve the ecosystems equilibrium
- Securitize and ameliorate the revenues of the population of the basin

(www.omvs.org (A))

For this project, the OMVS is the project owner.

2.2.4 Other actors

2.2.4.1 Royal HaskoningDHV

Royal HaskoningDHV is a leading Dutch international consultancy and engineering firm, providing services and innovative solutions in environment and sustainability, general buildings, manufacturing and industrial process, urban and regional development and water. The range of services covers the entire project cycle, including management consultancy, advice, design and engineering, project management, contract management and asset management. It has offices all over the world, with its headquarters located in Amersfoort, the Netherlands. Its most important clients are national governments, public sector and semi-governments, industry, commercial services, contractors, developers and international development agencies (www.dhv.nl).

2.2.4.2 Waterschap Rivierenland

Waterschap Rivierenland is a Dutch local governmental organisation responsible for the water management in the central part of the Netherlands. It takes care of for example the inspection of the dikes, the quality of surface water and surface water level control. Waterschap Rivierenland is not responsible for the bigger rivers in the Netherlands (Maas, Waal, Merwede and Rijn); this is the task of the Dutch ministry of Infrastructure and the Environment (www.waterschaprivierenland.nl).

2.2.4.3 Local population

The entire Senegal River basin has a total population of around 3.5 million inhabitants, 85% of whom live near the river. A large ethnic diversity characterizes the basin's population, with, amongst others, Peuls, Toucouleurs, Soninkes, Malinkes, Bambaras, Wolofs and Moors. There is a large emigration of the youngest generations towards the major cities. The annual population growth rate within the basin is about 3%, which is slightly higher than the individual average of the countries surrounding the River. In the project area, the areas are divided into sanitary districts. On the left river bank, the sanitary districts of Dagana, Richard Toll and Saint Louis are located. These districts have respectively about 24.000, 46.000 and 237.000 inhabitants.

Probabilistic Dike Design on the Senegal River

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Royal HaskoningDHV – Delft University of Technology

On the right bank, smaller villages are located and the total villagers in these communities are around 73.000. The main activities of these people are agriculture and fishery.

2.3 Typha and problems caused by its presence

2.3.1 Typha

The main infestation is caused in the Delta is by the native *Typha Australis* (syn.: *Typha Domingensis*), a kind of reed also known as *cattail*. The plant belongs to the family of the Typhaceae and its botanical characteristics allow a growth in a range of soil level comprised between 2-2.5m below water level. Typha is propagated through seeds and rhizomes (roots, see Figure 2-2) unisexual multiplication and grows in shallow and stagnant water. The seed can be transported by the wind and germinate in very low salt, anaerobic conditions, which explain the plant superiority over other aquatic species. Its seeds readily germinate in open wet areas. In wet soils, the rhizome has a high capacity of reproduction and mobilises its carbohydrates reserves just before the flowering period in April-May. The plant can not endure water salinity higher than 1,5 g/l. Moreover it is sensitive to dry periods longer than 6 weeks. Its eradication in the Delta is very difficult because the local growing conditions are good and the plant can grow back very quickly. Local applications of the plant are very limited. In Senegal it is used to make fences and baskets (Benoit Grandmougin, 2005). For more detailed information on *Typha Australis*, see appendix 1 (*Prota database*).



Figure 2-2: *Typha Australis* (left) and its root system (right) (Source: *Royal HaskoningDHV* and <http://plants.ifas.ufl.edu/node/459>)

2.3.2 The proliferation of *Typha* in the river Delta

Before the dams were built, no *Typha* was present in the Senegal River Delta. It was a river with no or very little discharge during the dry season and high discharge during the wet season, a so called ephemeral river. The construction of the dams and river dikes in the Senegal River has changed the hydraulic functioning of the river by creating a fresh water reservoir in the Delta, where formerly seasonal floodplains existed. As an unforeseen effect of the created new ecosystem aquatic weed pests have flourished exuberantly. Before the dams were constructed, salt water from the sea could flow upstream, into the river Delta. *Typha* couldn't grow because of this high salt concentration and the absence of year round water in the floodplains. It is estimated that *Typha* has invaded some 100,000 hectares in the Senegal Delta over the last 15 years into an almost monoculture cover (Frérotte, 2005). An overview of the presence of aquatic weeds is shown in Figure 2-3 and pictures of *Typha* present in the Delta is shown in Figure 2-4.

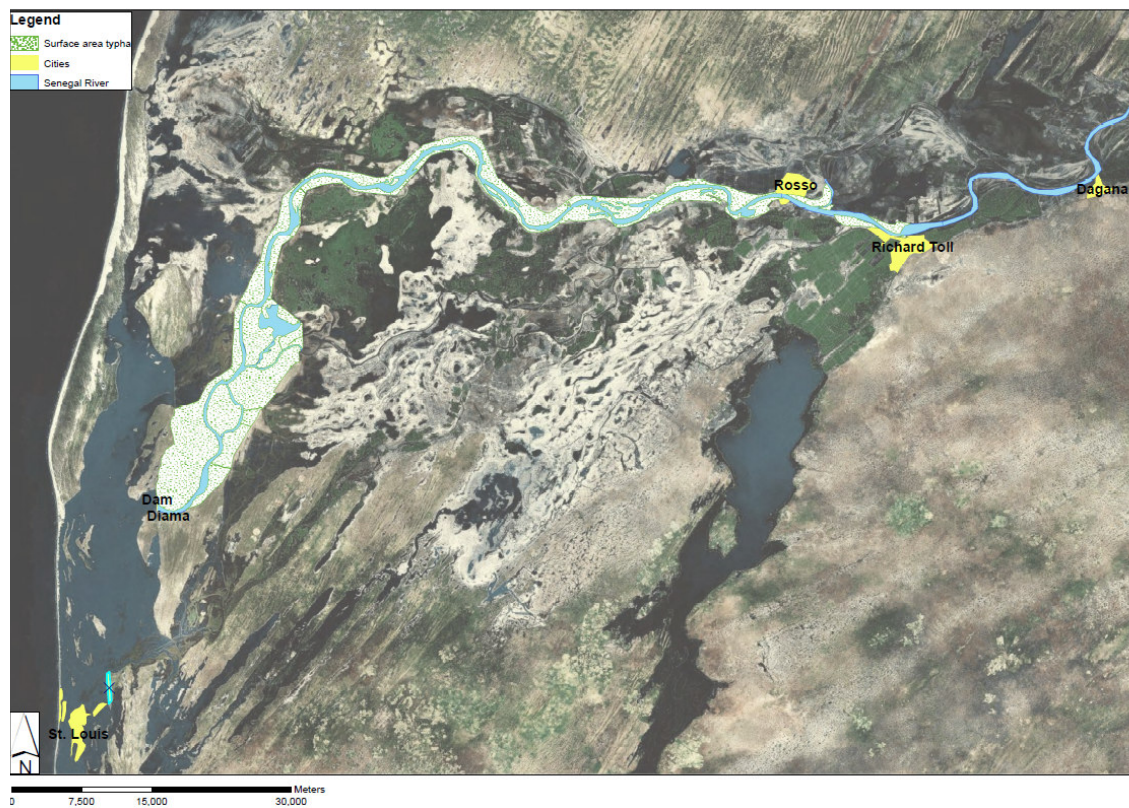


Figure 2-3: Aquatic weeds infestation in the Senegal Delta. *Typha* covers some 100.000 ha of floodplain.
(Source: AGRER-SOGED 2004)



Figure 2-4: Typha forms an almost monoculture and impenetrable cover in the Senegal River Delta. (Source: personal picture, Royal HaskoningDHV)

Also other kinds of aquatic weed grow in the Delta, however in much smaller amounts. The other kinds are for example *Salvinia molesta* and *Pistia stratiotes*. These other plants are not further addressed here.

2.3.3 Problems caused by the presence of Typha

The presence of *Typha Australis* has multiple negative effects in the river Delta:

- It forms an almost impenetrable wall between the river dikes already in place and the open water of the main channel, blocking direct access to the waterfront for local villagers living on the river banks. These people use the water for household tasks like washing; the water is also used as drinking water;
- It severely hampers the socio-economic development of the Delta, by for example the threat to public health by the development of water related diseases, in particular schistosomiasis, also known as bilharzia, bilharziosis or snail fever. Also molluscs, mosquitoes and many snakes live in the Typha fields;
- Food production is reduced (irrigation canals are blocked, which results in increasing difficulties to provide water to the irrigated zones) and biodiversity conservation is reduced (two national parks are located in the region).

2.4 General methods to remove Typha and earlier attempts

2.4.1 Methods

Various methods exist to remove Typha, as mentioned in chapter one. Most widely used methods are biological (use of cattle or livestock), chemical (use of pesticides) and mechanical (use of machinery). The chemical treatment is not an option at all in this case because of its unfavourable impact on the (ecological)

environment. On top of that, the majority of the population depends on drinking water supply from the river. For a more detailed explanation of the various methods, see appendix 1.

2.4.2 *Earlier attempts*

Over the past 10 years various methods to fight against the Typha have been studied and considered for the Senegal Delta case. According to (Frérotte, 2005) several methods for Typha control have been applied and/or considered already, all with limited or no success. Solutions range from temporarily restoring the original saline conditions, to biological and chemical control. Local people have been cutting away the Typha at smaller scale to get access to the river and to keep the 'Axes Hydrauliques' (irrigation canals) clean.

2.5 Project objective for Royal HaskoningDHV and its course

2.5.1 *Project objective and the proposed solution*

The overall objective of the project is the sustainable rehabilitation of the floodplains in the Senegal Delta between Richard Toll and Diama in order to restore the livelihood of the people living in the Delta and to protect the natural environment. The first "pilot" polders will be built between the villages of Rosso and Richard Toll, see Figure 2-5.

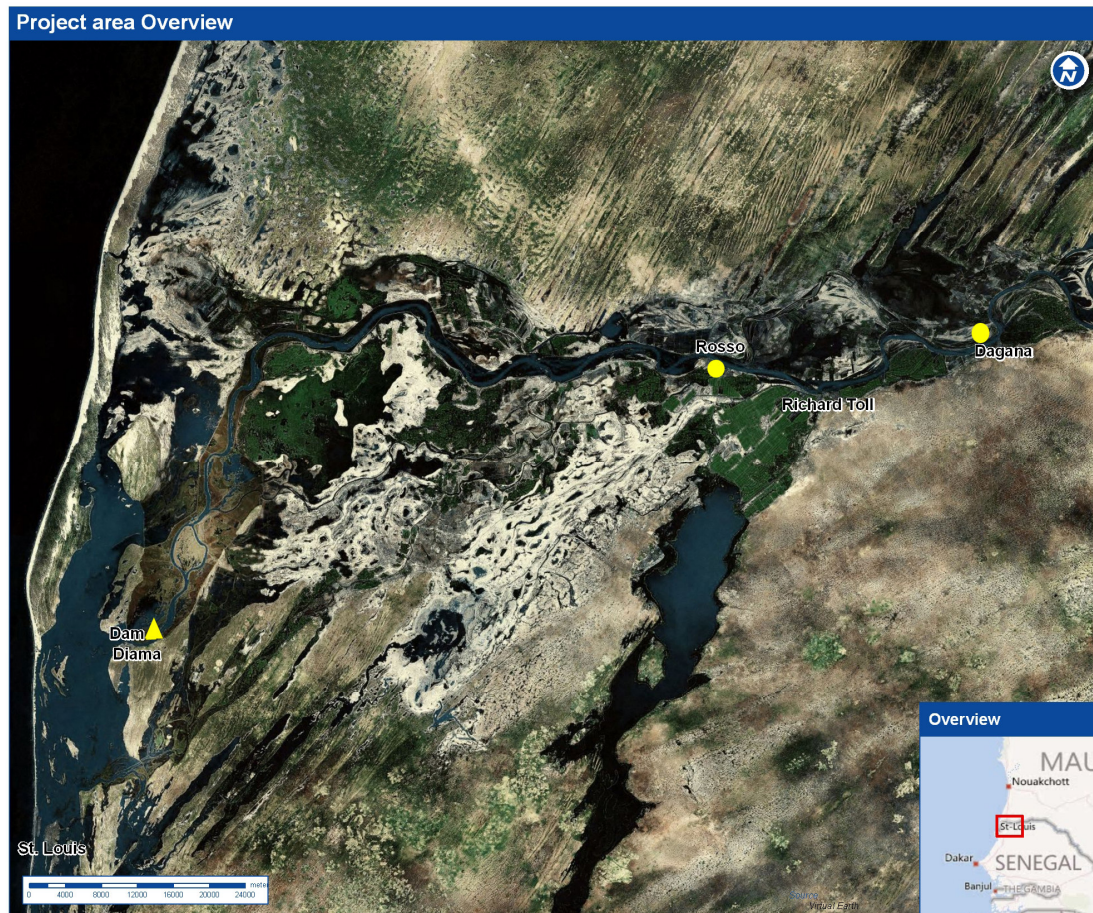
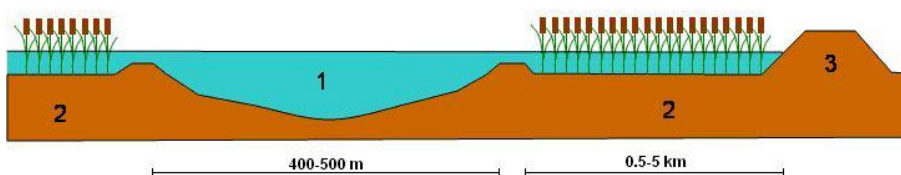


Figure 2-5: Close up of the Senegal River Delta (project area) between Dama and Dagana. (Source: Royal HaskoningDHV)

The solution proposed by Royal HaskoningDHV is to transform the floodplains of the river into polders. The charm of this solution is that the aquatic weeds will not only be removed, but also be replaced by new – mainly agricultural – development. The present situation is depicted in Figure 2-6, the situation with the proposed solution is shown in Figure 2-7. As the project is still in a start up phase, the exact location of the polders is not known yet and will have to be discussed with the OMVS. A more detailed proposal for the polder locations is given in chapter 4.

Side View

After Damming
Existing Situation



Top view

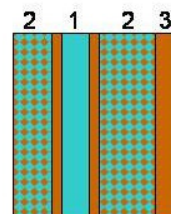


Figure 2-6: Present situation with Typha in the floodplains (not on scale). (Source: Royal HaskoningDHV)

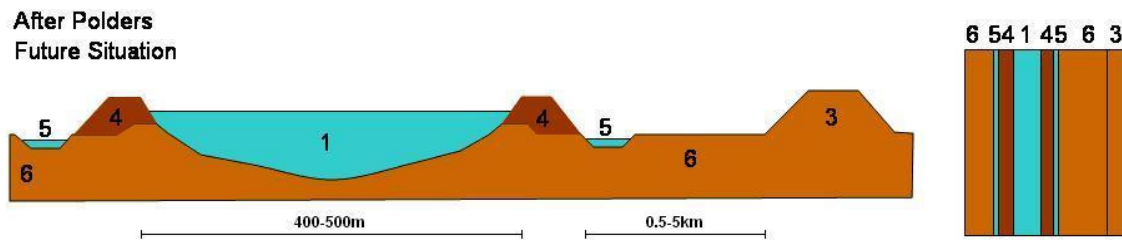


Figure 2-7: Situation with proposed solution and the Typha problem solved as the floodplains will not be inundated anymore (not on scale). (Source: Royal HaskoningDHV)

Explanation:

1 = main channel; 2 = floodplains; 3 = dike;
4 = polder dike; 5 = drainage channels; 6 = reclaimed land

The solution to eradicate the Typha is to further manipulate the river system by poldering the floodplains, as depicted in Figure 2-6 and Figure 2-7. The OMVS could also have decided not to further manipulate the system by looking for an alternative solution. In a very theoretical way of thinking, the OMVS could choose to go back to the original situation by demolishing the dams. Like this, the Typha problem would also have been solved (by bringing back the saline conditions). This is however not what the organization chose to do, because of other more important interests like the possibility of year round cultivation of land and navigation.

2.5.2 Project course

The project will be executed in the following phases:

0. Preparation and Inception Phase, resulting in contract arrangements, inception report with the approach and detailed work plan.
1. Master Plan at two levels:
 - Phase 1A: Strategic Integrated Master Plan: embedding of the flood polder area between the Diama Dam and Richard Toll, assessment and selection of polders
 - Phase 1B: Conceptual Polder Plan: Conceptual design for the selected flood polder area
2. Preliminary design and functional specifications: polder dikes, channels and in-and outlet structures, pumping stations, dredging and construction plan for the selected polders
3. Operations plan for the implementation phase
4. Operations plan for the Operation & Management phase

2.6 Side notes to the proposed solution

2.6.1 *Could the Typha problem persevere?*

The question is whether the construction of polders doesn't simply move the location of the Typha to the outer slope (river side) of the dike or that the Typha will grow in the polders. To make sure that the Typha is not coming back on the outer slope of the dikes, these dikes will have to be built with a relatively steep outer slope on the river side so that the Typha only has a limited space to grow. In this case, the Typha fulfils the function of bank protection which is an advantage. If the local people decide to grow rice in the newly created polders, these polders will have stagnant water and Typha could grow there. However, nowadays rice is already cultivated in the region and the Typha problem is not present in the rice fields. This is due to the active fight against the plant. This will also have to be done in the new polders.

2.6.2 *Disadvantages of the construction of polders*

Also possible disadvantages of the construction of polders were thought of. These disadvantages are to be investigated more profoundly at a later stage of the project.

- The Senegal River Delta is a place where birds hibernate. The birds not only stay in the national parks, but also in the Rivers floodplains. If all the floodplains in the Delta would be transformed into polders (which is the plan if the pilot polders seem to be a success), some 100.000 ha of floodplain would disappear and the birds will have to look for some place else to stay. So the polders influence the local fauna;
- The water from the Senegal River is also used as a source of drinking water. When the polders are built, the use of pesticides for agriculture in the polders will possibly increase. This might have a negative effect on the water quality;
- Due to narrowing of the river by building polders, the flow velocity in the river may increase and cause extra erosion of the river bed. This could cause problems downstream at the Diama dam, due to sediment accumulation;
- As the flow velocity in the river might go up due to the construction of the polders, the river can not be used in a safe manner by the local people during a period of the year for washing or swimming;
- It is likely the newly constructed polders influence the water level in the river (it may be higher than it is now), so the management of the Diama dam will have to change as well. If the management tries to keep the water level at for example 2.25 m IGN (Institut Géographique National, in The Netherlands this corresponds to NAP) and the water level increases because of the polders, this water level of 2.25 m IGN will be reached faster. The dam might have to be opened up earlier in the year and closed later in the year;
- On the long run, salt related soil degradation might occur in the vicinity of the Diama dam as there the salt seawater influences the soil characteristics the most. Three types of soil degradation are:
 - The soil becomes saltier (high salt concentration)

- Alkalisisation (high sodium and bicarbonate concentration)
- Sodication (higher sodium concentration)

These three processes occur due to:

- Natural causes: natural accumulation occurs in areas where the evaporation is more than the precipitation
- Human causes: irrigation.

As the irrigation will increase due to the irrigated agriculture which becomes possible as the polders are built, this can become a problem on the long run.

2.6.3 *Alternative solutions*

Various alternative solutions to the problem were sought for as well. For example:

1. Planting a different plant (which are useful to the local people) in the floodplains which will push aside the Typha. This alternative is not a very good solution because in this project, Typha itself is not the problem, but its presence is. So planting another plant is not a solution because this alternative doesn't give the access to the river back to the local population. Neither does it create valuable land;
2. Increase the water level in the river, so that the river becomes too deep for Typha to grow. To affect the Typha, this operation needs to be conducted over several months and controlled flooding needs to be repeated every 3 of 4 years (Frérotte, 2005). The elevation of the water level will however lead to flooding of a vast area, including irrigated zones which then temporarily cannot be exploited. Moreover, the dimensions of the Diama dam should be bigger to cope with the higher waters and dike heightening of the present dikes should be done. This option would only move the problem upstream;
3. Since Typha has a very low salt toleration level, temporarily restoring the initial salt conditions (from before the construction of the Diama dam) is an option by opening the dam to allow salt water to flow upstream from the sea. Yet, a lot of time is needed before the salt level reaches its initial value further upstream, and this action needs to be repeated every 3 or 4 years. It equally prevents exploitation of most of the irrigated perimeters during the year the treatment is conducted; an unacceptable price, economically as well as socially. On top of that, the Diama dam has not been designed to operate in two directions, and structural changes are needed to carry out such an operation;
4. If the problem is only the access to the river, one could think of making a bridge over the Typha fields, at various places, this is a relatively cheap solution. However, for the OMVS there is more at stake than just the access to the river, like irrigation or Typha elimination, so this solution is not further elaborated;
5. The Typha fields can be covered with a material not penetrable by light. This will stop the growth of Typha, as it needs light to grow. This is, however, not a sustainable solution because the favourable conditions for Typha to grow are only partly altered;
6. Fill the floodplains with soil, expelling the water and the land can be used for agricultural purposes. The difference with the proposed solution is that this solution will need a lot more soil. Building a

- dike needs less soil than filling a whole floodplain with soil, so this option is much more expensive than the proposed solution;
7. Make a bifurcation in the river which evacuates a certain discharge to lower the water level in the Senegal River. Like this, the water level will never reach the floodplain level in the river and the floodplains will always be dry. This solution will however need a lot of research before being executed. Also a lot of extra space is needed for this solution;
 8. Removing the Typha and putting a watertight geotextile on the ground which will hamper the Typha to grow. This could be an option which has to be investigated;
 9. Put the Diama dam out of order (all gates completely open), then also the water level in the river will decrease. Then however, the water can not be used for irrigation purposes anymore and the purpose of the dam disappears. There has been human intervention by constructing the dams and this is done with a goal.

Not all possible solutions given above have been tried out. So far, solutions have not proven to be sustainable in the sense that the physical conditions for abundant re-growth of Typha remained. Doing nothing is however not an option in view of the adverse socio-economic and environmental development in the Delta. The above mentioned alternative solutions need to be assessed in an Environmental Impact Assessment or a Social Impact Assessment. In the proposed solution, no attention is paid to possible effects of climate change on the river system or the polders.

3 Thesis problem analysis

This chapter gives an overview of the thesis goal and main objective. Also, a basic background is given on the theory used in the course of the thesis.

3.1 Thesis goal

The project as described in chapter 2 is quite broad and a lot of aspects are involved. The goal of this thesis is limited to defining the optimal dike design; and more in detail the optimal dike crest level, of the polder dikes along the Senegal River between Richard Toll and Rosso, taking into consideration the consequences of flooding. This optimal dike crest level is a relation between the failure probability of the dike (considering failure mechanisms), the investment costs for the (re)construction of the dike and the consequences of flooding.

3.2 Objective

The objective of this thesis is the following:

- Define the optimal economic dike height (crest level) of the polder dike along the Senegal River in a probabilistic manner

To be able to get to this objective, a hydraulic description and analysis of the Senegal River is carried out. It is expected that relatively little data is available for this project.

Various sub questions are linked to this research objective and the proposed solution method (several questions are repeated in chapter 5 as a hydrodynamic model is newly made to answer these questions).

- How does the river system work?
 - What are the main aspects that influence the water level in the river?
 - What are the water levels in the river and what is the discharge-water level (Q-h) relation for a certain location in the river (when the water level in front of the dam is kept at a fixed level)?
 - What are extreme values for discharges and water levels?
 - What happens to the water level in case of malfunctioning of the dam?
- What are the statistical distributions for the extreme values of discharges and water levels?
- What is a good value for the amount of economic loss due to flooding?
- What are the construction costs of a dike in Senegal?
- How will the water level in the system be influenced by the construction of polder dikes in the Rosso-Richard Toll area and in the whole Delta?

3.3 Proposed solution method

3.3.1 Definition

The proposed solution method is the probabilistic design method. The essence of the probabilistic design approach is that one not only looks at the probability of failure of a structure (like a dike), but also takes into consideration multiple failure mechanisms and the consequences of failure. For every failure mechanism of a dike, a range of failure probabilities exist. Examples of failure mechanisms are erosion of the outer slope or the instability of the inner slope of the dike. Every failure mechanism has its reliability function (Z), which is a relation between the load and the strength (resistance).

$$Z = R - S \quad [3.1]$$

In which:

R is the strength or more generally the resistance to failure (from French: Résistance)

S is the load or that which is conducive to failure (from French: Sollicitation)

R and S consist of various random variables. Each variable has its own distribution type (CUR, 1997). There is an uncertainty in the distribution type and in the parameter(s) of the chosen distribution. These uncertainties are called statistical uncertainties. Other uncertainties are model uncertainty (for example construction costs uncertainties) and inherent uncertainty. Inherent uncertainties represent randomness or variations in nature. For example, even with long history of data, one cannot predict the maximum water level that will occur in, for instance the coming year in the North Sea. It is not possible to reduce inherent uncertainties (Van Gelder, 2000). These will not be further addressed here. Two kinds of failure mechanisms exist: load-dominated failure mechanisms (e.g. overflowing) and resistance-dominated failure mechanisms (e.g. dike breach) (Vrijling, 2011). In case of overflowing, equation [3.1] results in a failure probability per dike height as the mechanism is considered to fail when the Z -function becomes negative ($Z < 0$), so when the load becomes bigger than the resistance (CUR, 1990). This failure probability is multiplied by the value of the loss and divided by the interest rate to get the present value of the loss due to failure. The present value is added to the construction costs of the dike (CUR, 1997). The final result by application of this method is an optimal economic risk-based dike height. For a further analysis of the problem solving method, see appendix 2 and chapter 7 and 8.

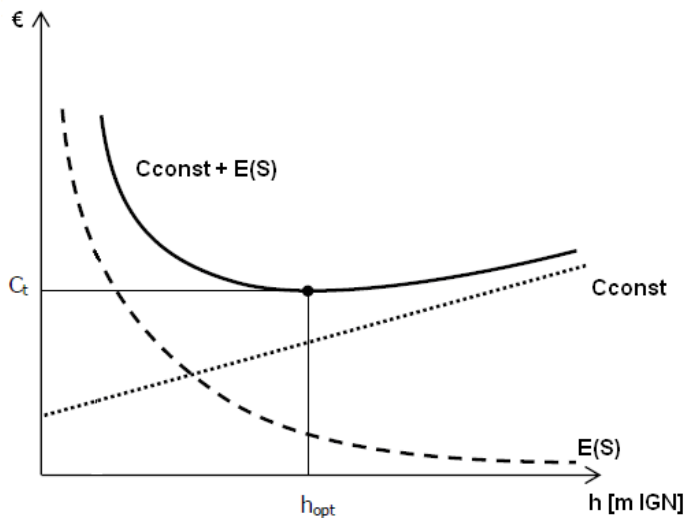


Figure 3-1: Optimal dike height considering the construction costs and the loss due to flooding. The optimal height is there where the total costs are minimal.

In Figure 3-1, C_{const} is the construction cost (investment) and $E(S)$ is the present value of the amount of loss due to flooding. Adding up C_{const} and $E(S)$ will result in the optimal crest height of the dike h_{opt} , as is shown in the figure.

3.3.2 The method applied to this project

For this project in particular, the water level in the river is influenced by the two dams, by the annual flood wave caused by rain and by the construction of the polders. The minimization of the total costs of the dike, taking into account the construction costs, the amount of loss due to flooding and the failure probability of a dike will lead to the optimal dike crest level (see Figure 3-1). Multiple failure mechanisms like instability or erosion exist with their failure probability, however only the overflowing failure mechanism will be investigated in this thesis. So the load is the water level in the river and the strength is the height of the dike. To be able to analyse this failure mechanism, an extensive analysis of the water level in the river will be performed. The consequences of flooding depend on the value of what is located behind the dike. If the local people will use the new land only for agricultural purposes, the consequences of flooding will be limited (i.e. the flooding of a tomato field is less dramatic than the flooding of a city with thousands of people getting killed). In this project however, this value is not yet defined. This thesis is based on the assumption that the land in the polders will be used for agricultural purposes. As this is a new project for Royal HaskoningDHV, the hydraulic functioning of the Senegal River will have to be investigated. Once the system is understood, the failure probability and the optimal dike crest level can be defined.

3.3.3 Theoretical background for open channel flow

As the project concerns a river, the hydraulic theory for open channel flow will be used. The basic equations for long waves in open channel flow (e.g. flood waves in rivers) are the continuity equation and the momentum equation, for an explanation of the two equations, see below. These two equations are better known as the equations of De Saint-Venant. As will be seen later, an annual flood wave comes down the river every year; that is why these long wave equations will be used. The assumption of hydrostatic pressure in the high water wave can be made because of the very slow fluctuations in water level and small depth (compared to the length of the wave). The continuity equation is a volume balance of water during a time interval of $t = t_1$ to $t = t_2 = t_1 + \Delta t$ which reads:

$$B \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial s} = 0 \quad [3.2]$$

in which B is the storing width, $\partial h / \partial t$ is the change of water level over time and $\partial Q / \partial s$ is the change of discharge in space.

The momentum equation is a balance between inertia (first two terms), propulsion (third term) and resistance (fourth term) which, in function of the discharge (Q), reads:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial s} \left(\frac{Q^2}{A_s} \right) + g A_s \frac{\partial h}{\partial s} + c_f \frac{|Q| Q}{A_s R} = 0 \quad [3.3]$$

in which $\partial Q / \partial t$ is the local acceleration of the water. This is the change in velocity in time of water at a fixed point in the river. $\partial(Q^2 / A_s) / \partial s$ is the advective acceleration, which is the change in velocity in space. $g A_s \partial h / \partial s$ is the force due to water level slope and $c_f |Q| Q / A_s R$ is the resistance factor.

In function of flow velocity U the formula is:

$$\frac{\partial U}{\partial t} + U \frac{\partial U}{\partial s} + g \frac{\partial h}{\partial s} + c_f \frac{|U| U}{R} = 0 \quad [3.4]$$

in which $\partial U / \partial t$ is the local acceleration, $U \partial U / \partial s$ is the advective acceleration, $g \partial h / \partial s$ is the driving force and $c_f |U| U / R$ is the resistance (Battjes, 2002 (A)).

Many simplifications (reductions) of these equations exist to approximate reality. One of the reductions in case of high water waves (flood waves) is the quasi-static approximation which neglects the inertia term in

the momentum equation. This can be done as the water level increase or decrease is relatively slow (in the order of 0.5 m/day). For high water/flood waves, the continuity equation keeps its original form (no reductions are made).

3.4 Possible side steps/further investigations

3.4.1 Hydraulic data

Only water level measurements at certain places along the river and discharge calculations at two locations in the river are available. One of the discharge calculation points is high upstream (about 800 km from the rivers embouchure, at Bakel) and the other is the Diama dam. Bakel is often mentioned in the following chapters, because at that location information is available; however, the use of this information is limited as the project focuses on the river Delta. Discharges at other places along the river in the Delta will be simulated. The losses of water due to irrigation, evaporation and the flow of water into side canals will have to be assumed as these values can change day by day. Also, the duration of a flood wave in extreme situations will be approximated. The development of the flood wave in case of high discharges is unknown because of the wide flow plains along the river. On these aspects, sensitivity analyses will be done in chapter 9.

Furthermore, the quality of the data is an issue. The discharges in the river are calculated by the OMVS and not measured, which means that it is uncertain whether the discharge really occurred or is correct. Moreover, the formulas used for the calculation are based on a limited amount of measurements. Also, measurements may sometimes not be done correctly or data from a dataset may be missing. Additionally, the available information is in some cases relatively old. For example, the manual on the functioning of the Diama dam was made in 2001 and describes the functioning of the dam until 1999. In the meantime, the dam operation may have changed. It is known that in the first years after the construction of the dam, the OMVS has done some tests by manipulating the dam.

Additionally, some phenomena just stay unclear. It is not possible to contact the OMVS for every little question that rises and even then they might not even know the answer. It is difficult to find the right person in Senegal who knows exactly what happened in a certain situation. It was experienced that different answers to one posed question were given asking various people.

3.4.2 Economic data

The information available on for example costs of land preparation, dike construction, interest rate and selling price of a certain crop and its yield is also sensitive to annual fluctuations. Moreover, the information is not always up to date.

PART 2: The River

4 *Senegal River system description*

In this chapter, the Senegal River system as a whole will be described and a more detailed description of the project area, the river Delta, will be given. In paragraph 4.1, the Senegal River system is described in general; paragraph 4.2 gives a hydraulic description of the river and paragraph 4.3 gives a description of the river Delta. Paragraph 4.4 gives a definition of the system used for this thesis.

4.1 Physical description of the Senegal River

The 1800 km long Senegal River is formed by the reunion of two major water streams; the Bafing and the Bakoye of which the confluence is situated in Mali at a distance of 1083 km from the Atlantic Ocean. After having crossed the eastern part of Mali, it forms, for the rest of its trajectory, the border between Senegal and Mauritania. The Bafing is 760 km long and takes source in the Fouta-Djalón mountain ridge in Guinea at an altitude of 800 m. It flows in northern direction crossing the plateau of the Soudan region before reaching Bafoulabé. The Bakoye River has his source in the Mandingue plateau in Guinea at an altitude of 706 m. Its length is 560 km and it confluent with the Bafing River after passing a great number of small drops. On the left bank of the Senegal River, the most important affluent is the Falémé river. It is 650 km long and sources in the northern part of the Fouta-Djalón mountain ridge at an altitude of 800 meters. The confluence with the Senegal River is located at a distance of 30 km upstream of Bakel. The basin of the Senegal River covers a total surface of 289.000 km^2 and is divided into three major parts (see Figure 4-1, Figure 4-2 and Figure 4-3):

- Haut-Bassin, which is defined as the region from the Fouta-Djalón mountain ridge to Bakel. This area provides nearly all the water as it is humid with annual precipitations between 700 and 2000 mm. These precipitations fall between April and October in the mountainous area in the south of the basin and cause the annual flood wave between July and October. At Bakel, the river has taken in all its major tributaries.
- Valley, defined as the region between Bakel and Dagana, is an alluvial plain surrounded by semi-deserted areas. It consists of a floodplain area with a width of 10-20 km, but can reach 25-35 km at some locations. This agricultural land is fertilised every year by the flood wave. The river shows many meanders in this region and forms a system of a main channel with floodplains and vast flow areas. The main channel has a width of 200-400m in this region and its bed is cut by various rocky or sandy steps.
- Delta, defined as the region downstream of Dagana. It is the terminal part of the river and has multiple arms, but only one river mouth. The water slope in the Delta is close to zero during low water and a slope of about $1.1E-5$ during high water. This region is almost completely flat and the

river is about 10 meters deep. In this part of the river, also Lake Guiers, lake R'Kiz many irrigation canals and Djoudj National Park are located. The river water fills these two lakes in the wet season. The national park has a variety of marshland habitats and many birds come to the national park every year to hibernate (www.omvs.org (B)).



Figure 4-1: The Senegal River with its three main parts: The Haut-Bassin, the Valley and the Delta
 (Source: Royal HaskoningDHV, OMVS; Source image of Africa: <http://images.wikia.com/althistory/images/6/64/Africa-large-BW.png>)

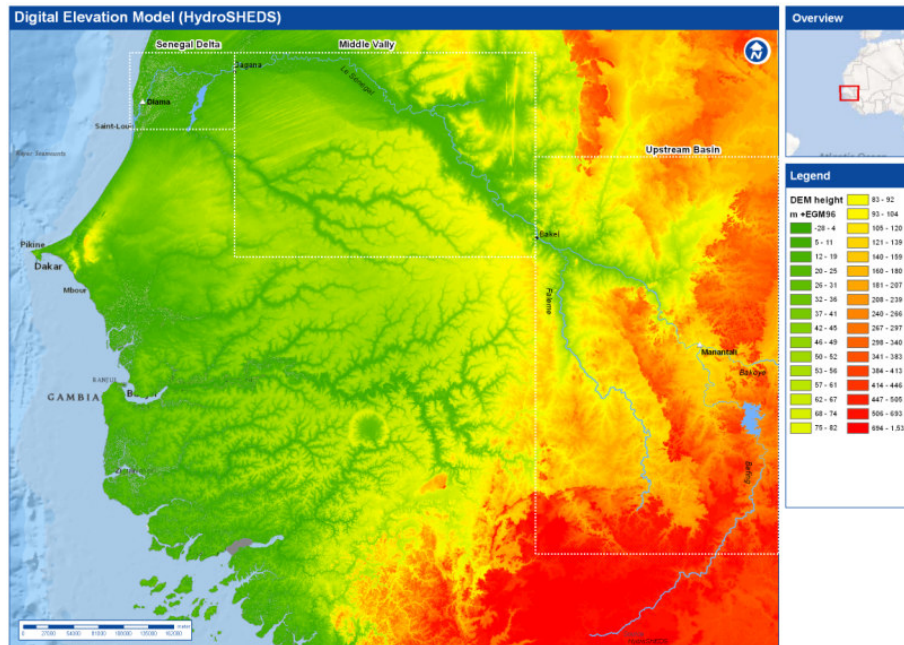


Figure 4-2: The three parts of the river showing the Haut-Bassin being located at higher altitude and the Delta being lower altitude. (Source: <http://hydrosheds.cr.usgs.gov/>)

In the 1980s, two dams were constructed in the river under the auspices of the OMVS: the Manantali dam and the Diama dam. A few places are important for the project, which are (with in brackets their distance to Diama in km): Diama (0), Rosso (105), Richard Toll (120), Dagana (140) (these three places are located in the Delta), Bakel (780), Manantali (1100). The Senegal River Basin is located for 31.000 km^2 in Guinee, for 155.000 km^2 in Mali, for 75.500 km^2 in Mauritania and for 27.500 km^2 in Senegal. See Figure 4-3 for an overview.



