Closure of the Gulf of Khambhat

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Site visits:	Area around Gulf of Khambhat, Gujarat (India, April 1998) La Rance tidal power facility (France, June 1998)

Preface

When Cornelis Lely in 1891 presented the final plans for the closure of the Zuiderzee, it took almost 30 years for the works to commence, and another 12 years before the Afsluitdijk in 1932 changed the Zuiderzee into the IJsselmeer. However, the first designs of closure were made in 1848.

The closure of the Gulf of Khambhat in India is a project of the same scale, with an extreme tide as the major difficulty. When this project is compared to the closure of the Zuiderzee, we live somewhere between 1870 and 1915. The image is born, but a lot of study has to be done, and many uncertainties, not only technically, should be clarified.

This report presents the results of the final thesis study of Erik Broos and Kees Wiersema. We hope that it can be a contribution in the insight in this interesting project.

We would like to thank all persons that have contributed to this project, especially the members of the committee of supervision and Gauri Wagenaar of the Netherlands Business Promotion Office Gujarat for support in India.

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Erik Broos and Kees Wiersema

Summary

This final thesis report describes the results of the feasibility analysis to the closure of the Gulf of Khambhat done by Erik Broos and Kees Wiersema, from Delft University of Technology. The study started with site visits to both Gujarat and La Rance where interviews were held to obtain insight in the desires of concerned parties as the Government and private investors, followed by a design study to the innovative methods of closure.

Context

The closure of the Gulf of Khambhat in the state of Gujarat, India, is the largest closure of a tidal estuary in the world so far. It is part of the multi-purpose Kalpasar project. This project solves the two main problems in Gujarat: the shortage of (irrigation) water and the shortage of electricity.

The closed Gulf will be divided into two reservoirs: a fresh water basin for irrigational purposes and a tidal basin for generation of electricity. Secondary aspects of the Kalpasar project are a shorter road connection and the possibilities of port development and land reclamation.

The dam alignment runs south of the Narmada river, to ensure inflow of fresh water into the basin. The total dam length is about 60 km, 30 km through deep water, 30 km through shallow water.

Design problems

Constructing a dam in this Gulf is very difficult, as the tidal difference is extremely large. The average difference between high and low water is about 8 meters, while during spring tide the difference can be more than 10 meters. Together with a water depth of sometimes more than 30 meters below Mean Sea Level and a 30 km wide closure gap, it is clear that this is a project unlike any other. A dam of 30 km closed off the Zuiderzee in the Netherlands, and the Tidal Power Station in La Rance (France) has a tidal difference of 13 meters at spring tide, but the Gulf of Khambhat combines these two problems.

Goal

Although several designs have been made, it has become clear that, to become feasible in India, the design should be 'cheap', fast to build and using local material as much as possible.

Design methodology

To reduce the scale of the problem without losing total overview, the closure dam is split into smaller components. These components each have similar design problems, but on a smaller scale. Integrating these in a total design while making use of each alternative's advantages creates an innovative design where problems of one component are reduced by the advantages of another. Interviews during the visit to India learned that a tidal power facility was one of the most desired components.

The main components are the tidal power facility (TPF), the spillway of the reservoir (Narmada spillway), the final closure gap and the secondary damsections.

Closure strategy

During an early stage the idea was born to incorporate the tidal power facility in the closure process. The tidal power facility requires a large orifice to fill and empty the tidal basin. This orifice proved to be very useful to reduce current

velocities during final closure (from 8 m/s to 6 m/s). In fact, the tidal power plant is used as a sluice. It is consists of 45 concrete caissons, which are constructed in a construction dock and floated into position. This same concept is used for the Narmada spillway that is needed to regulate the reservoir level. As this placing takes place in an early phase of the project, the current velocities are low.

Final closure gap
For the closure of the final gap, between the tidal power facility and the Narmada Spillway, two alternatives have been developed. The first is a temporary railway bridge from where rock is dumped. For this bridge two options have been considered: A conventional dumping process and a process where a certain amount of loss is accepted (overkill closure), to use smaller stones. The overkill principle is only applicable in the lower layers of the dam and just below Mean Sea Level. In the lower layers the required stone diameters are very small and a slightly larger diameter reduces, without extra cost per ton, loss considerably. In the upper layers wave attack demands bigger stones. Therefore the overkill closure is rejected. The maximum required stone diameter is 1,2 m (4700 kg).

The other alternative for closing the final gap is using sand-filled geotextile Superbags. These bags, with dimensions of length * width * height of 40 * 30 * 25 m, form a set of aramide-reinforced geotextile tubes, filled with densely packed sand. This filling and placing is done in one special operation. This method is very fast and cheap. It uses local material (sand). Combined with a bottom protection of sand-filled mattresses a good connection between the original bottom and the dambody is evident.

Secondary dam sections
The secondary damsection are the sections from Ghogha to the tidal power facility, constructed of rock, the work islands, constructed of sand, and the section from the Narmada Spillway to Hansot. This last section is made of clay, which can be found locally.

Costs and building time
The construction of the total dam takes 6 years and costs 22000 crores rupees (NLG 11 billion) when the final gap is closed with stones dumped from a temporary bridge. Constructing with Superbags takes 5 years and costs 20000 crores rupees (NLG 10 billion). These building times are without pre-producing of the quarries and all other startup works. It stops when the closure dam is finished and the hydraulic sandfill of the final dambody is constructed. All costs include the roads and railways and concrete works of the tidal power facility, but exclude the reconstruction costs of the tidal power facility, turbine costs and Kalpasar irrigation scheme.

Final closure technique
Two closure methods are discussed, one existing (rock) and one totally new. These Superbags seem to offer the best possibilities to close the Gulf of Khambhat 'cheap' and fast. Main problem is the fact that there is no experience with these bags. But, if after model tests the Superbags prove to be as good as they are assumed to be, the Gulf of Khambhat should be closed with these Superbags.

PART A

PART B

TABLES

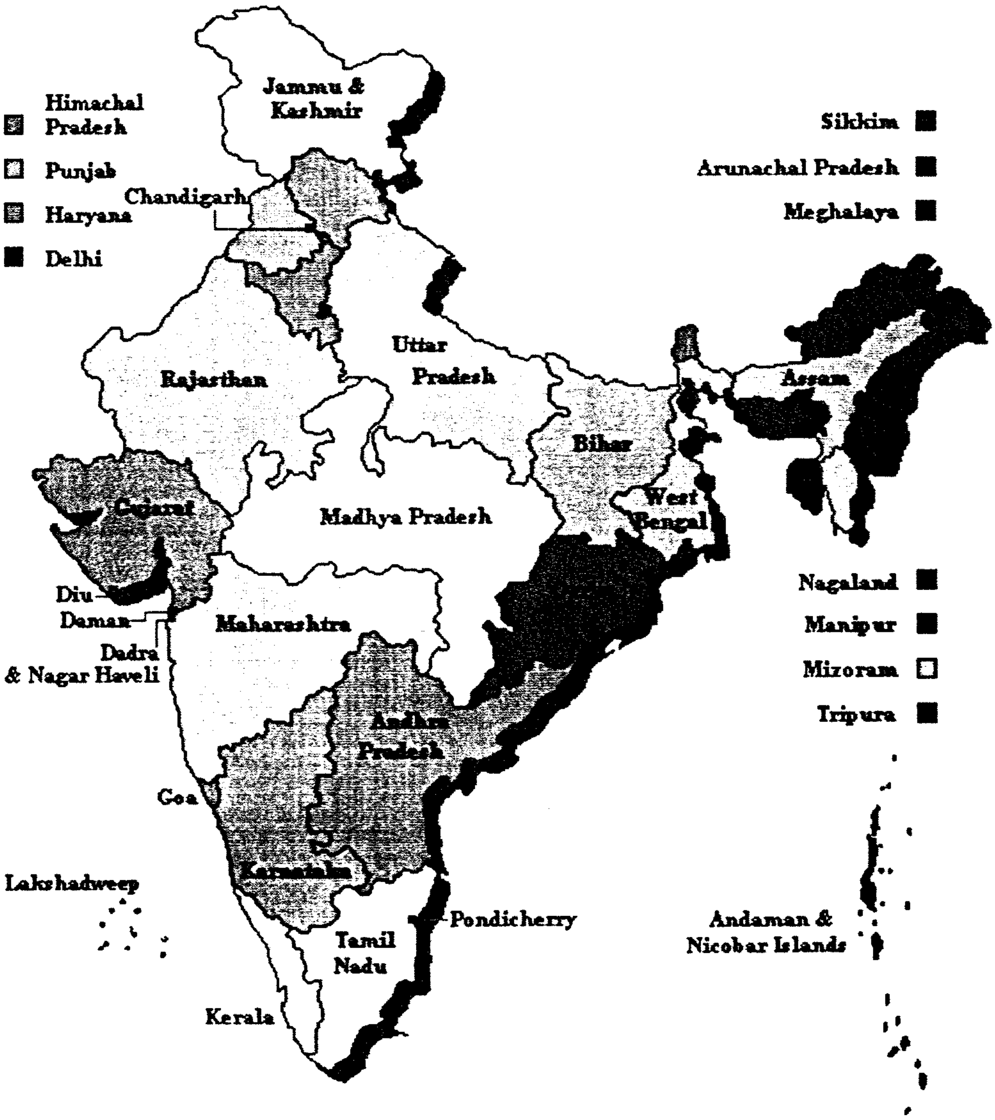
PART B

PART C

PART D

Part A Introduction

This Part provides general information about India and the Kalpasar project (chapter 2) and information about the design strategy (chapter 3).



1 Introduction

In the fast developing state of Gujarat, India, the demand to fresh water and (electrical) power is ever increasing. A standard solution that provides both demands is constructing a barrage in a mountain river. The big disadvantage of such a large dam, is the fact that good farming land (bordering a river) is flooded, and all the inhabitants have to be resettled.
There are however other methods to create fresh water lakes, in the Netherlands this is done by closing large estuaries. After the closure the salt water is replaced by fresh water. The largest and most well known closure in the Netherlands is the closure of the Zuiderzee, after closure renamed in IJsselmeer.
At some locations in the world it is possible to generate power out of seawater due to a large tidal difference. The only (real) tidal power-generating dam is located in France (La Usine Marémotrice de La Rance).
The salt Zuiderzee is turned into the fresh IJsselmeer, and the Gulf of La Rance is changed into a tidal power lake.

The Gulf of Khambhat is also an area with large tidal differences, offering the possibility to generate tidal power. But a large river flowing into the Gulf of Khambhat also offers the possibility to create a fresh water basin like the IJsselmeer. By combining these two alternatives a unique project is created providing fresh water and power to the people of Gujarat without resettlement of people. This project is called the Kalpasar project.

This report describes the closure dam that is needed to separate the Gulf of Khambhat from the Arabian Sea, to create the possibility to generate power and to catch the fresh water from the Narmada.

1.1 Main structure

This report is divided into five parts. Part A provides general information about India, the Kalpasar project and the exact goal of this report.
The second part, (Part B), describes all the structures that are needed to close the Gulf of Khambhat, to generate power and safely catch the fresh water (Chapter 4-7), a model (Chapter 8) to predict water level fluctuations and current velocities (Chapter 9). Part C describes the actual closure technique. Part D describes the costs and the building schedules of the project (Chapter 14) and the risks involved with the construction of the closure dam (Chapter 15). Part E contains the conclusions (Chapter 16) and recommendations (Chapter 17).
Part F, the last part, contains the literature list and site visit reports.

2 General project information

2.1 *India*

India is a federal parliamentary democratic republic with New Delhi as its capital. The number of inhabitants is currently estimated at about one billion, who live on an area of 3,3 million square kilometers. With its high birth rate, it is only a matter of time until it becomes the most populous country of the world.
The national currency is the Rupee, at this moment the value of a Rupee is about 20 to one Dutch Guilder (NLG) or 38 to one US Dollar.
Great differences exist in economic standard. On one side it is a quickly developing country, with great expansion of industry and technology and with its own aerospace and nuclear program. On the other side numerous people are illiterate and live in poverty.

2.1.1 Climate

The monsoons or seasonal winds dominate the climate of India. In winter, under influence of an area of high pressure over central Asia, winds are mainly blowing from the north or northeast with a high degree of constancy: northeast monsoon. Temperatures in New Delhi reach only 25°C during this cold season.
Early in the summer, temperatures rise to 40°C and more. After June an area of low pressure is lying over the mainland of Asia and winds are mainly blowing from the southwest: the southwest monsoon. This southwest monsoon brings cloudy, unsettled and very damp weather, with heavy rain on the west coast of India, and occasional gales.

The rainfall in India is divided in a very unequal way. South of the Himalayas the total rainfall is more than 3000 mm, and locally more than 10000 mm. In the rest of India the precipitation is less than 3000 mm. In the northwest region the precipitation is even less than 500 mm, the Harr desert.
These values are average values.

2.1.2 Land utilization

Most of the utilized soils are used for agriculture. In India this means arable farming. Most important are rice and wheat. The kind of crop is determined by the precipitation, where rice is cultivated in places with the highest precipitation.

2.1.3 Gulf of Khambhat

The Gulf of Khambhat is situated in the western part of India, in the state of Gujarat. Gujarat is one of the more thriving states of India. Industries, trading and business are taking place, partly because of the good shipping facilities. Distance from Ahmadabad to the economical center of Mumbai is about 450 km from, while closer ports like Hazira are growing.

Gujarat has a tropical continental climate, strongly influenced by monsoon winds. July and August are months with regular rainfall, causing temporarily high discharges in the rivers with a high probability of flooding and erosion. In the remaining part of the year there is usually not much rain. The annual precipitation is about 1000 to 1500 mm. 80 % of this falls in the period July-September.

In the same period the wind, in the other months very weak, speeds up to an average of 10 m/s from SW, creating waves with an average significant height of 1,5 m. In the monsoon period chances of tropical cyclones exist. The chance of a wind speed of more than 30 m/s passing the Gulf is calculated at 2,6 % per year.