Figure 4-3: The Senegal River with the important locations; Manantali, Bakel, Dagana, Rosso and Diama. (Source: Bonneau, 2001)

Catchment area

The catchment area of the Senegal River spreads over the four OMVS countries. The surface of the catchment area is 289.000 km^2 , which is about 7 times the Netherlands. The catchment areas of the Falémé and of the Bakoye are also of influence on the flow in the Senegal River. For an overview of the catchment area, see Figure 4-4.

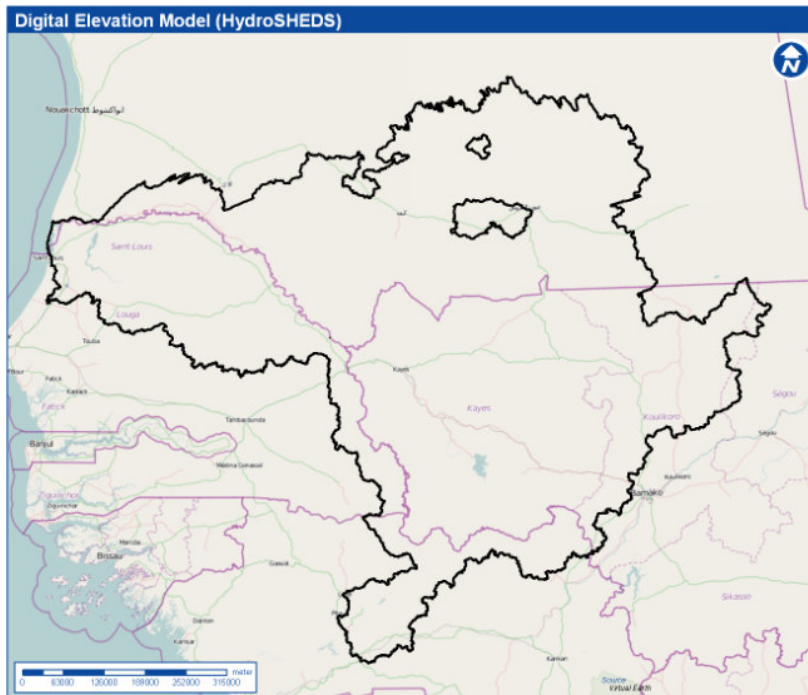


Figure 4-4: The Senegal River catchment area of 289.000 km^2 . (Source: <http://hydrosheds.cr.usgs.gov/>)

Climate, precipitation, wind and temperature

In the north of the country, where the river is located, a Sahelian climate exists. In the river basin three seasons exist: a rainy season from June to September, with temperatures between 23 and 35 °C; a cold, dry off season from October to February with temperatures between 12 and 34 °C and a hot dry off season from March to June with temperatures between 16 and 40 °C. The Senegal River is located in a region where the Harmattan wind blows. This wind is a dry and dusty West African trade wind. It blows south from the Sahara into the Gulf of Guinea between the end of November and the middle of March. In the 1970's and 1980's, "the great drought", longer periods of drought, were experienced. In the Valley and the Delta, rainfall is generally low and there is rarely more than 500mm/year, whereas in the Haut-Bassin the precipitation is between 700 and 2000 mm/year, as mentioned earlier. The rain falling in the upper part of the system causes the annual flood wave. The only available data on temperature and effective rainfall are the data in Figure 4-5 concerning Podor, which is located close to Rosso and Richard Toll (Zwarts et al., 2009). For the exact location of Podor, see Figure 4-3.

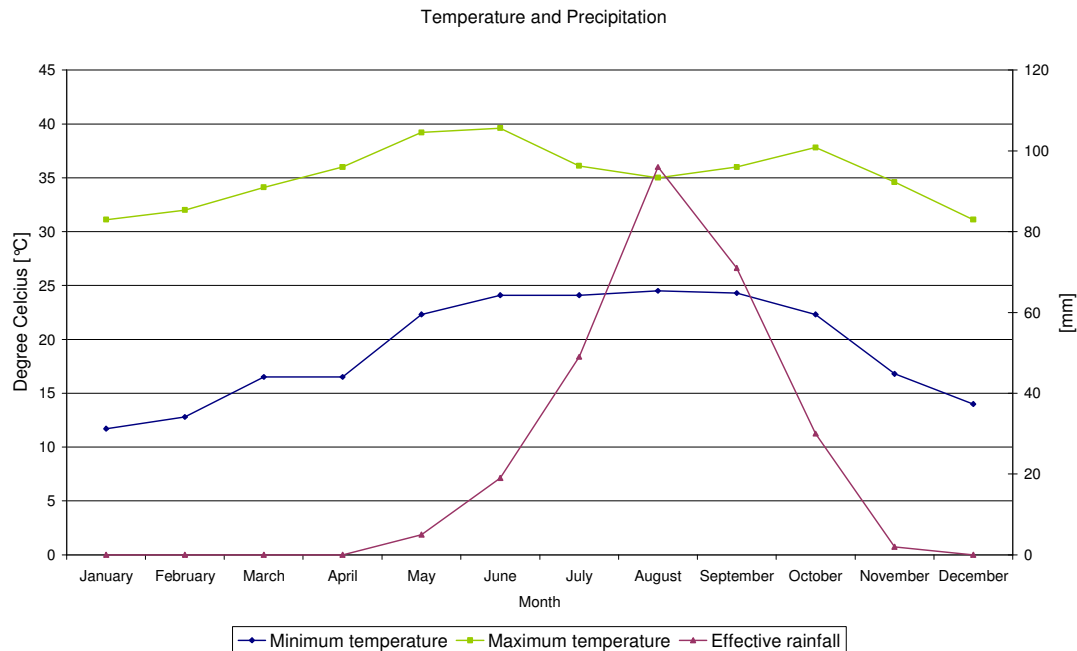


Figure 4-5: Minimum and maximum temperature and effective rainfall Podor. An increase in rainfall in the wet season occurs. (Source: CLIMWAT database)

Evaporation

In literature various figures on evaporation were found. Some articles state the average evaporation is 7 mm/day, some state 8 mm/day and some 10 mm/day (Zwarts et al., 2009, BCEOM 1999, Kosuth et al., 1999). It is not mentioned whether this evaporation is actual or potential, but considering the definitions of the two, it is assumed to be the actual evaporation. Potential evaporation is a measure of the ability of the atmosphere to remove water from the surface assuming no control on water supply. Actual evaporation is the quantity of water that is actually removed from the surface.

Water used for irrigation

For irrigation, various inlet structures are present in the Delta and in the Valley. Compared to the total volume of water in the river, this extraction of water is small. In (Diop, 1992) was stated that the water use for irrigation was only 3% of the total water losses between Dagana and Diama. The use of water for irrigation also depends much on rainfall. An example of a water extracting structure is given in Figure 4-6.



Figure 4-6: Water extraction structure for irrigation. (Source: personal picture)

Width

The width of the river varies a lot. In the Haut-Bassin, the width is about 150-250m. In the Valley, the river can reach a width of 10-20 km (including floodplains) and 25-35 km at some places. The main channel of the river has a width of 200-400 m in this area. In the Delta, the width of the main channel is 400-500 m. The width taken into account the floodplains varies because of the dikes from 1 km near Rosso to 6-7 km near Dama (distance between dikes) (Google Earth).

Depth

Depth measurements have been done by the OMVS in an earlier stage and have resulted in Figure 4-7. An average bed level is taken visualized by the brown line in Figure 4-7. The depth in the river increases going from upstream towards Dama, see Figure 4-7. This is because the Dama dam retains the water and forms a reservoir.

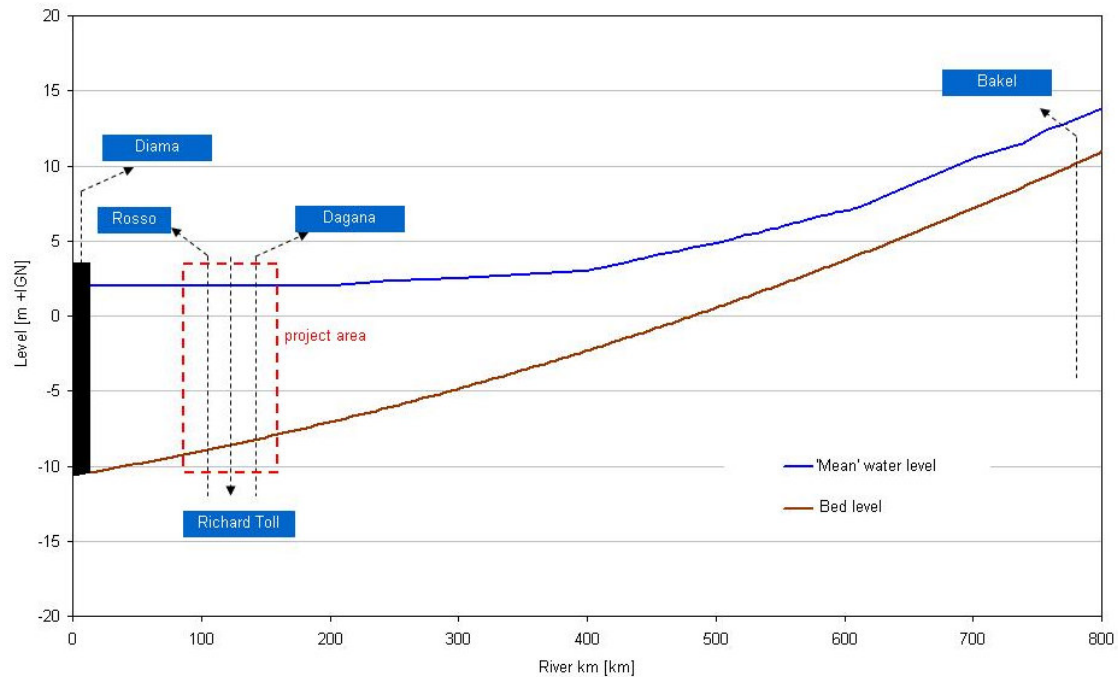


Figure 4-7: The river depth increases going downstream as a reservoir was formed by the construction of the Diama dam.

Table 4-1 gives an overview of the approximate length of the parts, average depth, average width and average bed level slope per river part.

Table 4-1: Overview of the main characteristics of the three main parts of the river.

	Part boundaries	Distance between cities [km]	Average depth [m]	Average width main channel [m]	Average bed level slope [-]
Haut-Bassin	Upstream-Bakel	1000	5	150-250	$3.2 \cdot 10^{-5}$
Lower Valley	Bakel-Dagana	680	7	200-400	$2.6 \cdot 10^{-5}$
Delta	Dagana-Diama	140	12	400-500	$2.4 \cdot 10^{-5}$

4.2 Hydraulic description of the Senegal River

4.2.1 Hydraulic description before the construction of the dams

The Senegal River discharges in a dynamic area where ocean currents, seasonal swell and littoral drift created sand barriers along the coast. The resulting strip of dunes was constantly being breached by the river (Zwarts et al., 2009). Highly dynamic conditions were characteristic of the natural estuarine floodplain and so the local wetlands had long adapted to these fluctuations. The river used to be a typical rainfall runoff river; a river mainly fed by rain water. In the wet season high precipitations caused the river to have a high discharge. During the dry season however, the river was (nearly) empty and no discharge was measured in

this time of the year. This kind of river, with no or very little discharge during the dry season and high discharge during the wet season, is a so called ephemeral river. Due to this difference in discharge, also a big difference in water levels used to occur (up to 13m difference). The water level rose in July, fresh water arrived in the Delta from August onwards. The salt water tongue from the sea moved upstream as soon as the river flow decreased, mostly from November onwards. Ethnic groups exploited the natural recourses, but the Delta remained sparsely populated. Heavy flooding and the silty, often saline, soils prevented the development of (flood recession) agriculture in the Delta, in contrast to the long-established (recession) agriculture employed by the people further inland along the river's mid-course (Zwarts et al., 2009).

4.2.2 *Hydraulic description after the construction of the dams*

4.2.2.1 *Manantali Dam*

The Manantali Dam is located on the Bifang river at 90 km South-East of Bafoulabé in the Republic of Mali, 1200 km upstream of the rivers embouchure. Construction works started in June 1982 and ended in 1990. It is a dam which allows the irrigation of around 255.000 hectares of agricultural land, navigation of the Senegal River between Saint-Louis and Ambidédi throughout the entire year and the production of 800 Gwh of electrical energy. The dam is 483 meters wide, 65 meters high and its reservoir can retain 11 billion m^3 of water. The three main goals are:

1. To produce energy for the neighbouring countries
2. To supply water downstream throughout the entire year which is also used for irrigation
3. To attenuate the flood wave going downstream by using the reservoir

One of the objectives related to goal number 2 is to reach a predefined discharge in Bakel (attenuation of the flood wave in the wet period). It is a controlled flood wave with a maximum of a certain discharge. The dam releases a varying amount of water so that an artificial flood wave occurs at Bakel with a maximum of $2500 \text{ m}^3/s$ to make (flood recession) agriculture possible in the Valley (see also Figure 4-12, the objective hydrogram). To be able to do so, the discharge is measured in the two other rivers, the Falémé and the Bakoye. If the discharge in the rivers is high, the dam will release less water so that the hydrogram at Bakel is attained. See

Figure 4-8 for a picture of the dam. For more information on the dam, see appendix 3.



Figure 4-8: Manantali hydropower dam. (Source: www.omvs.org)

4.2.2.2 Diama Dam

The Diama dam is located 27 km upstream of Saint-Louis (near the village of Diama). Construction works started in September 1981 and were completed in March 1988. It is a mobile dam which opens up in periods of high water to attenuate the high discharge in the river and closes in case of low discharge to minimize the salt-intrusion of the salted seawater. With the construction of the dam, irrigation became possible. The dam consists of 7 openings, which are gates of each 20 meters wide and a sluice of 175 m x 13 m for the passage of boats. These gates are rotating gates located in the upper part of the dam. When the dam opens, using electric winches, the gates rotate upwards and the water flows under the gates into the sea. The situation can occur that the gates are completely opened and that the water is not retained by the dam. The crest level of the lower part is located at -8.97 m IGN and the maximum opening of the gates is 11.25 meters. The construction of the Diama dam has caused the development of a reservoir just before the dam with a volume depending on the water level in the reservoir. For a water level of 1.5 m IGN or 2 m IGN, the volume of the reservoir is 250 Mm^3 and 400 Mm^3 respectively (BCEOM, 1999). The area of influence – with an almost horizontal water surface – extends beyond 350 km from Diama, according to [REF¹]. See Figure 4-9 for a picture of the dam and Figure 4-10 for a picture of the gates.



Figure 4-9: Diama dam. (Source: www.omvs.org)

¹ *Etude hydraulique des adducteurs du Delta*. Société Française d'Ingénierie, 1999.
Probabilistic Dike Design on the Senegal River
M.Sc. Thesis Alexander Henny
Royal HaskoningDHV – Delft University of Technology



Figure 4-10: Rotation point for dam gate (left) and the gate itself (right). (Source: personal picture)

For more information on the Diama dam, see appendix 3.

4.2.2.3 Dikes in place near the Diama dam

At the same time the Diama dam was built, dikes have been built on both sides of the river between Diama and Rosso. The goal of building these dikes was to better valorise the functioning of the Diama dam by being able to keep the water level in the river relatively high (2.25 m IGN, for irrigation purposes) without inundation of the villages along the river. The dikes have a height of 5 m IGN, a crest width of 3 meters and have taluses of 1:1. On the Senegalese side, 79.5 km of dike was built and on the Mauritanian side 76.5 km was built. Figure 4-11 gives an overview of the dikes in place.

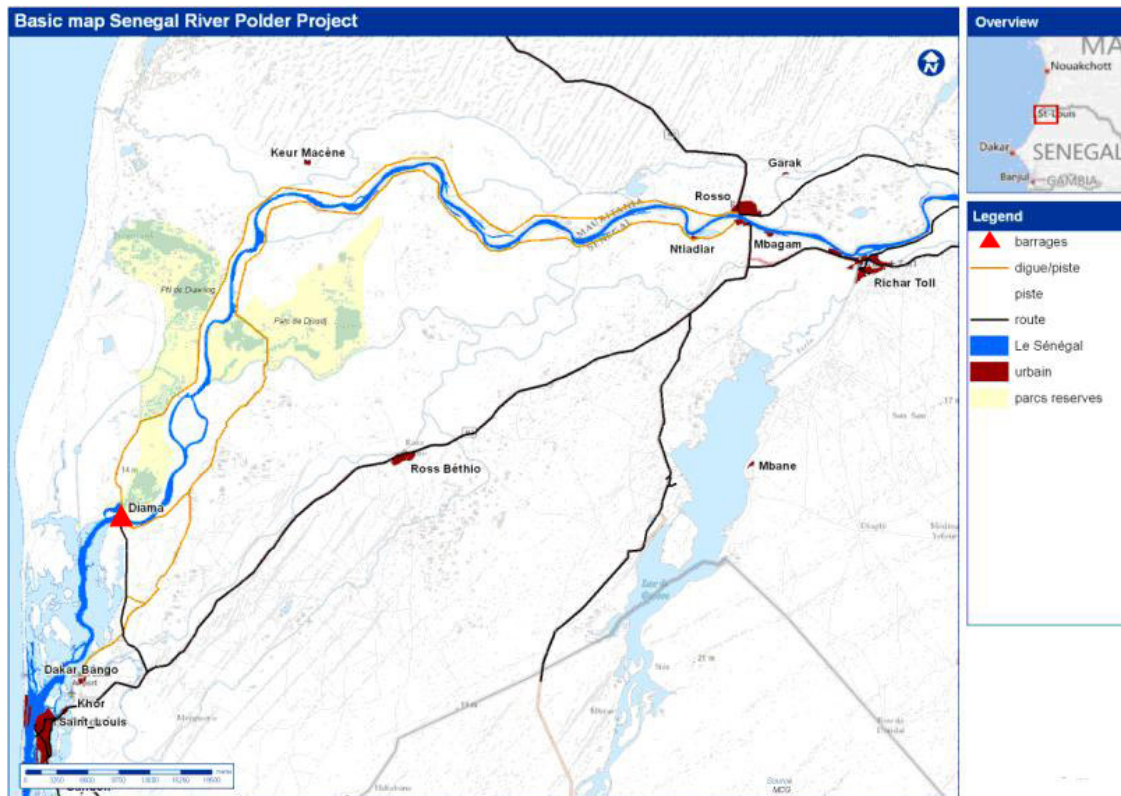


Figure 4-11: The dikes along the Senegal River are located between Diama and Rosso and have a total length (the two sides added up) of 156 km.

4.2.2.4 Hydraulic functioning

The hydraulic functioning of the river in the present situation is mainly determined by the construction of the Manantali dam and the Diama dam. As the Manantali dam is built in the Haut-Bassin in the upper part of the river, the consequences of this dam are felt more importantly in the upper part of the river (Haut-Bassin and Valley) where as the Diama has more influence on the Delta. As mentioned before, the area of influence of the Diama dam is about 350 km (BCEOM, 1999), which is not all the way upto the Haut-Bassin. In the Haut-Bassin and the Valley, the hydraulic functioning didn't severely change with the construction of the dams as a flood wave is still caused every year by the combination of the functioning of the Manantali dam and rainfall. As mentioned in paragraph 4.2.2.1, the Manantali dam regulates the water flowing downstream in order to get an objective hydrogram at Bakel (see Figure 4-12). The management of the Manantali dam analyses the rainfall in a particular year and then looks how much water should be released from the dam in order to get the objective hydrogram at Bakel.

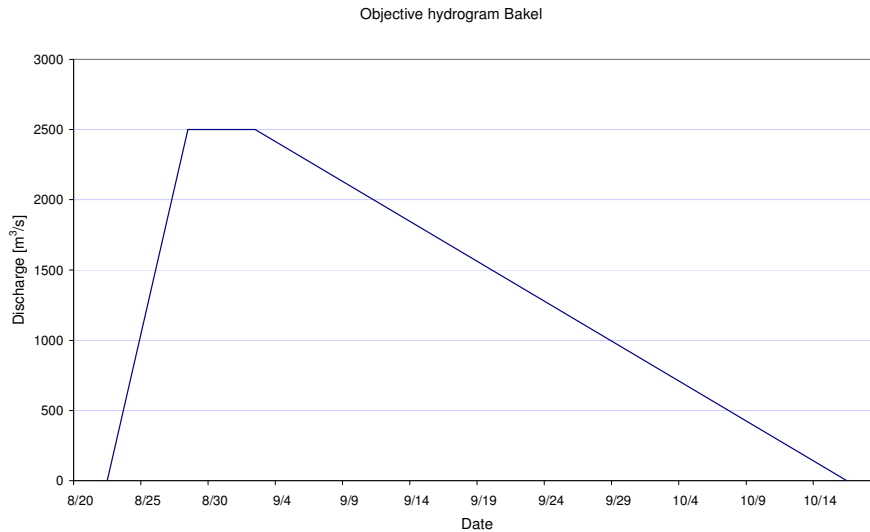


Figure 4-12: The Manantali dam releases as much water as needed (in combination with rainfall) to achieve the above discharge at Bakel. This is called an objective hydrogram.

The flood wave that travels downstream from Bakel is deformed significantly before reaching the Delta area. The flood wave is significantly slowed down and its maximum discharge is lowered (in other words, spread over a longer time period) when reaching Rosso in the Delta (see Figure 4-13). This modification of the flood wave when travelling downstream is likely to be caused by wide flow plains in the Valley area and by retention of significant water volumes in large off line connected water bodies along the river stretch between Bakel and Rosso. Offline water bodies are for example lakes that need distributaries to fill. These water bodies and the retention also cause the travelling time of the flood wave to differ from one year to the other. Furthermore, the high water level peak in the Delta area is reduced further by opening the Diama dam during extreme discharges.

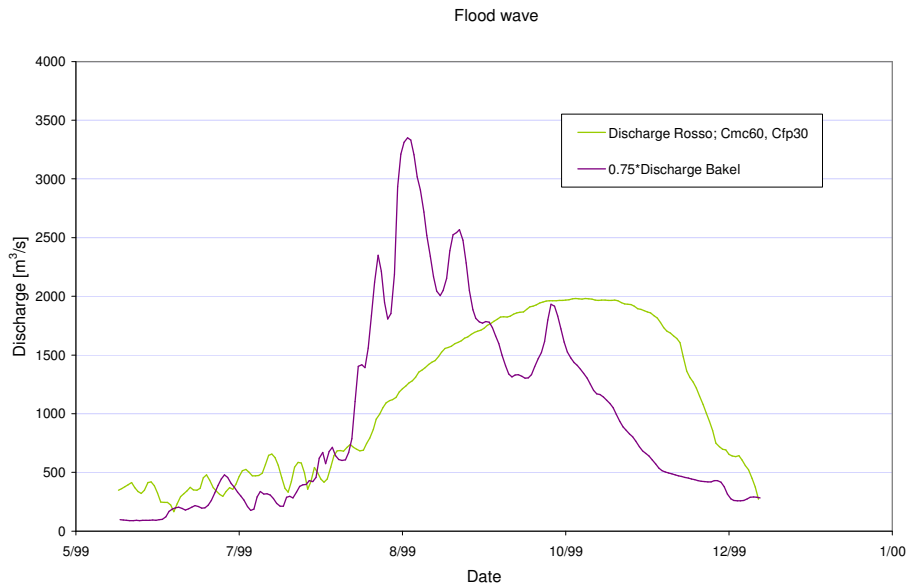


Figure 4-13: Flood wave calculated at Bakel (upstream) and simulated at Rosso (in the Delta). The maximum discharge of the flood wave becomes less and occurs later in the year at Rosso than at Bakel.

The data on discharges at the two places were investigated and it appeared that only a percentage of the volume of water coming by at Bakel flows through Diamia (see Figure 4-13 and Table 4-2). This is in principle not possible because of the volume balance that should match and an explanation was sought for. The explanation was found in the great evaporation (of 7-10 mm/day), the extraction of water for irrigation and drinking purposes and the extraction of water by distributaries and for the filling of the two lakes (Lac de Guiers and Lac R'kiz). In (Kosuth et al., 1999) it was stated that 1.5-2.0 billion m^3 of water is lost downstream of Bakel annually. Furthermore, the discharges at Bakel and Diamia are calculated and are error sensitive with missing data at one of the two locations for example, so the analysis is only done to give an order of magnitude of the amounts of water flowing through the two locations, see Table 4-2. This is the reason why in Figure 4-13, only 75% of the discharge of Bakel is taken. The purpose of the figure is to show the attenuation of the flood wave!

Table 4-2: Volume balance between Bakel (upstream) and Diamia (downstream); the volume decreases.

	Volume Bakel [$\times 10^9 m^3$]	Volume Diamia [$\times 10^9 m^3$]	Percentage [%]
1996	12.44	9.05	0.73
1999	24.22	19.3	0.80
2003	23.87	19.08	0.80
2005	16.36	13.28	0.81
2006	11.97	8.84	0.74

A more detailed analysis focused on the Delta using all available data has been carried out. It was found that the variation in water levels over the Delta area is almost negligible:

- During the dry period (9 to 10 months a year, from November to June), the water surface slope in the Delta area is around 0, and the discharge through the Diama dam is very small (about $300 \text{ m}^3/\text{s}$) and sometimes even $0 \text{ m}^3/\text{s}$. The water level throughout the Delta in this situation is kept at a constant level of around 2.0-2.25 m IGN by managing the Diama dam (opening up the dam when more water comes in and closing it when less water comes in). The minimal water level in the river is kept for irrigation and navigation purposes.
- During the flood season (2 to 3 months a year, July to October), the river discharge increases in the beginning of this period and the Diama dam is gradually opened further to keep the water level at a certain level, or to lower it. The water level at Diama is lowered to ultimately 1.5 m IGN (if needed and kept there as long as needed; if not needed, the water level at Diama doesn't reach 1.5 m IGN). At the end of the flood season, the goal is to get the water level as soon as possible back to 2-2.25 m. Closing the dam too early will result in a dangerous situation (too much water still coming down) and closing the dam too late will result in an unfavourable situation because there is too little water for irrigation.

The two situations are presented in Figure 4-14.

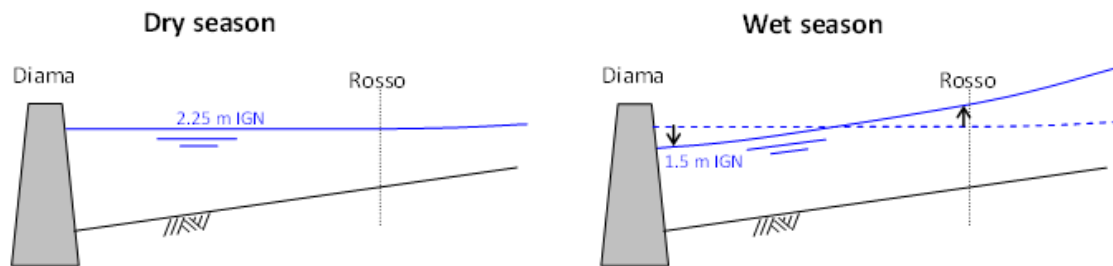


Figure 4-14: The water level in the Delta in the dry and the wet season. The water level in the wet season is lowered to cope with high discharges and a water level slope in the river is the result.

4.2.2.5 Water level

The water level is measured at various places along the river, at Bakel, Dagana, Richard Toll, Rosso and Diama (see Figure 4-3). The water level is measured by gauges, as depicted in Figure 4-15. An overview of the daily averaged water level at Bakel is given in Figure 4-16. Figure 4-17 gives an overview of the daily averaged water levels in the Delta.



Figure 4-15: Gauge at the Diama dam. (Source: personal picture)

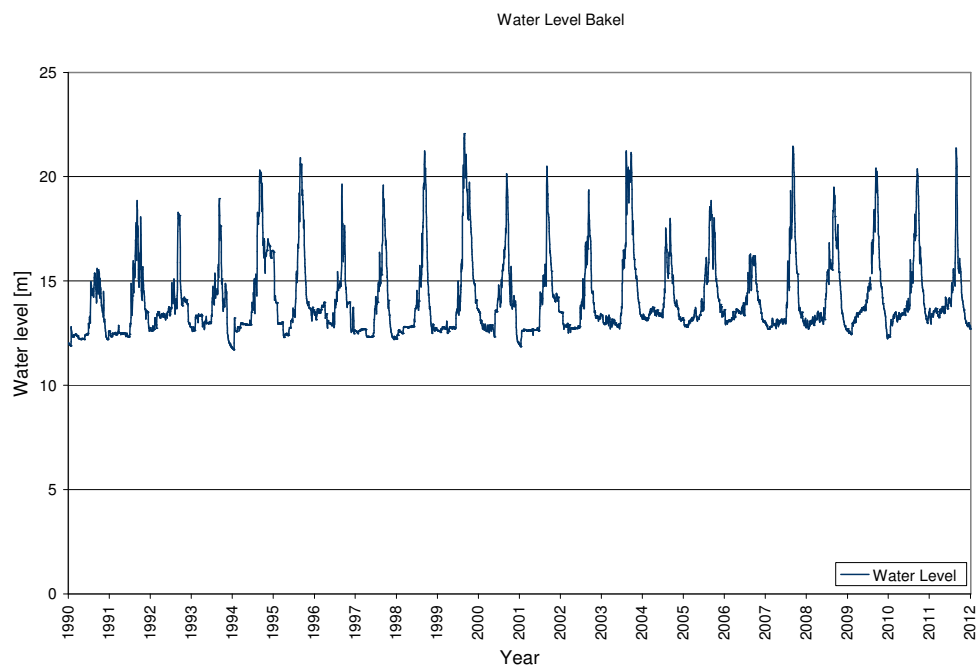


Figure 4-16: Daily averaged water level upstream at Bakel between 1990 and 2012. Every year a flood wave occurs.

The water level in the Delta has been measured at Richard Toll, at Rosso and at Diama. The water level is also measured in other locations like Dagana.

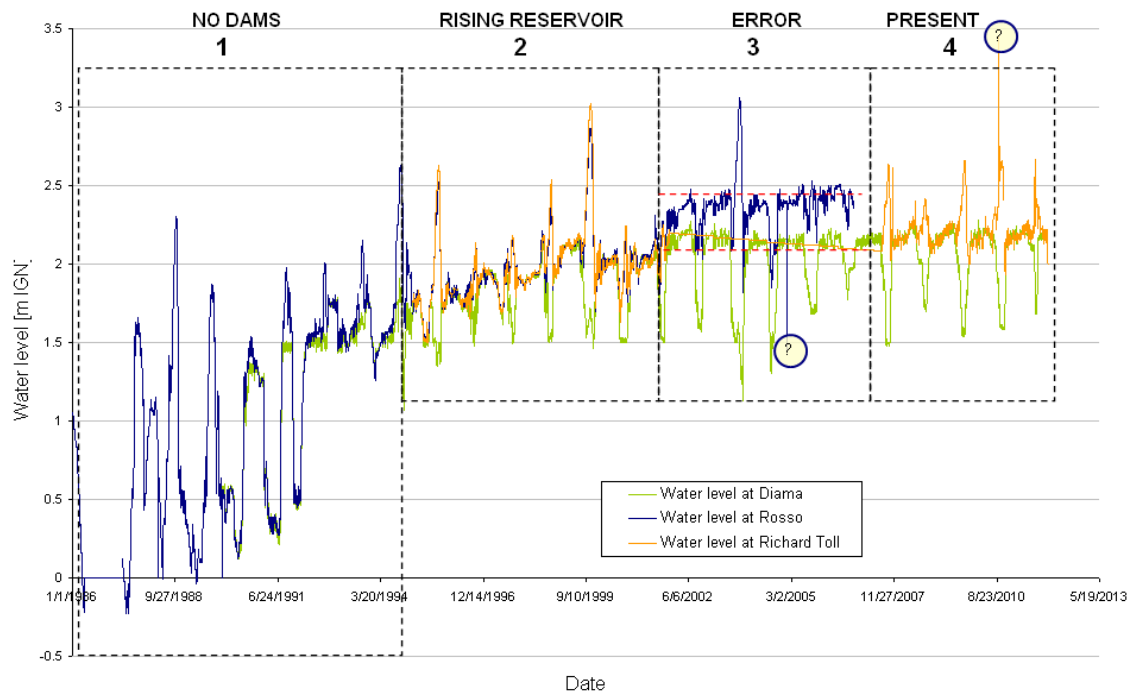


Figure 4-17: Daily averaged water level at Richard Toll, Rosso and Dama in the Delta between 1990 and 2012. The period without the dams present (part 1) shows the fluctuating water level; part 4 shows the present situation in the river. The water level at Dama is lowered during high discharges from upstream.

The water level measurement has been divided into four parts. Part 1 is the water level before the dams were constructed where the fluctuations in water level that used to occur are clearly visible, part 2 is the period when the water level was measured at all locations and when it was slowly raised to about 2.25m IGN. Part 3 is assumed to contain a lot of mis-measurements (like the water level for Richard Toll). Moreover, the measurements of Rosso are not trusted (as they are much higher than “normal”). Part 4 is the situation as it is in present day. The question whether this information is correctly measured remains.

4.2.2.6 Discharge

Bakel

The discharges in Bakel and through the Dama dam have been calculated and are daily averaged values. For the calculation for the discharge in Bakel, the formula mentioned below is used. See Figure 4-18 for the discharge at Bakel.

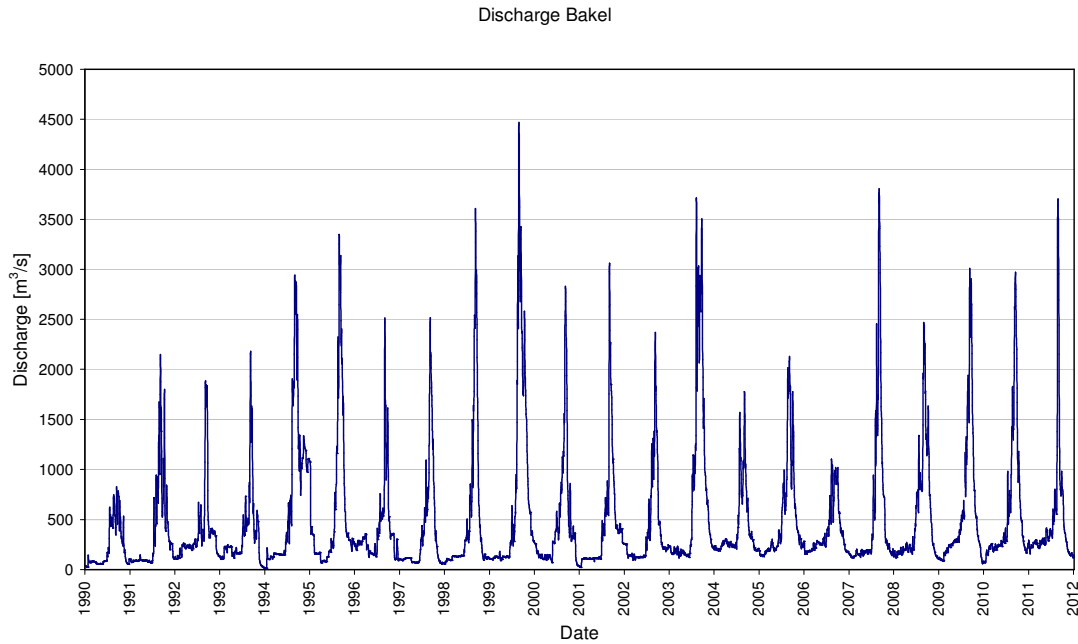


Figure 4-18: Calculated discharge at Bakel between 1990 and 2012. Every year an increase in discharge (flood wave) occurs.

The discharge at Bakel is calculated using the following theory. At Bakel, the discharge in the river depends on the water level and on the speed of the variation of this water level. If a certain water level H [cm] is measured, G is the speed of variation of h [cm/day], Q is the discharge in non permanent regime and Q_0 is the discharge in permanent regime. K is a coefficient of correction [day/cm] in function of the water level. In formula:

$$Q = Q_0 * \sqrt{1 + KG} \quad [4.1]$$

This formula was stated in the Manantali dam Manual and is given here to be complete on the discharge calculation. An analysis is done about the origin of this formula. It is very similar to the equation used to calculate the discharge for a high water wave with non-stationary flow taking into consideration the influence of a varying water depth (this is the case at Bakel):

$$Q = Q_e * \sqrt{1 + \frac{1}{i_b} \frac{1}{C_{HW}} \frac{\partial h}{\partial t}} \quad [4.2]$$

in which Q_e is the discharge during uniform flow, i_b is the bed level, C_{HW} is the velocity of the high water wave and $\partial h / \partial t$ is the change in water level over time.

This formula is called the formula of Jones. It is an approximation of the reduced momentum equation. The K factor in the formula from the manual should correspond to $1/i_b 1/c_{HW}$ and G should correspond to $\partial h / \partial t$. The OMVS uses a computer programme to calibrate the K factor. It is a correction factor which is (quoted) “eventually” (OMVS, 2001 (A) p. 31) used. It is however not known in which occasions this K factor is used or based on which information it is calibrated. So, based on the theory and looking at equation [4.1] and [4.2], equation [4.1] stems from equation [4.2] and the K factor should be $1/i_b 1/c_{HW}$, but to check whether this is correct is complicated due to the little information available on the use of the formula (when the K factor is used in the formula).

Additionally, the discharge at Bakel is not relevant for the project, as this calculated discharge will not be used anymore for further analyses. This is because the project focuses on the river Delta.

Diamma

The discharge through Diamma is calculated using two formulas which were calibrated based on 16 discharge measurements between 1998 and 2000. It is difficult to say something about the discharges upstream based on these values because the Diamma dam is opened up preventively to lower the water level. As the dam opens up, the water flowing through the dam is the stored water in the basin plus the water coming from upstream. When the dam is used to empty the basin, it doesn't mean that this water also comes into the basin at the upper end.

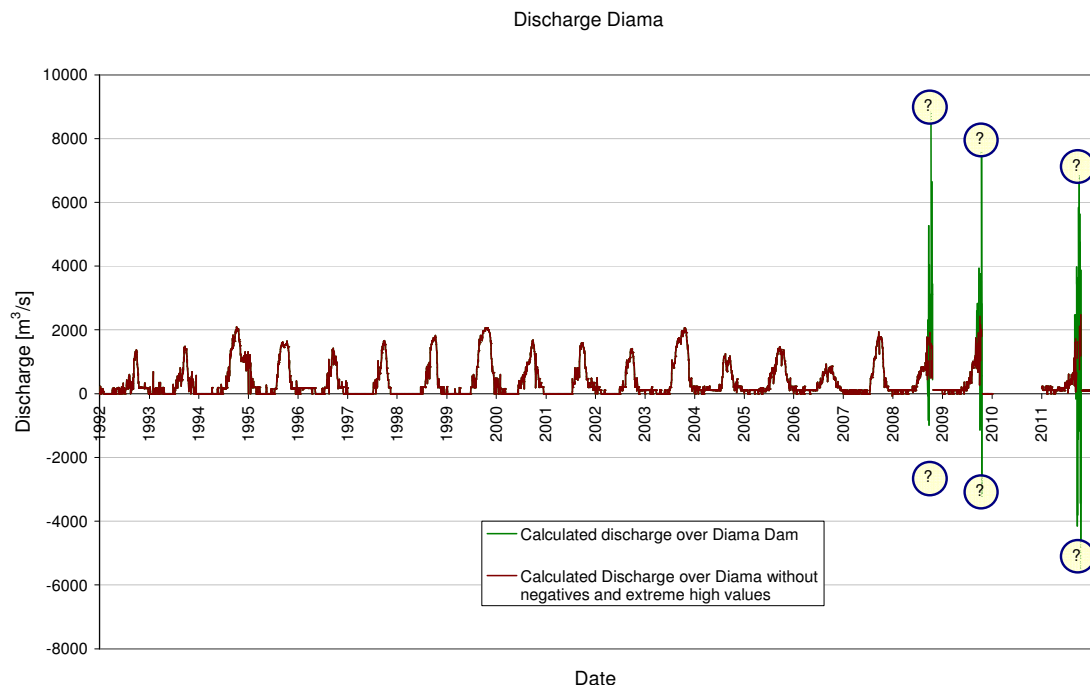


Figure 4-19: Calculated discharge through the Diamma dam. Every year an increase in discharge through the dam occurs during the wet season to lower the water level in front of the dam to cope with the flood wave.

As is visible in Figure 4-19, the discharge appears to be in line up to 2007; in 2008-2011 the calculations give higher and negative values (green line). The reason for these out of line data is not known. It could be assumed that in 2008 the measuring station for water levels was not done correctly (as from that year it gives results which are not in line anymore and in 2010 even no data are available).

4.2.2.7 Discharge-water level (Q-h) at Bakel

By using the measured water levels and the calculated discharges at Bakel, a Q-h relation is made by the OMVS (see Figure 4-23). The cross section of the river at Bakel is unknown as the exact measurement location of the water level is unknown. The cross section has been looked for using USGS Hydrosheds and there it was visible that the cross section varies a lot for every location, so the exact measurement location is important, see Figure 4-20 and Figure 4-21. It varies a lot from one location to the other, but the measurement location at Bakel is not known so it is difficult to say something about the profile used for the relation.

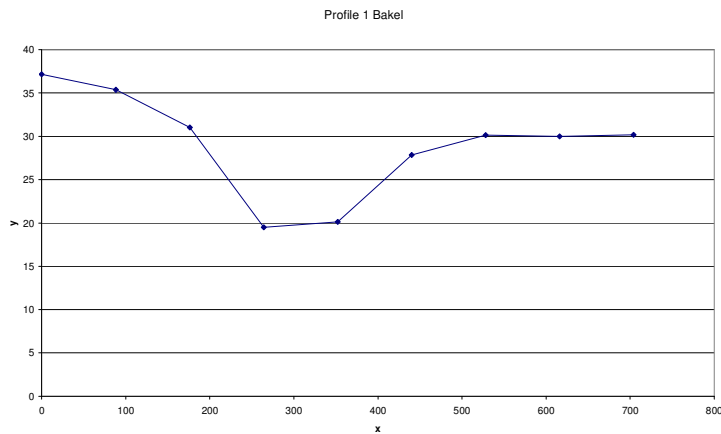


Figure 4-20: Cross section of the river at Bakel (profile 1).

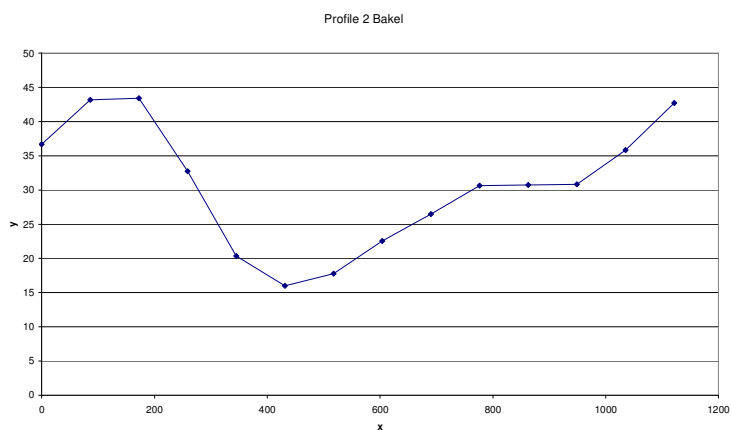


Figure 4-21: Cross section of the river at Bakel (profile 2). The cross section at Bakel appears to differ from one location to the other. As the water level measurement location at Bakel is unknown, the correct cross section is unknown.

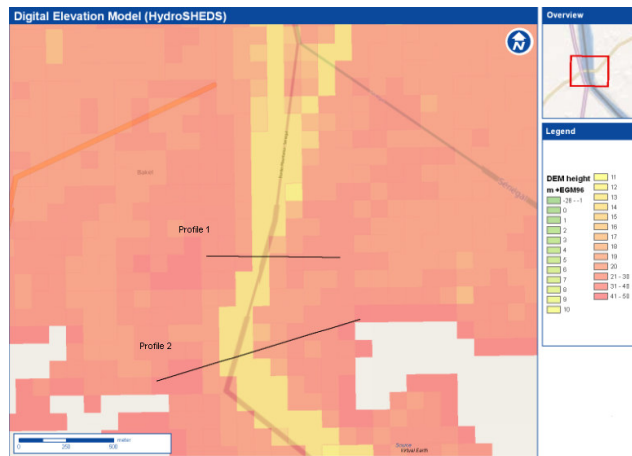


Figure 4-22: Location of the two profiles at Bakel. (Source: <http://hydrosheds.cr.usgs.gov/>)

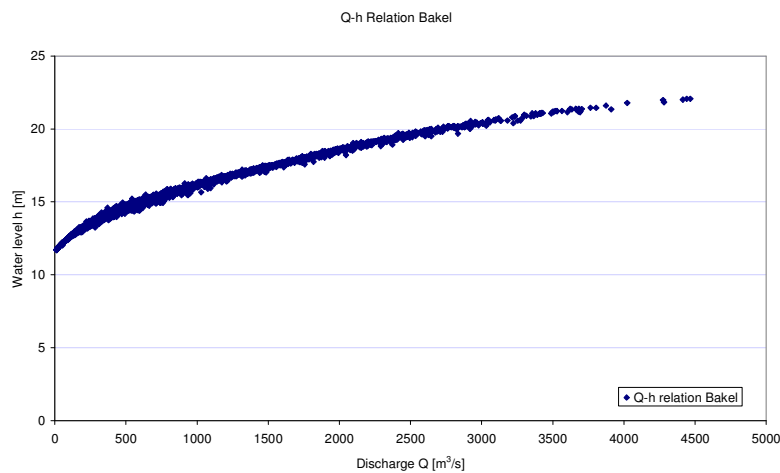


Figure 4-23: Q-h relation at Bakel. The water level increases as the discharge increases.

4.2.2.8 Flow velocity

The flow velocity in the Senegal River is not measured at all by the OMVS. To get an idea of the order of magnitude of the flow velocity, it is calculated at three different places (Bakel, Rosso and Diama) along the river using a common simplification (reduction) of the De Saint-Venant equations (as mentioned in chapter 3). Rough approximations are taken for the water depth and the water level slope; the calculations are not precise, they are made to give an impression.

In the Senegal River case, the De Saint-Venant equations should actually not be simplified like is done here (as is not done in hydrodynamic modelling programs like Sobek, see chapter 5) because of the fact that in theory, a translation wave could be formed by the fast manipulation of the Diama dam. However, the simplification is made here as the goal is to give an idea of the flow velocity. The flow velocity formula reads (Battjes (A), 2002):

$$u = C\sqrt{h \cdot i_w} \quad [4.3]$$

in which C is the Chézy coefficient [$m^{0.5}/s$], which is a bed roughness coefficient, h is the water depth [m] and i_w is the water level slope [-]. Actually, the hydraulic radius R, which is the wetted perimeter divided by the flow area, should be used in this formula as the water depth h can only be used in case of a very wide channel. As the calculation here is only done to give an idea of the order of magnitude of the velocity, the water depth may be used.

Two situations were considered; one with normal low water and one situation with extreme high water level (in 1999). When the water level at Diama is kept at a fixed level, the water level differences are very small, the water surface slope is close to 0 and it is nearly horizontally water. With extreme high water, the water level difference becomes a little bigger. During low water, the flow velocities in the Delta are in between 0 and 0.15m/s. At Bakel, the value is higher, see Table 4-3. During high water, the same appears to be the case, see Table 4-4.

Table 4-3: Flow velocities in the river during low water conditions. The flow velocities are between 0 and 0.15 m/s in the Delta.

Site	C [$\sqrt{m}/s$]	H [m]	i_w [-]	U (low water) [m/s]
Bakel	60	2.6	3.25*E-5	0.55
Rosso	60	11	3.5*E-7	0.12
Diama	60	12	0	0

Table 4-4: Flow velocities in the river during high water conditions. The flow velocities are around 0.7 m/s in the Delta.

Site	$C [\sqrt{m}/s]$	H [m]	$i_w [-]$	U (high water) [m/s]
Bakel	60	11	4.7E-5	1.35
Rosso	60	11.5	1.1*E-5	0.68
Diam	60	11.5	1.1*E-5	0.68

4.3 The project area: the river Delta

As mentioned before, the first polders will be built in the Delta area between Richard Toll and Diam. As the first polders will be built between Richard Toll and Rosso, this is the location where the input for the probability calculations will be collected. In Figure 4-24 the polder locations for the whole area are shown; in Figure 4-25 the polders for the area between Richard Toll and Rosso are shown.

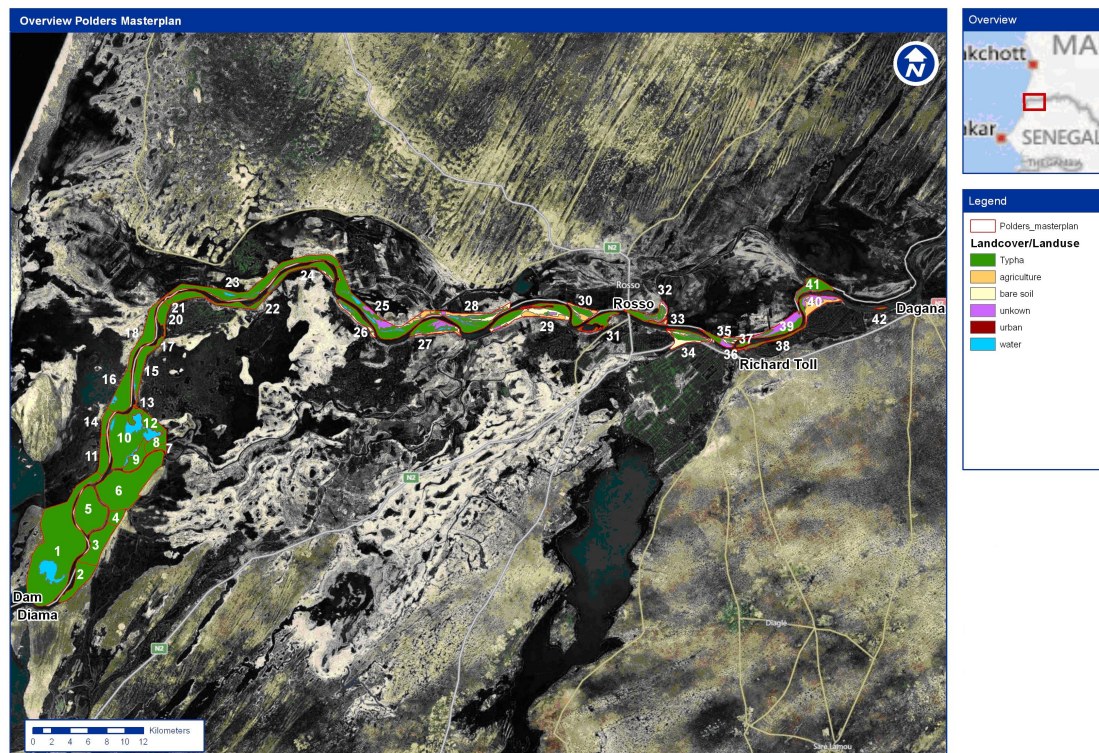


Figure 4-24: The river Delta with polders as proposed by Royal HaskoningDHV. (Source: Royal HaskoningDHV)



Figure 4-25: Polder locations of 6 polders between Rosso and Richard Toll as proposed by Royal HaskoningDHV. (Source: Royal HaskoningDHV)

4.4 Thesis system definition

For the hydraulic analysis of the Senegal River Delta, a Sobek model will be built (for detailed information on the model see chapter 5 and appendix 5). The system that will be analysed in this thesis is, as mentioned earlier, the Delta of the river between Dagana and Diama. The water levels in the project area will be measured at Rosso. The cross section of the profile at Rosso can be seen in Figure 4-26. This cross section was based on Google Earth and (topographical) maps received by the OMVS. Information could also be retrieved from USGS Hydrosheds, but after analysing this information, it was chosen to use the information from the OMVS as there was a big difference in floodplain height between the maps from the OMVS and those of USGS Hydrosheds. The difference was explained by the fact that USGS Hydrosheds “sees” the top of the Typha in the floodplains as the river bed, whereas this is not the case in reality. For a layout of the system, see Figure 4-27.

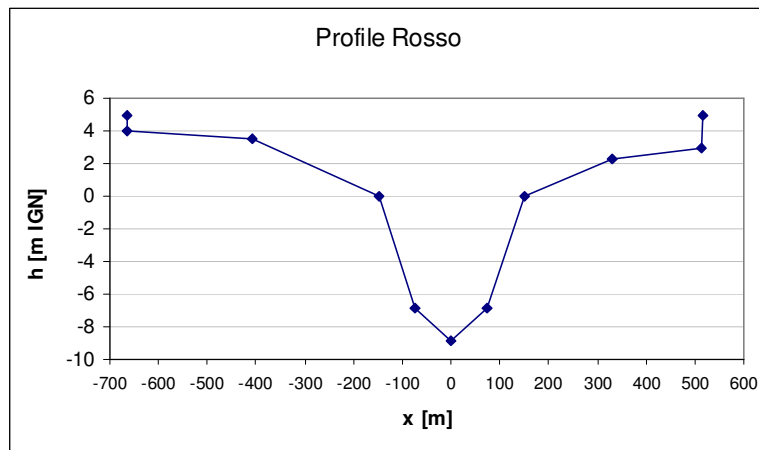


Figure 4-26: Cross section of the river at Rosso.

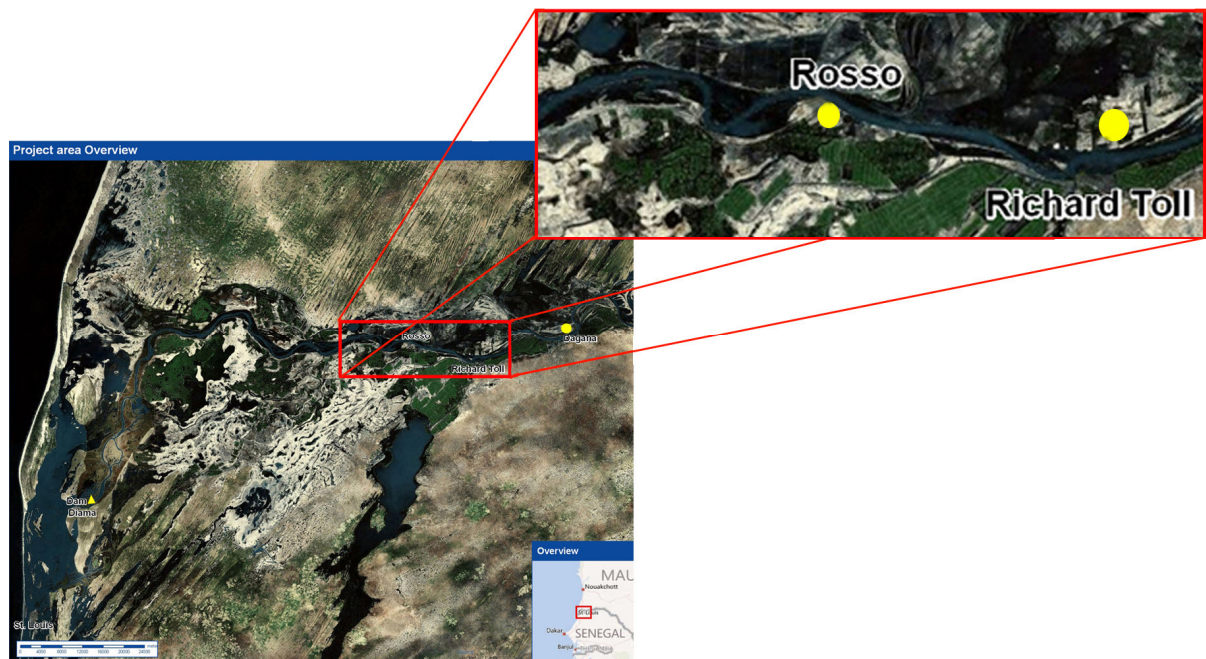


Figure 4-27: Layout of the system: the area between Dagana and Diama with the profile location at Rosso.

(Source: Royal HaskoningDHV)

5 *Hydraulic analysis of the Senegal River Delta*

This chapter will address the hydraulic analysis of the Senegal River Delta. This analysis aims to give a good picture of how the river system works. In order to be able to do so, a hydrodynamic model has been newly set up for the research. The model is described in paragraph 5.1. In paragraph 5.2, the hydraulic analysis is described. Besides the hydrodynamic model, hand calculations using the formula of Bélanger and the approximation of Bresse for the calculation of water levels and the making of discharge-water level relations, have been used.

5.1 The model for the Senegal River Delta from Dagana to Diama

5.1.1 *Model set up*

For the modelling of the Senegal River, a one dimensional Sobek model was set up. This is a numerical tool for simulating water movement in schematized open watercourses in one or two dimensions (www.helpdeskwater.nl). For more information on hydrodynamic modelling, see appendix 5. The stretch between these two locations is split up into various smaller parts and for every part at every beginning and end, a different cross section profile is defined. The cross section profile itself is divided into three parts, two floodplains (the left and right polder) and a main channel, see appendix 5. The real cross sections dimensions are not known, so approximated values are used for width and depth to define the profile. Various model set ups have been used:

- One set up without the dam at Diama
- One set up with the dam at Diama

The set up without the dam is used to be able to simulate the water level at Rosso in case the water level is kept at a certain height at the lower boundary. The Diama dam is modelled to be able to simulate the water level at Rosso in situations with a flood wave coming through the system. See paragraph 5.1.2 for detailed information on the use of the model. At the two boundaries, boundary conditions can be imposed. These boundary conditions can change for the lower boundary from a constant water level to the dam adjusting itself to try to keep a certain water level; for the upper boundary, this condition can change from a certain water level to a certain stationary or non-stationary discharge. See Figure 5-1 for an overview of the modelled river stretch. In this figure, the blue shapes are the profiles, the pink dots are the calculation points (separated 1 km from each other) and the pink rectangles are the boundaries of the model.



Figure 5-1: Modelled river stretch of the system area between Dagana and Diama. The boundaries, profiles and calculation points are indicated in pink, blue and red respectively.

5.1.2 Goal and use of the model

The model is used to serve two goals:

- To get a better understanding of the river system as it works nowadays after the construction of the dams.
- To simulate the water level in the Delta in situations where the Diama dam is not functioning as it should.

The model will be used to answer the following questions:

- What is the Q-h relation for a certain location in the river when the water level at the Diama dam is kept at a certain level? (5.2.2.3)
- What is the water level in the project area in extreme situations when flood waves with extreme high maximum discharges come into the system? These extreme situations are the following:
 - High discharges coming in from upstream (6.3.2.2)
 - Malfunctioning of the Diama dam (9.2.3)
- What is the effect on the water level when the polders are built? Will the building of the polders influence the water level in the Delta? (6.3.2.3)

5.1.2.1 Validation of the model

For the validation of the model, two attempts were done. The first attempt was done in an earlier phase of the project. For the details of the first attempt, see appendix 5.2.

At a later stage, the dam was put into the model and another way of validation was used. In case the dam is closed at the end of the wet season, the OMVS calculates the discharge coming in at Rosso using a hydraulic discharge calculation formula. This formula is based on the Strickler-Manning formula and is altered here to suit the situation.

$$Q_r = kl(H_r - H_f)^{1.629} ((H_r - H_m)/D)^{0.539} \quad [5.1]$$

in which Q_r is the discharge at Rosso [m^3/s], k is a Strickler-Manning coefficient of 7.799, l is the average width of the river between Rosso and Diama (=1440 m in the formula), H_r is the water level at Rosso [m IGN], H_f is the bed level at Rosso (= -13.88 m IGN), H_m is water level at Diama [m IGN] and D is the distance from Rosso to Diama (=105000 m). The k factor and the two exponents are based on discharge calculations at Diama. These are the discharges presented in Figure 4-19.

Taking the restrictions of the dam into consideration (see appendix 3.2) the discharge evacuated by Diama should be less than $1242 \text{ m}^3/s$ for the dam to close again. The OMVS assumes a loss of discharge of $120 \text{ m}^3/s$ between Diama and Rosso, and so when the discharge calculated at Rosso is less than $1362 \text{ m}^3/s$ (which is $1242 \text{ m}^3/s + 120 \text{ m}^3/s$), the dam is gradually closed.

For the validation of the model, this implicates that the discharge coming in at Rosso is the discharge through the Diama dam plus the losses of $120 \text{ m}^3/s$; the latter being a very rough estimation. For the model, the information can only be used when the water level at Diama is constant. The Diama dam adjusts its opening to keep the water level just upstream of the dam constant. When the Diama reservoir fills or empties, there is more (or less) water coming in at Rosso than Diama is discharging.

The dam was put into the model to simulate this situation. The dam manual states that the dam opens up completely when the discharge is (quoted) "about $1850 \text{ m}^3/s$ " (OMVS, 2001 (A) p.21). From then on, the water level rises in front of the dam in function of the discharge. For this discharge-water level (Q-h) combination, a relation was found, see Figure 5-2.

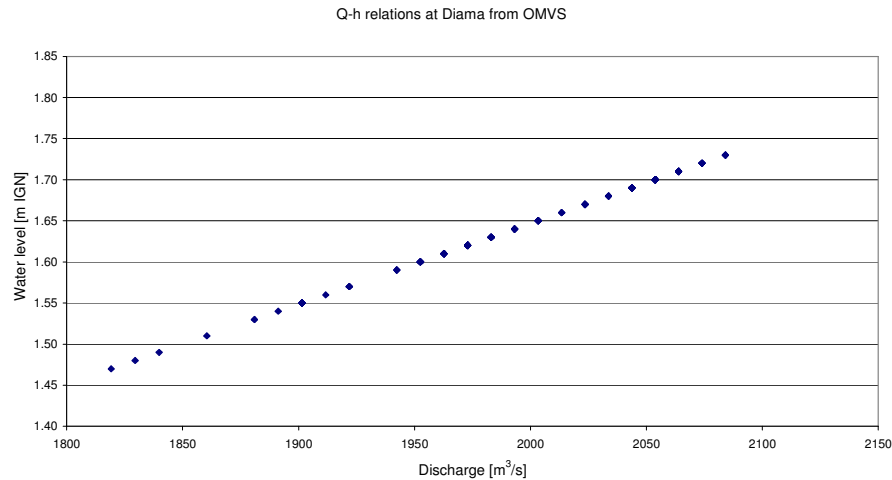


Figure 5-2: Q-h relation from the OMVS when the Diama dam is completely open. The water level starts to increase from a discharge of $1850 \text{ m}^3/s$. This relation should be followed by the model.

The modelled dam has to be calibrated so that this relation is followed in the model. As is visible in Figure 5-3, the dam doesn't completely follow the relation. This is due to the fact that the real situation is slightly different from the model situation (for example the pillars of the Diama dam are not put into the model and the river profile is also different). In the last 20 years, the dam was only completely open three times and the maximum measured water level was 1.73 m IGN corresponding to a calculated discharge of $2084 \text{ m}^3/s$ (see Figure 4-17, in the year 1999). Above this water level (or calculated discharge), it is unknown whether this linear relation will still occur. It could be that the water level follows this linear relation until it reaches the lower level of the gate (which is hanging above the water). From then on, the water level possibly increases faster as the water is retained by the dam again and extra storage of water occurs. In the model, a discharge was imposed at the upper boundary to see what the water level in front of the dam would be. Looking at Figure 5-3, the water level is underestimated by the model for water levels lower than 1.73 m IGN and overestimated for water levels higher than that. This overestimation is however not certain as the red line is not certain either. The underestimation could be due to many factors, for example the profile definition or the dam adjustment.

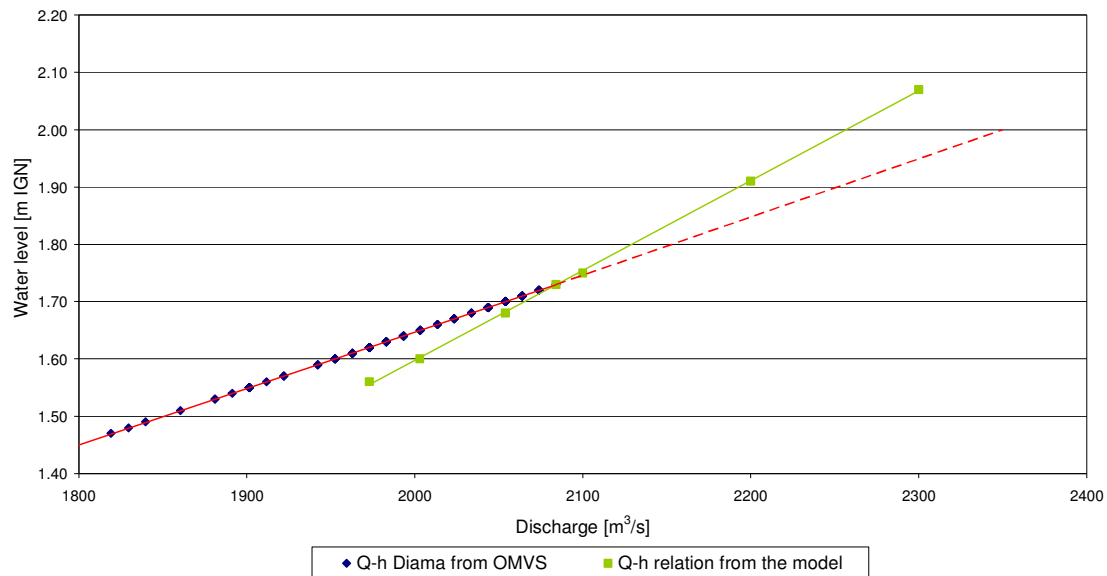


Figure 5-3: The modelled dam does not perfectly simulate the relation.

When the model was used with Chézy values of $40 \text{ m}^{0.5}/\text{s}$ for the main channel and $6 \text{ m}^{0.5}/\text{s}$ for the floodplains, the simulated water level at Rosso was too high compared to the measured level (it overestimated the measured water level with about 30 cm). It was found that a Chézy value of $60 \text{ m}^{0.5}/\text{s}$ for the main channel and of $30 \text{ m}^{0.5}/\text{s}$ in the floodplains was better. The model, however, still slightly overestimates the water level (see Table 5-1, last column). This could amongst others be attributed to the rough estimation of the losses of $120 \text{ m}^3/\text{s}$ or the adjustment of the dam. Other reasons are the dam adjustment or the rough approximations for the river profiles. In (Van Velzen et al., 2003) it is stated that Typha has a Chézy value of $6 \text{ m}^{0.5}/\text{s}$ at a depth of 2-2.5 m and that is why this value was first taken. This value changes with the water depth and becomes greater at greater depths. A Chézy value of $6 \text{ m}^{0.5}/\text{s}$ however does not give good results for the measured water level situations. The higher value for the Chézy coefficient for the floodplains could be attributed to the fact that the Chézy value is depth dependent and that in case of high water, the Chézy value is higher. An overview of the calculations is given in Table 5-1. What is of interest is imposing discharges at the upper boundary and look what is the resulting water level. This is further analysed in paragraph 5.2.2.3 and chapter 6.

Table 5-1: Simulated compared to measured water levels. The model overestimates the water level by up to 8 cm.

Discharge Dama [m^3/s]	Losses [m^3/s]	Measured water level [m IGN]	Simulated water level [m IGN]	Difference [m]
1955	120	2.7	2.75	0.05
2043	120	2.83	2.91	0.08
975	120	1.81	1.89	0.08
1073	120	1.89	1.96	0.07
947	120	1.8	1.87	0.07
1262	120	2.04	2.1	0.06

5.2 Hydraulic analysis of the Senegal River Delta

First, the hydraulic influence of the upstream basin on the Delta will be addressed, as this part of the river has great influence on the water flow in the Delta. Then, the Delta will be analysed using the backwater curve technique and analysing Q-h relations.

5.2.1 Upstream area with factors that influence the water flow in the Delta

The basin upstream of the project area greatly influences the discharge and the water level in the project area. For this reason, the basin upstream is described. The amount of water flowing into the project area is influenced by various factors:

- The rainfall upstream
- The Manantali dam
- Evaporation in the Manantali reservoir and in the river upstream of Dagana
- The amount of water that has delayed flow due to presence of wide flow plains
- Seepage of water into the ground
- Withdrawal of water to be used for irrigation and drinking water

As mentioned before, in the region between Bakel and Dagana, the floodplains are very wide (actually these are flow areas of up to 35km wide) where agriculture is important. Due to the wide flow areas, water can easily experience delayed flow. The water can stay behind in ponds, or in periods of very high discharges, the water can flow into the flow areas and it will take longer for the water to finally reach the Delta area. Seepage of water into the ground is also of importance for the amount of water flowing downstream. A certain amount of water is extracted from the river for irrigation purposes. Also water is extracted as drinking water.

5.2.2 The Delta

To get a good view on how the hydraulic system in Senegal River works, the river flow is approximated by using the backwater curve technique. In this technique, simplified versions of the De Saint Venant equations

have been used and assumptions have been made on the river's cross section (De Vriend, 2007). See appendix 4 for more information on the technique. The backwater curve has been simulated using a hand calculation and using Sobek. Firstly the hand calculation will be explained; secondly the Sobek calculation will be described in short. Afterwards, the results will be interpreted.

5.2.2.1 Construction of the backwater curves

For the hand calculation, the normal procedure of calculating the water level gradient line (in Dutch: verhanglijn) has been done using the formula of Bélanger and the approximation of Bresse. In this approximation, the flow into the system is considered to be stationary. Two cases have been calculated: at the downstream boundary (Diama), a fixed water level is set to 2.25 m IGN and 1.5 m IGN and for every 50km of river stretch, the backwater curve has been calculated. This is done by calculating the equilibrium depth, the half length and in the end the water depth (De Vriend, 2007) at a certain point along the stretch (see appendix 4.2). As mentioned, the river has been divided into stretches of 50km. This is done because the bed level slope is not constant. The water depth calculated at the end of the previous stretch is used as the lower boundary for the next stretch. When the calculated water depth is below the initial value (of 2.25m IGN or 1.5m IGN) the water level is set at 2.25m IGN or 1.5m IGN respectively (see Figure 5-4 and Figure 5-5). This is done because otherwise water would flow upstream, which is physically not possible. Also, the bed level is showed in the figures (red line).

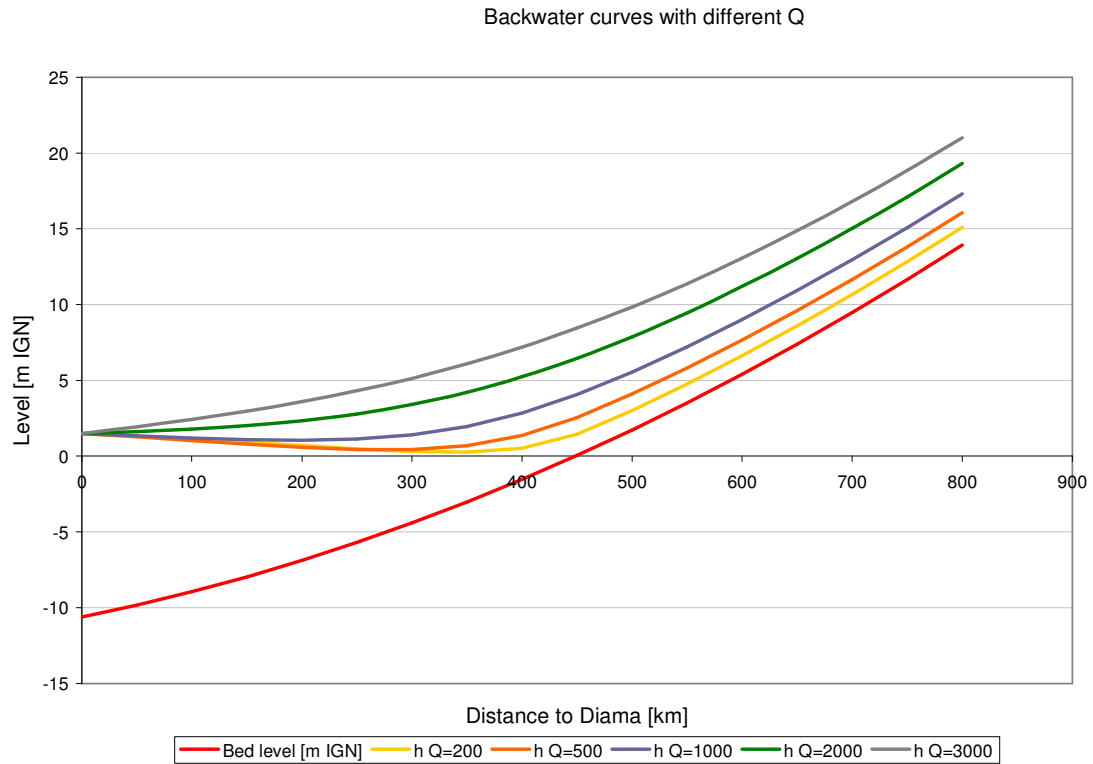


Figure 5-4: Backwater curve calculation without correction for water flowing upstream with $h=1.5\text{m IGN}$ at downstream boundary. As the discharge increases, the water level also increases.

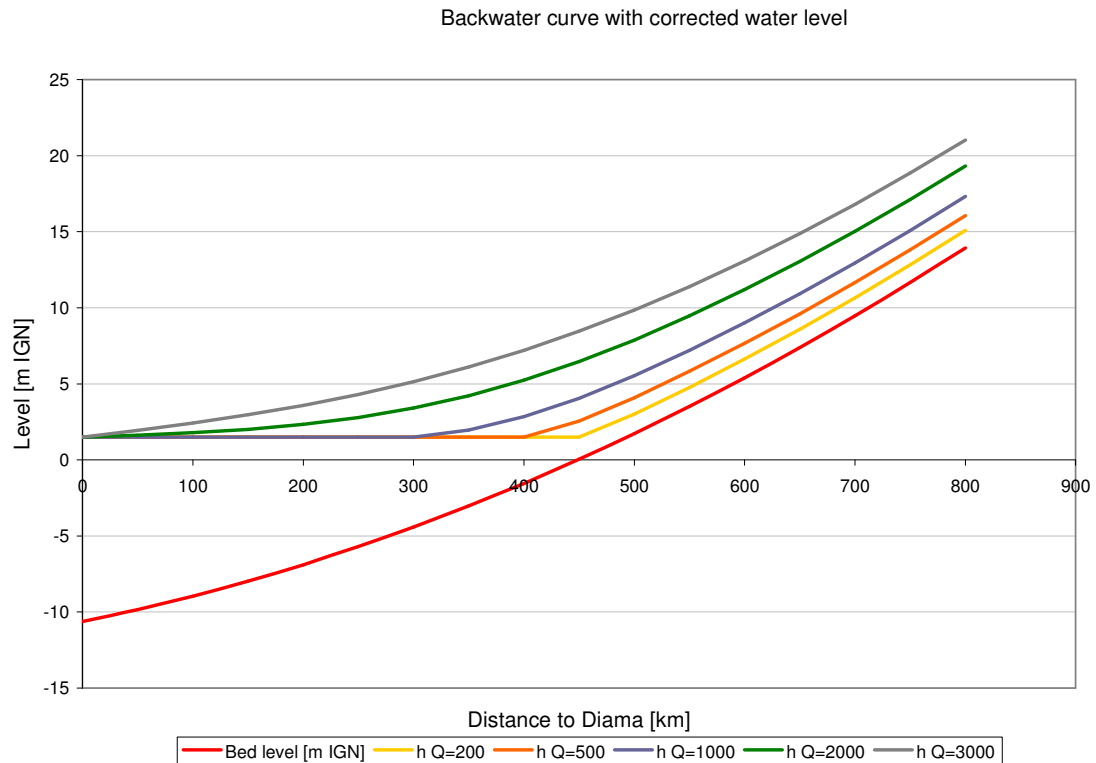


Figure 5-5: Backwater curve simulation with correction for water flowing upstream with $h=1.5\text{m IGN}$ at downstream boundary. The water level increases as the discharge increases.