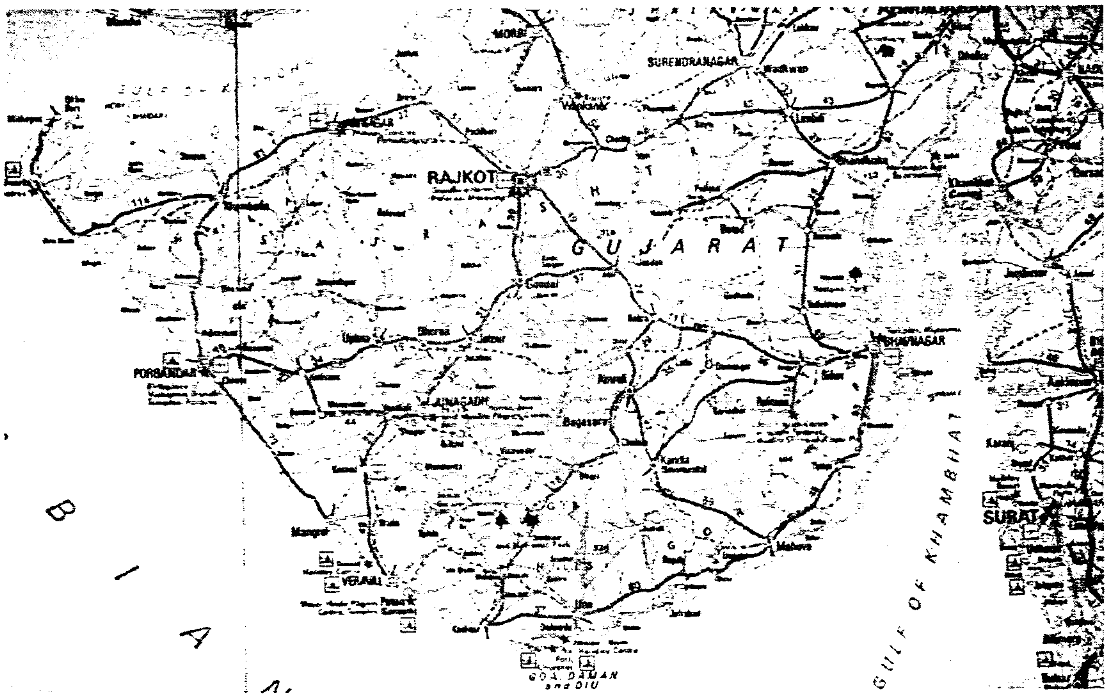


Figure 2.1 Map of Gujarat (scale 1:3200000)

2.2 *Kalpasar*

2.2.1 Historical background

Closure of the Gulf of Khambhat has been an issue since Wilson's study (1975) to the possibilities of tidal power. In that study, also the Gulf of Kutch (Gujarat) was suggested as a location for tidal power. Later, Khambhat was preferred because of some other developments that could take place simultaneously. Most important of these was irrigation.

In 1988 an agreement was signed between the Government of Gujarat, Water Resources Department and Haskoning Royal Dutch Consulting Engineers for the preparation of a reconnaissance report on the construction of a dam across the Gulf of Khambhat. In 1989 a report was presented which presented the technical viability and additional benefits.

In 1992 two students, P.L.M. Jansen and I.C. Vreeburg (Delft University of Technology), studied the tidal movement in the Gulf of Khambhat and made a design for the closure and tidal power station. In 1996 another student of Delft University of Technology, T.P. Hafkamp studied a quarry stone closure.

In 1998 Haskoning Royal Dutch Consulting Engineers presented a pre-feasibility study in which this so-called Kalpasar project was described technically and economically viable. This Kalpasar project, which stands for "a lake which fulfills one's wishes", aims at solving the four vital problems of the state of Gujarat in one mega-project. These are: fresh water, electrical power, road and rail transport and port development. Also possibilities of land reclamation, development of fisheries exist.

2.2.2 Scope of the Kalpasar Project

As has been mentioned above, Kalpasar is definitely a multipurpose project. Of the four vital problems, the shortage of fresh water and the (predicted) lack of electrical power are the most important. The area west of the Gulf is drying up. To obtain fresh water, groundwater is raised from great depths. This results in a (saline) groundwater flow from the Gulf inwards. This salt groundwater makes agricultural development impossible. A basin where fresh water is stored and used for irrigation can reverse this process. The fresh water comes from three rivers that flowing into the Gulf. These are the Mahi, the Sabarmati and the Narmada, of which the latter is by far the greatest. Only the east and south of Saurasthra (west of the Gulf) will be irrigated with water from the Kalpasar reservoir. The areas around Ahmadabad will be provided with fresh water from the reservoir behind the Sardar Sarovar dam. This dam is currently under construction, despite financial withdrawal of the World Bank. The directions in which fresh water will be transported are indicated with arrows in figure 2.2.

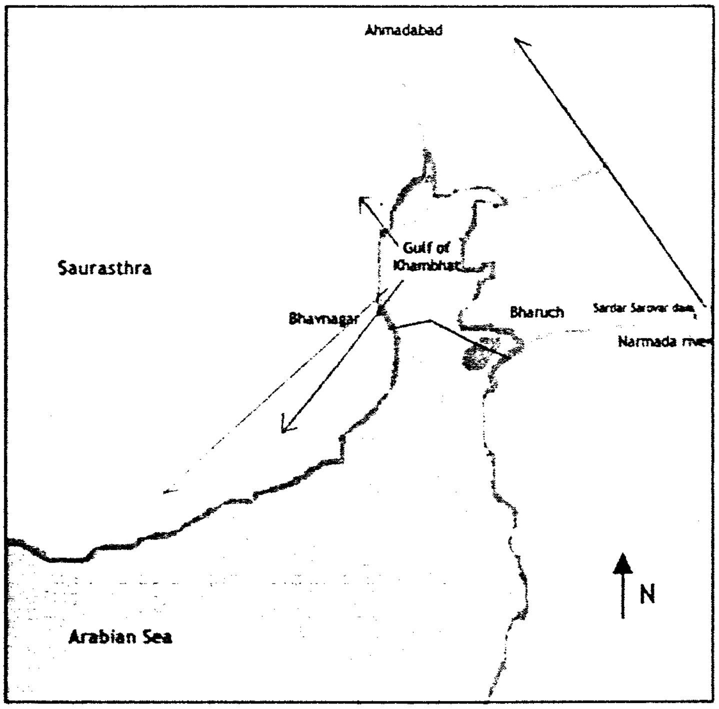


Figure 2.2 Distribution of irrigation water

Generating tidal power

Moreover, if the Gulf has to be closed off to create a reservoir, why not use a part of this reservoir to generate electricity out the enormous tidal range. This difference between high and low water has an average of about 8 m, but during spring tide it can reach a value of 10,4 m. The energy that is generated can be used for various developments, but also for pumping fresh water out of the reservoir and use it for irrigation.
From investigations by Haskoning an output of 5000 MW for a tidal area of 570 km^2 has been calculated.

Improving infrastructure

A dam across the Gulf of Khambhat reduces the distance from the economical center of Mumbai (Bombay) to Saurasthra considerably. There is no ferry service, so all transport has to go by road to Ahmadabad and onwards to Bhavnagar. A dam offers the opportunity to connect both sides of the dam to the existing road and railway network, saving 10 hours traveling time.

Port development in Gujarat
Ports can be developed close to the dam, both seawards or inside the reservoir. The ports that are currently in use (Dahej and Bhavnagar) are at the reservoir side of the proposed dam. This becomes a sheltered tide-free (only fresh water basin) area, but each ship will have to pass a shiplock. A new port on the seaside of the dam offers excellent infrastructure, but will have the disadvantage of the tide.

Other options as land reclamation, which has been done in the Netherlands, and development of fisheries and recreation can be useful in the nearby future.

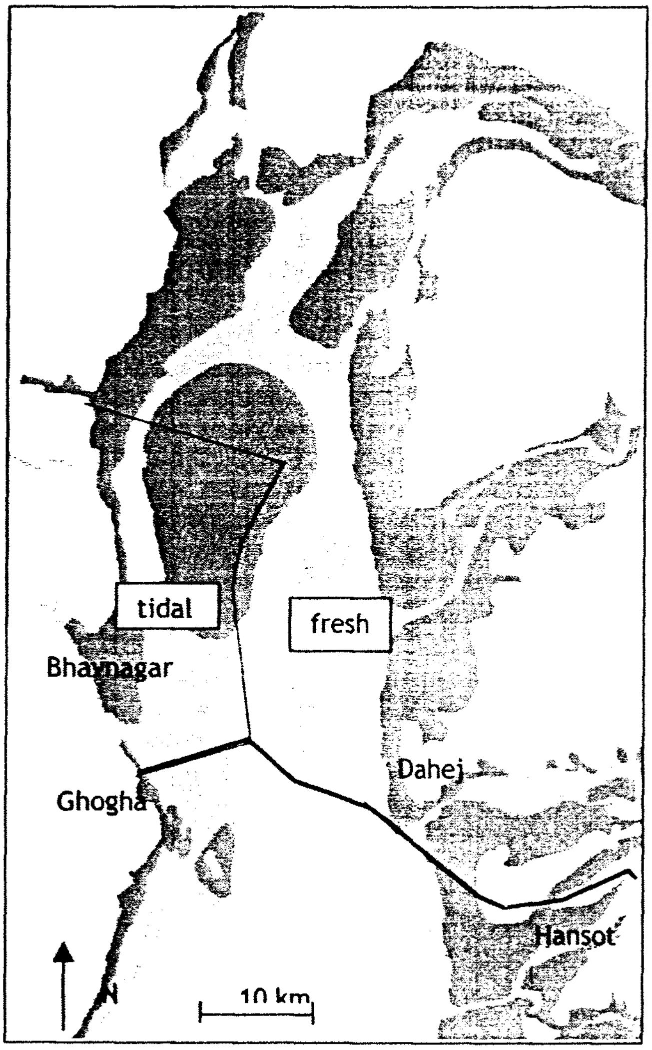
Consequences of the closure dam
The project looks very promising, especially because the reservoir does not require resettlement of people as has proved to be a terrible problem in the Narmada project.
However, the project has great environmental impact as it changes the Gulf from a saline estuary into a (partly) fresh water basin. This turn is expected to take about four years, ruining current wildlife in the Gulf.
Around the Gulf is a basic need for fresh water, benefits as recreation do not have priority.

It should be clear that such a mega-project needs tremendous investments, which need to be earned by selling electricity, water and land. Although Gujarat is one of the wealthiest and most developed states of India, the profits are not guaranteed.

2.3 Scope of this study

2.3.1 Functions of components

In the previous paragraphs it is stated that the two most important functions of the Kalpasar project are fresh water storage and tidal power generation. In these a contradiction can be seen, as tidal power generation requires a basin where seawater can flow in and out. This means that in fact two reservoirs have to be constructed, separated by a dam. The fresh water basin should be on the east side, as this is the place where the Narmada enters the Gulf. This locates the tidal basin on the southwest. The closure dam, which alignment is south of the mouth of the Narmada, borders both reservoirs. All this is indicated in figure 2.3.



In the damsection separating the tidal basin from the sea a structure has to be built with some sort of powerhouse and separate inlet sluices. These make it possible for the seawater to flow in and out the basin. This structure will be referred to as **tidal power facility**.
In the dam section separating the fresh water basin from the sea a structure has to be built that makes it possible to spill a surplus of fresh water into the sea. This can be a huge amount of water, as most rain falls in a short period, and in this monsoon irrigation is not needed. Although the reservoir is designed to *store* that water, there should be a facility to *spill* some of it, the **Narmada spillway.**

It should be possible to reach the Gulf of Khambhat by ship, especially since great investments have been done on the east side near Dahej. Recently a 2,5-km jetty (see figure 2.4) has been finished, while 2 others at almost the same location are currently under construction. Therefore shiplocks should be built between the sea and each basin.

Figure 2.3 Location of Kalpasar basins

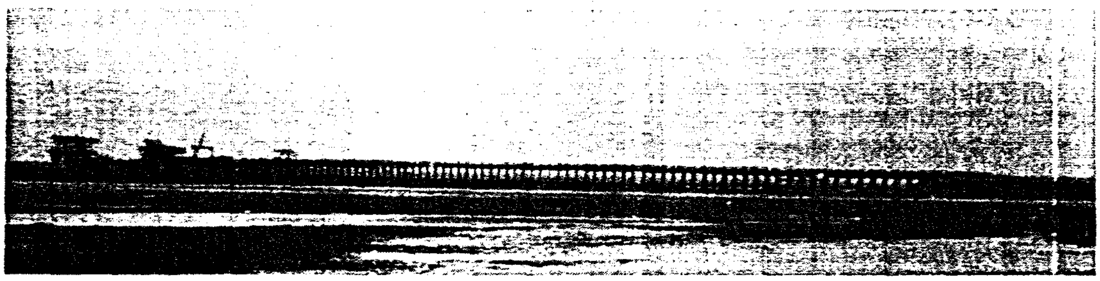


Figure 2.4 Dahej jetty

2.3.2 Problem description

In this study a closure method for the Gulf of Khambhat will be designed. Although this is only a part of the whole Kalpasar project, it faces probably the biggest difficulties of all: the extreme tide. The current velocities that develop when the Gulf is (partly) closed reach very high values. For example, when a dam is built up with sand, the velocities reach the value where all sand is immediately washed away when the gap has still a width of 20 km.
Several studies have been made, including a (sluice) caisson closure and a combined closure. The latter required stones of more than 20 ton for the final part.

A caisson closure has been used by Haskoning Royal Dutch Consulting Engineers to prove that closure is technically possible. In this design a combination of two caisson sections, with a length of 5 km each, and a stone closure in between those sections is proposed.
The bottom of the Gulf consists of sand and clay, which causes an additional problem. When the current velocities increase, the bottom will erode. This can have devastating consequences, as the scour of the bottom deepens the gap. The scour holes that develop can easily reaches depths of 30 m and more. This means that the bottom has to be protected.
However, this should be done in an integrated approach with all other works. Capacities of production are limited, and because other items use the same material (e.g. stones of 60 kg), the planning is crucial. Above all, the local circumstances require special attention. Mentioned before is the monsoon, which makes waterborne operations practically impossible. It should also be taken into account that in India materials like rock, sand, and clay but also concrete and steel are locally available and labor is (still) inexpensive.

2.3.3 Goal of this study

Goal of this study is to design an innovative closure method for the Gulf of Khambhat that is fast and as inexpensive as possible.
This study will not provide a final design, but only a rough design, not suitable for construction purposes.