For the Sobek simulation, the downstream boundary condition is set on 2.25m IGN or 1.5m IGN and the backwater curve is made by using the side view option with various discharges flowing into the system.

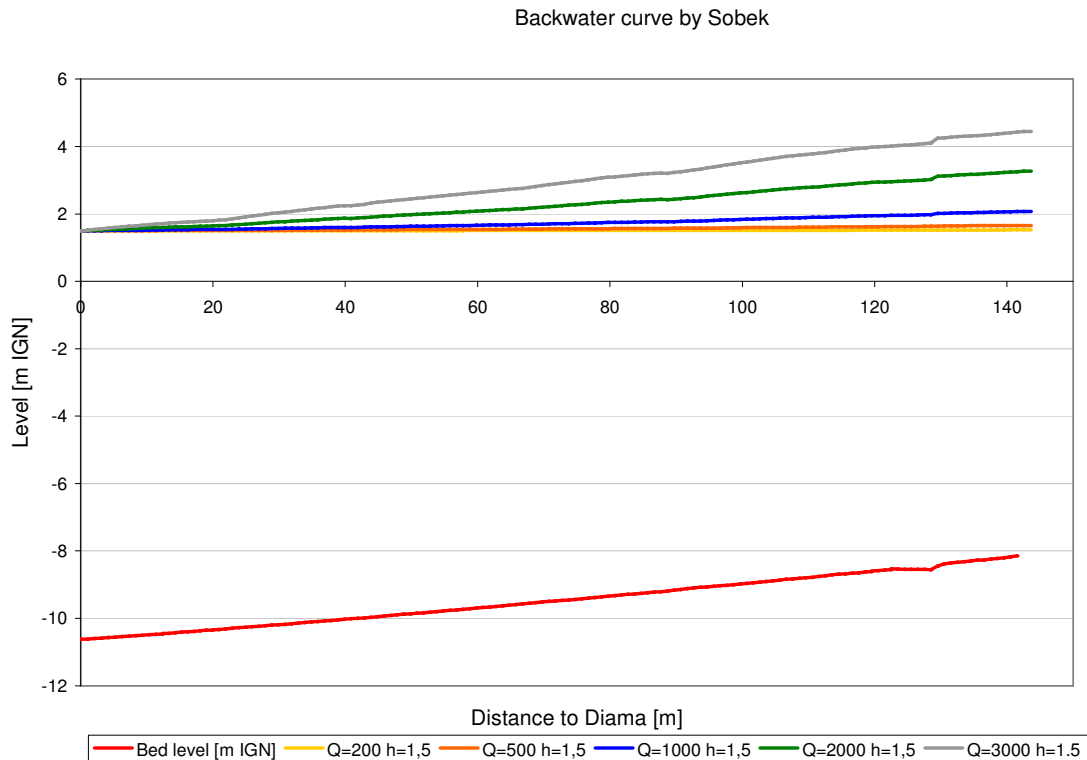


Figure 5-6: Sobek side view simulation with $h=1.5\text{m IGN}$ at downstream boundary. The water level increases as the discharge increases and the water level slope is gentle.

5.2.2.2 Interpretation of the backwater curve

First the hand calculation will be evaluated and then the Sobek calculation will be interpreted.

The hand calculation is a very rough approximation. The method used assumes a straight channel with a shallow, rectangular cross section (De Vriend, 2007). At the Senegal River, this is not the case at all. Moreover, the backwater curve has been calculated for every 50km of the river. This also influences the course of the curve in the way that for every 50km a new equilibrium depth is calculated and a new half length is calculated. Based on these new calculations, new values for the equilibrium depth and water levels are generated.

For lower discharges, in the way the backwater curve has been modelled, water will flow upstream as the equilibrium depth is very low and the difference between the equilibrium depth and the water level; is big (Imagine the difference between the water level and the equilibrium depth is big in Figure 5-7, then the water level will be “pulled down” by the equilibrium depth as the water level asymptotically goes towards this depth, this is what happens in Figure 5-4.). This is not the case in reality because of the presence of the Dama dam. This is why the water level has been corrected for levels lower than the initial value (see Figure 5-5). Here, an offset is visible in this figure at the moment the water is higher than the initial water level, which in reality is not possible either. As mentioned earlier, it is a theoretical approximation. In this approximation, the

influence of the equilibrium water depth on the actual water level is not felt anymore at a certain discharge (the water level doesn't have to be corrected anymore).

These hand calculations are made to show the phenomena happening in the system; the numeric values are not important. It can be seen that only from a certain discharge, the water level will increase substantially. This same phenomenon will later be seen in the analysis of the Q-h relations in paragraph 5.2.2.3.

In Figure 5-7, the area of influence of the Diamo dam upstream is presented. This area reaches till the location where the difference between the (corrected) water level and the equilibrium depth is small (defined as $<0.5\text{m}$). It was stated that the area of influence of the Diamo dam was about 350 km (BCEOM, 1999). In Figure 5-7 this is also visible. At about 450 km, the difference between the water level and the equilibrium depth becomes small. This area is longer than 350 km, but as the model is a rough approximation, it is a good approximation for the area of influence of the Diamo dam.

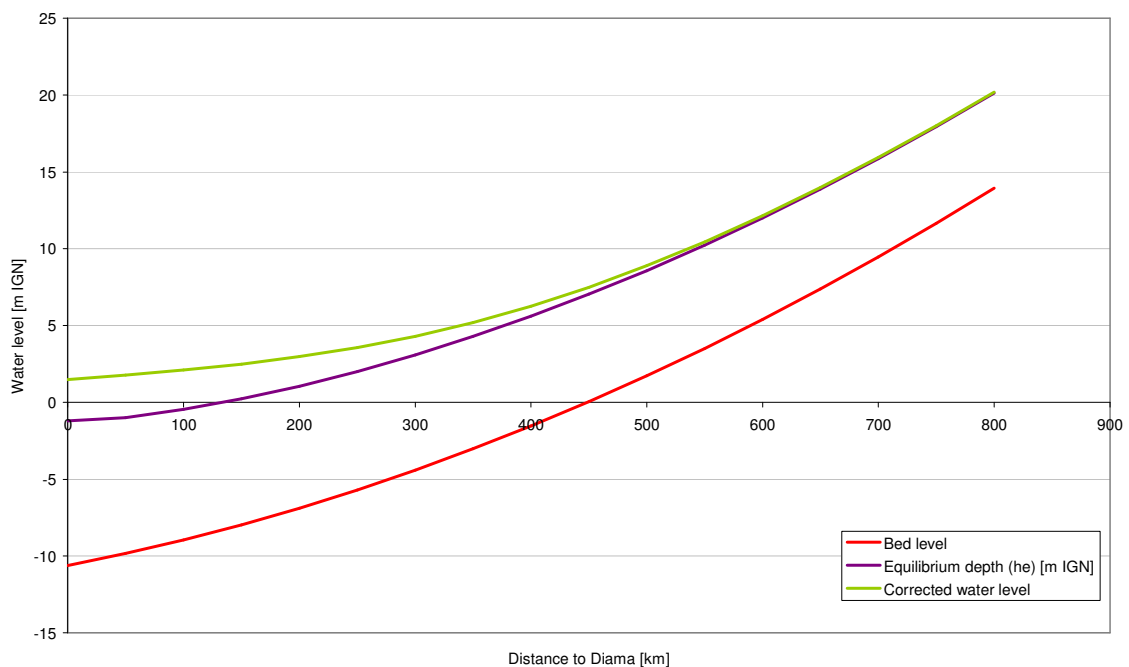


Figure 5-7: Backwater curve with equilibrium depth. The influence of the Diamo dam reaches up to where the difference between the equilibrium depth and the water level is small; about 450 km in this case.

The water level slope generated by Sobek gives a more realistic view of the situation as the assumptions made in the hand calculation (of a straight channel with a shallow, rectangular cross section) are not done. However, this is also only an approximation as from a certain discharge, the water level at Diamo will not be 1.5 m IGN anymore as the dam can not cope with the discharge coming down. As visible in Figure 5-6, the slope of the water level will increase when the discharge increases; however, the slope is still gentle. This figure is made to get an idea of how the water level changes over the first 140km of the river.

5.2.2.3 Q-h relation at Rosso

For the construction of the Q-h relations, the profile presented in paragraph 4.4 at Rosso has been chosen (all different profiles have their own Q-h relation). The Q-h relation has been made by hand using various methods and using the Sobek model. For the exact way the calculation of the relations by hand and in Sobek was done, see appendix 4. Firstly, the Q-h relation in case of free flow made by hand and made by Sobek is analysed and secondly the Senegal River case is analysed.

Free flow

When no structure or other object is hampering the flow of water downstream, the water can freely flow through the river and the water level is low at low discharges and higher at higher discharges. To simulate an empty channel without influence of any structure downstream in the model, an extra reach of 1000 km has been added in the model. The Q-h relation for the chosen profile then looks like Figure 5-8.

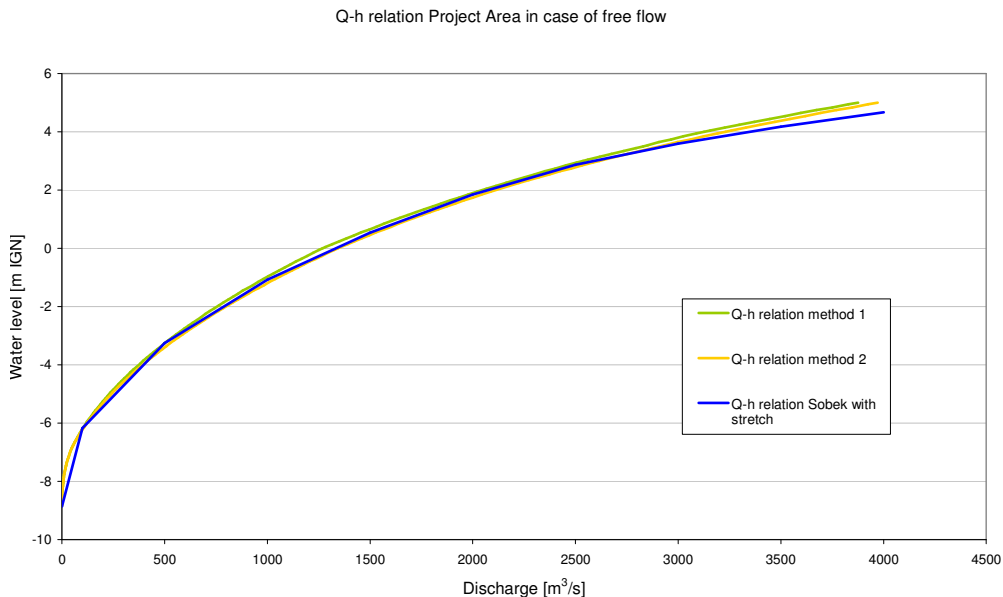


Figure 5-8: Q-h relation calculated using different calculation methods in case of free flow. There is good agreement between the methods.

For this calculation, a Chézy coefficient value of $60 m^{0.5}/s$ in the main channel and a Chézy coefficient value of $30 m^{0.5}/s$ in the floodplains has been used. For calculations with different Chézy coefficient values, see Appendix 4. As can be seen in Figure 5-8, the Sobek calculation finds itself a little bit lower compared to the hand calculation. The modelled relation was defined at a calculation point at a certain distance from the profile definition point. The hand calculation used the profile as defined, without interpolation. The small difference in the figure can be explained by Sobek using slightly different values for bed level slope as the model interpolates this value in between the two defined profiles. Also the cross section area is interpolated

and for this reason is different from area used in the hand calculation. The expected offset in the relation due to the sudden participation of the floodplains is less visible because of the floodplains being relatively rough.

The Senegal River case

In the Senegal River situation however, there is no free flow. The water is retained by the Dama dam and a reservoir is formed (see Figure 5-9).

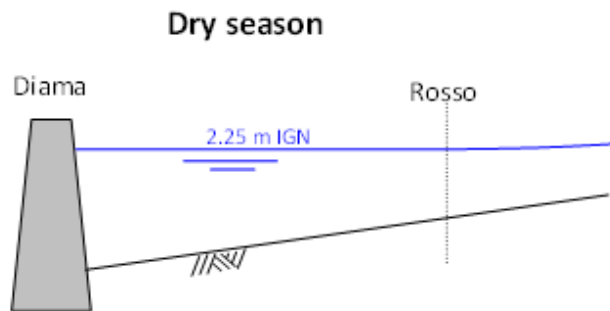


Figure 5-9: The Dama dam retains the water in the Senegal River and no free flow occurs.

In this case, the Dama dam influences the Q-h relation of the project area. This is visible when looking at Figure 5-10; the lower boundary is set at various values and the Q-h relation changes considerably. Another conclusion from Figure 5-10 is that putting the water level at the lower boundary at for example 2.25m IGN, a discharge up to about $300\text{-}400\text{ m}^3/\text{s}$ has minimal influence on the water level increase in the project area. This aspect has earlier been seen in the backwater curve analysis, with different values for the discharge however. Multiple lines have been drawn to show how the system works; however, in reality, the water at Dama is held between 1.5m IGN and 2.25m IGN and so only the lines of $h=1.5\text{m IGN}$ and of $h=2.25\text{m IGN}$ should be used (blue lines). During the wet season, the water level is lowered by the Dama dam when high discharges come from upstream. It is not the case that exactly one of the two blue lines is followed. The lines lower than 1.5 m IGN are actually not used; they are however shown to indicate that in the Senegal River case with a dam, the Q-h relation is different.

As the situation of free flow through the system is not applicable because of the presence of the Dama dam, the Chézy formula for discharge calculation is not applicable either! This is why the model will be used to simulate water levels.

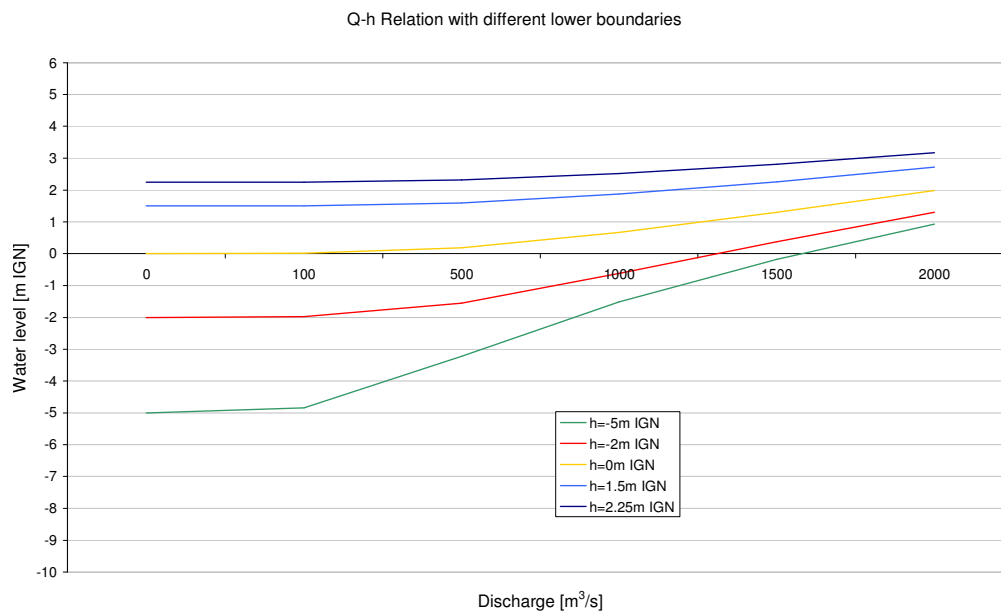


Figure 5-10: Q-h relation at Rosso using Sobek with different downstream boundaries. The two blue lines should be used and there is a clear difference with Figure 5-8. This is because of the Dama dam which causes no free flow in the river.

PART 3: The Dike

6 Failure mechanism and the failure probability

In this chapter, the overflowing failure mechanism is addressed. The extreme discharges and the extreme water levels are defined and consequently the probability of failure is defined for a dike height between 2 and 6 meters.

6.1 Overview of failure mechanisms

6.1.1 Fault tree

A dike can fail due to multiple failure mechanisms, as presented in Figure 6-1.

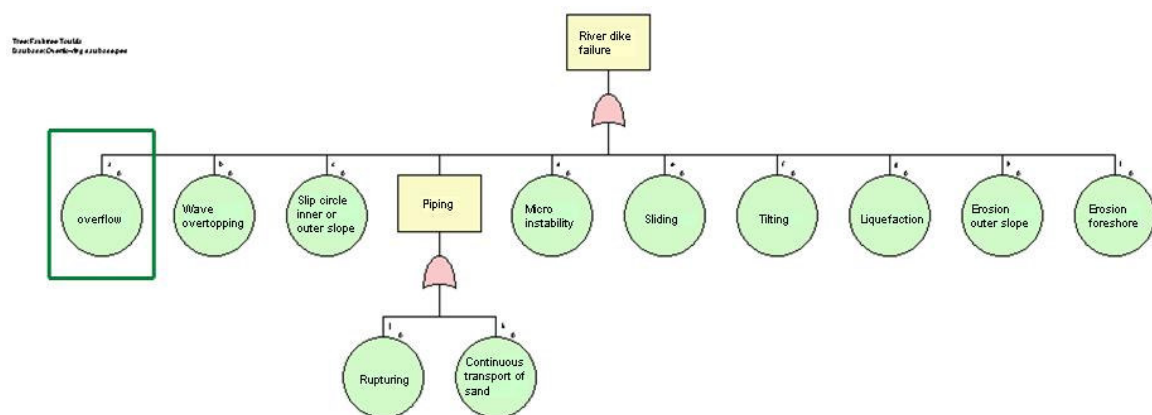


Figure 6-1: Dike failure mechanisms; only overflowing will be investigated.

In this thesis, only the overflowing failure mechanism is considered. The other failure mechanisms are not considered because in the first place the hydraulic system of the river had to be understood and data for other failure mechanisms were not (yet) available.

In Figure 6-2, a fault tree for the failure mechanism overflowing is given. The malfunctioning of the Dama dam in combination with the flood wave will be addressed in chapter 9 by performing a sensitivity analysis; the flood wave in the Delta will be addressed in this chapter and chapter 7 and 8. In Figure 6-2, the combination of the flood wave and the malfunctioning of the dam is not further elaborated as the sub-trees are the same for the flood wave and the malfunctioning of the dam as already presented in the figure. In the tree, the subsidence of the dike itself could for example also be taken into account, but the focus lies on the water level in the river.

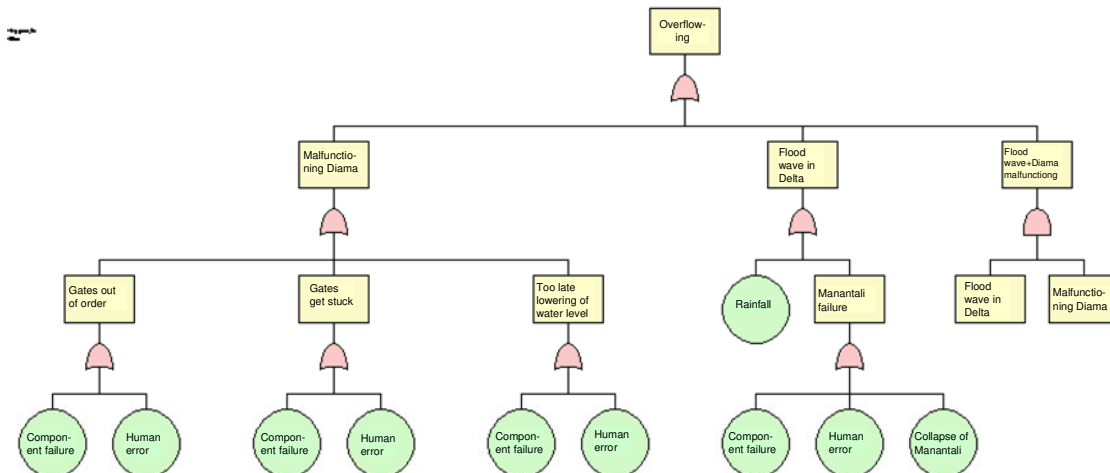


Figure 6-2: Fault tree for the failure mechanism overflowing. The flood wave and the Dama dam are the most influential factors on the water level.

6.2 Cross section dike

For simplicity, the slopes on both sides of the dike are assumed to have the same angle. The slope of the talus will be fairly steep (more in the order of 1:1, 1:2 or 1:3 than in the order of 1:5 for example). This is, as mentioned earlier, done because of the Typha. If the talus of the dike becomes mildly sloped (on the river side), the Typha will have space to grow again in a too wide area and the access to the river will still be blocked. If the slope of the talus is steep enough, the Typha will still be able to grow back, but on a smaller area. Like this, the Typha will only function as bank protection, which is a positive result. The slope of the talus can be influenced by for example the failure mechanism piping, however, this failure mechanism is not investigated here. A study was found on the extension of the existing river dikes upstream of Rosso and these dikes would be designed with a crest width of 3 meters. This is the same as the crest width of the dikes in place. That is why the new polder dikes will have the same crest width as these dikes. A picture of the dike to be constructed is shown in Figure 6-3.

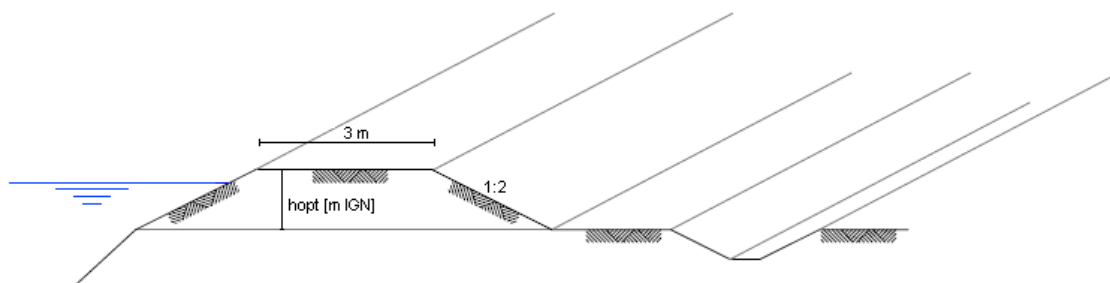


Figure 6-3: The dike with a crest width of 3m and a slope of 1:2.

6.3 Overflowing

6.3.1 Reliability function Z

This failure mechanism occurs when the water level in the river exceeds the crest level of the dike. The dominant load which drives this failure mechanism is the maximum water level in the river (see appendix 2). The high water levels during the wet season have to be taken into account when designing the dike. The most general reliability function (limit state function) for the overflowing case reads:

$$Z = H_{cr} - H_{wl} \quad [6.1]$$

in which H_{cr} is the dike crest level [m IGN] and H_{wl} is the water level in the river [m IGN].

6.3.2 Working method

The working method is to analyse the reliability function for this failure mechanism. In the probabilistic approach, the type of distribution for each variable of the reliability function has to be determined. Looking at the reliability function for this failure mechanism considered here, a distribution type and parameters will have to be determined for the water level in the river at Rosso. To do so, the hydrodynamic model is used. Only the water level at Rosso is known, and the discharge yonder is unknown, so that the latter will first have to be found in the situation with measured water levels and in extreme discharge situations. In the upcoming paragraphs, the different steps of the calculation will be explained in more detail; each paragraph explains one step. First, the discharge at Rosso will be defined, and then the water level is defined. Afterwards, the distribution and parameters of the water level are found.

6.3.2.1 Discharges in the Delta

To get to a discharge at Rosso, three methods have been investigated. As the project advanced, it became clear which method was the best to use. The three methods are explained below.

First method

The first method investigated was to define the discharge in the project area as a percentage of the discharge at Bakel. As the discharge is known at Bakel and at Diama, the maxima are compared and a statement was sought for on which percentage of the maximum of Bakel flows into the Delta. This, however, is an inaccurate way of getting to a discharge in the project area because of the following reasons:

- Comparing the maximum discharges of Bakel and Diama gives a feeling on which part of the discharge of Bakel goes out through Diama (!). This does however not give an idea of the discharge flowing into the Delta at Dagana as Dagana is located 140 km upstream of Diama, and in the Delta losses due to evaporation, irrigation and distributaries occur (Diop, 1992).
- The area between Bakel and Dagana, as described in paragraphs 4.2.2.4 and 5.2.1, influences the maximum discharge considerably.

- An unambiguous factor as a relation between the two locations is not easy to define, looking at Figure 6-4 and Figure 6-5. One of the explanations for this unambiguity is that when a maximum discharge at Diama is calculated, this in some cases is a discharge for emptying the reservoir (and not to cope with water coming from upstream). Another explanation is that the wide flow plains influence the flood wave coming down.

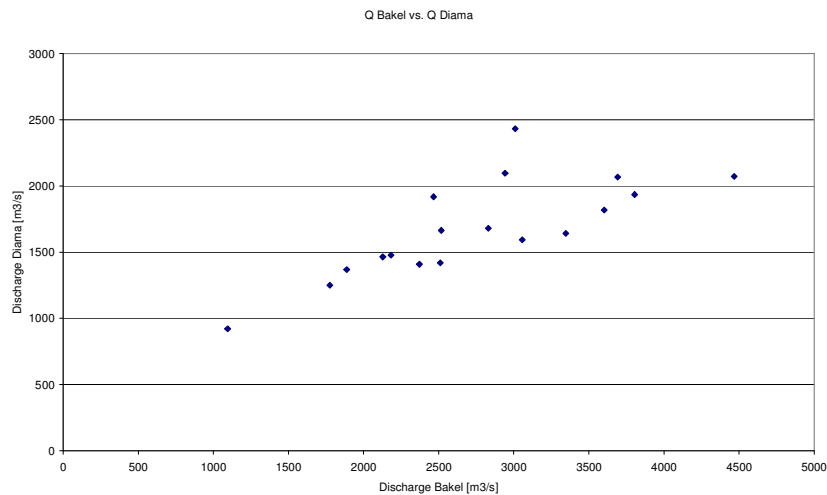


Figure 6-4: The relation between the maximum discharges at Bakel and at Diama is not unambiguous so one multiplication factor between the discharges at Bakel and Diama is not definable.

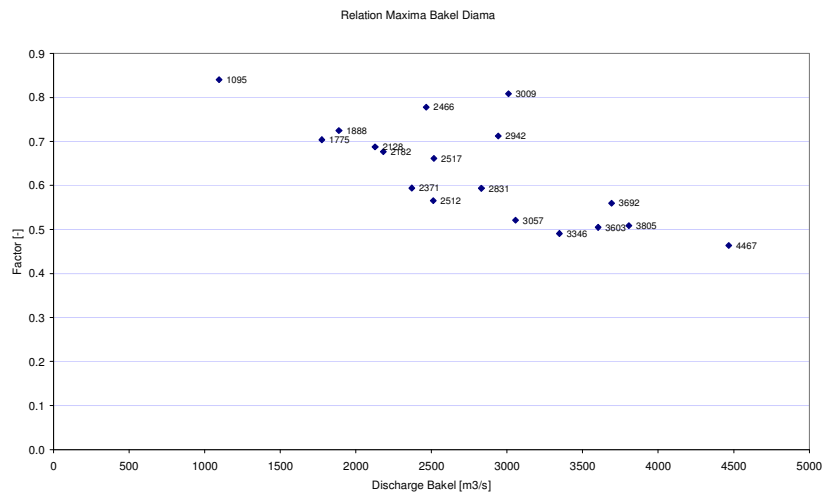


Figure 6-5: Also considering Figure 6-4, one factor between the two discharges is not definable.

It was also considered to extend the hydrodynamic model to Bakel, so that the model would calculate the maximum discharge itself. The information for the model could be found in USGS Hydrosheds. The model is however not extended to Bakel because of the following reasons:

- The profiles (analysed from USGS Hydrosheds) vary strongly the one from the other in width (10 to 30 km) and in depth (tens of meters) that the Valley basin can't be modelled correctly with a limited amount of cross sections;
- The area between Bakel and Dagana consists of so many creeks, (old) (dis)tributaries, gullies and vast areas where water can flow that the river should be modelled in 2D. The Sobek model would correctly simulate if the floodplains did not contain gullies, which is not the case here;
- The information on discharge at Bakel is calculated, and is influenced by the Manantali dam;
- The OMVS itself doesn't use the information at Bakel to give an expression about the discharge at Dagana or at Rosso. The OMVS uses the formula related to the water level at Diama to calculate a discharge at Rosso (see paragraph 5.1.2.1) instead of using the available information of Bakel. The OMVS gives preference to the use of a formula based on information at Diama instead of using information at Bakel for the management of the dam.

Second method

A second method to get to the discharge in the project area was to take the discharge calculated at Diama (for which the calculation formula was based on measurements, see paragraph 4.2.2.6) and add a certain discharge for the losses in the area. So the discharge at Dagana (upper boundary of the model) would be:

$$Q_d = Q_{Di} + Losses \quad [6.2]$$

in which Q_d is the discharge at Dagana, Q_{Di} is the discharge at Diama and Losses are the losses between Dagana and Diama. Here, the discharge at Dagana is needed as this is the boundary of the model.

The inconvenience of this method is that it can only be used when the water level at the Diama dam is constant (otherwise the calculated discharge might be "used" to empty the reservoir and this discharge is not coming in at Dagana). Moreover, the losses are a first very rough estimate done by the OMVS in 1999. Between 1999 and nowadays, the losses are likely to have increased as also the amount of irrigated agriculture has increased. That is why this method is rejected.

Third method

The best way to give a realistic statement on the discharge in the project area is to use the validated model. The water level upstream and downstream could be imposed to the model; however, this is preferably not done because when two boundary conditions of the same kind are imposed, the model could give wrong answers. As the water level slope is imposed. This is wrong, because the model should calculate this slope itself.

That is why the discharge should be found by iteration. At the lower boundary (Diama), the measured water level is imposed and at the upper boundary (Dagana), a discharge is imposed. The goal is to impose the discharge for which the model will give the measured water level at Dagana. These steps are repeated until the correct (measured) water level at Dagana is found. For this case, the two ways of discharge simulation

are followed and they result in a maximum difference in discharge of $10 \text{ m}^3/\text{s}$. The problem with this method is that at every time the water levels are measured, the dam is in a different position. This results in an inhomogeneous dataset of point as the conditions are not the same when calculating the water levels. However, considering the situation, this is the best procedure to follow.

6.3.2.2 Water level in the Delta at Rosso

In order to get to a water level in the project area, an extreme value analysis with periodic maxima is done on the discharges to be able to give an expression on the low frequency water levels. Three distributions were used in the analysis: the exponential distribution, the Gumbel distribution and the Weibull distribution. These three distributions are investigated because these are common extreme value distributions for river discharges. The function that describes the dataset best was sought for and it appeared that all three distributions describe it well, as is visible in Figure 6-6. The exponential distribution and the Weibull distribution give the most conservative outcome for the extreme values and the exponential distribution is used for further analysis. This is done because this distribution is the most conservative of the three distributions for low frequency discharges. Moreover, the pragmatic reason that for the exponential distribution the parameter uncertainty can be analytically investigated is considered. This parameter uncertainty will be elucidated in paragraph 9.2.2.

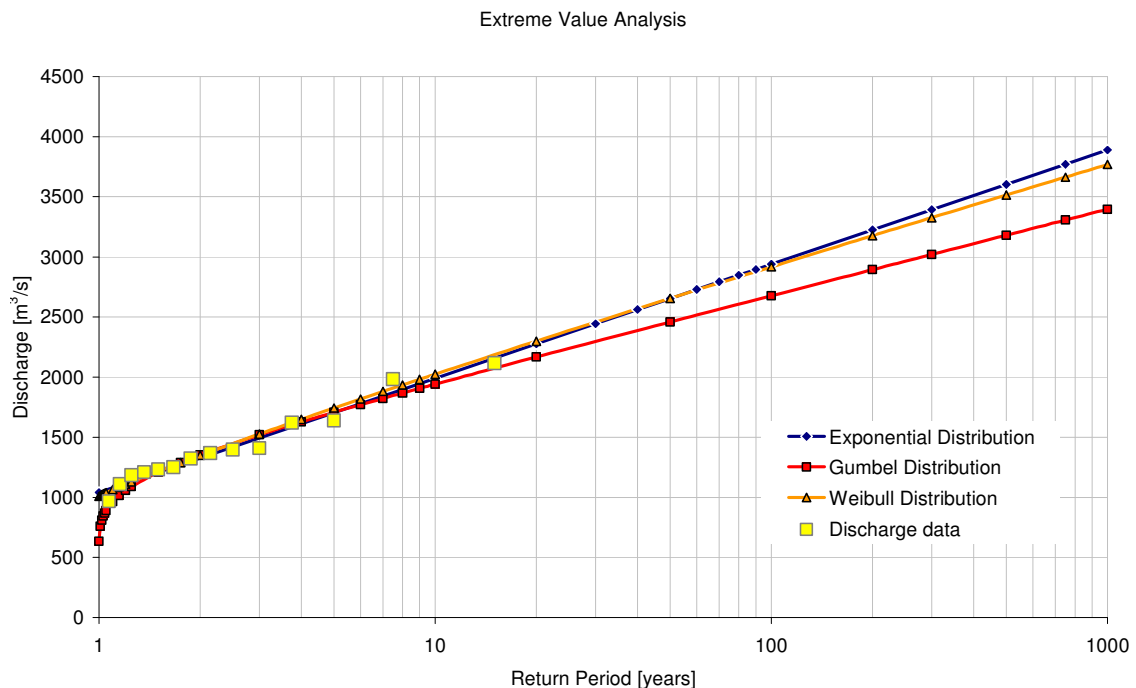


Figure 6-6: Extreme value analysis on the discharges. The three distributions appear to describe the data well; the exponential distribution is further developed as it is most conservative.

Based on the extreme value analysis, the water levels were simulated by the model. To do so, the following was done:

- The dam was put into the model keeping the water level just in front of the dam at 1.5m IGN. If the dam can not cope with the discharge, the water level will rise, following the relation from Figure 5-3 (see chapter 5 how this is done)
- The values of the extreme discharges were transformed into flood waves (with a maximum and duration).

To give a good approximation of the flood wave at Rosso, an analysis of the historical flood waves was done at Rosso. The goal is to find a relation between the maximum discharge and the flood wave duration in order to be able to scale the flood wave to higher discharges. As mentioned in paragraph 5.1.2.1, the discharge at Rosso was calculated using a Strickler-Manning formula. The OMVS uses the discharge calculations at Rosso, done by this formula, to manage the dam. The discharge was calculated for the years 1992 and 1999 and an approximation formula was looked for. It appeared that an increase of the maximum discharge of $1000 \text{ m}^3/\text{s}$ corresponds with a duration increase of 90 days. See table Table 6-1 for an overview.

Table 6-1: Flood wave analysis, a flood wave maximum difference of $1000 \text{ m}^3/\text{s}$ corresponds to a 90 day longer flood wave.

	Max. discharge [m^3/s]	Duration [days] ($Q > 500 \text{ m}^3/\text{s}$)
1992	1400	40
1999	2400	130

The duration of the flood wave is defined by $D = \frac{(Q_{\max} - 1400)}{11.1} + 40$. Now that the relation is found, approximations can be done. Three flood waves were approximated, those of the years 1992, 1995 and 1999. The flood waves are approximated using a second order polyfit (parabola). For the duration of the flood wave it was said that when the discharge was over $500 \text{ m}^3/\text{s}$ the flood wave started. This can be done as at a discharge of $500 \text{ m}^3/\text{s}$, the Diam dam can easily cope with the water coming down. It appeared that the parabola and the found relation between discharges is a good approximation for the flood waves. As can be seen in the figures below, the approximation is not perfect, however. For example for the year 1995, the flood wave is overestimated by the approximation at decreasing discharges. However, this is not a problem, as by then the dam can cope with the inflowing discharges (of less than about $1850 \text{ m}^3/\text{s}$). In Figure 6-7 through Figure 6-9, the calculated discharges are depicted as well as the approximated waves.

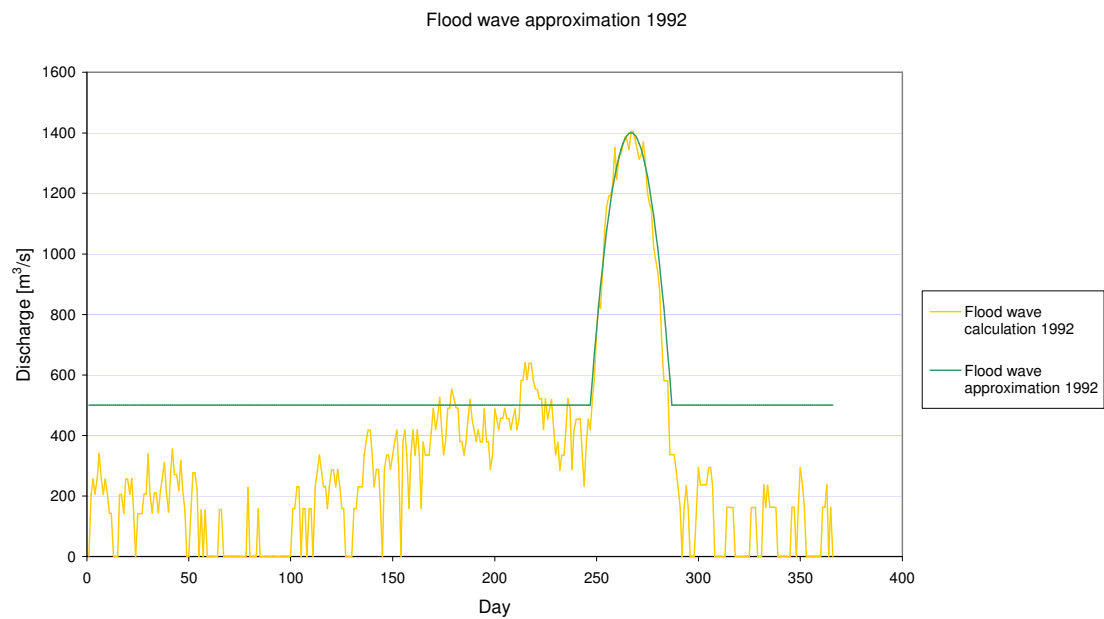


Figure 6-7: Flood wave approximation at Rosso in 1992, a good approximation.

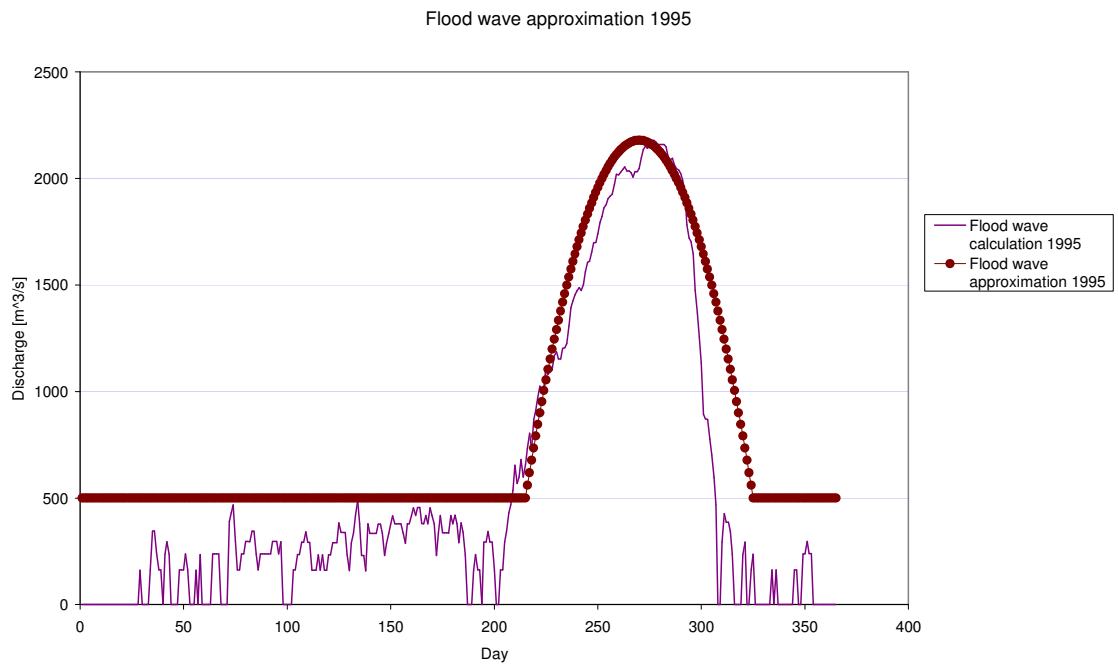


Figure 6-8: Flood wave approximation at Rosso in 1995, the approximation overestimates the duration of the flood wave.

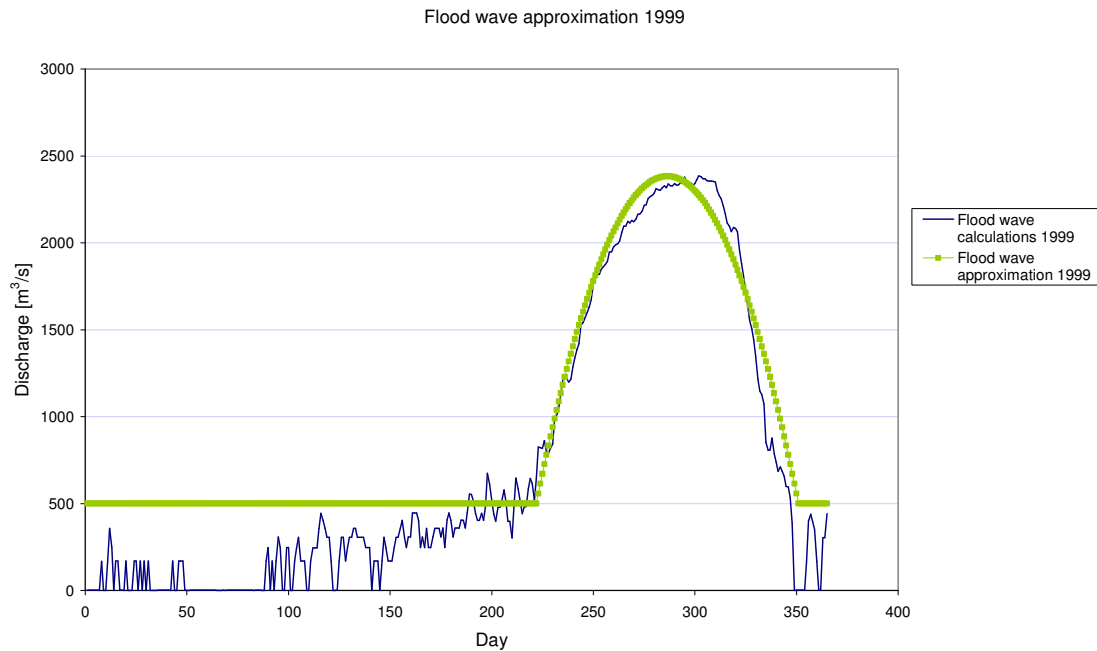


Figure 6-9: Flood wave approximation at Rosso in 1999, a good approximation.

These approximations can also be used for higher discharges from the extreme value analysis from Figure 6-6. This is of course a very rough approximation as it is unknown how the river upstream of the Delta will act when such an amount of water is coming through. The approximations of the flood wave for high discharges based on the exponential distribution are depicted in Figure 6-10.

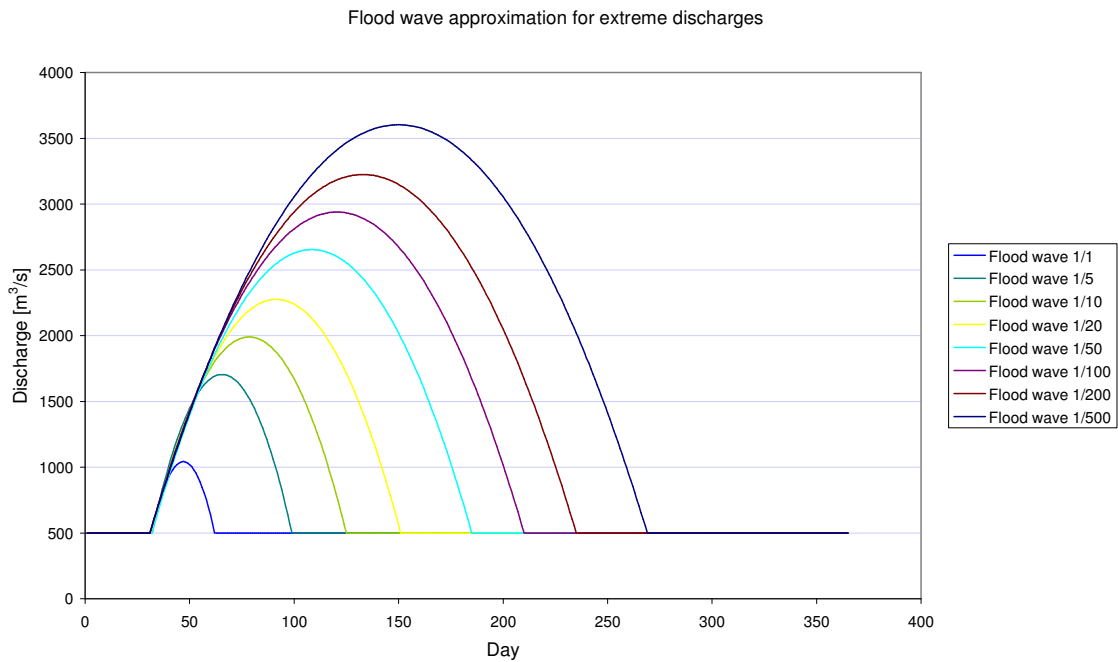


Figure 6-10: Flood wave for extreme discharges at Rosso based on the exponential distribution. The duration increases as the maximum increases.

By imposing these flood waves to the hydrodynamic model, corresponding extreme water levels are found. The once per year flood wave gives the once per year water level and so on to 1/500. The result is depicted in Figure 6-11.

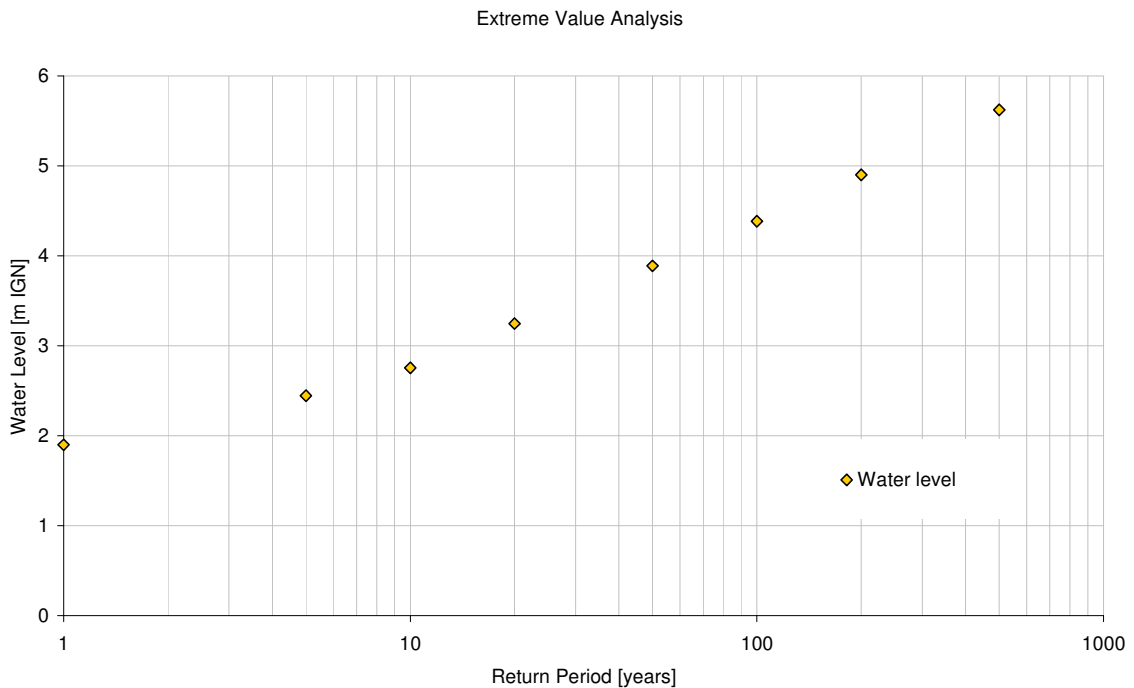


Figure 6-11: Water levels at Rosso generated by the approximated flood wave. The water level ranges between 1.9m IGN and 5.60m IGN.

6.3.2.3 Distribution and parameters for the water level at Rosso

To this series of water levels, the Exponential distribution and the Weibull distribution have been fitted to see which one would best describe the dataset. The Weibull distribution appears to fit the dataset best, as presented in Figure 6-12, and is therefore chosen for further analysis.

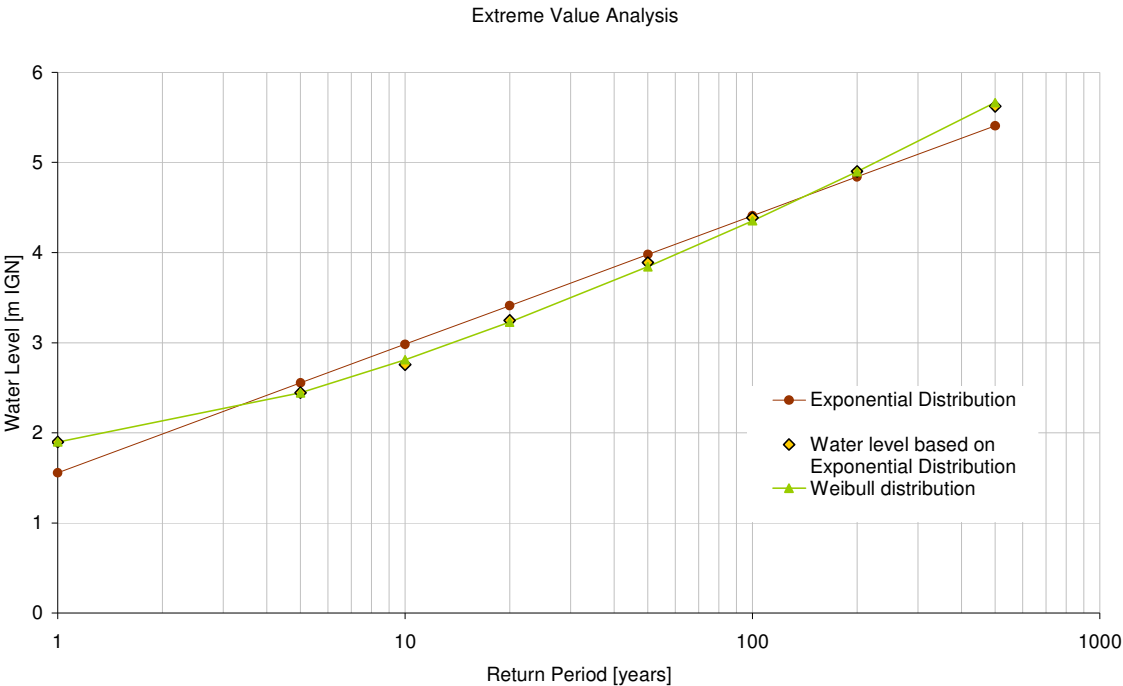


Figure 6-12: Distribution fit to the water level dataset at Rosso. The Weibull distribution fits best.

The results here are shown in the present situation, when no polders are built and so the correction for water level increase due to narrowing of the river is not taken into account. Two more scenarios are investigated: one scenario with only the area between Richard Toll and Rosso being poldered (so the increase in water level due to building polders is taken into account) and one scenario with all the floodplains being poldered. These three scenarios are investigated to show the difference between the three. The same steps are followed as in the no polder case to get to the water levels. The water level simulations are shown in

Table 6-2. It can be concluded that making polders only in the Rosso-Richard Toll area has a limited influence on the water level yonder (15 cm at the most, see column 1 and 2 in

Table 6-2), but making polders in all the floodplains does have a significant influence on the water level at Rosso (71 cm at the most, see column 1 and 3 in

Table 6-2).

Table 6-2: The influence of polder construction on the water level is maximum 71 cm.

Flood wave return period [per year]	Present situation [m IGN]	Polders between Rosso and Richard Toll [m IGN]	Polders in all floodplains [m IGN]
1/1	1.90	1.90	1.93
1/5	2.45	2.46	2.54
1/10	2.75	2.78	2.89
1/20	3.25	3.29	3.48
1/50	3.89	3.95	4.26
1/100	4.39	4.48	4.86
1/200	4.90	5.01	5.49
1/500	5.62	5.77	6.33

6.3.3 Results for the failure probability

Now that the distribution and parameters for the water level are defined, the reliability function can be analysed. This is done using two methods: the FORM method and the Monte Carlo simulation method. For a detailed explanation on these methods, see appendix 2.4. Figure 6-13 shows the failure probability $P(F_i)$ as a function of the dike crest level calculated with Matlab based on the present situation. Matlab is a mathematical programme that is often used for mathematical computations and suited for probability calculations. The influence of the construction of dikes on the water level is not taken into account to, at a later stage, show the difference in water level and optimal dike height with the future situation when polders are built. This influence will be taken into account later. The figure shows that the failure probability decreases as the crest level increases. Figure 6-14 shows the failure probability calculation based on water levels in the three scenarios of the present situation when no polders built, polders built in the area of Rosso and Richard Toll and polders built throughout the entire Delta (between Richard Toll and Diama).

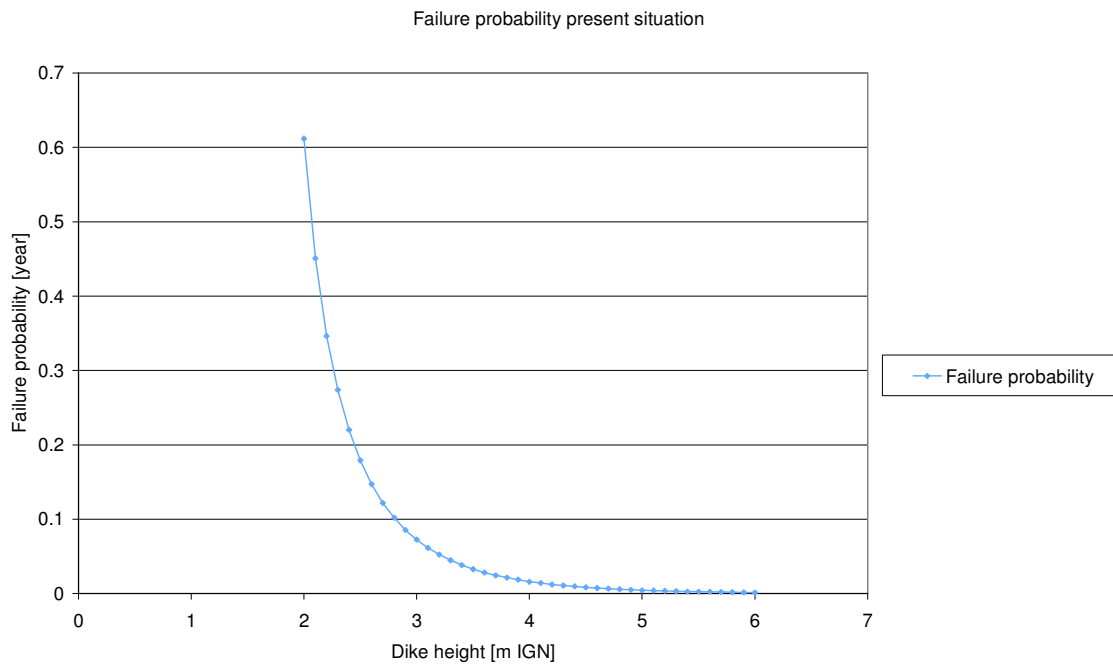


Figure 6-13: The failure probability as function of the dike crest level according to the Monte Carlo method based on the water levels in the present situation. The failure probability decreases as the dike height increases.

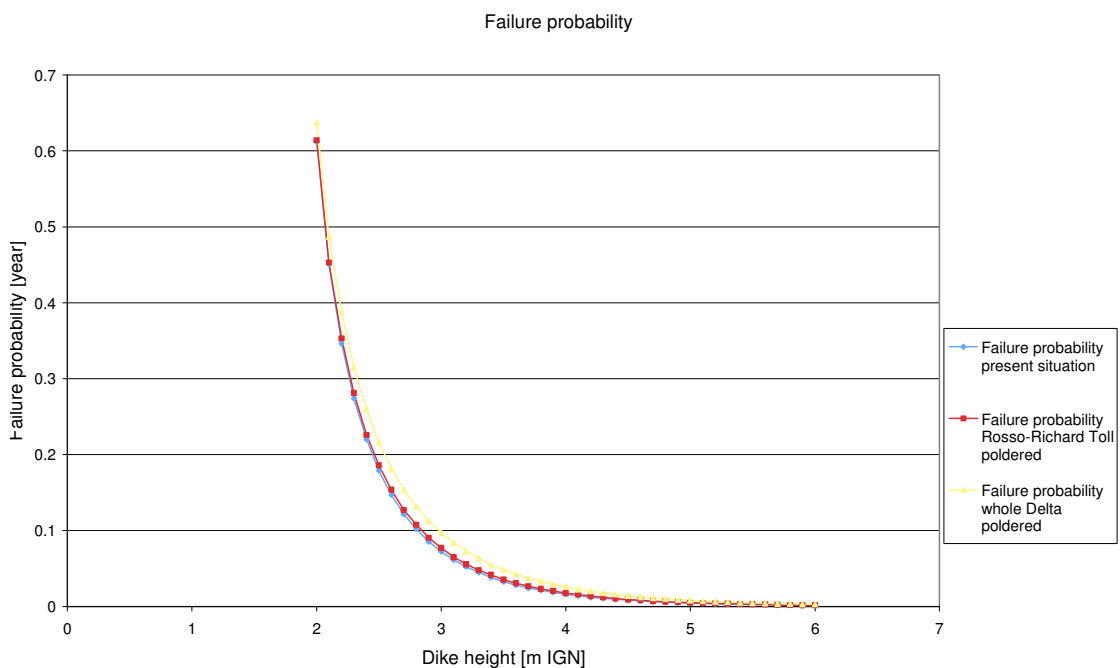


Figure 6-14: Failure probability as function of the dike crest according to the Monte Carlo method. The failure probability decreases as the dike height increases, and it increases as polders are built.

Figure 6-14 shows that the failure probability increases little when polders are only built in the Rosso-Richard Toll area. When polders are built in the whole Delta area, the failure probability increases more importantly. This is related to the water increase in water level: as the water level increases little when polders are built between Rosso and Richard Toll (see Table 6-2), the failure probability will also increase little. The water level increases more when the whole Delta is poldered (see Table 6-2), so the failure probability will also increase more.

7 Defining the consequences of flooding

In this chapter, the amount of loss due to flooding is defined. In order to do so, the yield of various crops is defined and the present value of the loss per crop is determined.

7.1 Working method

Defining the consequences of flooding is a part of defining the economic optimal dike height. The goal is to minimize the following function:

$$C_{tot} = C_{const} + E(S) \quad [7.1]$$

in which C_{const} are the construction costs of a dike and $E(S)$ is the present value of the expected loss due to flooding. This chapter will address the latter and the whole equation will be solved in the next chapter. The equation for $E(S)$ is the following:

$$E(S) = P(F_i) * S \quad [\text{€/year}] \quad [7.2]$$

in which $P(F_i)$ is the probability of failure in year i and S is the amount of (economic) damage or loss. In this case, $E(S)$ is also called the risk [€/year]. If both the actual interest rate r' (rate of return that could be earned on an investment with similar risk) and the intended service life of the dike N are considered, the capitalized loss expectations, or the present value of risk, can be written as:

$$E(S) = \sum_{i=1}^N \frac{P(F_i) * S}{(1 + r')^i} \quad [7.3]$$

If N is large and $P(F_i)$ is constant over time, $E(S)$ can be written as:

$$E(S) = \frac{P(F_i) * S}{r'} \quad [\text{€}] \quad [7.4]$$

(CUR, 1997) $P(F_i)$ was calculated in chapter 6, and in this chapter S is defined. Once the dikes are built, the Typha will first have to be removed and the polder has to be prepared before agriculture can be done; these costs are not taken into consideration in chapter 7 and 8. For the definition of the consequences of flooding, it is assumed that a certain polder is full of one kind of crop for one harvest. A possible second harvest in a year can be a different kind of crop. The calculations here are done for polder 29 which is the

biggest polder; the other polders can also be calculated, with the same calculation procedure. Various scenarios will be calculated, for example if different kinds of crop, like rice or tomatoes are cultivated in the polder.

It will not be possible to use the entire polder surface which is due to the presence of for example irrigation canals, roads or paths. Moreover, the topography of the newly created polders does not allow the polder to be used in its entirety. That is why the usable surface of the polders is reduced to 75% in the calculation. This value could change and is not sure. Also, some loss will occur when harvesting the crop. That is why this yield is reduced to 95% (Van Berkum, 2012). The interest rate r' was assumed at 5% as agriculture is widely applied (Royal Haskoning, 2007). Since the local people know how agriculture is done, harvest failure due to mismanagement is not likely. It is also assumed that once the dike fails, the entire harvest is lost.

7.2 Various scenarios of crop in the polder

As mentioned, polder 29 will be used as the pilot polder. The polder has an area of 1000 hectares and will need 11556 m of dike. The amount of loss due to flooding S is defined as follows: for every crop, the cost of production is calculated by multiplying the production cost in €/kg with the yield in tonnes per hectare. This yield is also multiplied by the selling price of crop in €/kg. Finally the margin between production costs and yield in €/ha is defined; see Table 7-1 for an overview. The information on production cost and selling price in the table was received from a French agronomist who has been living in Africa since the 1980's and who knows the region very well (Cadelan, 2012). In these calculations, the amount of loss due to flooding is defined as the value of the crop which could not be harvested due to flooding of the polder. The value of for example the loss of inlet structures or left machinery in the polder in case of flooding is not taken into account. Moreover, the possibility that people are going to live in the polders is not taken into account. This can be investigated at a later stage.

Table 7-1: Production costs, yield and margin for various crops. The margin for tomatoes is double of that of rice.

	Rice	Tomatoes
Production cost [€/kg]	0.13	0.05
Yield [tonnes/ha]	5	20
Selling price [€/kg]	0.19	0.08
Total cost [€/ha]	650	1000
Yield [€/ha]	950	1600
Margin [€/ha]	300	600

The amount of loss due to flooding is then calculated as follows:

$$S = Y * X * 0.75 * 0.95 \quad [7.5]$$

In which Y is the yield in €/ha for a particular crop and X is the area of the polder (=1000ha in this case).

Table 7-2: Amount lost per crop in case of flooding. The amount of loss is higher for tomatoes.

	Rice	Tomatoes
S [€] (for 1000 ha)	214.000	427.500

In this case, the uncertainty in expected damage (loss due to flooding) is not taken into account. For completeness, this can however be included in the calculations (Slijkhuis et al., 1997). Now that S is known, $E(S)$ can be calculated. Along the Senegal River the crops are harvested once or twice per year. Various scenarios are considered; a once and twice per year harvest for rice and a once per year harvest for tomatoes. The second harvest of rice has a cropping intensity of 80%, so the harvest is not doubled, but multiplied by 1.8. Two harvests of tomatoes are not possible, however, a once per year rice harvest in combination with a tomato harvest is realistic. The tomato harvest in combination with rice has an intensity of only 60%. These values are according to (Cadelan, 2012).

$E(S) = \frac{P_f * S}{r}$ and P_f is dependent on the dike height. Table 7-3 presents the dike height given in steps of 0.5 meter for the present situation. A table like Table 7-3 can be made for all three scenarios of poldering. Figure 7-1 gives an overview of $E(S)$ in these scenarios.

Table 7-3: The present value of the loss as a function of the dike height. The present value of the loss decreases as the dike height increases.

Dike height [m IGN]	Failure probability [year]	Present value of loss for rice ($E(S)$) [€]	Present value of loss for tomatoes ($E(S)$) [€]
2	0.61	2,615,287	5,230,574
2.5	0.18	765,977	1,531,955
3	0.07	309,831	619,661
3.5	0.03	139,985	279,970
4	0.02	67,960	135,919
4.5	0.008	35,448	70,897
5	0.004	18,630	37,261
5.5	0.002	10,504	21,007
6	0.001	5,677	11,354

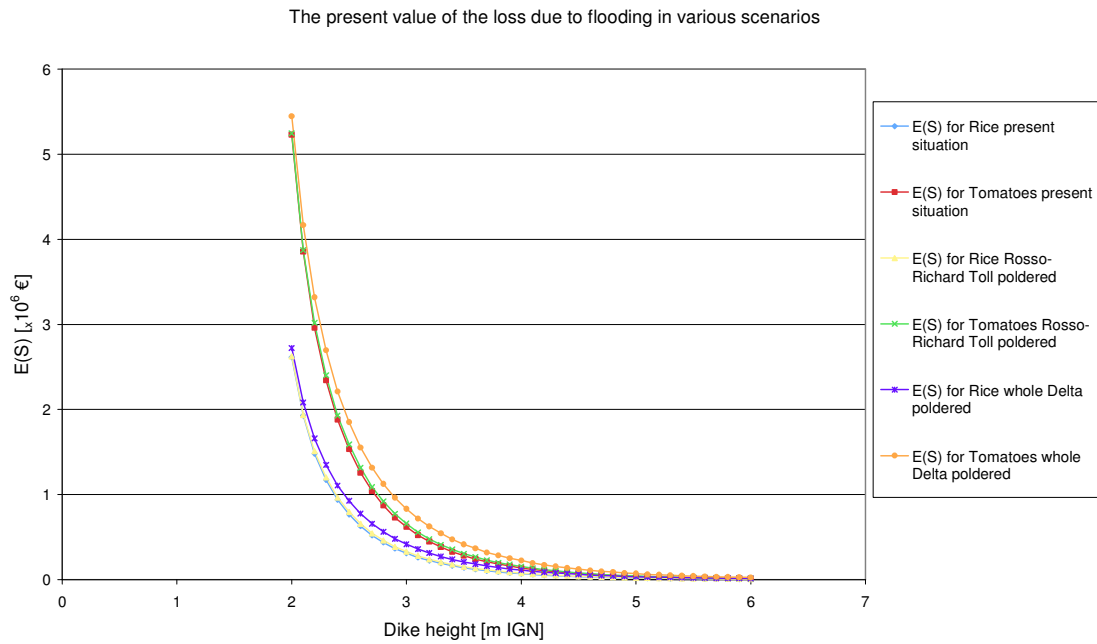


Figure 7-1: Calculation of $E(S)$. The present value of the loss is higher for tomatoes than for rice. Also, $E(S)$ is higher when polders are built.

Considering Figure 7-1, the value for $E(S)$ does not increase that much when only the Rosso-Richard Toll area is poldered. However, when the whole Delta area is poldered, the value of $E(S)$ increases more importantly (see also Figure 6-14). The value of $E(S)$ is different for rice and tomatoes. This is because the S value for the two crops is different; that for tomatoes is higher than that of rice. It will be seen in chapter 8 that a higher value for $E(S)$ will result in a higher dike crest level.

8 *Economic optimal dike design (economic risk-based design)*

In this chapter, the equations for the total costs of the construction of the dike are further elaborated. First, the construction costs of 1m dike with a certain height are defined. Then the total costs for the dike are calculated and an optimum is found. The optimum is situated where the total costs are minimum, see Figure 3-1.

8.1 The construction costs of the dike

The construction costs of a dike are calculated using the following formula:

$$C_{const} = L_d * h_0 * (h_0 * \cot \alpha + b_k) * f_d \quad [8.1]$$

(Van Gelder et al. 2004) in which L_d is the length of the dike (for polder 29, this is 11556 m), h_0 is the crest height (meters above IGN) which is a design parameter. $\cot \alpha$ is the cotangent of the slope of the inner and outer taluses, also a design parameter. b_k is the crest width and f_b is the construction cost per unit volume (defined hereafter).

In 1999, the OMVS has investigated the possibility to extend the dikes in place upstream of Rosso towards Dagana. The value of the various variables in the calculation of the construction costs of the dike are based on (SOGED, 2005). The total cost for 63.8 km of dike was 10.507.168 euro so that the price of one meter dike is 164.69 €/meter dike. The total volume of one meter dike is 31.5 m^3 so that the costs per unit volume is $5.23\text{€} / \text{m}^3$ (f_d). The dikes in the study have a crest width of 3 meters (like the ones in place now), an inner slope of 1:2 and outer slopes of 1:3. In Figure 8-1, the construction cost is defined per dike height and slope angle.

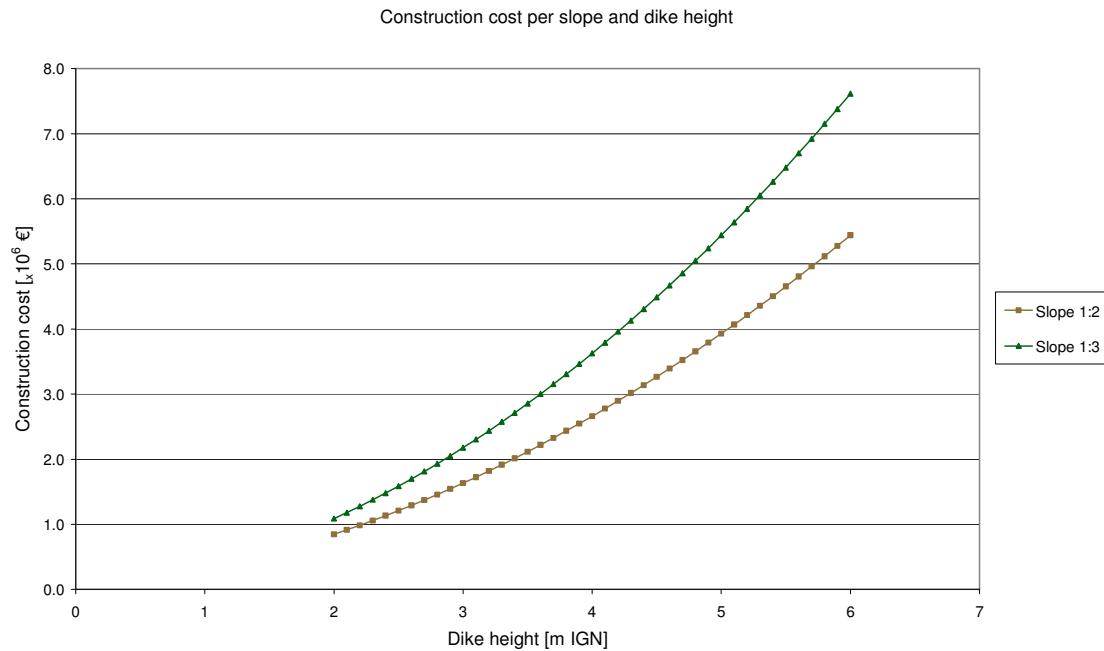


Figure 8-1: Construction cost in function of the dike height and slope. The construction costs go up as the dike height increase and when the slope of the talus becomes gentler.

8.2 The economic optimal dike height

The goal is to find the optimal dike height, using the following formula:

$$C_{tot} = C_{const} + E(S) \quad [8.2]$$

(Van Gelder et al. 2004). The optimal dike height is situated, as said, at the point where C_{tot} is minimal. In Figure 8-2, the optimal dike height is defined for several cases of slope and harvest frequency for rice. The optimal dike height increases as the harvest frequency is higher (as the amount of loss due to flooding will be higher for higher harvest frequency) and the optimal dike height decreases with increasing construction costs (as the costs increase compared to amount of loss due to flooding).

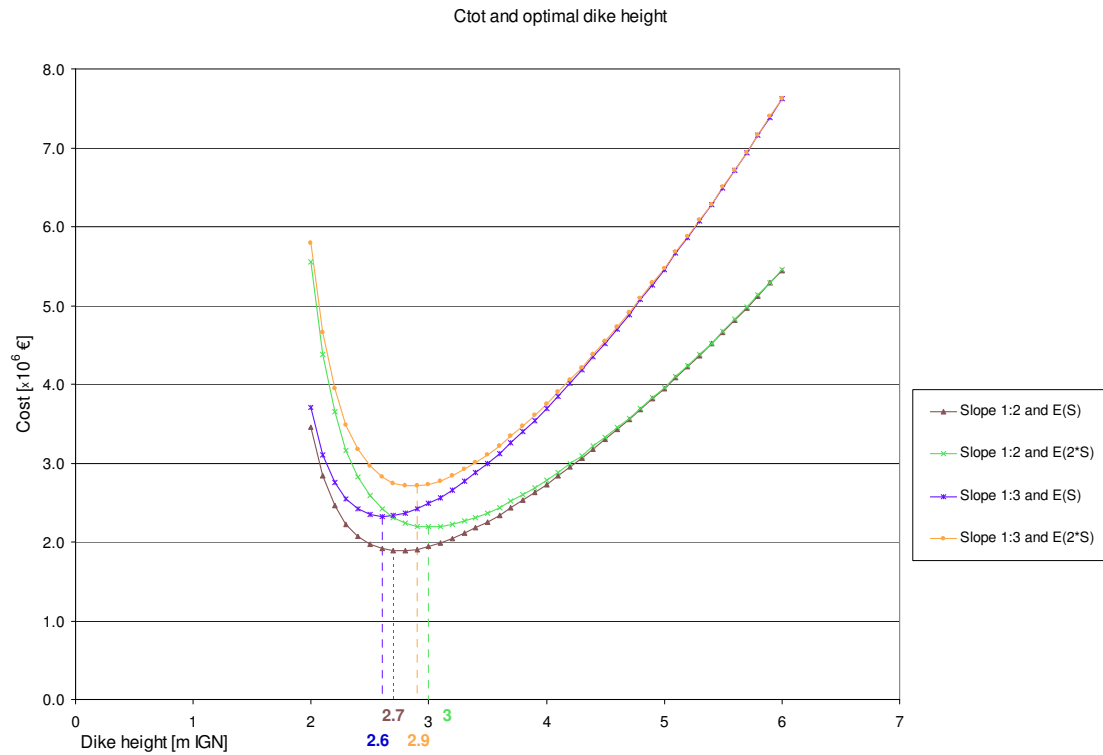


Figure 8-2: The total cost of the dike resulting in the optimal dike height (based on rice harvest). The economic optimal dike height increases as the amount of loss due to flooding increases and it decreases as the dike slope becomes gentler.

An overview of the expected loss due to flooding, the construction cost and the total cost is given in Figure 8-3.