2.4 Design methodology

This study can be divided in two parts. The first is a visit to the project area in India to investigate the local situation and to do several interviews. The goal of this inventory study is to investigate what exactly is desired in India and how the chances of realization are (at what period this project is expected to commence). A review of this visit can be found in annex 2.

The second part is the actual designing of the closure method. This is done in the Netherlands.
To ensure a structured study, a step-by-step approach is followed. This approach is described in figure 2.5.

Concepts of alternatives that have been generated during this study will be developed and adapted to meet the specifications. This process is repeated until the closure method is technically feasible and acceptable.

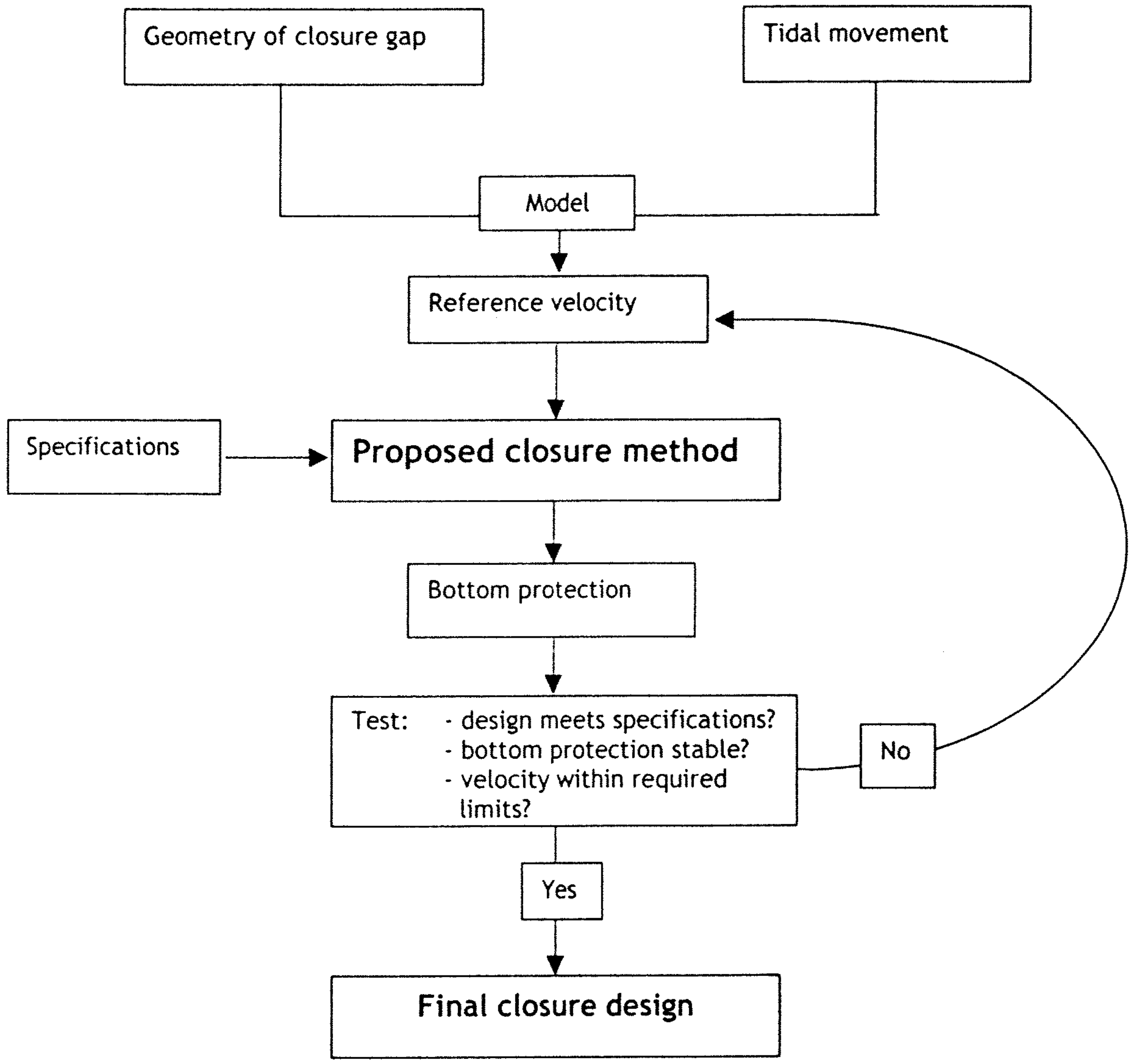


Figure 2.5 Design schedule

Out these alternatives the best will be chosen, by the following procedure.

Description of steps in scheme
According to the scheme above the following steps will be carried out for each design:

1. On a big scale a concept of the closure design is made. This is described in chapter 3, Design strategy.
2. Each conceptual design will be split into the most important components. This is described in Part B.
3. For each component alternative designs are made. These alternatives are worked out to obtain first dimensions and insight in building time, costs and risks. A storage area approach will be used to calculate current velocities. This is described in Part B and Part C (only final gap).
4. Out several different designs, the most promising are further engineered. A selection is made. This is described in Part B and Part C.
5. The selected alternatives of components are joined together into a total closure design.
6. For the two most promising designs a planning and cost estimate is made. This is done in Part D.
7. A risk analysis is made. This contains a sensitivity analysis of calculations to boundaries, sensitivity in planning, risk and damage Part D.
8. Finally, a choice is made between the alternative methods.

2.5 Boundary conditions and assumptions

2.5.1 Boundaries

- Mean Sea Level (MSL) is at Chart Datum (CD) +5,15 m, but for reasons of simplicity a MSL of CD +5 m is used in this study.
- The maximum tidal amplitude is 10,4 m. The four main tidal components are:

Tidal component	Amplitude in m (Bhavnagar)	Period
M2	3,14	12h25m
S2	0,96	12h
K1	0,76	23h56m
O1	0,34	25h50m

- Determined water area at Chart datum (CD): 630 km^2 (Kalpasar study);
- Determined area including all the sand and mud areas as indicated on the map: 2187 km^2 (MSL +15 m) (Kalpasar study);
- Maximum design Narmada discharge of 67000 m^3/s;
- Bottom geometry from Admiralty Chart no. 1486 and no. 3460 will be used;
- The dam will start at 21°48', from Howarth Point near **Ghogha** in the west, via the most Southern point of the Mal bank to the southern point of the Dhadhar river near **Hansot.** (See Admiralty Chart no. 1486). This implicates that the Narmada is enclosed by the dam;
- Tidal power generation is required;
- Representative wave height is 3,5 m with a period of 7 seconds. These waves only occur during monsoon, with a chance of exceeding of 2% per year;
- Combining tidal power and irrigation requires separate basins;
- Shiplocks are needed to connect the basins with the sea;
- To spill excess water during monsoon, a spillway is required.

2.5.2 Assumptions

- The area needed for power generation is at least 570 km^2;
- With a tidal area of 570 km^2, 5000 MW of tidal power can be generated;
- One turbine has a diameter of 9 m and produces 42 MW ;
- Average monsoon period is two months;
- Complicated waterborne placing and positioning is not possible during monsoon;
- The strength of the sand bottom is sufficient for founding concrete structures;
- The sand that can be found in areas indicated by S on the Admiralty Chart has sufficient quality for land reclamation;
- The clay on Alia Bet is suitable for dam construction;
- Effective working time during continuously operations is 20 to 21 hours per day, 140 to 147 hours per week (not 168 hours);
- The bottom material in the alignment is sand with a D_{50} of 200μm;
- The sand is not sensitive to liquefaction.

3 Design strategy

3.1 Introduction

The closure of the Gulf of Khambhat, at a scale unrivaled in the world, can not be done easily (fast and cheap) using traditional methods. The main dam has a length of more than 30 km, in an area with tidal differences up to 10,4 m and depths up to 30 m below lowest astronomical tide. A sand closure is out of the question, and stone or caisson structures cause design problems.

It is of greatest importance to reduce the scale of the problem without losing total overview. In this study an attempt is made to split the closure dam into smaller components. These components each have similar design problems, but on a smaller scale. Integrating these in a total design while making use of each alternative's advantages creates an innovative design where problems of one component are reduced by the advantages of another. Interviews during the visit to India learned that a tidal power facility was one of the most desired components.

The main components are the tidal power facility, the Narmada spillway, the final closure gap and the secondary damsections. Designs for these components are made in chapters 4-7. The final gap is further engineered in Part C.

3.2 Incorporation of tidal power facility

During an early stage the idea was born to incorporate the tidal power facility in the dam design. A tidal power plant requires a huge orifice to fill and empty the tidal basin and this orifice could be very useful to reduce current velocities during final closure. In fact, the tidal power plant is used as a large discharge sluice. Two completely different designs are made for this component, one in the very deep gully near Piram Island, and one caisson-based design in the dam alignment itself (see chapter 4). The latter uses extra large temporary gates during closure. This same concept was used for the spillway of the reservoir (Narmada spillway) which is needed to regulate the reservoir level.
In figure 3.1 arrows indicate the flows through the (temporary) openings. It should be clear that only one of the two alternatives for the tidal power facility will be chosen.

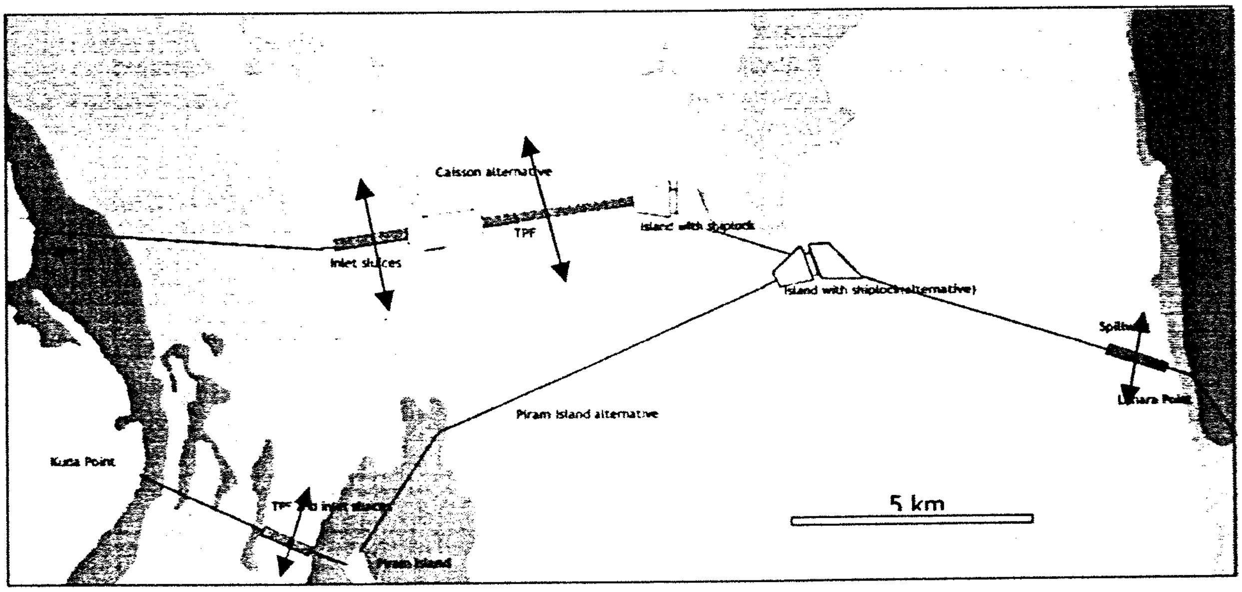
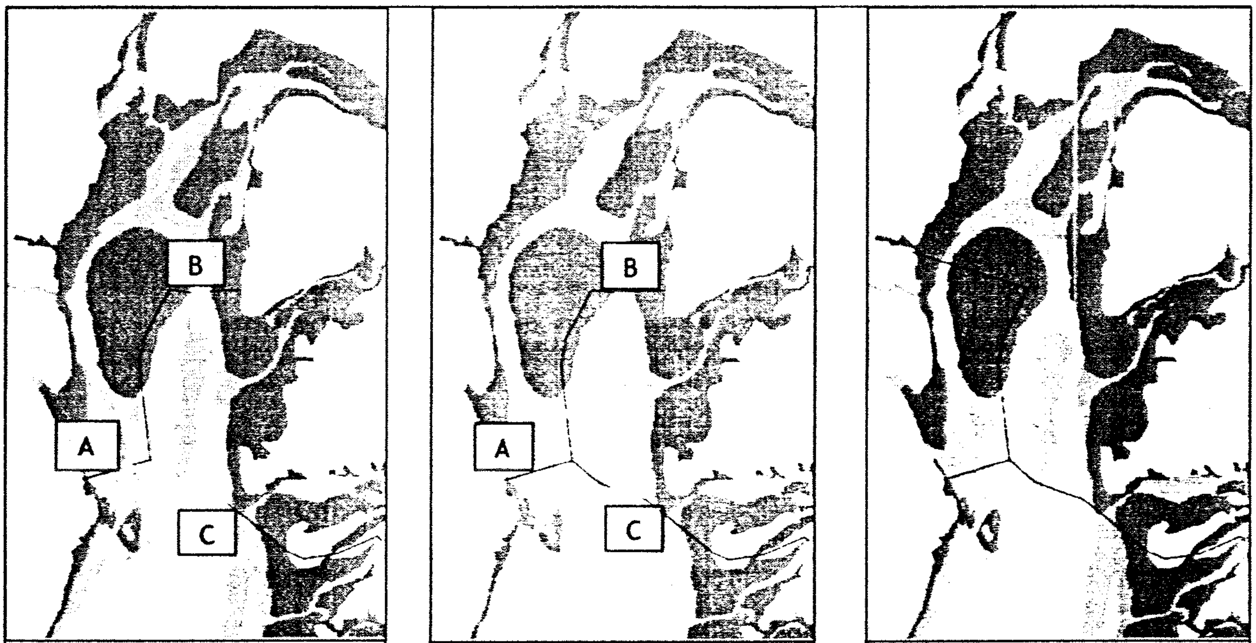


Figure 3.1 Flow through temporary openings

3.3 Split up strategy

The idea behind splitting up the reservoir into smaller sections is that the storage area (and thus the volume that has to be filled during a tidal cycle) will be reduced. This results in two closures, together easier than one big closure.
In the Gulf of Khambhat it is possible to make use of the two basins (tidal and fresh) that have to be built. Thus, a separation dam is needed anyway. When this dam is finished in an early phase, it will be possible to divide the basin in two parts, one with an area of 2/3 of the total area, and the other with the remaining 1/3. This leads to a situation (see figure 3.2) with a basin where 2/3 of the storage area has to be dammed by a dam with a length equal to the original (undivided) design. Besides that, use can be made of the tidal power facility to reduce flow velocities in the other gap.
The other 1/3 has to be closed off by a dam with half the length of the original closure dam.
It can be expected that, although the total length of the dams that have to be made is larger, the two closures will be easier to construct. Also the risks are expected to be smaller.