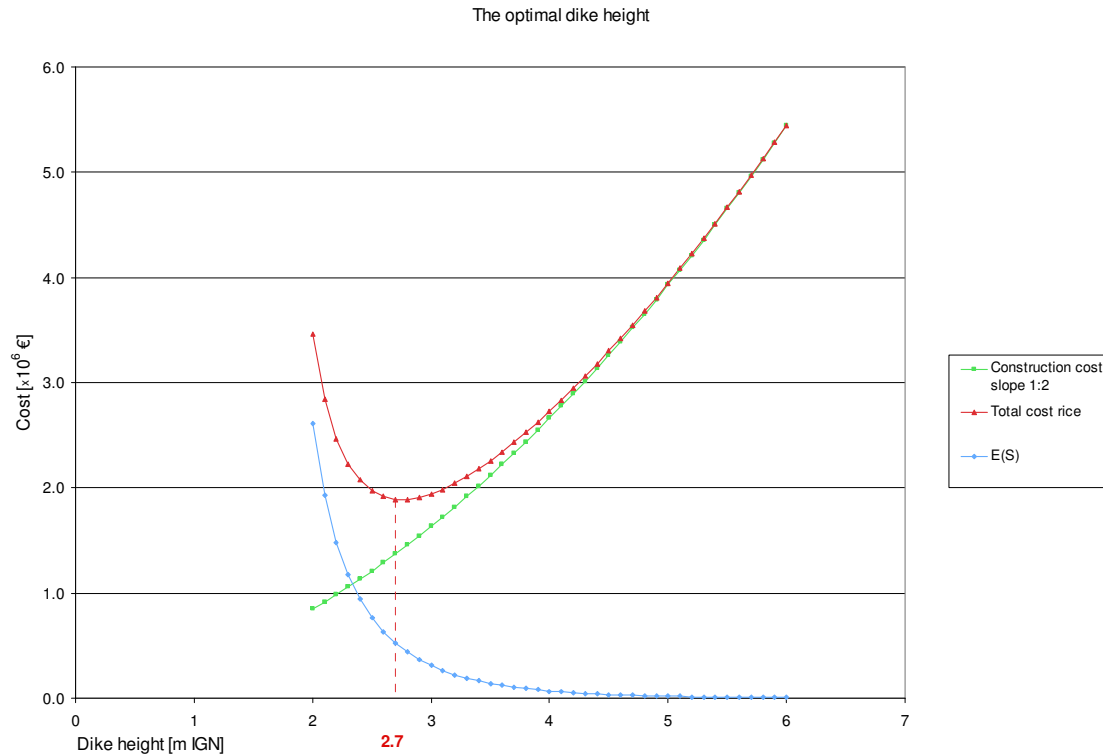


Figure 8-3: An overview of the failure probability, the construction cost and the total cost. The economic optimal dike height is 2.7 m IGN in this case.

The optimal dike height is defined for the three scenarios in case of one or two harvests of rice per year, for one harvest of tomatoes per year and for a combination of rice and tomatoes. For a slope of 1:2, the results are presented in Table 8-1, for a slope of 1:3; the results are presented in Table 8-2.

Table 8-1: The optimal dike height for dikes with a slope of 1:2. The dike height increases as the amount of loss due to flooding increases.

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} [m IGN]	2.7	3	3.1	3.1	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2

Table 8-2: The optimal dike height for dikes with a slope of 1:3. The dike height increases as the amount of loss due to flooding increases.

	Present situation			Rosso- Richard Toll poldered				Whole Delta poldered				
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} [m IGN]	2.6	2.9	2.9	2.9	2.6	2.9	2.9	3	2.7	2.9	3	3

Looking at these results, various conclusions can be drawn. The optimal dike heights in Table 8-1 are higher than those in Table 8-2; which is because the construction cost for a gentler slope are higher. Also, the optimal dike heights are higher for multiple harvests per year for rice, as “there is more to protect”. The difference in amount of loss due to flooding appears to be close to each other for two harvests of rice per year and one harvest of tomatoes (as the optimal dike height is the same) and the difference in amount of loss due to flooding for tomatoes and the combination of tomatoes and rice is small as well. The influence of poldering on the economic optimal dike height appears to be 10 cm at the most. This is due to the fact that the flooding frequency is relatively high. As the difference in water level is not big, the effect on the optimal dike height will be small as well.

For a slope of 1:2 and in the case of rice being harvested once per year, the optimal dike height is a 2.7 m IGN, and 3.1 m IGN for a slope of 1:2 in case of tomatoes being harvested once per year based on the present situation. These values for the dike height correspond to a failure probability of 12.1% and 6.1% respectively and consequently to a flooding frequency of 1/8.2 year and 1/16.3 year as can also be deducted from Figure 8-4.

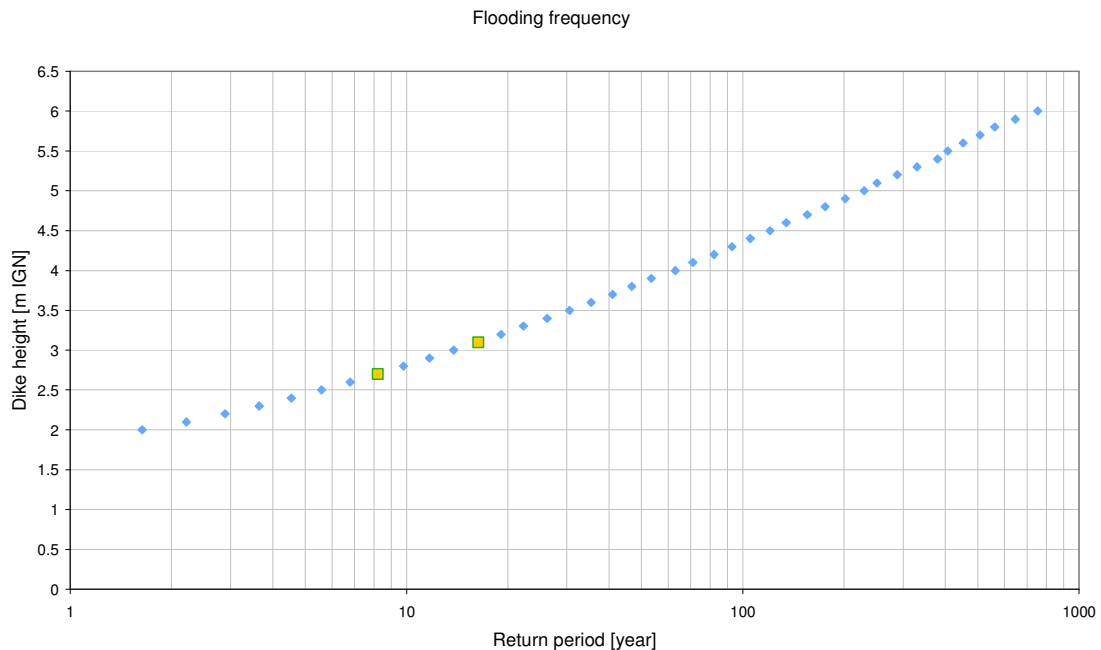


Figure 8-4: Flooding frequency in function of the dike height. The failure probabilities for the frequency calculation are based on water levels in the present situation. The flooding frequency decreases as the dike height increases.

A figure like the one above can be made for every scenario (every scenario has its own flooding frequency figure as for every scenario the failure probabilities are different) and like that the flooding frequency can be determined for every optimal dike height.

The flooding frequencies (of 1/8.2 per year and of 1/16.3 per year) are relatively high (compared to the Netherlands for example where the river dikes are built at a safety level of 1/1250 per year (De Haan et al., 2001)). In the Netherlands however, the value of the goods behind the dikes (like people activity, factories, homes, etc...) is much higher than in this case.

Length effect and system functioning

The length effect is defined as the increase of the probability of failure with the increasing length of the dike (Vrijling et al., 2011). The relative contribution of load and resistance and the special variety in the subsoil are two factors that determine the magnitude of this effect. The failure probability of a dike section is the minimum failure probability of the entire dike ring, which refers to full correlation of all sections and consequently all sections should fail simultaneously (as the length effect is not taken into account). A dike can be considered as a chain of dike sections where the weakest section is governing for the failure probability. In the calculations made here, the length effect was not taken into account. To incorporate the length effect in the failure probability, the dike ring should be considered as a series system of independent elements. In general, taking into account the length effect will consequently increase the failure probability. For load-dominated failure mechanisms however, like overflow as is the case here, the probability of failure

of a dike ring is close to that of a cross section (so no section of a certain length, but a cross section with no length). This is due to the fact that the hydraulic load will probably be more or less the same for adjacent sections (Mai et al., 2006). The choice of the length of the dike section is of course of importance. Normally, the dike section is about 1-2km of length. In the case presented here, the hydraulic load in the Rosso area will be more or less the same; however, at Diama (approximately 100 km downstream) the hydraulic load will be different. To be able to incorporate the length effect for resistance-dominated failure mechanisms (which are not investigated here) in a correct manner, the subsoil should be investigated, which is not yet done at the moment.

System functioning is the increase or decrease of the failure probability of a certain dike ring as another dike ring fails elsewhere (Immink, 2007). In the case polders are only built in the Rosso-Richard Toll area, this effect will be limited as the amount of polders built is also limited and the size of the polders is relatively small. The system of dikes is limited in size so to say. For now, the dikes neighbouring the river side have been considered. The dikes neighbouring the irrigation canals between the polders are designed at the same crest level as the river dikes, but the focus does not lie not on these dikes in this study. The effect of system functioning will be more important when polders are built in the whole Delta. Especially in the lower Delta, the influence will be more important as the surface of the polders over there is larger.

9 Sensitivity analysis to the optimal dike height

In this chapter, a sensitivity analysis is done on the water levels and their corresponding optimal dike height for various cases of model sensitivity, discharge uncertainty and dam malfunctioning in combination with a flood wave coming through the system. The same steps as described in chapter 6, 7 and 8 are followed to get to the water levels, failure probabilities, values for $E(S)$, construction costs and total costs.

9.1 Types of sensitivity

The sensitivity of the water level and the optimal dike height to various aspects is investigated. The following three types of aspects are discerned:

- A model aspect: the dam is put into the hydrodynamic model differently from the way determined by the OMVS (see Figure 5-3). The consequences are investigated.
- A discharge aspect: the exponential distribution with its parameters is chosen for the extreme values of the discharge, as presented in Figure 6-6. The extreme value analysis for the discharge is based on a relatively small dataset of discharges which can lead to an error sensitive parameter definition. For this reason the parameter uncertainty of the exponential distribution is investigated. This leads to different values for the discharge.
- A dam aspect: various situations are modelled for a Diama dam not functioning as it should, for example if one or multiple gates of the Diama dam break down, or the gates get stuck.

To all the newly simulated water levels, the exponential distribution has been fitted and the probability of failure was calculated.

9.2 The analysis

9.2.1 Model aspect

As mentioned in paragraph 5.1.2.1, the model doesn't follow the relation as depicted in Figure 5-3. The model relation (relation 2) is steeper compared to relation 1 (see Figure 9-1). Relation 1 is however calculated with a formula which was based on "only" 6 measurements. So the correctness of this relation can be questioned as well. Three different dam situations are investigated: one situation like used before (discharge of $2084 \text{ m}^3/\text{s}$ gives a water level of 1.73 m IGN) (relation 2), a second situation in which a discharge of $1973 \text{ m}^3/\text{s}$ giving a water level in front of the dam of 1.62 m IGN (relation 3) and one situation in which the relation is based on a discharge of $1860 \text{ m}^3/\text{s}$ giving a water level just in front of the dam of 1.51 m IGN (relation 4).

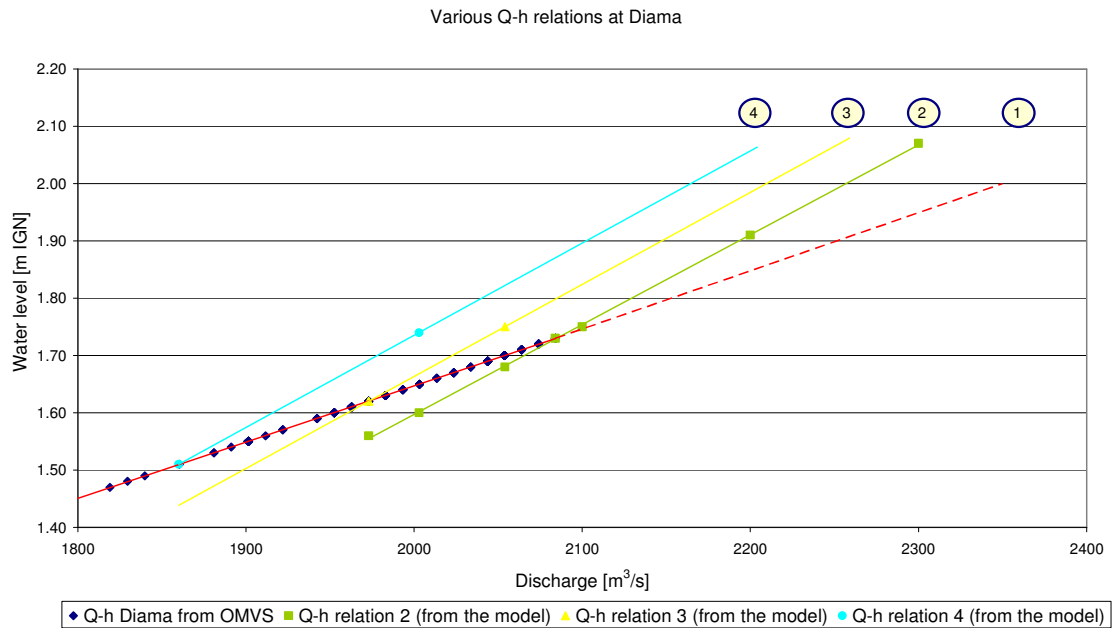


Figure 9-1: Various Q-h relations at Diama. Relation 2 will be used for further simulations as it fits best for high discharges.

The dam adjustment has an influence on the water level in the river. The water level was simulated in these three situations at Rosso, presented in Figure 9-2. The water level increase between relation 2 and 4 at a discharge with a return period of 500 years, is 30 cm, which is over 5%. For lower discharges, the difference is less. Relation 2 is preferred and used as this one gives better results for high discharges (the water level at a discharge of $2084 \text{ m}^3/s$ is 1.73m IGN and not 1.86m IGN) and even this relation seems to overestimate the water level for high discharges (discharges higher than $2084 \text{ m}^3/s$).

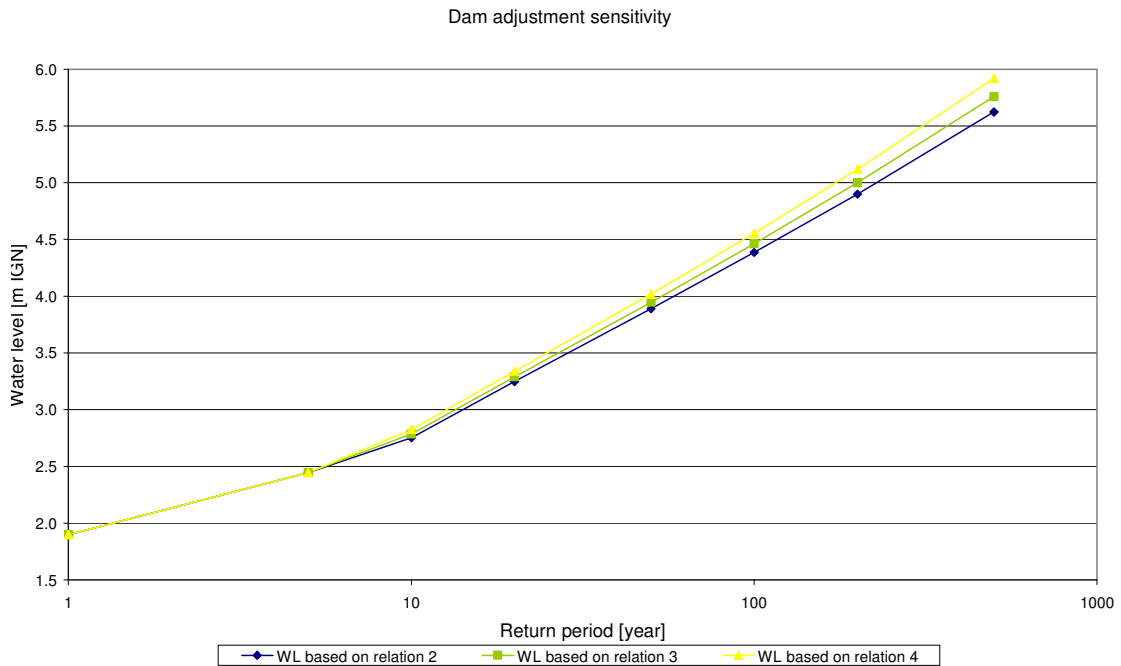


Figure 9-2: Water levels simulated in case of different dam adjustment. Relation 4 gives the highest water levels; relation 2 however is the best suited relation for high discharges.

The result of the model aspect on the water level is now known and the effect on the optimal dike height will be investigated. To get to a conclusion for this dike height, the total cost of the dike is calculated.

Table 9-1: Influence of the dam adjustment on the optimal dike height. The maximum influence is 10 cm. The results are shown for a dike with a 1:2 slope.

	Present situation			
	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)
h_{opt} dam at relation 2 [m IGN]	2.7	3	3.1	3.1
h_{opt} dam at relation 3 [m IGN]	2.8	3	3.1	3.1
h_{opt} dam at relation 4 [m IGN]	2.8	3	3.1	3.1

Table 9-2: Influence of the dam adjustment on the optimal dike height. The maximum influence is 10 cm. The results are shown for a dike with a 1:3 slope.

	Present situation			
	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)
h_{opt} dam at relation 2 [m IGN]	2.6	2.9	2.9	2.9
h_{opt} dam at relation 3 [m IGN]	2.6	2.9	2.9	3
h_{opt} dam at relation 4 [m IGN]	2.6	2.9	2.9	3

From Table 9-1 and Table 9-2 can be concluded that the dam adjustment does not have major influence on the optimal dike height (maximum 10 cm). This is also expected as the influence of the dam adjustment on the water level in the river is also very limited (Figure 9-2). Moreover, as seen in paragraph 8.2, the optimal dike height has a flooding frequency under the 1/10 per year (for rice), so if the influence on the 1/10 year water level is negligible, this will also be the case for the dike height.

9.2.2 Discharge aspect

Parameter uncertainty occurs when the parameters of a distribution are determined with a limited number of data (Van Gelder, 2000). In the extreme value analysis described in paragraph 6.3.2.2 and shown in Figure 6-6, the dataset of discharges has got a limited amount of data points and therefore the uncertainty in the parameters is investigated. Furthermore, in the area between Dagana and Dama, losses of water due to evaporation, irrigation and distributaries occur. These losses change a lot over time as the water supply for irrigation and the distributaries is regulated by inlet structures. It is not known when these inlet structures are opened exactly and that is why these losses are incorporated into the parameter uncertainty. As mentioned earlier, the exponential distribution is used. This distribution has two parameters, a shift parameter ξ and a scale parameter α . The cumulative distribution function of the exponential distribution is given by (Van Gelder, 2000):

$$F(x) = 1 - e^{-\frac{x-\xi}{\alpha}} \quad x \geq \xi \quad [9.1]$$

The influence of statistical uncertainty in the shift parameter can be considered by replacing the original cumulative distribution function of the exponential distribution by that original function multiplied by a factor. This is done by writing the shift parameter ξ as $\xi + \epsilon$ in which ϵ is normally distributed $N(0, \sigma_\xi)$. The function then looks like this (Van Gelder, 2000):

$$F(x) = 1 - e^{-\frac{x - \xi + \frac{\sigma^2 \xi}{2\alpha}}{\alpha}} \quad [9.2]$$

The same could be done for the scale parameter. However, the multiplication factor is then different. Finally, the cumulative distribution function for the scale parameter uncertainty looks like this:

$$F(x) = 1 - e^{-\left(\frac{x - \xi}{\alpha}\right)} * e^{-\frac{\sigma^2 \alpha^2 x^2}{2\alpha^4}} \quad [9.3]$$

For more detailed information on the exact calculation method, see (Van Gelder, 2000).

This uncertainty in parameters was investigated for the shift parameter as well as for the scale parameter. Equation 1 (shift parameter) was applied for the parameters $\xi = 1040.7$ and $\alpha = 412.31$ and an uncertainty in ξ of $\sigma_\xi = 10\%$, $\sigma_\xi = 20\%$, $\sigma_\xi = 30\%$, $\sigma_\xi = 40\%$ and $\sigma_\xi = 50\%$ was distinguished. The result is presented in Figure 9-3.

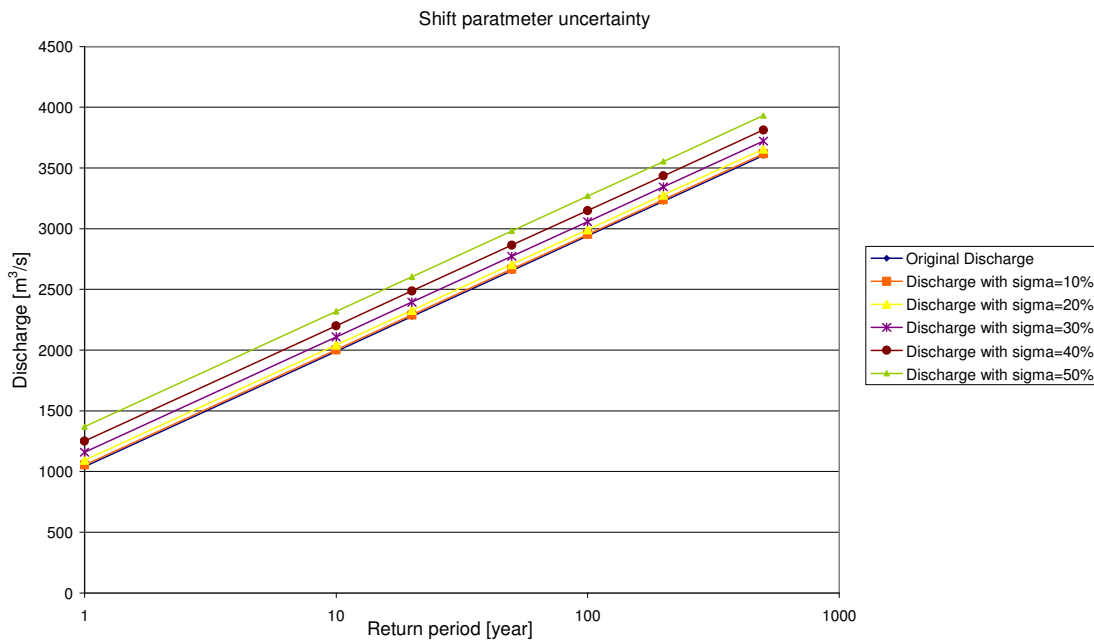


Figure 9-3: Shift parameter uncertainty for the Exponential distribution. The values of the random variable discharge increase as the standard deviation of the parameter distribution (σ_ξ) increases.

Equation 2 (scale parameter) was applied for the parameters $\xi = 1040.7$ and $\alpha = 412.31$ and an uncertainty in α of $\sigma_\alpha = 5\%$, $\sigma_\alpha = 10\%$, $\sigma_\alpha = 15\%$ and $\sigma_\alpha = 20\%$ was distinguished. The result is presented in Figure 9-4.

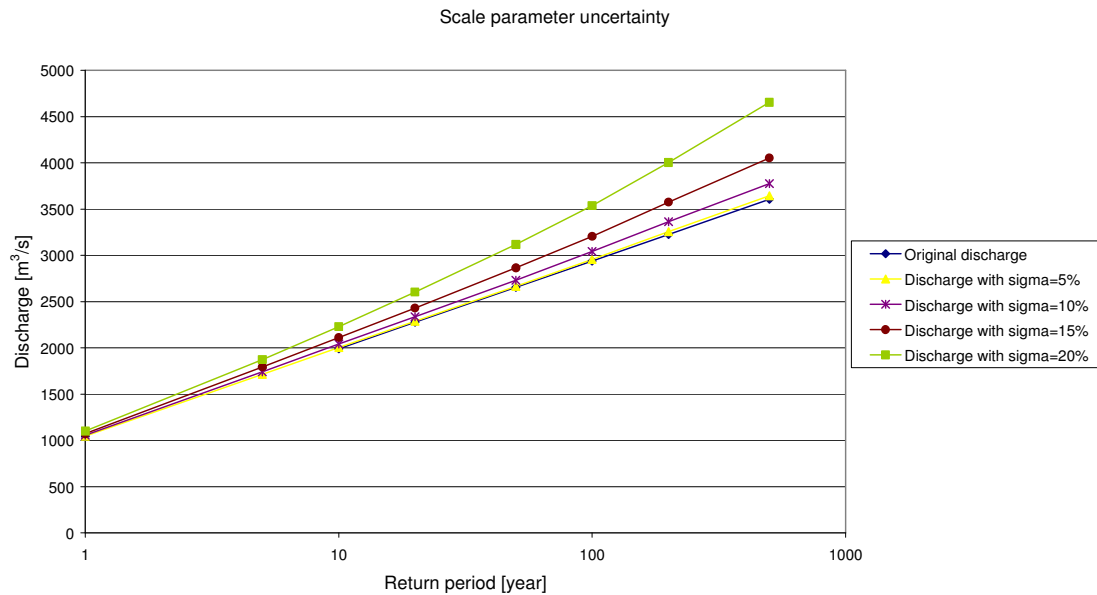


Figure 9-4: Scale parameter uncertainty. The increase in discharge is higher for discharges with lower return periods.

As the number of data points is limited in this case, a relatively big standard deviation for the uncertainty level in the scale parameter is taken. That is why the discharges based on the $\sigma_\alpha = 20\%$ line have been used in further analysis. From Figure 9-4 can be concluded that the difference in discharge is bigger for discharges with a lower return period. Taking the scale parameter uncertainty into account not only results in a shift, but also a slope increase. The latter is due to the fact that the multiplication factor for the scale parameter contains σ_α and x .

For a particular return period, the discharge value increases with this shift and slope increase. However, if a certain discharge value is considered, the return period of this discharge value will increase. Considering the original discharge value for the once in 100 year return period (the blue line in Figure 9-4 with a value of about $3000 \text{ m}^3/\text{s}$), now becomes discharge value with a once in 40 year return period. This is a considerable change. For discharges with a low return period (over the once in the 10 years return period), this parameter uncertainty causes a big increase in discharge. For discharges with a high return period, this difference is less important.

These new values for the discharge have been transformed into flood waves, the same way this was done in paragraph 6.3.2.2, and water levels were simulated. The result is presented in Figure 9-5. The increase in water level between the blue and the green line in Figure 9-5 depends on (the corresponding discharges of) the choice of σ_α . With a smaller value for σ_α , the increase in water level is smaller. For $\sigma_\alpha = 20\%$, the water level increase for the highest discharge is 2.28 m, which is about 40%. This is however much lower for for example the 1/10 year discharge; here the water level difference is only 0.41 m, which corresponds to a 14% increase approximately. For discharges with a higher frequency, this is even lower.

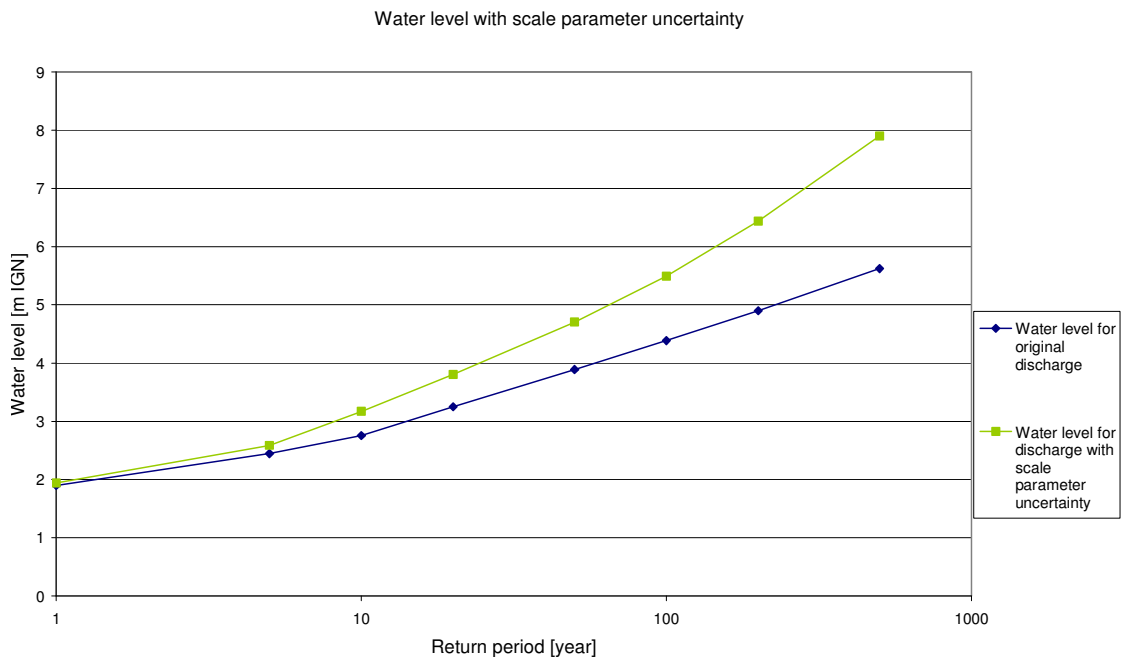


Figure 9-5: The simulated water level increases as the parameter uncertainty is taken into account.

The effect of the parameter uncertainty on the optimal dike height for a 1:2 sloped dike is presented in Table 9-3.

Table 9-3: The optimal dike height with parameter uncertainty for the discharge for dike with a slope of 1:2. The economic optimal dike height increases (by max. 20 cm) when taking into account the discharge parameter uncertainty.

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt}	2.7	3	3.1	3.1	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2
Original water levels [m IGN]												
h_{opt} with par. uncertainty [m IGN]	2.8	3.1	3.2	3.3	2.8	3.1	3.2	3.3	2.9	3.2	3.3	3.4

From Table 9-3 can be concluded that the economic optimal dike height is higher as the parameter uncertainty is taken into account. This is logical as a higher discharge results in a higher water level. The increase in optimal dike height is 20 cm when the amount of loss due to flooding is high (the comb. situation).

The aforementioned aspects are influential on the water level independently of the dam functioning. Actually, the new discharge series (green line in Figure 9-4) should be used to determine the optimal dike height. A sensitivity analysis can be done on the combination of new discharge series and dam adjustment in combination with dam malfunctioning. For the dam malfunctioning however, the parameter uncertainty was not taken into account and it was assumed that the dam was working following relation 2. The influence of the dam adjustment and the parameter uncertainty compared to the original situation has been investigated.

9.2.3 Dam aspects

The Diama dam malfunctioning is considered here. First a list of scenarios which could lead to faster water level increase than when the dam works normally is given. Thereafter various dam aspects are investigated.

Situations that could have an influence on the water level in the Senegal River Delta:

- One or multiple (or all) gates can't be used and stay closed
- Various gates of the Diama dam get stuck at a certain opening height
- The dam starts lowering the water level much later than it should
- The collapse of the Diama dam, or the collapse of the Manantali dam
- The collapse of a river dike
- Change in the river, for example the construction of an extra dam, a bypass in the river or the realisation of an extra lake
- The increase of navigation and using big boats

The first three scenarios are analysed hereafter. It is also investigated whether the assumption on the length of the flood wave (so not the maximum value but the duration) is of influence on the water level.

Gates out of order

It has been investigated what the effect of one or two gates breaking down is on the water level. By breaking down is meant that the gate stays fully closed at the moment it should open up. The result of the simulations for the water level is presented in Figure 9-6.

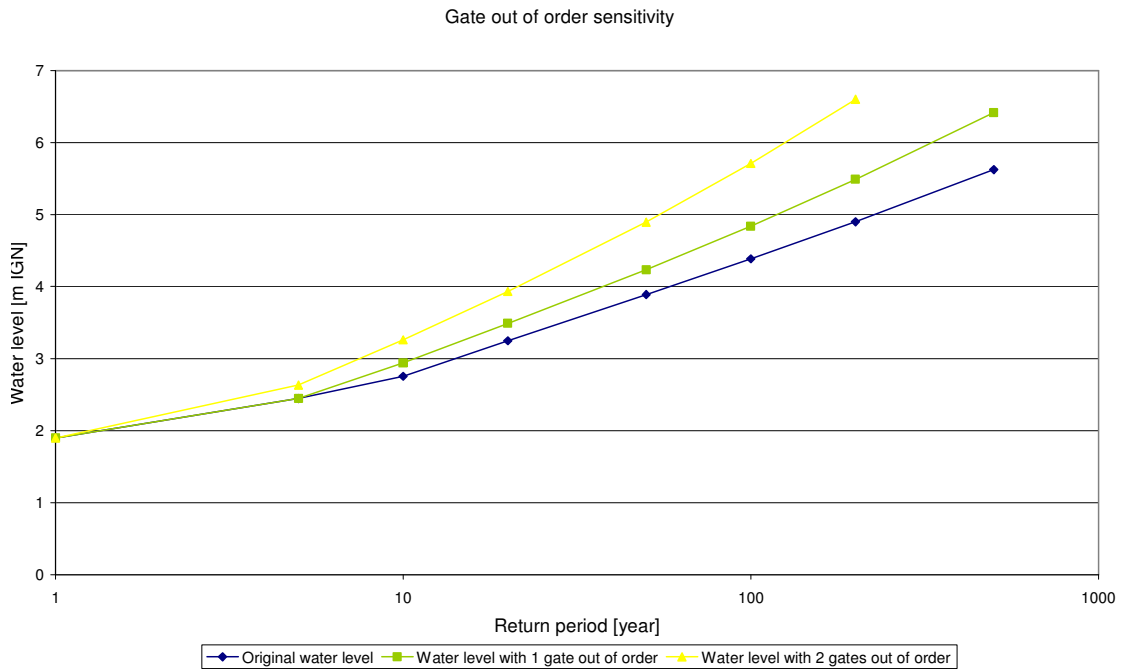


Figure 9-6: The water level increases as one or two gates break down. The water level increases more when the amount of gates out of order increases.

Here, the water level difference at the 1/500 year discharge when one gate is out of order is about 80 cm, which corresponds to a water level increase of about 14%. It can be concluded that the influence on the water level for discharges with a higher return period is higher. The increase in value of the once in 100 year water level is bigger than the increase for the once in 10 year water level. The effect of gates being out of order on the optimal dike height for a dike with a slope of 1:2 is presented in Table 9-4.

Table 9-4: The influence of gates out of order on the optimal dike height. The economic optimal dike height increases as gates of the Dama dam are out of order (by max. 30cm).

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} Original water levels [m IGN]	2.7	3	3.1	3.1	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2
h_{opt} one gate out or order [m IGN]	2.8	3.1	3.1	3.2	2.8	3.1	3.2	3.2	2.8	3.2	3.2	3.3
h_{opt} two gates out or order [m IGN]	2.8	3.2	3.3	3.3	2.8	3.2	3.2	3.4	2.9	3.3	3.4	3.4

From Table 9-4 can be concluded that the breaking down of one gate has an influence of 10 cm at the most. In some scenarios, there is no influence. The breaking down of two gates has an influence on the optimal dike height reaching from 10 cm to 30 cm.

Gates get stuck at opening height

As mentioned earlier, the gates of the Dama dam can be opened or closed. It is investigated what the influence would be on the water level in the river as the gates get stuck at a certain opening height. The maximum opening of 11.25 m is reduced to 10.25 m, 9.25 m and to 8.3 m. The result is presented in Figure 9-7.

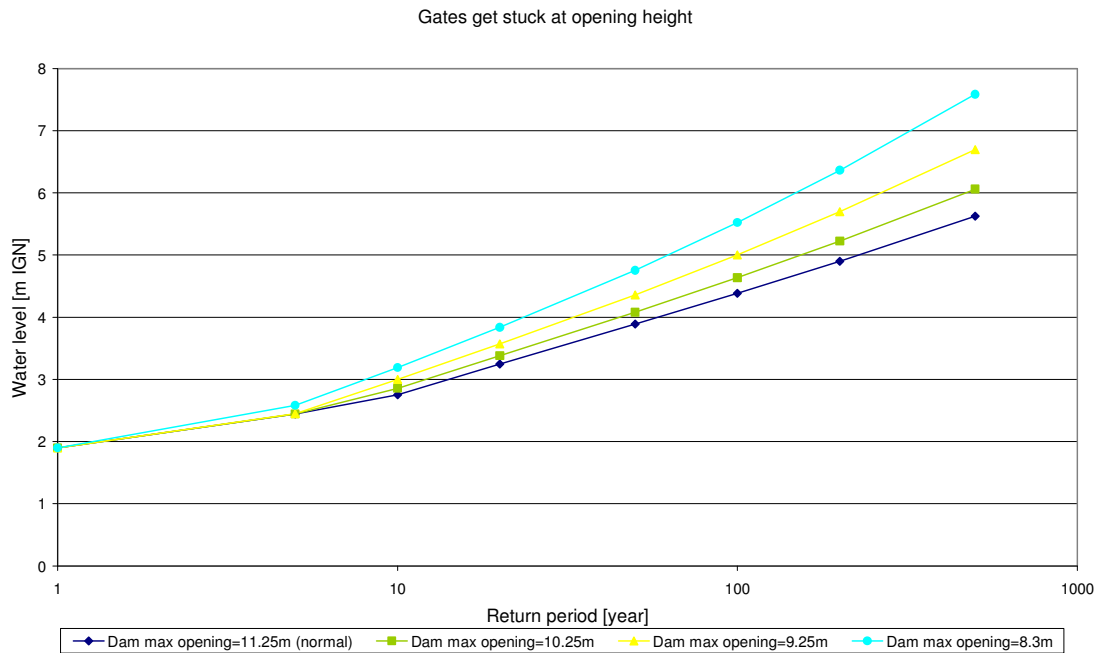


Figure 9-7: As the maximum opening on the dam gates decreases, the water level in the river increases.

From Figure 9-7 can be concluded that the influence of a gate getting stuck at a certain opening is of importance for the water level. For the highest discharge, the increase in water level between the original water level and the water level when the dam is stuck at 8.3 m is 35%. The difference in water level appears to increase as the return period increases. Differently said, the slope of the line of simulated water levels at various maximum openings is steeper than the one for the original water levels.

It also appears that the increase in water level between 11.25 m opening and 10.25 m opening is smaller than the increase in water level between 9.25 m opening and 8.3 m opening. This means that at a smaller maximum opening, the water level increases more. An overview of the effect of the gates getting stuck on the optimal dike height is presented in Table 9-5.

Table 9-5: The influence of gates getting stuck on the optimal dike height. The economic optimal dike height increases as the maximum opening height of the gates decreases (by max. 20cm).