1-First closure | 2-Second closure | 3-Final situation

Figure 3.2 Split-up schedule

In Part B this alternative closure strategy will be calculated with the help of a storage area approach.

3.4 Final closure gap

Because the actual closure of the Gulf of Khambhat is the main problem, several alternative designs will be made, based on traditional and new techniques. These designs can be found in Part C.

3.5 Bottom protection

All these components need protection of the original bottom. Construction of each component has undoubtedly consequences for other components. Therefore for each component a bottom protection program is developed. This consists of a study of the maximum design values of the current velocities, whether the protection is temporary or permanent. Four types of protection will be chosen and adapted to each section.

3.6 Secondary damsections

The other parts in the closure dam, not being the Tidal power facility, Spillway, islands or final gap (10 km), are referred to as secondary damsections.
Most of these secondary damsections, with a total length of about 40 km, can be built in relatively shallow water.

Part B Components

In Chapter 3 (Design strategy) is explained that the total design in this study is divided in components, and also the main components and their functions are described. In this section all components are engineered further. A prelimenary design is made of a tidal power facility (chapter 4), the dimensions for the Spillway are determined (chapter 5), the secondary damsections are designed (chapter 6) and the functions of bottom protection are described (chapter 10).The closure of the final gap is not described here. Only the dimensions and an introduction to several closure techniques are given (chapter 7). In Part C the closure of the final gap is discussed. In the figure below the locations of all components in the alignment are indicated.

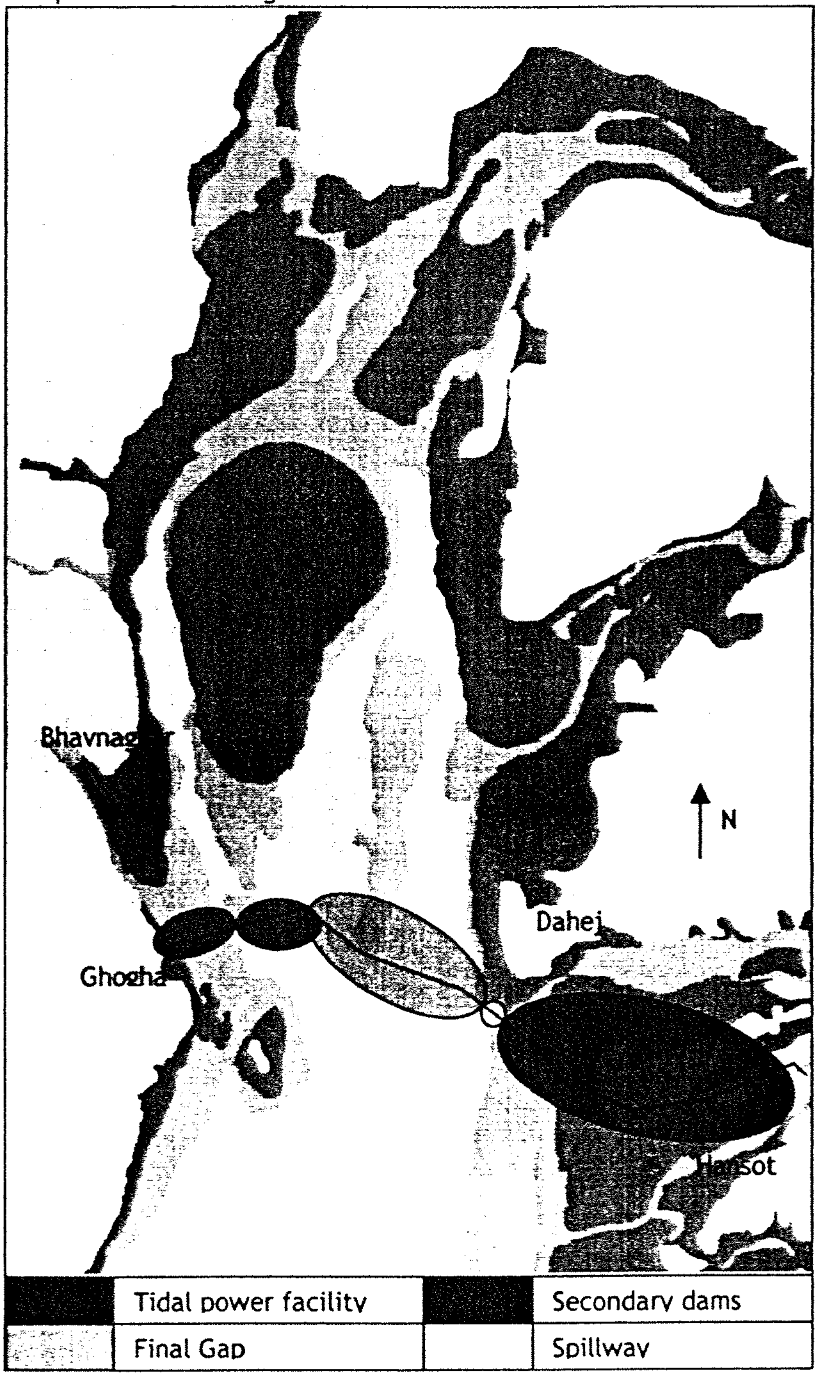


Figure 4.1 Location of components

4 Tidal power facility and spillway

4.1 Introduction

From the beginning one of the main issues of the Kalpasar project has been the possibility to produce electricity out tidal power. The Gulf of Khambhat is one of the few locations all over the world where tidal power development should be possible, this because of the high tidal difference (due to the shape of the estuary, which amplifies the amplitude), the large tidal prism and the great (future) demand of India. Moreover, there is not a lot of nautical traffic in the Gulf.

In this chapter the various types of tidal power facilities will be discussed. It should be known that there are only few tidal power stations worldwide, of which the tidal power facility in La Rance is the only non-experimental one.

Use has been made of 'Dictaat Energiewaterbouwkunde (1996)' and 'Jansen & Vreeburg (1992)'.

4.1.1 Power and production

Comparable to generating hydropower in a tidal power facility the potential energy of the water is used. With help of the following formula the amount of power (P) is given:

$$P = \eta \times \rho \times g \times H \times Q$$

In which:

P = generated power (Watt)
η = efficiency
ρ = density of water (kg/m^3);
g = gravity acceleration (9,8 m/s^2);
H = head difference (m);
Q = discharge of water (m^3/s).

To generate power, it is necessary to store the in flowing water until the water outside the basin is considerably lower. Then the water flows back through the turbines before the water rises again.

The difference with conventional hydropower is clear:

- H varies with the water level outside and inside the basin. This variation is caused mainly by the cycle of 12h25min and the fortnightly cycle;
- Q is depending on the tidal area, tidal range, capacity of structures and time available for filling;
- The minimum and maximum water level varies daily;
- Power can only be generated during a limited period.

4.1.2 Types of tidal power facilities

In a tidal power station three modes of operation are possible to generate energy. These are:

- Ebb-generation
- Flood-generation
- Two-way generation

4.1.2.1 Ebb-generation

In the ebb-generation mode sluices are opened on the flood tide allowing the basin behind the dam to fill (indicated with F in figure 4.2). On the ebbing tide, this retained water is used to produce power with the turbines until the operating head becomes too low (indicated with G). The turbines are then closed down until the rising flood reaches the level of the drawn-down basin. Then the cycle restarts. This mode of operation has been shown to be the most cost-efficient method of tidal energy production as it combines high energy productivity with comparatively low-cost turbines and sluices. In figure 4.2 the reservoir is kept at the maximum level until a higher head difference is obtained (indicated

with S). It is clear that the larger the sluice gates will be, the higher the reservoir level will be, and therefore more power will be generated.
It is possible to increase the power generation by pumping water into the basin just before the generation period. This increases the head difference. In the particular case of the Gulf of Khambhat the tidal area might be too large for doing this.

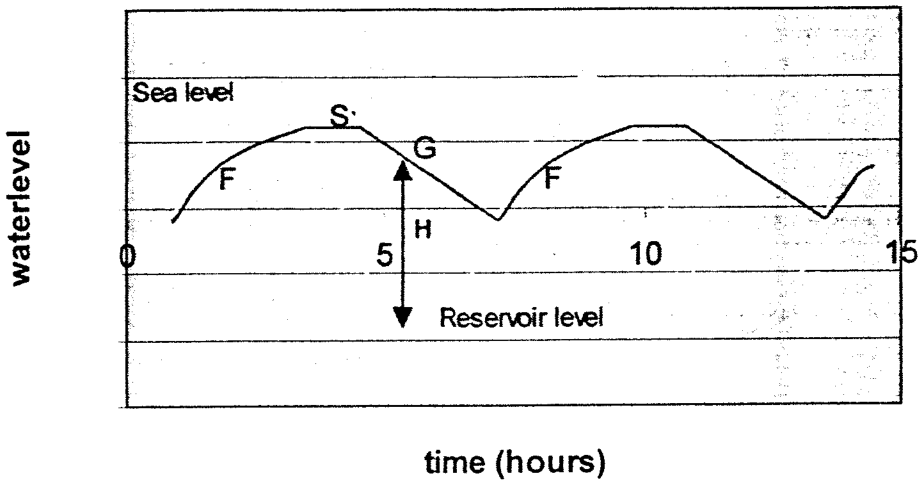


Figure 4.2 Principle of energy generation

4.1.2.2 *Flood-generation*

In the flood-generation mode the direction of flow through the turbines is reversed. This results in a reduced energy output when compared with ebb-generation. The reason is that the basin is used in a permanently drawn down condition, which reduces the basin volume. The reduction in energy output depends upon the character of the enclosed basin, but may be around 5 to 10%.

4.1.2.3 *Two-way generation*

Combining these modes seems logical and has two advantages:

- Power is produced during a longer tidal window, which meets electricity demand better;
- Since the generating head is generally reduced, delivery of large 'blocks' of energy into the electricity network is avoided. This is characteristic for single-effect operation and causes problems inside the power system.

Despite these advantages the turbine-efficiency is substantially less than for a one-way system. This is caused by compromises on blade design, water passage geometry and distributor positioning which have to be made to allow the turbines to operate in both directions.

Furthermore, it is possible to generate energy over the whole tidal cycle if there are two tidal basins available (see figure 4.3).

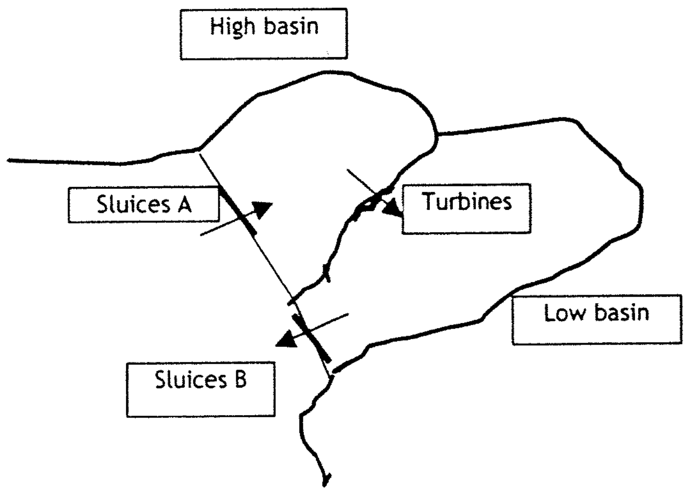


Figure 4.3 Two-basin system

A high basin is filled through the intake structures 'A' during the rising tide until some time after high tide. When during ebb the water level at sea has the same level as in 'the high basin (point C in figure 4.4) the intake structures are closed. During the whole tidal cycle the water flows from basin 'A' through the turbines to the low basin. Therefore the water level in the low basin rises. When the level at sea drops below the level F the sluices in the low basin are opened. These sluices are closed again when the level at sea has risen to the water level of the low basin. In the meantime water keeps on flowing through the turbines from the high to the low basin. When at sea point D is reached the cycle restarts.

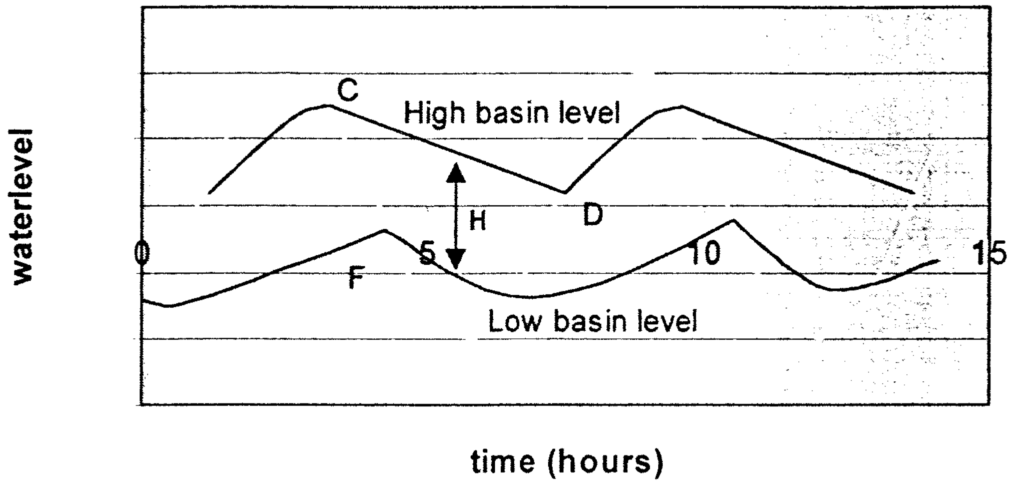


Figure 4.4 Principle of energy generation for a two-basin system

There are two major drawbacks for this scheme:

- Two basins and two sluices cause higher costs;
- Only 50% of the energy that can be generated with an ebb-generation mode can be generated. Production is continuously, but the amount of power still varies with the head difference over the powerhouse.