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} Original water levels [m IGN]	2.7	3	3.1	3.1	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2
h_{opt} dam max opening= 10.25 m	2.8	3	3.1	3.1	2.8	3.1	3.1	3.1	2.8	3.1	3.2	3.2
h_{opt} dam max opening= 9.25 m	2.8	3.1	3.2	3.2	2.8	3.1	3.2	3.2	2.9	3.2	3.2	3.3
h_{opt} dam max opening= 8.3 m	2.8	3.2	3.2	3.3	2.8	3.2	3.3	3.3	2.9	3.3	3.3	3.4

From Table 9-5 can be concluded that the influence of gates getting stuck at 10.25 m is 10 cm at the most. Most influential is when the maximum opening is reduced to 8.3m; then the optimal dike height increases with 20 cm. The tendency of the water level increase is to go up as the gates get stuck at smaller opening heights (smaller than 8.3 m). The exact effect on the water level and the optimal dike height is to be investigated.

Timing of lowering the water level

An analysis was executed on the timing of lowering the water level at Dama. It can happen that the management of the dam only starts lowering the water level in front of the Dama dam 10, 20 or 30 days after it should do so. The reason for this is for instance that the person managing the dam is not available, or that some element of the dam is out of order and only repaired after a certain number of days so that the water level can only be lowered after a number of days. The influence of this late lowering is investigated here. The goal is to regulate the water level in front of the dam to 1.5 m IGN as quickly as possible. In Figure 9-8, the influence on the water level in the river is presented.

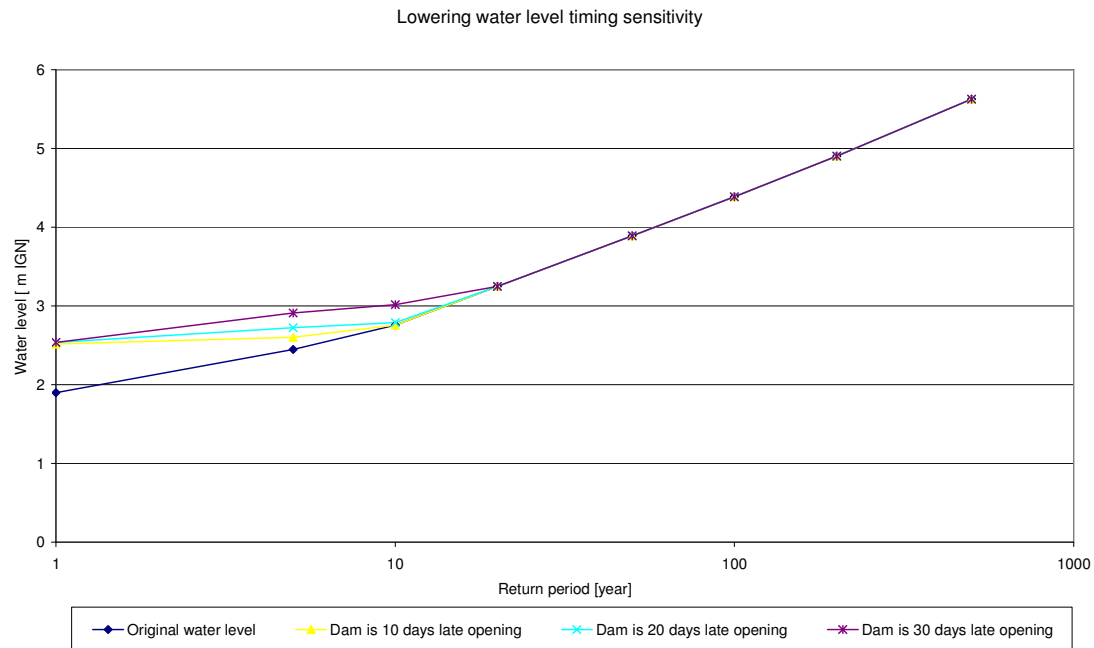


Figure 9-8: For lower discharges, the timing of the lowering of the water level is important.

Figure 9-8 shows that the timing of starting to lower the water level is only important for discharges with a relatively high return period (above a return period of 1/20 year). This is because as the water level is kept at 2.25 m IGN and the flood wave arrives, the water level will start to increase. Then, as the dam is opened up, the water level is lowered so there is more water going out at the dam than there is coming in from upstream, but there is still water coming downstream. This will cause the water level to increase again. In case the delay of lowering the water level is long enough, the maximum water level in the river will have occurred, before the maximum water level due to the flood wave, has presented itself. For higher discharges, the maximum water level will in all cases be caused by the flood wave, due to the fact that the increase in water level because of late opening is smaller than the maximum water level caused by the flood wave. The effect of too late lowering of the water level in front of the dam on the optimal dike height is presented in Table 9-6.

Table 9-6: The influence of late lowering of the water level in front of the dam on the optimal dike height. The optimal dike height increases by max 40 cm as the water level in front of the dam is lowered too late.

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} Original water levels [m IGN]	2.7	3	3.1	3.1	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2
h_{opt} 10 days late [m IGN]	2.9	3.1	3.2	3.2	2.9	3.2	3.2	3.2	3	3.2	3.2	3.3
h_{opt} 20 days late [m IGN]	3	3.2	3.2	3.3	3	3.2	3.3	3.3	3.1	3.3	3.4	3.4
h_{opt} 30 days late [m IGN]	3.1	3.3	3.3	3.3	3.1	3.3	3.3	3.4	3.2	3.4	3.4	3.5

From Table 9-6 can be concluded that lowering the dam 30 days late has the most influence on the optimal dike height (up to 40 cm dike height increase). This corresponds to the fact that lowering the water level 30 days late also has the biggest influence on the water level.

Duration of the flood wave

The flood wave has been approximated as described in paragraph 6.3.2.2. The influence of the length of the flood wave on the water level in the river was investigated. An increase in flood wave duration of 20 and 30 days was simulated. It was found that an increase in duration does not cause the water level in the river to increase. If the maximum discharge stays the same, but the duration of the flood wave increases, the Dama dam should be opened for a longer period of time.

Overview

An overview of the different dam aspects and their importance is presented in Table 9-7. The influence of the duration of the flood wave is left out as it doesn't have any influence on the optimal dike height.

Table 9-7: Overview of the dam aspects and their influence on the optimal dike height.

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} Original water levels [m IGN]	2.7	3	3.1	3.1	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2
h_{opt} one gate out of order	2.8	3.1	3.1	3.2	2.8	3.1	3.2	3.2	2.8	3.2	3.2	3.3
h_{opt} two gates out of order	2.8	3.2	3.3	3.3	2.8	3.2	3.2	3.4	2.9	3.3	3.4	3.4
h_{opt} dam max opening= 10.25 m	2.8	3	3.1	3.1	2.8	3.1	3.1	3.1	2.8	3.1	3.2	3.2
h_{opt} dam max opening= 9.25 m	2.8	3.1	3.2	3.2	2.8	3.1	3.2	3.2	2.9	3.2	3.2	3.3
h_{opt} dam max opening= 8.3 m	2.8	3.2	3.2	3.3	2.8	3.2	3.3	3.3	2.9	3.3	3.3	3.4
h_{opt} 10 days late [m IGN]	2.9	3.1	3.2	3.2	2.9	3.2	3.2	3.2	3	3.2	3.2	3.3
h_{opt} 20 days late [m IGN]	3	3.2	3.2	3.3	3	3.2	3.3	3.3	3.1	3.3	3.4	3.4
h_{opt} 30 days late [m IGN]	3.1	3.3	3.3	3.3	3.1	3.3	3.3	3.4	3.2	3.4	3.4	3.5

For every combination of polder occupation, i.e. the amount of harvests per year and the amount of polders built, a prioritized list of factors of influence can be made. For instance, in case of one harvest of rice per year and Rosso-Richard Toll poldered (highlighted in blue in Table 9-7), this list is the following:

1. 30 days late lowering the water level
2. 20 days late lowering the water level
3. 10 days late lowering the water level
4. gates get stuck at 10.25, 9.25 or 8.3 m, 1 or 2 gates out of order

In the analysis done here, the flooding frequency is high as the value of the area behind the dike is limited. Also, the increase in optimal dike height is maximum 40 cm. A few tests were done to get a good view on the factors with the most influence on the dike height when the amount of loss due to flooding increases by raising the value of the behind laying area. As the value of this area increases, the optimal dike height will be higher. It appeared that the blocking of the gates at 8.3 m and 2 gates being out of order became more important.

9.2.4 Additional remarks

Combination of factors

To be able to give one single statement on the optimal dike height (and not to be left having to say that if a certain scenario occurs, then x is the optimal dike height), a probability of occurrence is attributed to every of the dam aspects described above. Like this, the different failure modes are incorporated into the failure probability. This probability of occurrence has to be further analysed, however, for the moment, probabilities of occurrence are arbitrarily chosen to show the result.

The analysis of the malfunctioning of the dam can for example be done looking at the reliability of the dam as a system. The gates of the dam function as a parallel system in which the elements (gates) can compensate for each other. This is so because if one gate fails to open or gets stuck at a certain opening height, it does not mean that the function of the dam (retaining water) fails. Moreover, what is the failure probability of a second gate being out of order/ getting stuck if it is known that one gate is already out of order/got stuck? What is the relation between these events? Or what is the probability that the dam is opened up 10, 20 or 30 days after it should be? These are questions that the analysis of the dam as a system is able to answer.

Table 9-8: Probabilities of malfunctioning; arbitrarily chosen.

Situation	Probability of occurrence [%]
Good functioning of the Dama dam	75
1 gate out of order	5
2 gates out of order	5
Gate gets stuck at 10.25 m	3
Gate gets stuck at 9.25 m	3
Gate gets stuck at 8.3 m	3
Dam 10 days late	3
Dam 20 days late	2
Dam 30 days late	1

In case of the probabilities of occurrence taken as shown in Table 9-8, the optimal dike height is determined as presented in Table 9-9.

Table 9-9: The optimal dike height in case of probabilities attributed to the dam aspects

	Present situation				Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt} [m IGN]	2.8	3.1	3.1	3.1	2.8	3.1	3.1	3.2	2.9	3.1	3.2	3.2

Additional tests were done by taking a probability of occurrence of 25% for each of the dam aspects of the Diama dam. They were analysed for water levels based on the present situation and rice being harvested once or twice per year. The results are presented in Table 9-10.

Table 9-10: Influence on the economic optimal dike height at 25% probability of occurrence of a dam aspect (horizontal in table!). The optimal dike height increases with maximum 20cm.

	Normal functioning	1 gate out of order	2 gates out of order	Dam max opening= 10.25 m	Dam max opening= 9.25 m	Dam max opening= 8.3 m	Dam 10 days late	Dam 20 days late	Dam 30 days late
h_{opt} [m IGN] Rice E(S)	2.7	2.8	2.8	2.8	2.8	2.8	2.9	2.9	2.9
h_{opt} [m IGN] Rice E(2*S)	3	3	3.1	3	3	3.1	3.1	3.1	3.1

From Table 9-10 can be concluded that when assuming high probabilities for the dam aspects to occur, the optimal dike height will increase by maximum 20 cm. The probabilities of occurrence should be investigated in more detail, however. This increase of 20 cm occurs when the dam is opened up too late.

In the cases described here above, the only value behind the dikes taken into account is the value of the agricultural activity (the value of the crops). For the local people, the disappearance of the Typha itself is also of high importance. This value could be taken into account determining the optimal dike height. The annual value of the absence of Typha to the local people is however difficult to define. Moreover, if the dike fails and flooding will occur, the harvest will be lost, but that doesn't mean that Typha will be back immediately. The optimal dike height will in any case be higher as the amount of loss due to flooding will increase.

10 Conclusions and recommendations

Conclusions

The research presented in this thesis is an attempt to improve the understanding of the Senegal River system, of its Delta in particular, and gives insight in the economic optimal dike height of the polders to be constructed considering various kinds of crop in the polders.

To get to an economic optimal dike height, the Senegal River system first had to be fully understood with very little information being available. The newly set up one dimensional hydrodynamic model of the Delta using Sobek appeared to be well applicable to generate data and to understand the behaviour of the river. The discharge in the river occurs predominantly in the main channel and the storage of water in the Typha fields (floodplains) does not significantly influence the water level (for water levels occurring up to 1/20 years).

It appeared that making polders in the Rosso area or making polders in the whole Delta has limited influence on the economic optimal dike height, whilst considering future agricultural activity in the polder (10 cm at the most). This optimal dike height corresponds to a high flooding frequency (in the order of 1/10 years), the latter being the result of amongst others the relative low amount of expected loss due to flooding. The results shown here are for one polder of 1000 ha. See Table 10-1 for an overview.

Table 10-1: The optimal dike height for various scenarios. The dike height increases as the expected amount of loss increases. These dike heights corresponds to a flooding frequency of around 1/10 years.

	Rosso- Richard Toll poldered				Whole Delta poldered			
	Rice		Tomatoes	Comb.	Rice		Tomatoes	Comb.
	E(S)	E(2*S)	E(S)	E(S)	E(S)	E(2*S)	E(S)	E(S)
h_{opt}	2.7	3.1	3.1	3.1	2.8	3.1	3.1	3.2
Original water levels [m IGN]								

A sensitivity analysis of various aspects that have an influence on the water level and on the optimal dike height was performed. These aspects were divided into three kinds: a dam aspect, a discharge aspect and the modelling of the dam. A prioritized list of the dam aspects with effect on the optimal dike height was established for every scenario (kind of crop and amount of polders built). The list for Rosso-Richard Toll poldered and one harvest of rice per year is given here:

1. 30 days late lowering the water level
2. 20 days late lowering the water level
3. 10 days late lowering the water level

4. gates get stuck at 10.25, 9.25 or 8.3 m, 1 or 2 gates out of order

It appeared that taking into account the malfunctioning of the dam results in a higher optimal dike height (up to 40 cm increase in dike height). This is because when the dam is malfunctioning, it can discharge less water and therefore the water level will increase which needs a higher dike height. The discharge aspect was investigated by taking into account the parameter uncertainty of the discharge distribution type. A parameter uncertainty with a standard deviation of 20% was investigated for discharges based on the exponential distribution. Considering the parameter uncertainty, the outcome was a higher discharge and thus a higher water level and a higher economic optimal dike height (10cm). The way the dam is put into the hydrodynamic model, resulted in an optimal dike height increase of 10 cm.

Recommendations

The following recommendations are made:

- Collection of more data for more reliable understanding of the river and model improvement
 - Discharge measurements at Rosso
 - Evaporation, extraction of water for irrigation and drinking water
 - Detailed bathymetrical and roughness data
- Examine other dike failure mechanisms (for example piping, sliding or stability of inner and outer slope).
- Take into account the value of the absence of Typha for the optimal dike height calculation
- Other factors contributing to failure probability of the dike
 - More extreme failure of the Dama dam (for instance, all gates out of order)
 - Breach of the Manantali dam
 - Failure of inlet structures or pumping stations in the dike system
- Investigate other kinds of crop and other uses of the polder (for instance aquaculture) for the amount of loss due to flooding
- Further investigation of the yield per kind of crop and polder use
- A sensitivity analysis on the assumptions for harvest loss and polder design
- Take into account the removal of Typha and site preparation (canals, levelling...)
- Treating the Dama dam as a system that can fail in order to determine the probability of occurrence of dam failure.

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Persons

Michel Cadelan (via Royal HaskoningDHV), acelandx@yahoo.fr
Siemen van Berkum, siemen.vanberkum@wur.nl

Appendices

1 Appendix 1: *Typha Australis* or *Typha Domingensis*

(www.database.prota.org)

1.1 Characteristics

Typha Australis

Typha Australis is the water plant which grows in the floodplains of the Senegal River. The plant has got several synonyms like *Typha australis* Schumach. (1827), *Typha javanica* Schnizl. ex Rohrb. (1869), *Typha angustifolia* auct. non L. and it belongs to the Typhaceae family. Its vernacular names are Bulrush, cattail, Santo Domingo cattail, southern cattail, Indian reed mace or 'Massette australe' in French.

Origin and geographic distribution

Typha Domingensis is distributed in the subtropical and tropical zones of the world. It is very widespread in tropical Africa. It occurs in several Indian Ocean islands where it possibly has been introduced. *Typha* comprises about 15 species, most of which are widely distributed throughout the temperate, subtropical and tropical zones of both hemispheres.

Uses

The leaves are widely used in tropical Africa for making mats, hats and baskets. The leaves are sometimes used for caulking barrels and to plug seams of canoes. They are also used as bedding for domestic animals and can be used for making paper. The stems are made into mats and fences. The mature silky female florets are used for stuffing pillows. In parts of DR Congo the stems and rhizomes are eaten throughout the year. In other countries, for instance Nigeria, the rhizome is eaten as a famine food. Immature leafy spikes are eaten as a vegetable and the soft core of these spikes is appreciated as a sweet snack. *Typha* as a fiber plant is only collected from the wild. No production statistics are available.

Properties

The stems and leaves of *Typha Domingensis* are tough and fibrous. The leaves are suitable for caulking, because they swell when wet. Paper made from *Typha* is fairly strong but difficult to bleach. The floss from the female part of the inflorescences has high buoyancy and good insulating properties, both for heat and sound.

The rhizome contains starch and a toxic substance with purgative and emetic properties. Stems (moisture content 88%) from the Zambezian region contained per 100 g dry matter: energy 1253 kJ (299 kcal), protein 1.4 g, fat 0.5 g, carbohydrates 81.9 g, fiber 3.2 g, Ca 20 mg, P 30 mg and Fe 10 mg. The seeds contain a drying oil similar in quality to linseed oil. Aqueous extracts of *Typha Domingensis* have shown phototoxic properties, inhibiting the germination of lettuce and radish seeds and the growth of the water fern *Salvinia minima* Wild. *Typha Domingensis* is propagated by rhizome division or seed. The 1000-seed weight is 0.02–0.03 g. Seeds do not germinate in the dark.

Other botanical information

Typha comprises about 15 species, most of which are widely distributed throughout the temperate, subtropical and tropical zones of both hemispheres. The taxonomy is still not clear and identification of the taxonomical units often difficult. In tropical Africa 4 *Typha* species occur. *Typha angustifolia* L. is a species of temperate regions of Europe and America and has been recorded erroneously for Madagascar. *Typha elephantine* Roxb. is distributed in Asia, the Mediterranean, Senegal, the Sahel, Ethiopia and in oases in the Sahara. The leaves are used for making mats, clothes, ropes, windbreaks and shelters. Young roots and leaf-bases are eaten as a vegetable and plants are grazed by sheep, goats and cattle. In India it is planted to control erosion, the pollen is made into bread, and the rhizome is used for the treatment of ulcers, dysentery and gonorrhea.

Growth and development

Typha seeds readily germinate in open wet areas, but mortality is high and few seedlings reach the reproductive stage. The seedlings can survive when submerged in water as well as when emerged. Once the plant is established, within a year after germination, rhizome growth starts and becomes the main mechanism maintaining a stand. Buds in 2 rows on each side of the rhizome apex may develop into new rhizomes or shoots. *Typha* species are perennials, with their rhizome enabling them to survive periods of cold and drought. Individual shoots do not live longer than 1 year. The pollen is transported by wind. On windy days cross-pollination may occur, but on calm days self-pollination is more likely. Estimates of the number of seeds per inflorescence are up to c. 680,000 and the number of seeds per m² has been estimated at 17 million for *Typha Domingensis*. The fruits are easily transported by wind, with the hairs serving as parachutes. Within minutes of contact with water the follicular tissues saturate and split open, and the seed is released. Seeds remain viable for a long period if conditions are unfavorable for germination.

Ecology

Typha Domingensis grows in marshy locations, in shallow pools and along the margins of often stagnant, fresh or brackish water and along irrigation channels. It is known to grow at a water depth of up to 2-2.5 m. In East Africa it is found from sea-level up to 1500(–2300) m altitude. At higher altitudes it is usually growing mixed with *Typha latifolia* L. *Typha* species are often associated with disturbed, fertile environments. They grow on a variety of soil types, but are usually found on fine-textured organic muds and silts, which have a high nutrient content and water-retention capacity. They are considered moderately salt-tolerant, but growth is significantly reduced at salinities higher than 3–5 ppt (parts per thousand) =0-1.6%. Permanent salinities of 7 ppt (2%) or higher exceed the tolerance limit. The success in brackish environments seems to stem from their ability to grow rapidly when fresh water is available and to persist in a dormant state under saline conditions. *Typha* species are often considered weeds. They are able to dominate vegetations, because their bulky rhizome and their tall, dense canopy give them a competitive advantage. *Typha* species readily colonize disturbed areas where water is available, and may block irrigation and drainage channels. In tropical Africa the dense *Typha* swamps can harbour dormitories with hundred-thousands of red-billed quelea (*Quelea quelea*) and offer protection to malaria mosquito larvae.

Yield

Typha Domingensis is highly productive and the aerial biomass production has been estimated at 13–15 t dry matter per ha per year.

Handling after harvest

The stems may be split before being woven. For pulping, the fibre can be chemically extracted from stems and leaves together by treatment with sodium hydroxide.

Genetic resources

In view of its wide distribution and occurrence in disturbed habitats, *Typha Domingensis* is not threatened with genetic erosion. No germplasm collections or breeding programmes of *Typha* species are known to exist. (document van de Prota database, Gaby Schmelzer)

1.2 Typha removal

Biological solution

Biological solutions to the Typha problem are for example cattle that will eat the Typha. This is a more generic solution, because Typha lives in wetted areas, so the cattle will not be able to eat the Typha. Other biological solutions are inundating the Typha fields (increasing the water level) or introducing a kind of insect or larva that can damage the Typha and that it will die in the end.

Chemical solution

The most widely used chemical solution is the use of glyphosate. This herbicide is applied as a spray directly onto exposed foliage and is then translocated to the rhizomes. Thus, it kills the whole plant and control can last for several seasons (normally 3). Only those plants which are treated with the spray are controlled so that localised control of selected areas or individual weed beds can be achieved. Plants which are to be controlled with glyphosate should have well-developed, undamaged foliage exposed above the water surface at the time of spraying. Plants which have been damaged by cutting or grazing, or which have been bent over or broken by flooding or other forms of mechanical damage are less likely to be controlled. Glyphosate is a relatively slow acting herbicide and treated plants do not regrow in the following spring. Early season control has the advantages that the risk of summer flooding and other problems caused by the weeds during the growing season are reduced. Also, the weeds die back and decompose more rapidly if sprayed when still young and tender. If the plants are sprayed early in the season, there is also less likelihood of the plants being damaged before the spray is applied. Typha is not susceptible to early season control and should be sprayed in August or September. Amitrol, Rodopan, and Doupon are other herbicides used to fight Typha (CEH, 2004) (Apfelbaum).

Mechanical solution

Reeds can be cut by hand, using a scythe, or by machine, using weed cutting buckets or boats. The choice of technique depends on the area involved and on factors such as water depth, ease of access and

availability of suitable equipment. Cutting only removes the emergent shoots and does not affect the buried rhizomes from which new shoots will emerge. If cutting is carried out early in the season in May or June, a second cut may be necessary before the end of the season. Dredging often removes the rhizomes as well as the emergent shoots and so produces longer control, but is generally too expensive to be used purely as a method of weed control (CEH 2004).

2 *Appendix 2: Theoretical background of problem solving method*

This chapter of the appendix gives an overview of the method used in this thesis to solve the problem. This method is the probabilistic design method. The chapter considers the problem from very general to more detail starting with the general idea and ending with all the different parts of the approach. Paragraph 2.1 gives an explanation of the probabilistic approach in general, paragraph 2.2 gives a definition of a failure mechanism and the related Z-function, paragraph 2.3 considers different methods how to solve a Z-function. Paragraph 2.4 talks about the consequences of flooding and paragraph 2.5 combines the foregoing paragraphs by defining risk. Paragraph 2.6 gives an explanation of a method how to define the economic optimum in design taking into consideration construction costs and failure probability.

2.1 Probabilistic approach to a problem

2.1.1 *General idea*

The general idea of a probabilistic approach to a design problem is not to base the design on so-called design values for the load and the strength parameters, but to determine an acceptable probability of failure of a structure. This approach does not only look at the failure probability, but also at the consequences of this failure and like this determines the risk, the latter is a multiplication of the probability of failure and the consequence of that failure. The difference between probability of failure and risk is that risk is a multiplication of the probability of failure with the consequences (Mai, 2010).

Concept of failure and collapse

A structure fails if it can no longer perform one of its principle functions. In the case of a dike, this function is, in general, the prevention of inundation, which means preventing a protected region from being flooded, attended by loss of human life and/or damage of property.

A structure collapses if it undergoes deformations of such magnitude that the original geometry and integrity are lost (CUR, 1990).

SLS vs. ULS

The boundary between failure and non-failure, or between collapse and non-collapse, is generally called a limit-state (LS); this is the state just before failure occurs. A distinction has to be made between ultimate limit state (ULS) and serviceability limit states (SLS). The SLS are limit states in which the functions can barely be fulfilled within the so-called usability boundaries. The ULS is the utmost limit state by which through failure and collapse an object permanently ceases to function (CUR, 1990 and CUR, 1997).

2.1.2 *Application of the solution method in the Netherlands*

The Netherlands has a long lasting history of the defence of land against water. The first polder (droogmakerij) was constructed in the Netherlands in 1533 and ever since, the Netherlands has been

fighting against water. After the big flooding of 1953 in the provinces Zeeland and Zuid-Holland, the Delta Committee was appointed later the same year and proposed a series of protection measures that came to be known as the Deltaworks. The flood surges in 1993 and 1995 in the Rhine and the Meuse lead to the formulation of the Delta plan for the Major Rivers which enters into effect in 1995.

In 2001, the project FLORIS (flood risk and safety in the Netherlands) is launched and tries to answer two questions: What is the probability of flooding? And what are the likely consequences? The project seeks to identify the failure mechanisms that contribute to the occurrence of a flood. Nowadays, the probabilistic design method is used in the Netherlands by taking into consideration the strength of the retaining structure and the consequences of flooding. This project is also related to as VNK1 (Veiligheid Nederland in Kaart 1, the Safety of the Netherlands on Map 1). It ended in 2005 and since 2007 VNK2 is used (Van Eijsbergen et al., 2008).

2.1.3 *Other possible solution method*

The conventional design method in civil engineering design is the deterministic approach. The basis of this approach is to use so-called design values for the load and the strength parameters. Using design rules according to codes and standards, it is possible to determine the dimensional parameters of the civil engineering structures like bridges, tunnels and dams. These design rules are mostly based on limit states of the structure's elements. In the deterministic approach it is assumed that the structure is safe when the margin between the design value of the load and the characteristic value of the strength is large enough for all limit states of all elements. Therefore, the safety level of the structured system is not exactly known. However, this is only correct in the theoretical case where the dike fails as soon as the design level is exceeded, but not below that level. This method has various shortcomings (for example in this method the magnitude of damage has got no influence on the design of the dike, etc...) and therefore, the probabilistic solution method has been developed. (CUR, 1990 and Mai, 2010)

2.2 Failure mechanisms

2.2.1 *Fault tree*

A failure mechanism is a mechanism by which a structure can not fulfil its function anymore. Failure mechanisms differ very much looking at different industries. For example in ports and waterways, a failure mechanism could be the non-workability in a harbour due to waves that are temporarily too high. In the building industry, a failure mechanism could be a building which can not be used anymore due to cracks in the façade or due to settlements in the subsoil. In order to give an overview of the different failure mechanisms, several techniques are available. In the event tree technique, the procedure consists of going from an undesirable initial event to the response of the system and the consequences. Fault trees are based on the opposite procedure and are often used to give an overview of all the failure mechanisms considered. It also shows the relationship between the different events which lead to failure. A fault tree starts from an undesirable top event and looks at what different events can cause this failure. In a fault tree, AND and OR

gates are used to show the relation between the events. If two (or more) events all have to occur for failure to take place, these events will be linked by an AND gate; this corresponds to a parallel system. In case of one event being enough for failure, an OR gate can be used; which corresponds to a series system. OpenFTA is a commonly used programme to draw fault trees (CUR, 1990); an example of a fault tree is given in Figure 2-1.

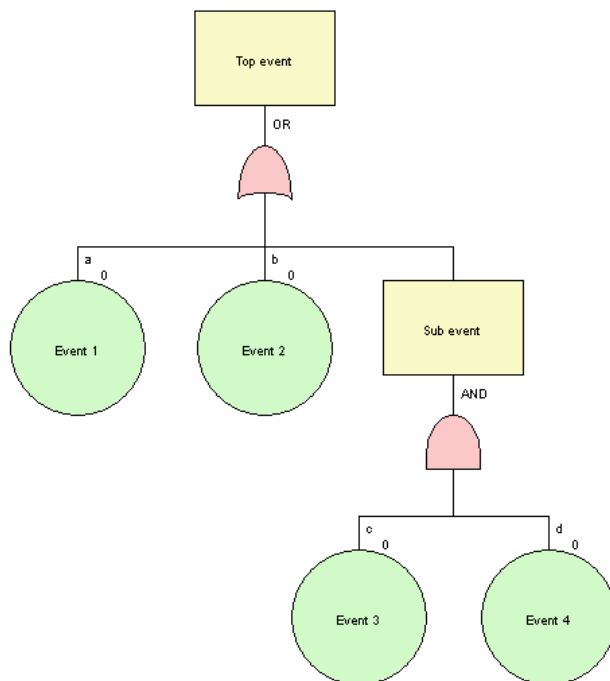


Figure 2-1: Fault tree example

2.2.2 Dike failure mechanism overview

Considering a dike, various failure mechanisms exist, see Figure 2-2.

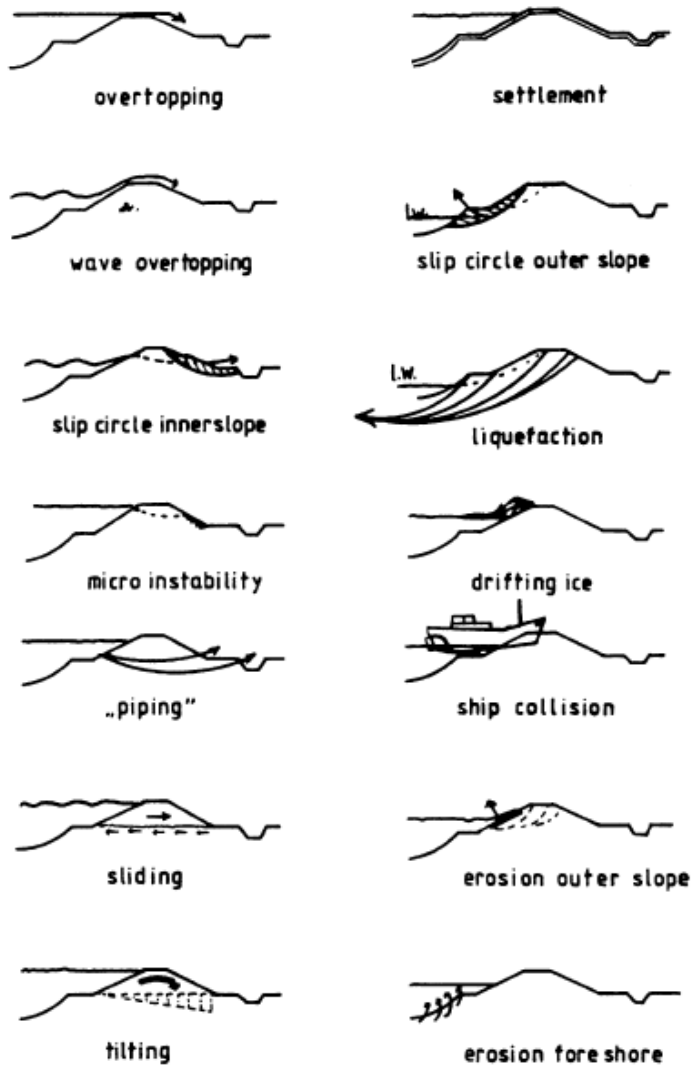


Figure 2-2: Failure mechanisms of a dike

(Source: Vrijling, 2001)

Figure 2-2 gives an overview of many kinds of failure mechanisms of a dike. A few mechanisms are elucidated here which need more explanation:

- Overtopping, often referred to as overflowing as well, occurs when the water level in a river or at sea is higher than the crest height of the dike. The water will then flow over the dike

- Wave overtopping is actually the same as overtopping; the only difference is that the water level in a river or at sea is influenced by the presence of waves. These waves can manipulate the water level. The two mechanisms described here could also lead to consecutive failure mechanisms like erosion of dike crest, inner slope and dike body; breaking of crown walls; functional failures due to too much overtopped discharges, if duration and intensity of storm are large enough
- Piping is the flow of water under or through a dike body due to a water level difference at both sides of the dike. This water will take soil particles with it, which could aggravate the situation. Piping starts by seepage by which soil particles will be displaced on the inner side of the dike. This process starts off slowly, but the longer piping continues to occur, the faster this process of erosion will go. Like this, the dike body will be weakened and could result in settlements or even dike breach
- Liquefaction: refers to the process by which saturated, unconsolidated soils are transformed into a substance that acts like a liquid. The pressures between the soil grains diminished and the material could lose its strength. In case of a dike, water could flow into the dike which will lead to strength loss and finally to dike breaching
- Slip circle inner/outer slope: this touches upon the mechanism of macro-stability. Here, on the one hand the weight of the soil plays a role and on the other hand the shear along the circle is important.
- Settlement: due to its own weight, the dike could be lowered. This phenomenon is called settlement.
- Micro instability: with micro instability, the stability of the outer grains on a slope is meant, in contrast to the stability of the slope as a whole (macro-instability)

2.3 Reliability function

In principle, there are two approaches in ascertaining the probability of failure due to a particular mechanism. One approach is to make a direct estimate of the probability on the basis of experience and intuition. Alternatively, a probabilistic calculation of the failure probability may be performed. In the probabilistic approach, every failure mechanism has its own reliability function; a so-called Z-function. Other names used for this function are the limit state function or the limit state equation (LSE). The reliability is the probability that a limit state is not exceeded. The reliability function is a mathematical expression describing the resistance (R) and the load (S). The general form of the reliability function is:

$$Z = R - S \quad [\text{II.1}]$$

In which:

R is the strength or more generally the resistance to failure (from French Résistance)

S is the load or that which is conducive to failure (from French Sollicitation)

The mechanism is considered to fail when the Z-function becomes negative, so when the load becomes bigger than the resistance. In Figure 2-3, the reliability function is plotted in the RS-plane. The failure space is indicated in this plane (space where $Z < 0$). The limit state is described by $Z = 0$.

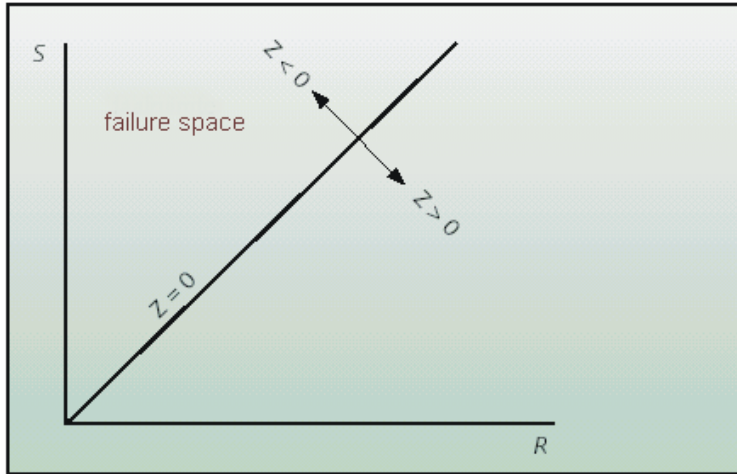


Figure 2-3: Reliability function in the RS-plane

The probability of failure is defined as:

$$P_f = P(Z < 0) = P(S \geq R) \quad [\text{II.2}]$$

And the reliability is the probability $P(Z > 0)$. The point in the failure space with the greatest probability density is called the design point; here the probability density has a local maximum. Generally this point is located on the border between the safe and the unsafe areas. More than one design point in one failure space is also possible. The design point plays an important role in the different techniques for estimating the probability of failure (CUR, 1990 and CUR, 1997).

2.4 Calculating the Reliability function

A reliability function can be analysed using different methods. The Joint Committee on Structural Safety proposed a level-classification on calculation methods. These methods will be explained below. The classification goes from the most probabilistic approach (level III) to a non-probabilistic approach (level 0).

2.4.1 Level III

The level III methods calculate the probability of failure by considering the probability density functions of all strength and load variables. The reliability of an element is linked directly to the probability of failure. The foundation of the level III calculation is the mathematical formulation of the subset of the probability space, which involves failure according to the following equation (here it is assumed that the load and the strength are functions of one or more random variables):

$$Z = g(X_1, \dots, X_n) \quad [\text{II.3}]$$

If the joint probability density function $f_{R,S}(R,S)$ of the strength R and the load S is known, the probability of failure can be calculated by means of integration (see Figure 2-4):

$$P_f = \iint_{Z < 0} f_{R,S}(R,S) dR dS \quad [II.4]$$

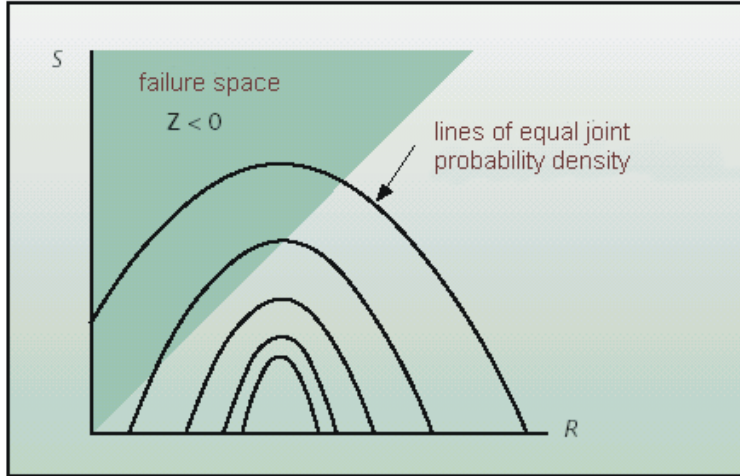


Figure 2-4: Area over which the joint probability density function $f_{R,S}(R,S)$ has to be integrated

If the variables $X_1, \dots, X_n$ in equation [II.3] are statistically independent, the equation can be simplified to:

$$P_f = \iiint \dots \int f_{X_1}(X_1) f_{X_2}(X_2) \dots f_{X_n}(X_n) dX_1 dX_2 \dots dX_n \quad [II.5]$$

This integral can seldomly be determined analytically. The solution is therefore found using numerical methods. A possible solving method is numerical integration; another method is the Monte Carlo Method. Here, the Monte Carlo method will be explained in more detail.

Monte Carlo Method

This method uses the possibility of drawing random numbers from a uniform probability distribution density function between zero and one. The non-exceedence probability of an arbitrary random variable is uniformly distributed between zero and one, regardless of the distribution of the variable. The formula reads:

$$F_X(X) = X_u \quad [II.6]$$

In which:

X_u is the uniformly distributed variable between zero and one

$F_X(X)$ is the non-exceedence probability $P(X < X)$

Using this formula a random number X can be generated from an arbitrary distribution $F_X(X)$ by drawing a number X_u from the uniform distribution between zero and one. This way of drawing random numbers is generally applicable. However, for distributions, for which the inverse probability distribution function $F_X^{-1}(X_u)$ is not known analytically, this method can lead to a lot of iterative calculations.

By inserting the values for the reliability function(s) it can be checked whether the obtained vector $(X_1, X_2, \dots, X_m)$ is located in the safe area. By repeating this procedure a large number of times the probability of failure can be estimated with:

$$P_f \approx \frac{n_f}{n} \quad [11.7]$$

In which:

n is the total number of simulations ($n < m$ draws from the uniform distribution, in which m is the number of base variables);

n_f is the number of simulations, for which $Z < 0$.

One hundred simulations of a two-dimensional random vector have been plotted in the $F_{X_1}(X_1), F_{X_2}(X_2)$ – plane in Figure 2-5. The figure shows that six realisations are located in the failure area. An estimate of the probability of failure is $P_f = 0.06$.

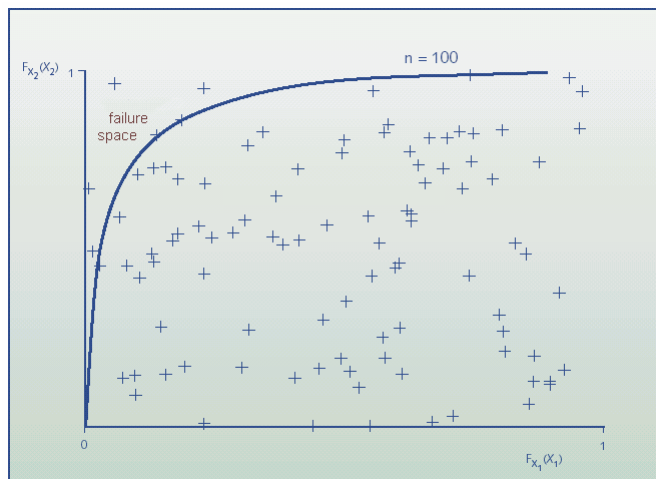


Figure 2-5: Monte Carlo simulation

(CUR, 1997)

2.4.2 Level II

If the reliability function is linear and the base variables $X_1, \dots, X_n$ are normally distributed and statistically independent, Z is also normally distributed. The probability that $Z < 0$, can then be determined using the standard normal distribution:

$$P(Z < 0) = \Phi\left(\frac{0 - \mu_z}{\sigma_z}\right) = \Phi\left(-\frac{\mu_z}{\sigma_z}\right) \quad [II.8]$$

The reliability index is referred to as:

$$\beta = \frac{\mu_z}{\sigma_z} \quad [II.9]$$

Once the reliability index is found, the failure probability can be found by using the following equation:

$$\Phi_N(-\beta) \quad [II.10]$$

The value of $\Phi_N(-\beta)$ can be found in tables.

For more detailed information, see (CUR 1997)

2.4.3 Level I (quasi-probabilistic approach)

In practice, the problem is often that the strength is unknown, but that it has to be determined for a given reliability. The determination of the required strength can be carried out with the help of both level III and level II methods, by iteratively adjusting the strength in the calculation until a sufficiently small probability of failure is found. The most common way of creating a design is by means of regulations and guidelines. The essence of the standards is that a certain representative value of the strength is divided by a factor and that the representative value of the load is multiplied by a factor, for which the following must apply:

$$\frac{R_{rep}}{\gamma_R} > \gamma_S S_{rep} \quad [II.11]$$

The factors γ_R and γ_S are known as partial safety factors. The representative values of the strength and the load are generally calculated with:

$$R_{rep} = \mu_R + k_R \sigma_R \quad [II.12]$$

$$S_{rep} = \mu_S + k_S \sigma_S \quad [II.13]$$

In which k_R can be negative and k_S can be positive or negative.

A link was sought with probabilistic design methods with the help of level II failure probability calculation. The link is found in the definition of the design point. The design point is the point in the failure space with the greatest joint probability density of the strength and the load. It is therefore plausible that for failure the values of the strength and the load are close to the values for the design point (CUR, 1997).

2.4.4 Level 0 (deterministic approach)

Some maximum load and minimum strength is taken based on experience and/or intuition and one overall “safety factor” is applied. This is not a probabilistic approach at all. An overall safety coefficient, usually γ , usually says nothing about the safety. Another approach can be to work with characteristic values for load and strength, with a small chance that these values are too low or too high.

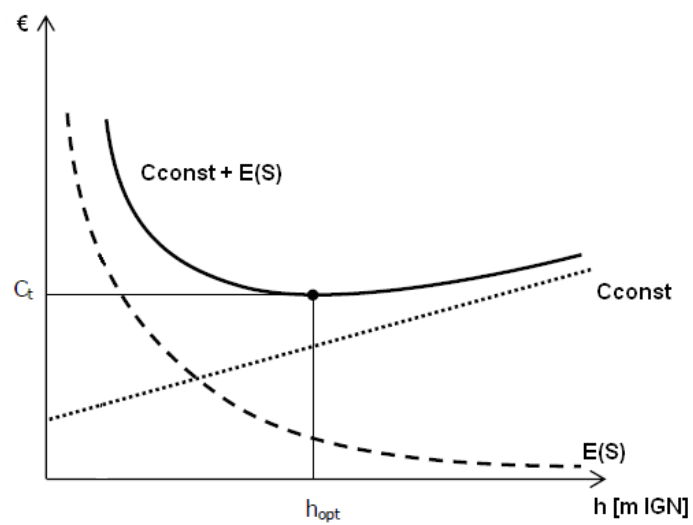
For more detail on the design levels, see (CUR, 1990 and CUR, 1997).

2.5 Consequences and the concept of risk

When a flood defence structure fails, water from the sea, lake or river flows into the region the structure was intended to protect. In the great majority of the cases, inundation of the region will occur. If inundation occurs, “damage” or “loss due to flooding” will result. In case of making a polder, benefits will be created by the creation of usable land, but once this land is in use and the dike fails, still damage will occur (CUR, 1990). Risk is the multiplication of the probability of failure and the amount of “damage”.

2.6 Economic risk-based design (Van Dantzig Model)

The (Van Dantzig, 1956) model takes the present value of the loss due to flooding and relates that to the construction cost of the dike. Like this, the total cost of the dike is calculated. The point where the total cost is minimal, the economic optimal dike height is located.



2-6: The economic optimal dike height using the Van Dantzig model.

3 Appendix 3: The dams

3.1 Manantali dam

As mentioned in paragraph 4.2.2.1, the Manantali dam is of importance for the Senegal River system. Its goals were mentioned in paragraph 4.2.2.1 and its management is based on three groups of constrictions/instructions.

- i. Management constrictions related to the dam characteristics
- ii. Safety restrictions of the dam so that the dam will not be damaged
- iii. Instructions for regulating the discharges and energy production

These three constrictions/instructions result in a list of about ten rules which have to be followed. In function of the water level in the Manantali reservoir, the constrictions of the dam give a total discharge that can go through the dam which has to be located in an interval $[Q_{\min}, Q_{\max}]$. The way these two discharges are determined is unknown. In function of the safety restrictions and the instructions for regulation, a $Q_{\min}$ and a $Q_{\max}$ are defined. The $Q_{\min}$ depends on the water level in the reservoir, the energy production, the artificial flooding, the minimal discharge in the downstream part of the dam and the minimal available space in the Manantali reservoir in order to be able to attenuate a possible flood wave. The $Q_{\max}$ depends on the minimal water level in the reservoir for safety reasons, the immediate attenuation of a flood wave at Bakel and the minimal water level in the reservoir needed to ensure a minimal discharge in the dry period. The precise calculation of $Q_{\min}$ and $Q_{\max}$ is not known. The information comes from the Manantali manual which was made in 2001 and the dam only started functioning in 2003. So all the information in the manual is theoretical and discharge measurements are not done. Of course, $Q_{\min}$ and $Q_{\max}$ have to lie within the $[Q_{\min}, Q_{\max}]$ interval (OMVS, 2001 (B)).

3.2 Diama dam

The Diama dam has got major influence on the water level in the project area. The available information concerning the dam is from 2001 and is very theoretical. The exact way the dam is managed is unknown as the gentleman who designed the management of the dam has passed away and he wanted to keep the information to himself (for financial reasons). The formulas stated in the Diama dam manual are based on 16 discharge measurements between 1998 and 2000. Two different scenarios are defined: partly opened gates and completely opened gates. For the partly opened gates, the discharge through the dam is

$$Q = 1.0566ENL(2g(H_m - H_v))^{0.3761} \quad [\text{III.1}]$$

and for the completely opened gates the discharge is

$$Q = 9.494NL(H_m - 0.01)^{0.8239} \quad [\text{III.2}]$$

in which E is the vertical opening of the dam [m], N is the amount of open gates [-] and L is the width of the gates equal to 20 m, g is the gravitational acceleration [m/s^2], H_m is the water level just in front of the Diama dam [m IGN] and H_v is the water level downstream of the Diama dam [m IGN].

In the manual, a lot of information is given on how the dam should be working and about the calibration of formulas. The energy dissipation over the dam is calculated by multiplying the discharge over the dam with the water level difference between upstream and downstream of the Diama dam. This value can not be higher than $1000 m^4/s$, as also mentioned previously (OMVS, 2001 (A)).

4 Appendix 4: Volume balance, Backwater curves and Q-h relation

4.1 Volume balance

To do a volume balance calculation, the (daily averaged calculated) discharges at Bakel and those at Diama were taken per year. These were multiplied by 3600*24 to get the volume coming through in one day. Then all the daily volumes were added for one year and the sum of Bakel was compared to that of Diama.

At the Diama dam, for some periods during the year, the discharge was unknown. In that case, also the discharge at Bakel was not taken into account in order not to get wrong calculations. It was assumed that the daily average discharge would flow during 24 hours.

4.2 Backwater curve calculation

For the backwater calculation by hand, the “normal” calculation procedure has been followed by using Bélanger and the approximation of Bresse. These approximations are derived from the complete De Saint-Venant equations using the assumption of stationary flow. It is a rough approximation because the flow is not stationary in the river. It is however a good way to show the main functioning of the river. The stationary flow in a straight canal with a shallow and rectangular cross section can be described as if it were a very wide canal. In that case, the stationary over depth averaged flow is described by the continuity equation (De Vriend, 2007):

$$\frac{d(uh)}{dx} = 0 \quad [\text{IV.1}]$$

and the momentum equation:

$$u \frac{du}{dx} = gi_b - g \frac{dh}{dx} - \frac{g}{C^2} \frac{u^2}{h} \quad [\text{IV.2}]$$

The approximation of Bresse is used when the Froude number is relatively small (a lot smaller than 1). The Froude number equals

$$Fr = \frac{u}{\sqrt{gd}} \quad [\text{IV.3}]$$

and is a relation between the flow velocity of the water and the propagation speed of a long, low surface wave. It is actually a relation between the particle velocity and the wave velocity. In this case, the exact flow velocity in the river is only approximated, and the depth is an average value, but to give the order of magnitude, the following calculation is done:

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M.Sc. Thesis Alexander Henny

Royal HaskoningDHV – Delft University of Technology

$$Fr = \frac{0.5}{\sqrt{9.81 \cdot 10}} = 0.05 \quad [\text{IV.4}]$$

This is a relatively small Froude number and so the approximation of Bresse may be used. The Froude number is used to give a statement on the kind of flow of water. The flow is called subcritical if the Froude number is below 1, critical if $Fr=1$ and supercritical if $Fr>1$.

First, the equilibrium depth has been calculated by

$$h_e = \sqrt[3]{\left(\frac{q^2}{C^2 i_b} \right)} \quad [\text{IV.5}]$$

then the half-length has been calculated by

$$L_{1/2} = \frac{0.24 h_e}{i_b} \left(\frac{h_0}{h_e} \right)^{4/3} \quad [\text{IV.6}]$$

and finally the water depth at a certain point has been calculated by (De Vriend, 2007):

$$h = h_e + (h_0 - h_e) \left(\frac{1}{2} \right)^{\frac{-x+x_0}{L_{1/2}}} \quad [\text{IV.7}]$$

The equilibrium depth is the depth in the situation when the acceleration in the river is 0 m/s^2 and the flow is uniform, the half length is the distance from the downstream boundary to a location at which the difference between the water level value at the downstream boundary and the equilibrium depth value at this boundary is halved. The total length of the backwater effects is much larger than the double than the backwater half-length, because the water level curve asymptotically goes to the equilibrium depth.

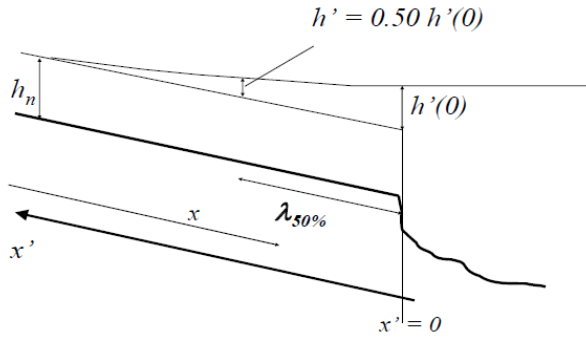


Figure 4-1: Backwater curve with half-length (Source: Crosato, 2007)

4.3 Q-h relation

Firstly, the hand calculation will be explained and interpreted, secondly the model simulation will be explained and interpreted and finally the two methods will be compared.

Calculation by hand

For the Q-h relation made by hand, the cross section is filled with water from 0 m water depth until the whole cross section is full of water. The water depth is increased by 0.1 m at the time and for every depth, the area of the cross section, the wetted perimeter, the hydraulic radius and the flow velocity is calculated. With the flow velocity and the area, the discharge through the cross section has been determined for every water depth.

Various methods have been used to calculate the relation by hand:

1. Separate channels for the floodplains and the main channel with zero internal shear stress at the vertical interface of the channels are assumed. The floodplains and the main channel have different Chézy values as the floodplains are filled with Typha and the bed of the main channel consists of clay. In this method, and overall C-value for the whole river is used, calculated as follows:

$$C = \frac{P_1 C_1 + P_2 C_2}{P} \quad [IV.8]$$

The hydraulic radius is calculated by:

$$R = \frac{(A_1 \sqrt{R_1} C_1 + A_2 \sqrt{R_2} C_2)^2}{(AC)^2} \quad [IV.9]$$

This method yields a more realistic R-value than the traditional $R = A/O$ approach which is only accurate for a simple cross section with uniform roughness (which is not the case here). The discharge Q is then calculated by

$$Q = AC\sqrt{Ri} \quad [IV.10]$$

Here, i_w should be used. As this value is unknown (in this theoretical calculation), i_w is approximated by i_b (uniform flow). Other parameters (the profile is not precisely defined, so A and P could be different and the Chézy value is assumed) have a lot more influence on the result, so the assumption of $i_w = i_b$ can be made.

This method (Van Rijn, 2011) is called method 1.

2. To determine the flow discharge of an irregular cross section, the latter can be divided into a series of elements with known width b_i and depth h_i . The discharge is then obtained by summation, as follows:

$$Q = \sqrt{I} \sum b_i C_i (h_i)^{1.5} \quad [IV.11]$$

In the case of calculating the discharge for every depth, the width of the river is expressed in the depth following geometrical rules. The result of the two calculation methods is shown in Figure 5-8 in paragraph 5.2.2.3. The hand calculation has been repeated for various values of the Chézy coefficient in main channel and floodplain in order to analyse the influence of this coefficient on the discharge through the channel.

This method (Van Rijn, 2011) is called method 2.

The sensitivity of the choice of the C-value for the main channel and for the floodplains is analysed to show the influence of the C-value on the simulated discharges. As can be seen in Figure 4-2, a higher C-value results in a higher discharge. This is logical, due to the fact that the Chézy coefficient actually is a smoothness coefficient. A higher C-value means that the water can more easily go through the channel, resulting in a higher discharge value. Also the difference in C-values for the floodplains is shown in the figure. Here also, a higher C-value will result in a higher discharge. The difference in discharge caused by the C-value for the floodplains is logically more noticeable for high discharges as in this case a large portion of the water will flow through the floodplains.

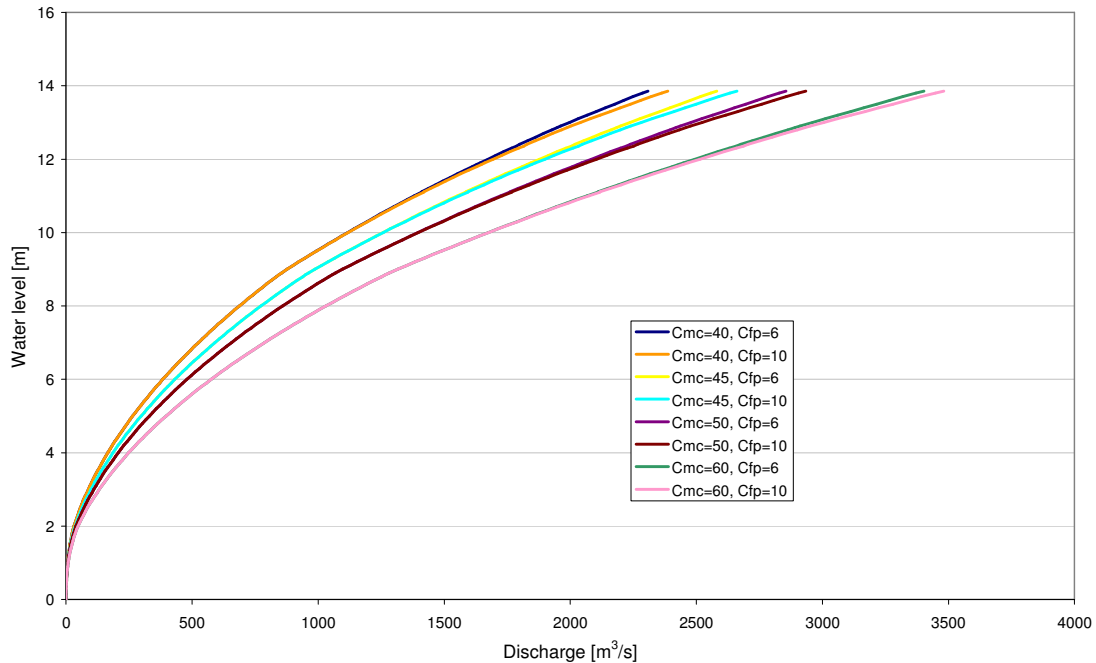


Figure 4-2: Sensitivity of the hand calculation based on method 1. For a value of the water level, the discharge increases as the Chézy value increases.

Calculation by Sobek

As mentioned in paragraph 5.2.2.3, an extra reach of 1000 km is added to the Sobek model in order to minimize the influence of the downstream boundary on the project area and to ensure free flow of water (in case of the dam present at the downstream boundary, the water is not freely discharges as it is held by the dam). At the downstream boundary in the model a very low water level is imposed and at the upstream boundary a discharge is imposed. The discharge set at the upstream boundary changes from $0 \text{ m}^3/\text{s}$ to $4000 \text{ m}^3/\text{s}$ in steps of $500 \text{ m}^3/\text{s}$ and like this, the model calculates the water level in the project area at the different discharges. This method is the exact opposite of the method used in the hand calculations. The result can be seen in Figure 5-8 in paragraph 5.2.2.3.

5 Appendix 5: The hydrodynamic Sobek model

5.1 Hydrodynamic modelling

5.1.1 Hydrodynamic models

For the purpose of modelling the flow in a river system, many different software programmes exist. The choice of which model to use depends on the application, the available data and the envisaged result. A hydrodynamic model simulates the motion or flow of for example water, in for example a river. It is a tool to describe or represent in some way the motion of water. Examples of hydrodynamic modelling programmes are Delft3D, Sobek 2D and Sobek 1D.

In this case, Sobek 1D Rural will be used for the following reasons:

- The results of modelling in Sobek 1D meet the required results. No more detailed modelling is required;
- The data available don't allow a more detailed model. For more advanced modelling, more detailed information is required, which is not available in this case. For example for modelling in 2D, the modelling grid should be very fine, which is done with information that is not available;
- Sobek 1D is a programme which can be used in case of open channel flow with a high water wave coming through, as is the case here.

Sobek 1D is a numeric programme which uses the equations of "De Saint-Venant". Many simplifications of this set of equations exist, but Sobek 1D does not make any simplifications to these equations. Sobek 1D uses the so called "Dynamic-wave" approach (i.e. using the complete set of equations) (Battjes, 2002 (A))

5.2 Application to the Senegal River case

5.2.1 The profile

The river was divided into multiple stretches with different lengths and widths. For every stretch, a profile at the lower end and at the upper end has been defined. As can be seen in Figure 5-1, the river profile is divided into three parts: the main river channel and the two floodplains. The profile has been defined between the two dikes (the summer and winter bed on Senegalese and Mauritanian side). An average width was calculated based on Google Earth images and the length of the stretch. The topography of the floodplains was defined based on a map received from the OMVS. The bathymetry of the summer bed was defined based on the longitudinal section.

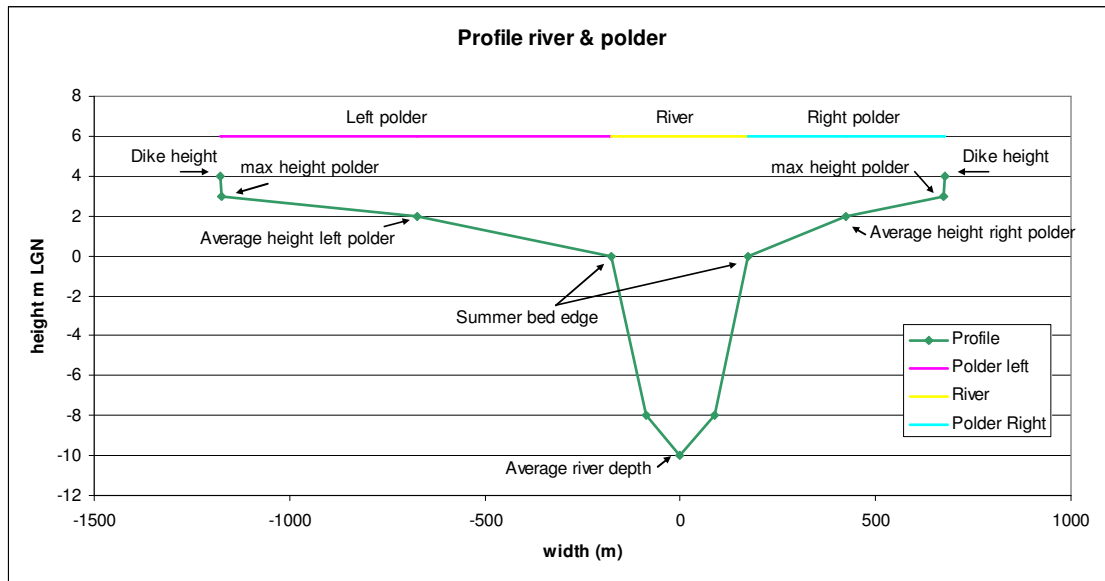


Figure 5-1: Cross section as used in the model.

The profile used in the Q-h relation is a profile defined in the project area near Rosso, see Figure 5-2.

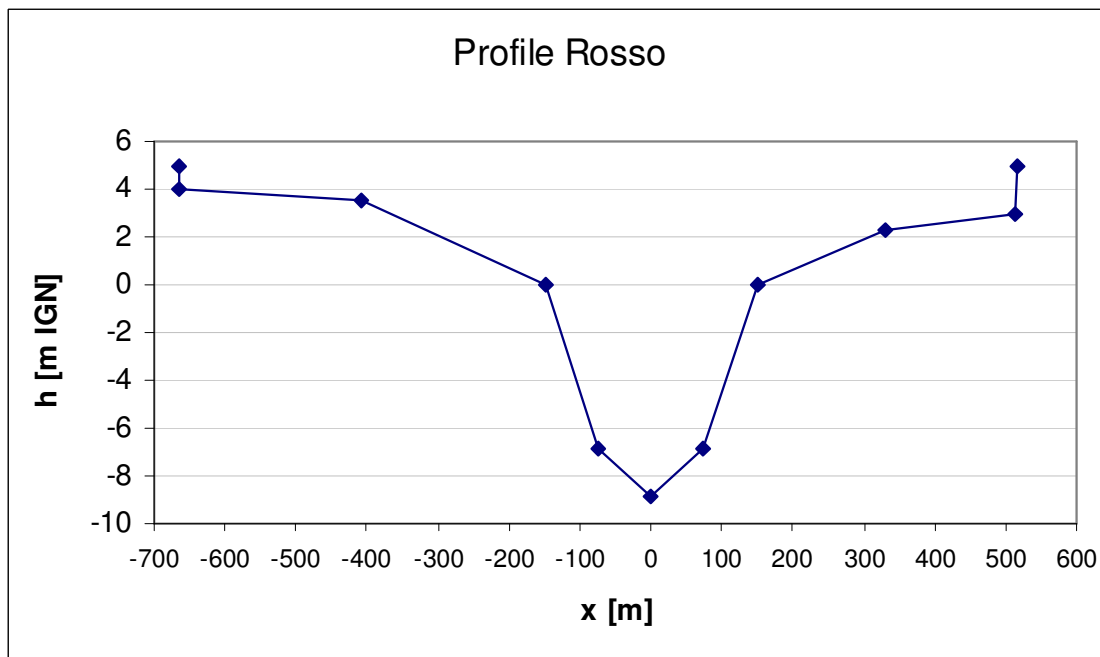


Figure 5-2: Profile near Rosso used for Q-h relations.

5.2.2 Model validation

In the beginning stage of the project, the model was validated by imposing water level measurements at the upper and lower boundary and investigating whether the simulated water levels in between these boundaries would coincide with the measurements at these locations. This is done for the flood wave in 1999 and for the year 2000. The water level measurements were imposed at the boundaries and the water level was compared to the measurements at Rosso. It is visible in Figure 5-3 and Figure 5-4 that the simulated water levels are fairly close to measured water levels. Based on these validation data, the model appears to give good results. In 1999, the average difference between measurement and simulation is 2.2 cm and in 2000 this value is 2.3 cm. The differences between the measured water levels and those simulated could be due to several reasons:

- The location of water level measurement done by the OMVS is not exactly known and might be different from the location of the simulation point in the model
- The model works with fairly rough estimates for the river profile

However, looking at Figure 5-5, this way of validation does not tell us anything about whether the Chézy values are correct or not (as they all give similar results for the water level). A Chézy coefficient is a coefficient that gives an indication on the roughness of the river bed. It actually is a smoothness coefficient meaning that the higher the C-value is, the smoother the river bed is. A sensitivity analysis was done for the Chézy values for the main channel and the floodplains for the simulation of discharges, see Figure 5-6. The water level measurements were imposed and the discharge was simulated. Here it is visible that the Chézy value does make a big difference.

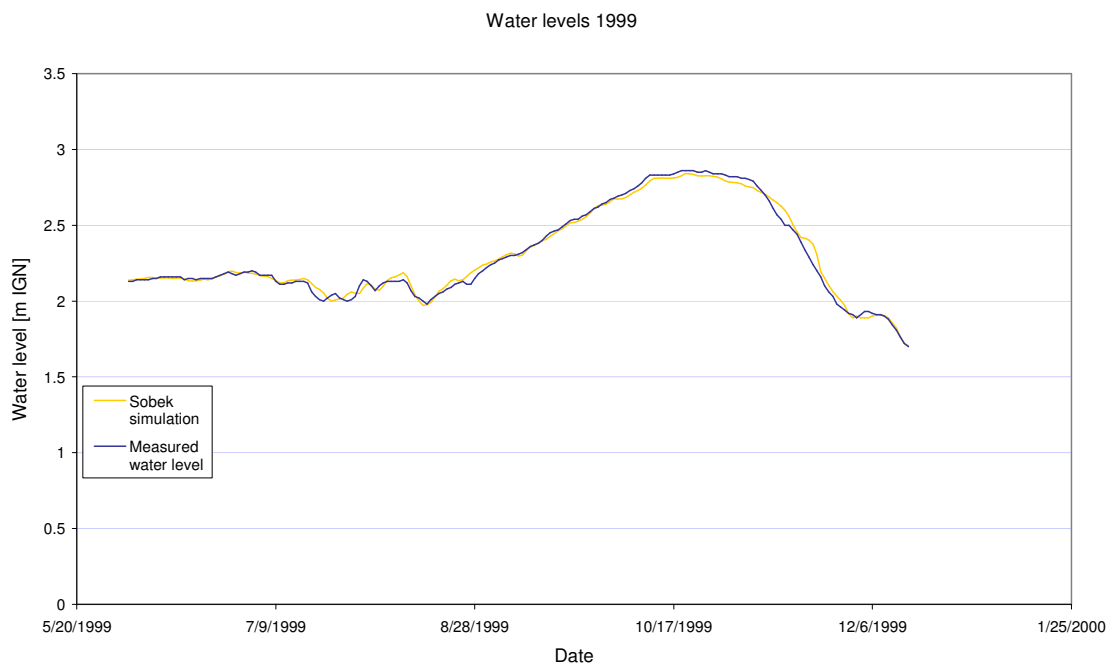


Figure 5-3: Validation of the model. The model shows good agreement between the water levels measured and those simulated.

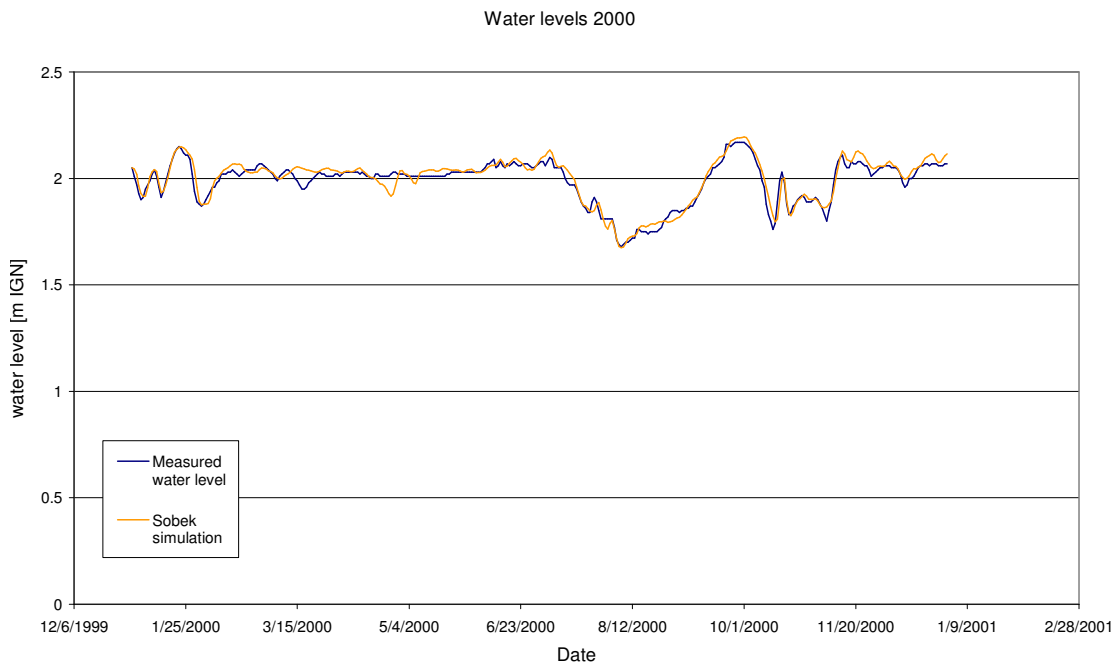


Figure 5-4: Validation of the model. The model shows good agreement between the water levels measured and those simulated.

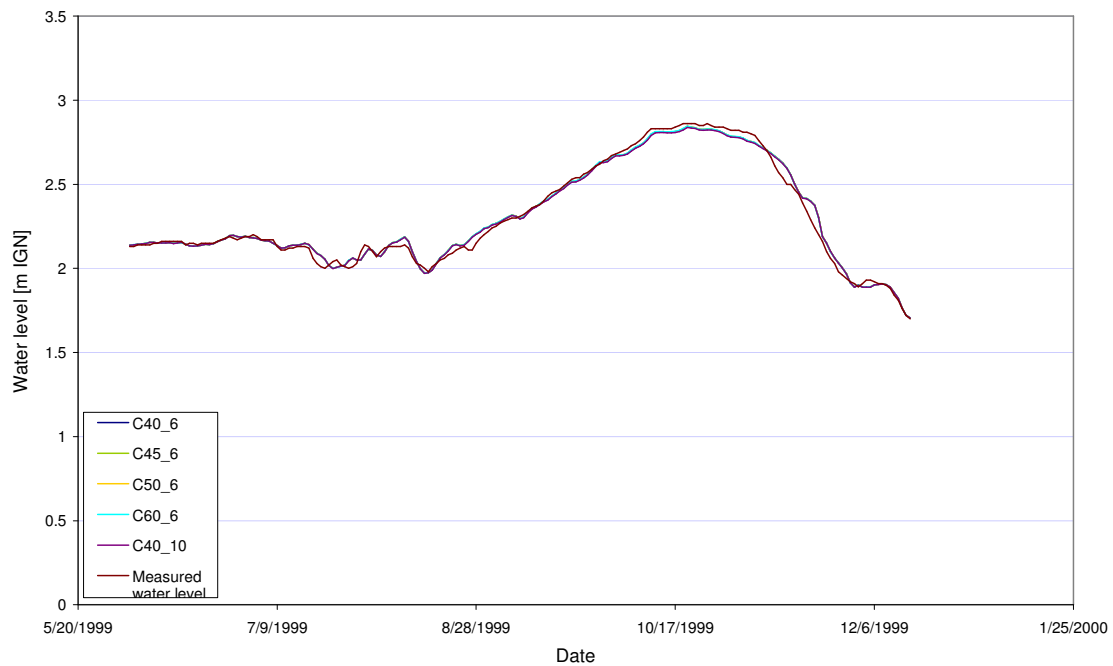


Figure 5-5: Influence of various Chézy values on water levels. The Chézy value has not much influence on the water level simulation.

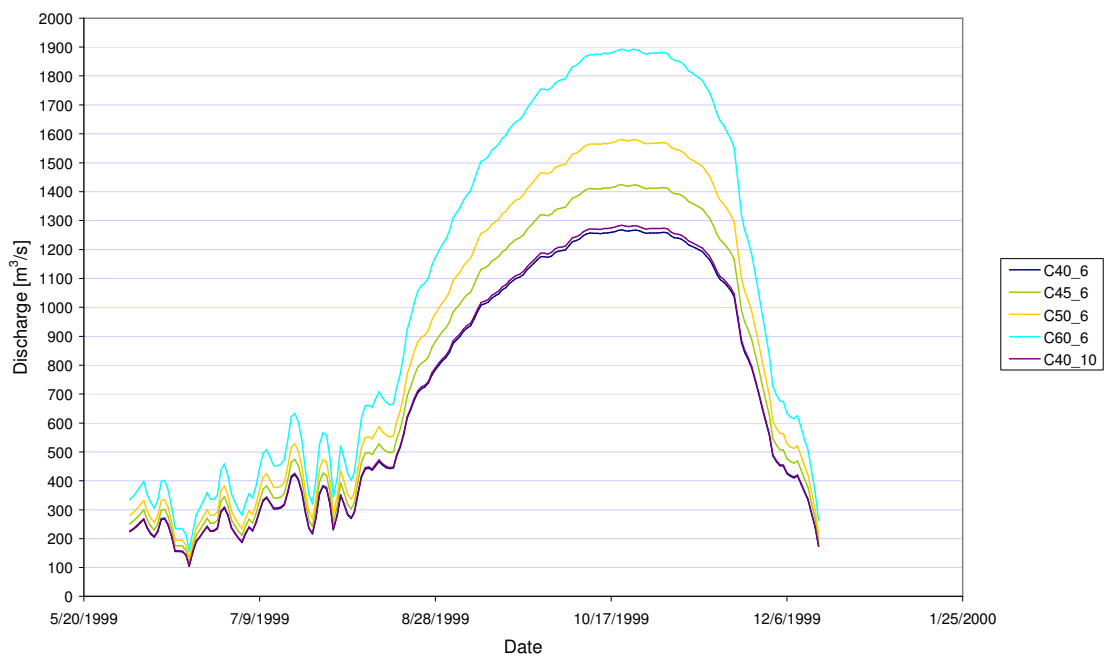


Figure 5-6: Influence of various Chézy values on the simulated discharge. The Chézy value does have influence on the simulated discharge.