It seems that especially the higher costs of creating two basins (with the required high capacity) will not outweigh the benefits of continuity for the Khambhat power facility.

4.1.3 Required turbine orifice

Interviews in India learned that it is desirable to produce as much energy as possible. This means that, instead of the 1500 MW schemes that have been investigated earlier, there is a demand for a 5000 MW scheme (570 km^2 basin, Haskoning). 6200 MW is also possible but requires a 2187 km^2 basin, thus no fresh water storage.
When 40 MW generators are used (the highest capacity known), 125 to 155 turbines are needed. Assuming a diameter of about 9 meters, this leads (with good hydrodynamic shape so the contraction μ is minimal) to an orifice of 8000 to 10.000 m^2.
Length of the total structure (including sluice gates) will then be around 5,5 to 6 km.

4.1.4 Costs of tidal power facility

This will be the largest tidal power facility in the world, making it impossible to compare the whole structure to existing ones. Therefore it will be very difficult to estimate the costs. This will be done when the dimensions have been calculated roughly and with reference to the Haskoning design.

4.2 *Local situation*

In the inventory report it is stated that the tidal power facility in the Khambhat dam should have a generating capacity of 5000 MW, with a tidal area of 570 square kilometers. Also mentioned is the suggestion that the flow area of the tidal power facility should be used during closure, to reduce current velocities.

It is estimated that a state-of-the-art bulb turbine unit (same type as used in La Rance) can produce up to 40 or 50 MW. The diameter should be about 9 meters.
For the Khambhat Tidal power facility, a number of 120 turbines is chosen.

Two locations have been developed.

- The first is located in the dam alignment, north of Piram Island. Here the depth of the gully is 20 to 25 m below CD. The bottom material consists mainly of sand.
- The second is located in the deep trench on the West Side of Piram Island. The deep water is advantageous for the turbines. The bottom material, rock, requires no bottom protection and makes a sound fixation possible. On the other hand, construction in 70-100 m deep water can be hazardous.

Both locations are indicated in the following figure:

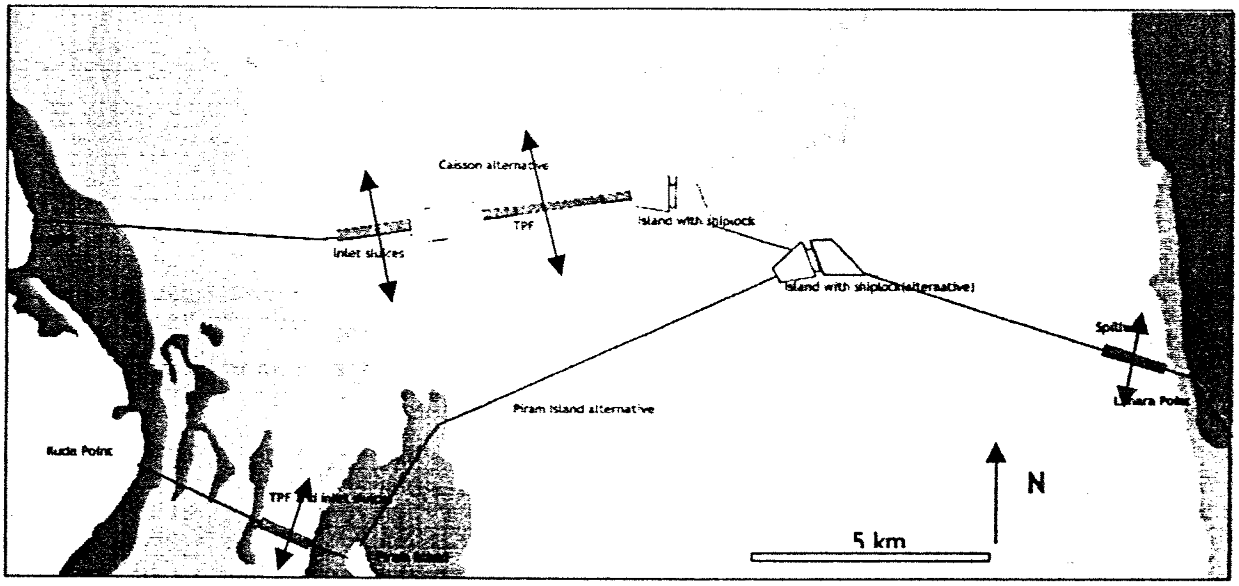


Figure 4.5 Location of Tidal power facility alternatives

4.3 Caisson alternative for tidal power facility

4.3.1 Introduction

The design for the first location is a caisson building method. Sections of 4 turbines are prefabricated as a floating caisson of about 100 m long and 50 m wide. Temporary sluice gates are constructed on top of the turbine sections, to ensure the greatest possible temporary flow area.

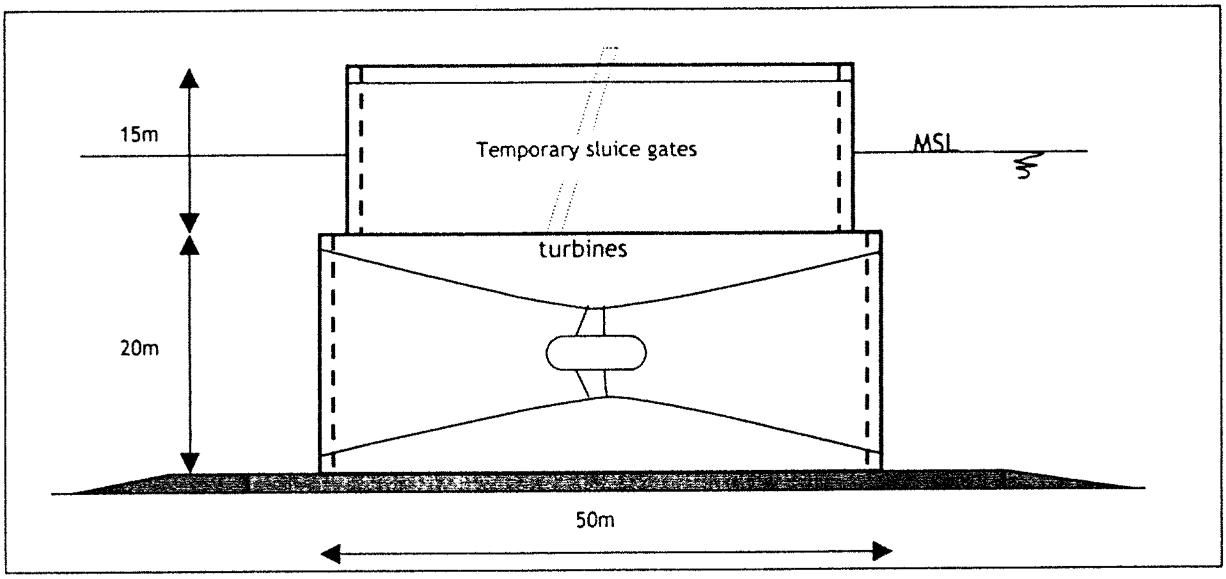


Figure 4.6 Turbine caisson

The flow area can be divided in a permanent and a temporary area. Permanent are the turbines and the sluice gates (the latter will be used to fill the reservoir in operational situation). Temporary are the sluices that are placed on top of the turbines and the sluices.

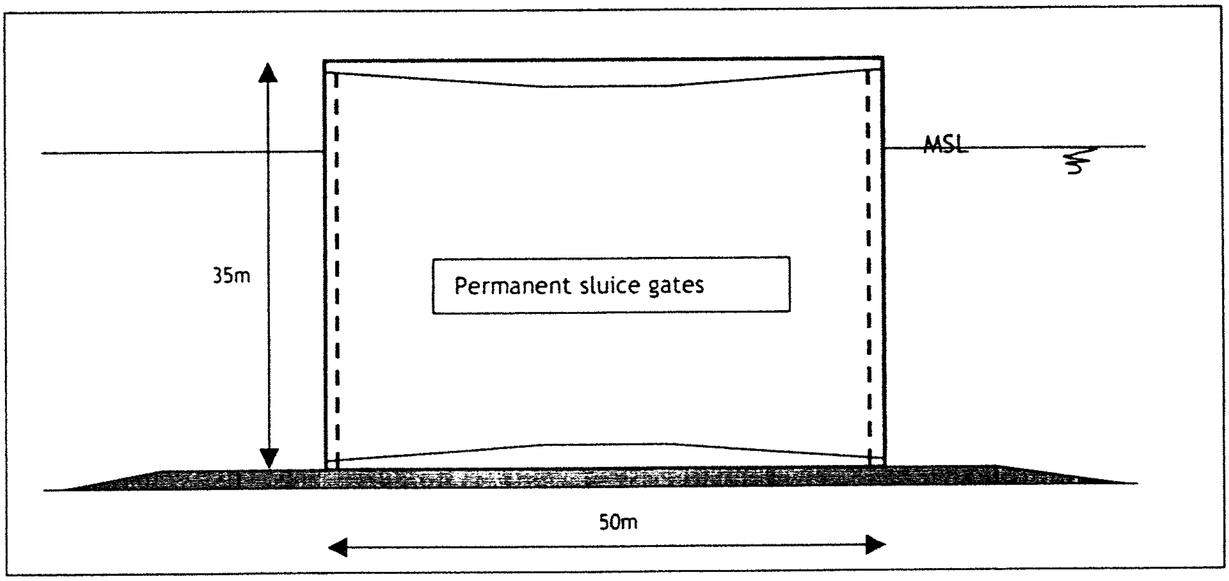


Figure 4.7 Sluice caisson

4.3.2 Estimated dimensions

Turbine diameter is 9 m. Turbine section will have a height of 20 m and a width of 25 m, including walls. Temporary sluice gates are 15 m high, including floor and roof. The effective flow height will be 10 m maximum.

The permanent sluice gates will have a height of 20 m and a width of 25 m, including walls. It is suggested to construct caissons with 4 sluice gates each. The required total length of the sluice gates is about 1000 m. However, extra capacity is advantageous during closure and also when operational. Moreover, it can be useful during maintenance. Therefore a length of 1500 m is chosen, in which 60 gates are placed.

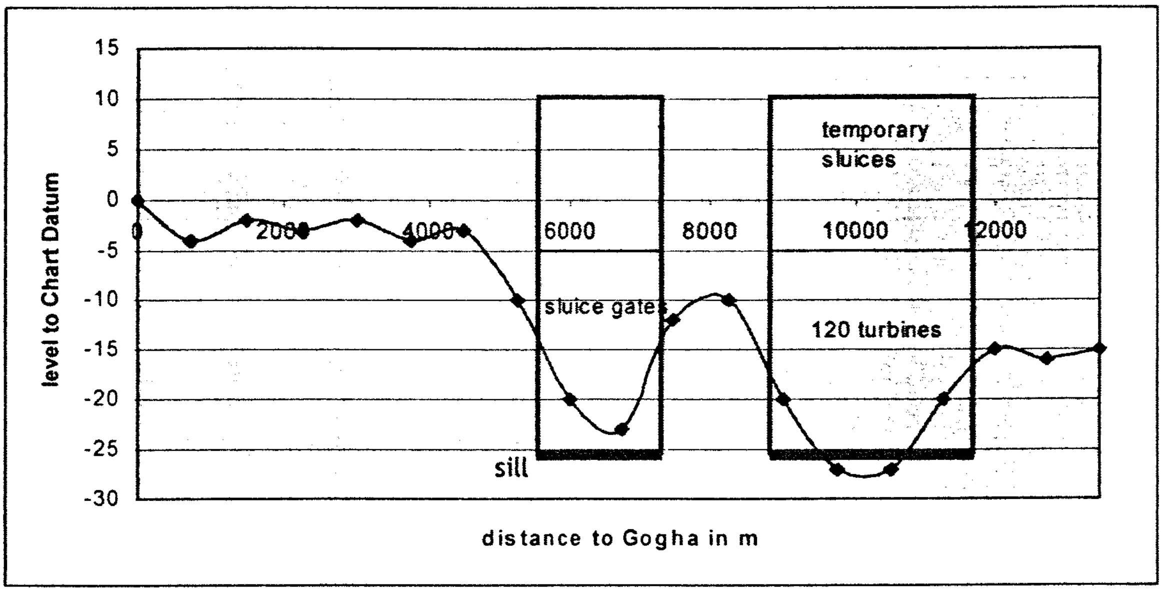


Figure 4.8 Position of caissons in profile

4.3.3 Orifice dimensions

Turbines: $0{,}25 * \pi * 9^2 * 120$ = 8000 m^2.
Temporary sluice gates below CD: 20 * 5 * 120 = 12000 m^2.
Temporary sluice gates above CD: 20 * 10 * 120 = 24000 m^2.

Permanent sluice gates below CD: 20 * 20 * 60 = 24000 m^2.
Permanent sluice gates above CD: 20 * 15 * 60 = 18000 m^2.

The caissons are placed in the gully at CD -25 m. The height will be about 35 m, which results in a top level of CD +10 m (MSL +5 m). When placed, the level will be raised to final dam height.

The Tidal power facility will not be completely finished directly. Once the caissons have been placed the tidal power facility serves as a temporary sluice caisson in the closure process. When the actual closure is completed, the gates of the tidal power facility will be closed. Then (in dry conditions) the installations (turbines, generators etc.) will be installed.

4.3.4 Placing of tidal power caissons

4.3.4.1 Introduction

In previous sections it is stated that the placing of caissons for the tidal power facility is one of the first tasks that have to be carried out. When bottom protection is placed and the construction of work islands is in progress, the gap width is about 14 km. The maximum current velocities (calculated with the storage area approach) are 2,2 m/s (spring tide, tidal range of 10,4 m). With a range of 7,5 m (mean tide) the maximum velocity is 1,5 m/s.

As these caissons are placed in the deepest gullies, the values will be higher, estimated 20%. This leads to 2,7 m/s for spring and 1,8 m/s for mean tide.

When a current velocity of 0,5 m/s is accepted as a maximum for final placing and sinking of a caisson, the tidal window is about 1 hour. This is the available time around slack water, where the velocities are below 0,5 m/s.

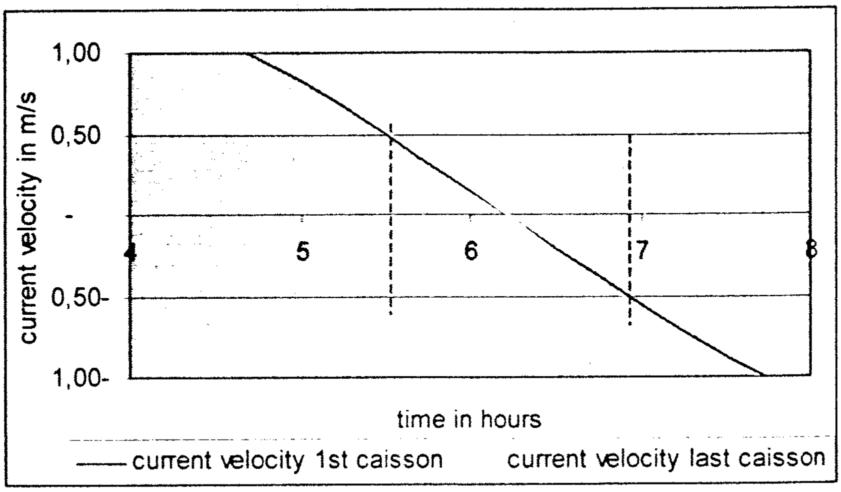


Figure 4.9 Tidal window for final placing

4.3.4.2 Placing restriction by current velocities

The placement operation can start when the current velocity is about 2 m/s and has to be finished when the velocity is about 0,5 m/s in the opposite direction (Closure of tidal basins, p 639). This means that the caissons have to be towed from the construction site to an anchoring site from where the actual placing can begin. The tidal window for the transport, placement and sinking operation is 2h20min under spring conditions and not limited during mean tides.

This may seem strange, but it should be kept in mind that this is not a 'conventional' caisson closure where the final gap is closed with caissons. Here the caissons are placed much earlier, almost in an undisturbed area.

4.3.4.3 Restriction by available depth

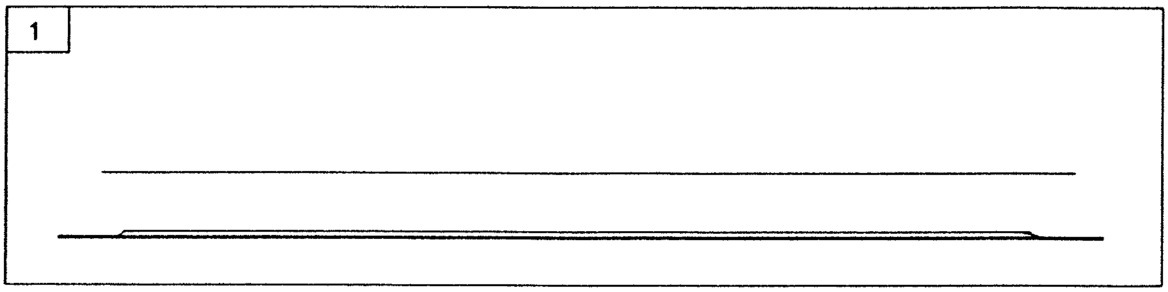
The draught of a tidal power facility caisson is calculated at some 25 m (when the roofs are placed afterwards). It is therefore crucial that the route from the construction site to the final location is sufficiently deep. Depths around the gully are some 20 to 30 m below CD. This means that dredging will be needed in the vicinity of the placing location to ensure that transportation can take place anytime. Then it is suggested to sink the caissons at HW slack. When this would be done at LW, the space between the sill and the bottom of the caisson will be very small.

It seems best to sink the caisson during HW slack. The process is controlled best and the risk of touching the sill too early is avoided. The actual sinking procedure takes about 15 minutes.

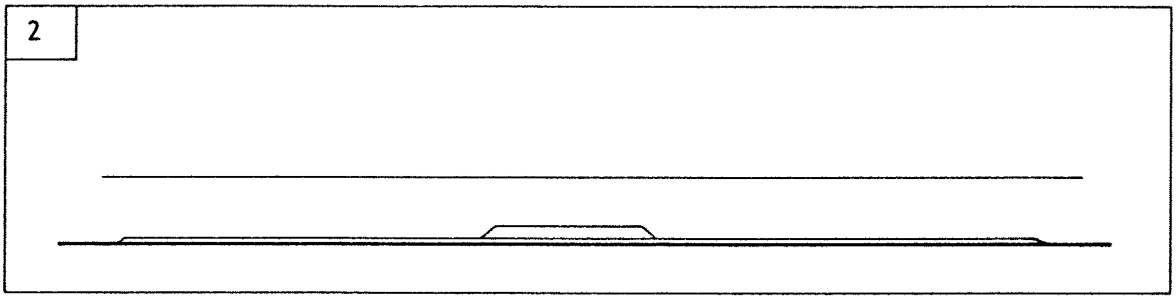
4.3.4.4 Sill for the caissons

The caisson will be placed on a previously constructed sill. Because the caisson has to be placed as deep as possible this sill should not be very high. A height of 3-4 m is suggested, on a thin bottom-protecting layer of 0,5 m.

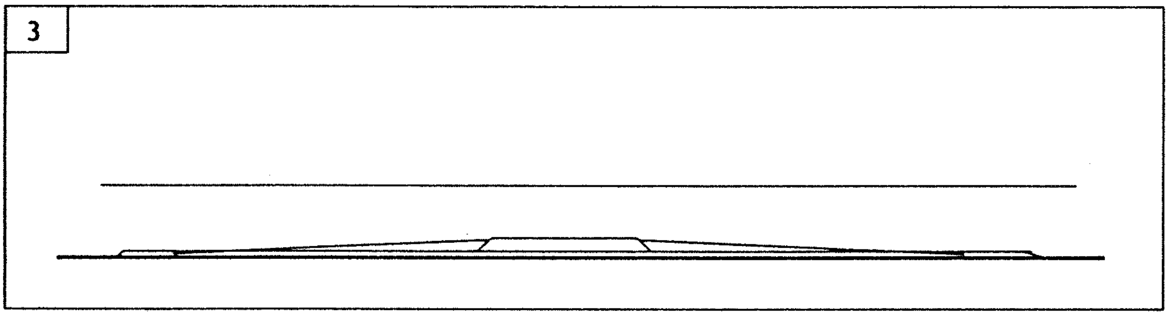
To withstand the extreme load from the current during the final stages of closing, the bottom protection close to the caissons should be considerably stronger. The following procedure is recommended. The drawings, and especially the sill height, are not to scale.



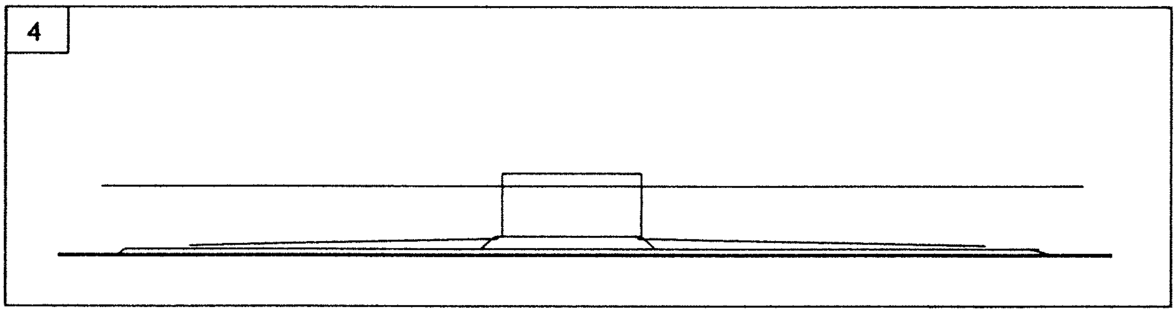
Placing of bottom protection. This will be a thin construction. The purpose is to fix the bottom while working on the sills.



Constructing of the sill on which the caissons are actually placed. This is the permanent foundation of the tidal power facility, and therefore has to be done precisely.



Placing final bottom protection. This is the permanent bottom protection that has to resist the forces of the current in both closing and operational mode.



Placing of caisson on top; afterwards finishing foundation and connection between caisson and bottom protection. A crane that operates from the caisson can do this.

4.3.4.5 Placing and foundation of the caissons

After the caisson is sunk into position it is necessary to fix the caisson (by filling hollows with sand or concrete) to ensure that the placing is permanent.
For the final placing the most commonly used option is a three-point support. This means that the caisson will be placed on three points to avoid torsion forces in the caisson. The exact positions of these points are known and by using special placing equipment, unequal settlement of the caisson (which would cause either direct damage or an unwanted unequal placing) can be avoided. After placing the caisson, the space between the caisson and the sill can be grouted.

A problem that is encountered is flow under the caisson. As both the sill and the sandy bottom are porous, flow will inevitably exist. However, there is a difference between piping, which is not allowed anyway, and some porous flow through the sill itself due to head differences when the tidal power facility is operational. This will not endanger the structure as a whole, but it will somehow reduce the efficiency of the turbines.

Piping occurs when the piping length is to short for the occurring gradient, and can be avoided by increasing the piping length. The gradient is the head difference over the structure divided by the watertight length. As the width of the caisson is approximately 50 m, and the head difference will be maximum 10,4 m, the gradient is 10,4/50 = 0,2. This large gradient can cause problems. However, during normal operation this gradient does not occur. A head difference of 10,4 implicates spring flood and the lowest possible water level inside, or vice versa. This situation will almost never occur.

If the occurring gradient is too large, the gradient should be reduced. Extending the watertight length vertically, by an impervious core below the caisson, or horizontally by an impervious bottom protection (asphalt) can reduce the gradient. Prevention of piping is not further studied.

4.3.5 Financial aspects of the caisson tidal power caissons

Using a caisson structure for a tidal power facility has never been done before. However, the most important phase in constructing (positioning of the tidal power facility elements), can be seen as a combination of techniques from tunnel and closure caisson construction.
In the planning of the works it is crucial that the structure of the tidal power facility is finished before the actual closure works begin. This means that the constructing time will be large. The building (and placing) time are estimated at 3 years. In those years preparation works and bottom protection of the rest of the works take place.

4.3.6 Risk

Although the combination of techniques (turbines in caissons) is totally new, the techniques are not. The sinking and placing of caissons has proved to be a reliable technique.

4.4 *Tidal power facility Piram Island*

4.4.1 Introduction

Theoretically, the best natural location for tidal power generation is in the deepest gullies. The deeper the **turbines** are placed, the more efficient they work because cavitation on the turbine blades is less at greater depths.

The proposed alignment crosses the gullies at the shallowest points. However, quite close to the alignment a small island is located, Piram Island. In between this island and the coast of Saurashtra lies a gully with its deepest point 104 meters below CD (Admiralty Chart 3460). The shape of this gully (steep slope), and the nearby rocky reefs and island, suggest that the bottom of this gully does not consist of sand, but probably of rock.
Presence of rock would mean that **bottom protection is not necessary**, which would be an enormous advantage.
It is however also possible that the bottom material in this gully is something between rock and loose sediment. Laterite for instance, occurs in Gujarat and could also be the bottom material. In that case, a chance exists that the bottom cannot withstand the forces of the water during the operation of the Tidal power facility. There is also a chance that the bottom cannot withstand the vertical forces of the construction and fluidize (when wet, fine laterite behaves like clay). Soilmechanical investigations should clarify this.
For the time being the bottom material is assumed to be rock.

4.4.2 Location of Facility

The chosen location is the place in the gully where the width of the section deeper than 50 m is at its maximum. That width, according to Admiralty Chart 3460, is 750 meters. At CD the width is 1100 meters. The deepest point at this location is not known exactly, but estimated at CD -80 m. With these facts the following profile can be drawn:

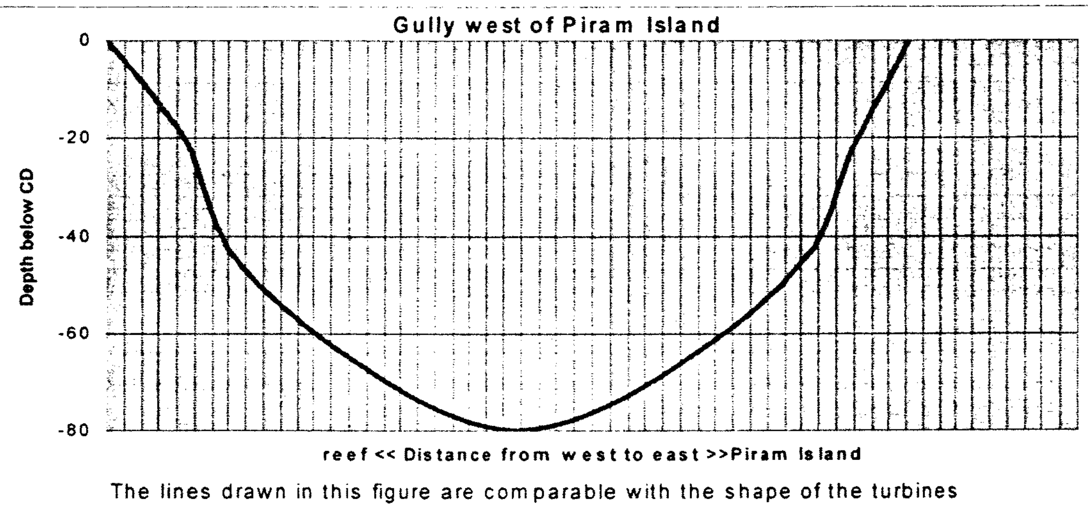


Figure 4.10 Gully west of Piram Island

4.4.3 Turbines

It is stated earlier that the proposed turbines have a diameter of 9 meters and require a free space from 20 m * 25 m (height * width). For the proposed power production 120 turbines are needed. This will lead to the following results:
It is possible to place 30 turbines in a row of 750 meters. This requires four rows of turbines placed above each other. In the top row, the depth is the minimum required depth to prevent cavitation.
From the profile determined with AC 3460, it seems possible to place a few turbines outside the 750 meters. There will be at least space for 20 turbines.

4.4.4 Construction of the tidal power facility near Piram Island

For construction several options can be considered:

1. Constructing the structure elsewhere. For example: building of large caissons in a building pit. Since the depth of the Gulf does not allow the transport of an 80 m deep caisson, the caissons should be built on its side. This implicates a rotation operation at the location where the caissons will be placed.

2. Constructing the structure at the location, in a building pit. A very large building pit is needed. Disadvantages of such a building pit are:
   - The high vertical walls. It is recommended to construct the walls of the building pit only in shallower water. This avoids the high walls, but requires a very large building pit;
   - The enormous dry area, requiring very much pumping capacity; The high water pressure on the rock bottom, with serious risk of collapsing;

3. Constructing the structure at the location. Without a building pit the high current velocities make working impossible. But when a simple dam blocks the flow, the working conditions will be much better. This temporary closure dam in the gully should be as cheap as possible. This dam can be a rockfill dam, still being a bit porous, only a bit higher than the maximum water level. Behind this dam an almost current free area is created, the tide however is still present.
   The Tidal power facility can now be built using the technique used to construct large gravity offshore constructions. This means a floating construction yard around the tidal power facility, comparable to the construction of the Troll platform (Norway). The construction slowly sinks deeper in the water when it becomes higher. At a certain critical point, the construction can be placed on the bottom without a difficult placing operation.
   This operation is very critical. When placing fails, it is simply not possible to rebuild or even remove the structure. This means that a whole new design will have to be made for another location. This implicates that the costs will be more than double.

- This method will be complicated and probably more expensive than the other two options. The advantage is that some experience exists with this kind of method.

The methods should be studied in more detail to be able to recommend the best.

4.4.5 Dam alignment with the tidal power facility near Piram Island

Placing the Tidal power facility west of Piram Island requires a change in the dam alignment. There are two options:

- A direct line from Piram Island to Alia Bet;

This alignment has the disadvantage of crossing another very deep gully, east of Piram Island, with depths up to CD -50 m. This requires enormous amounts of material.

- Optimal connection with the existing alignment

In this case there will be a junction made from Piram Island to the existing dam alignment. This junction has to be as cheap as possible (=short and through shallow water). By following the Kalpasar alignment as much as possible it is guaranteed that it is the most economical alignment. This option is recommended.

4.4.6 Inlet sluices in the Piram Island tidal power facility

Another point of attention is the necessity of inlet sluices in the alignment. It is possible to place the inlet sluices in the gully west of Piram Island on top of the turbines. But there is no space for temporary sluices that are necessary for the proposed strategy.

4.4.7 Costs

It seems that the direct costs of this tidal power facility itself will be higher than the costs of the Tidal power facility with caissons. The advantage from this tidal power facility should be the better energy production, better foundation possibilities and the absence of bottom protection.

4.4.8 Shape of the structure

Concrete structures at a depth like used here, cannot have flat walls. The shear force will reach very high values, requiring very thick and reinforced walls. Cylindrical shaped structures are not opposed to shear forces, but to pressure, offering the possibility to construct without reinforcement. Therefore it is recommended to use round shapes that only have to withstand pressure forces. This results in the basic shape for the concrete around one turbine as drawn in figure 4.11.

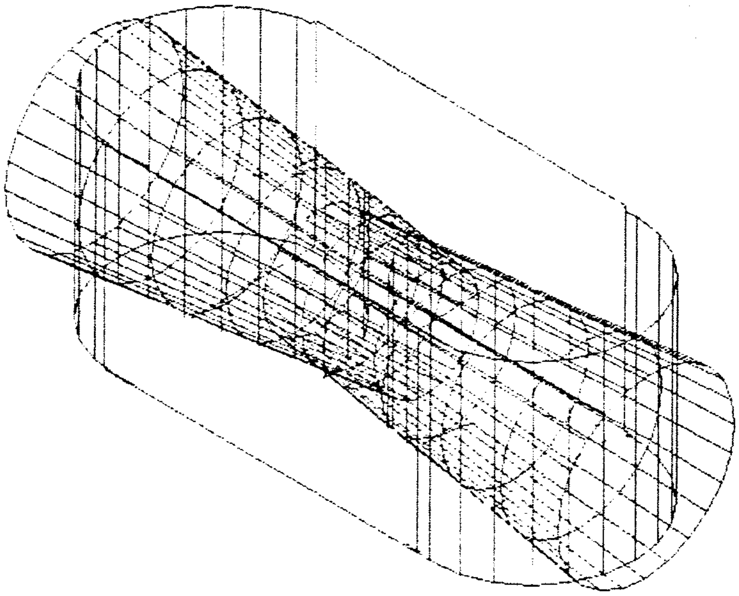


Figure 4.11 3D view of turbine section

These structures are not built individually but are the components of the bigger structure. For one turbine unit around 8000 m^3 concrete is necessary. 120 Turbines are needed (8000 * 120 = 960.000 m^3) and a top structure from approximately 1100 meters long and 20 meters high, two walls and a roof of 1 meter thick (1100 * 20 * 2 * 1 + 1100 * 25 * 1 = 71500 m^3). For fixing the whole structure in the gap (connecting with rock, connection with bottom etc.) a volume off 1100 * 2 * 50 = 110.000 m^3 is taken (maximum width gap * length of Venturi tubes* arbitrary thickness). The total volume of concrete needed for this tidal power facility is about 1,1 million m^3.

4.5 *Choice of Tidal power facility*

Comparing the two alternatives, the tidal power facility in caissons, or the tidal power facility in the deep gully west of Piram Island, leads to the following:

Building phase

- The building method at Piram Island will be very unconventional;
- The construction of the tidal power facility at Piram Island will be more expensive than the construction of the tidal power facility with caissons in the Kalpasar alignment;
- The caisson method will offer a large temporary flow area, which is favorable for the rest of the closure dam;
- The tidal power facility near Piram Island does probably not require bottom protection. This depends on the bottom material, which is assumed to be rock.

Operation phase

- Piram Island is a better location for most of the turbines, deeper means less risk to cavitation. This means better electricity production;
- Before operating the tidal power facility much work has to be done in the caisson alternative (breaking walls and building walls in the temporary gates. In the Piram Island Tidal power facility only the turbines have to be placed;
- The tidal power facility near Piram Island requires less bottom protection. There is no expensive bottom protection needed in the front and on the backside of the tidal power facility.

The possibility of the caisson alternative to provide a temporary orifice has a high priority in this study. This is not possible with the Piram Island alternative. As this temporary orifice will undoubtedly ease the rest of the closure, the caisson alternative is chosen.
Moreover it can be concluded that, however promising the alternative at Piram Island seems, the many uncertainties in this design point out that building a Tidal power facility with caissons is much safer.

5 Narmada spillway

5.1 Introduction

Besides the tidal power facility, at least one other important structure has to be designed. It has been mentioned before that rainfall and water from three major rivers, of which the Narmada is by far the biggest, fill the reservoir. Irrigation is one of the main issues of the Kalpasar project, besides land reclamation and port development. Therefore, it is clear that the level inside the reservoir has to be regulated by discharge sluices.

5.2 Discharge sluices

The main function of the discharge sluice is to pass excess water from the basin into the sea after the completion of the closure dam. Not only will it be necessary to maintain the water level in the basin within certain limits, but irrigation schemes upstream of the sluices will also require the proper regulation of the water levels in the basin.
In view of possible damage from flooded fields critical maximum levels in the basin will have to be determined.

The discharge capacity of the sluice is determined by the maximum floods from the rivers flowing into the area, and taking into account permissible water levels in the reservoir, the storage capacity of the reservoir, local rainfall and the tidal water levels at the seaside. It is expected that peak values for both river discharge and rainfall coincide with the minimum of needed irrigation water. This will be in the Monsoon, roughly between June and September.
The Narmada has a design discharge of 67.000 m^3/s, which is the 1/100-year value.
This flood wave is schematized as a parabolic function described as follows:

$$Q(t) = \frac{t(T-t)}{\left(\frac{T}{2}\right)^2} Q_{max}$$

Where: T = total duration of the flood wave (chosen at 60 hours);
t = time (hours);
$Q(t)$ = momentary discharge (m^3/s);
Q_{max} = peak discharge (m^3/s).

This formula has been integrated over the 60 hours period, leading to a flood wave with a content of $9{,}6 \cdot 10^9$ m^3. Without spilling this raises the basin level about 5 m. To control the reservoir level a spillway with a width of 700 -800 m is required.

From the required discharge capacity the main dimensions of the spillway can be determined. It is recommended that the discharge sluices will be used during the final closure as an extra orifice. In this way it may be possible to reduce the head difference over the dam during the closing operation.

5.3 Dimensions of the spillway

In the design for the dam a spillway for regulating the reservoir level is needed. Just like the tidal power facility it is possible to construct this spillway before closure of the gulf. This way an extra flow area can be used during closure.
The spillway will have a length of about 800 m. The total length of the construction is 1000 m. When radial gates with a span of 20 m and a height of 10 m are chosen at a sill level of MSL -10 m (Kalpasar report), an extra flow area of 8000 m^2 can be obtained.

5.4 *Narmada channel*

As can be seen on the maps, the Narmada spillway is divided from the main gully in the Narmada by the tidal flats near Dahej. To guarantee safety during extreme flood (the spillway can not be reached by the water and might lead to collapse of the dam over Alia Bet), but also to guarantee filling of the Kalpasar Lake by the Narmada, a connecting channel is needed. This channel has to be dredges across the tidal flats before the Narmada mouth is closed completely.

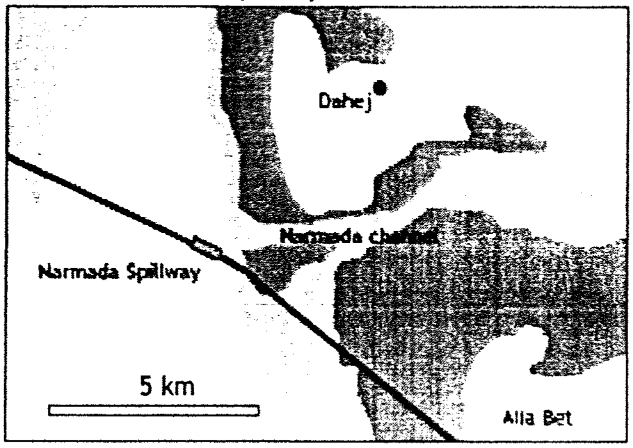


Figure 5.1 Proposed Narmada channel

6 Secondary dam sections

6.1 *Introduction*

The dam alignment contains a few main sections that are described extensive (tidal power facility in chapter 4 and final closure gap in Part C). The dam obviously is longer than the tidal power facility and the final gap. This chapter deals with the secondary dam sections left in the alignment. They are:

- The connection from the Saurashtra peninsula to the Tidal power facility, section **Ghogha-Tidal power facility;**
- The connection between the **Tidal power facility and the final gap**, including the shiplocks;
- The connection between the **Spillway and Hansot.**

It is not expected that a major cost reduction can be achieved in these dam sections. Combining this assumption with the fact that the alignment of the dam is the same as the Kalpasar alignment it is suggested to use the designs for the "normal dam sections" made by the Dutch Ministry of Transport, Public Works and Watermanagement for the Haskoning Kalpasar study.

6.2 *Ghogha-tidal power facility*

This dam has to connect the tidal power facility with the Saurashtra peninsula. The length of this dam section is around six kilometers. The averaged water depth in this area is between 2 and 5 meters below CD, only close to the Sluices of the tidal power facility the depth increases. At the tidal power facility side of this dam the large caisson building pit will be created. To ease the supply of equipment and material to this site, it is essential to build this dam section at the beginning of the project and directly make it a permanent dam. The cross section for this dam profile is given in the drawing below: Figure 6.1
The layout for the section that borders the building pit will be designed in combination with the design of the whole pit. This is not done in this report.

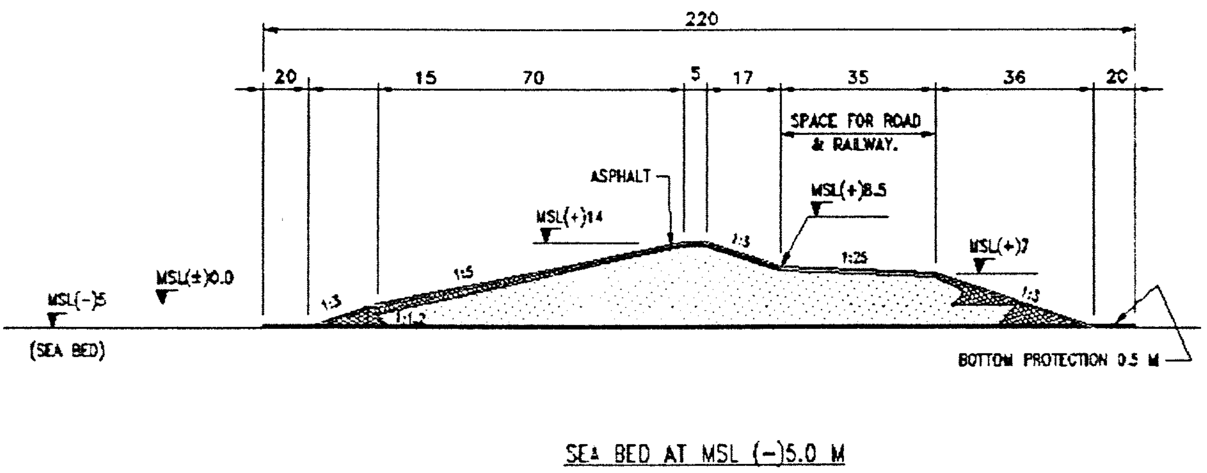


Figure 6.1 Cross section dam Ghogha-Tidal power facility (source Kalpasar study)

The road will be constructed directly after the construction of the dam body, offering the possibility to transport whatever one wants to transport. Whether the railway has to be constructed immediately or not depends strongly on the chosen method for the final gap.

6.3 *Tidal power facility - final gap*

This section connects the Tidal power facility with the final closure gap. The length of this section is around three kilometers. This section contains the two shiplocks. It starts at the end of the turbine section of the Tidal power facility. The average depth is around 10 meters below CD. This section has to be built as fast as possible because the construction of the shiplocks will take much time and the closure of the final gap demands a work-free area at the beginning. Either because the free space is required for storage of the geotextile boxes or because of the heavy train traffic. This makes it almost impossible to manage large building activities. The layout of this dam section is given in the drawing below (figure 5.2).

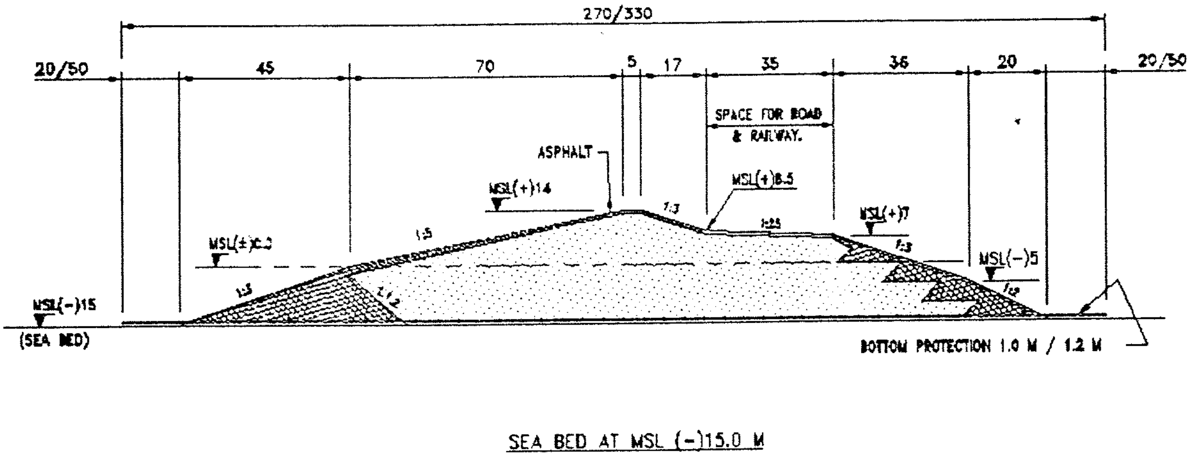


Figure 6.2 Cross section dam tidal power facility-final gap

6.4 *Spillway - Hansot*

This is the longest secondary dam section (30 km) and runs across the Narmada mouth. This section is exactly the same as in the Kalpasar report.

The construction of this phase can be done parallel to the construction of the tidal power facility. This part of the alignment does not cross any of the Khambhat gullies, only the gullies in the Narmada mouth. This section therefore will not influence the flow pattern in the Gulf and thus will not increase the velocities at the tidal power facility. Only the crossing of the real mouth (north of Alia Bet) might have some influence, this is also the end of this dam section and therefore it is suggested to start construction in Hansot.

In contrary to the other dams, this dam will be constructed of clay instead of rock. This has a few reasons.

- The local bottom consists of clay;
- The bottom is very shallow, and most of the alignment is temporary dry during the tidal cycle;
- The velocities are lower than in the rest of the dam, as the whole Narmada mouth is located outside the gully system of the Gulf;
- The less rock is used the better; constructing this dam of rock would increase the required amount of rock significant.

The alignment is drawn in the following figure.

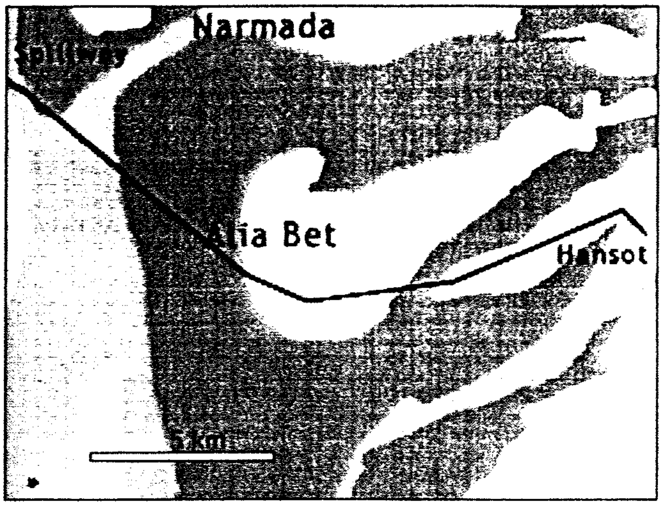


Figure 6.3 Dam alignment Spillway-Hansot

6.5 Work islands & construction dock

Two work islands are planned in the design. One is situated between the sluice and the turbine section of the tidal power facility, with dimensions of 1500 * 300 m. The second is situated east of the tidal power facility and has the same dimensions. In this second island the shiplocks are situated. The islands are constructed by hydraulic sandfill. All dimensions are preliminary.

Also a construction dock for caissons (building pit) is planned. The best location seems west of the gully in which the tidal power facility is placed. The inner dimensions are estimated at 1000 * 1000 m. In the following figure the locations of all work islands are indicated.

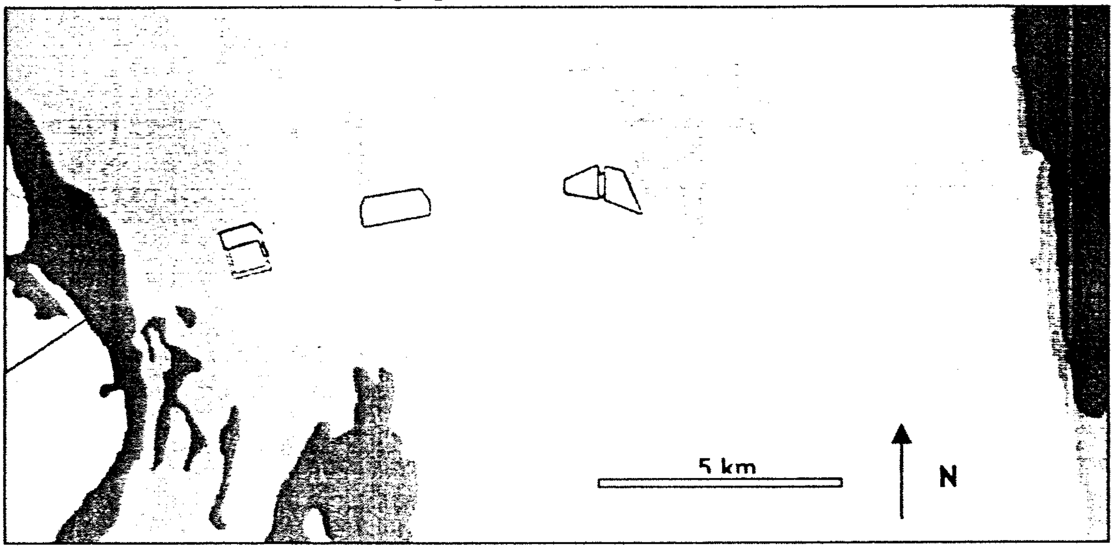
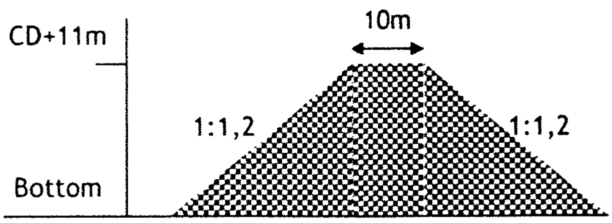


Figure 6.4 Location of work islands & construction dock

6.6 Volumes of rock

The amount of can roughly be determined with the drawn cross-section.



The required volume is:

Cross section * length of dam section
=(Bottom level under CD, + 11m) x (((Bottom level under CD + 11m) x 1.2) + 10) x dam length

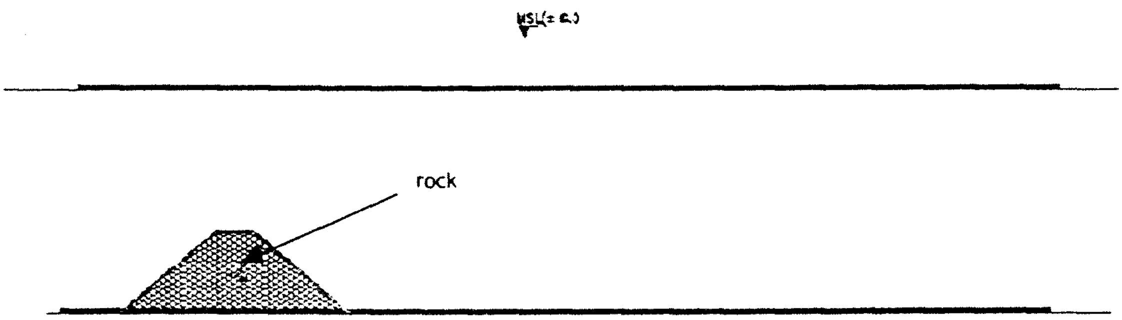
Table 6.1 Rock volumes secondary damsections

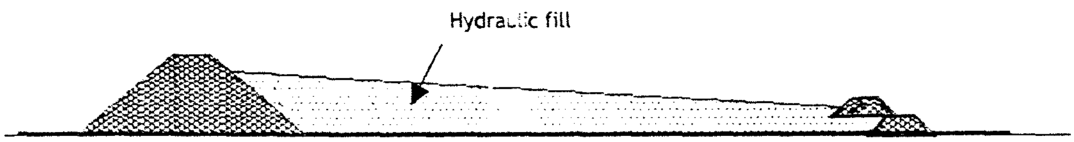
	Length	Average depth	Cross area	Volume
Ghogha-Tidal power facility	6 km	CD -3,5 m	397 m^2	2382000 m^3
Tidal power facility - final gap	3 km	CD -10 m	739 m^2	2217600 m^3
Total quarry run	9 km	-		4599600 m^3
Armor layer (D_{50} = 1,2 m)	9 km	-	43 m^2	387000 m^3

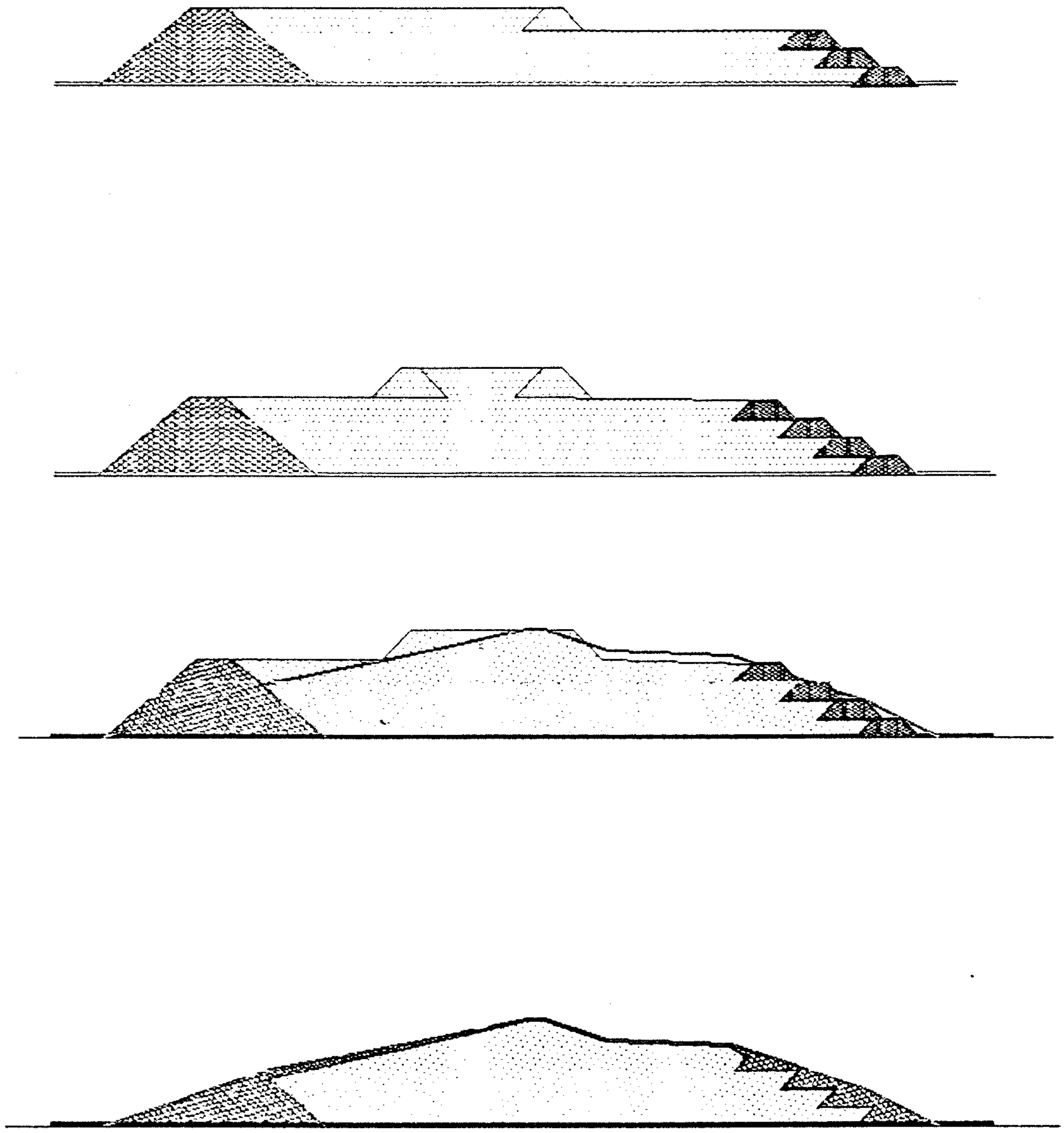
The maximum occurring velocity during the construction of these dams will not be higher than 4 m/s, this is the velocity (including 15% turbulence) in the final gap. Most of the time the velocities are considerably lower. The stone that is stable according to the Shields relation has a D_{50n} of 0,17m and a mass of 14,3 kg. This can be considered to be quarry run.

Large stones are only needed to protect the dams against wave attacks. The design storm requires stones with a diameter of 1,2 m. These stones have to cover the layer that is attacked by the waves, roughly from CD to the top, this implicates a layer width of ±18 m. This layer will have a thickness of (minimal 2 x D_{50}) 2,4m. This layer is needed at the seaside and would result in a volume of 12000 x 18 x 2,4 = 387.000m^3 stones. This volume has to be added to the volume in table 6.1. In this way the extra stones needed to construct the final dam are compensated. (See drawings below, the little dams on the right are assumed to have a similar volume.)

The following drawings show the proposed construction phases for the secondary rockfill sections. This are uniform designs, only the top level will be the same (CD +19 m).







6.7 Diversion dam

The tidal basin will have to be separated from the larger fresh water basin after closure of the dam. Since there are no heavy currents in the closed basin, this dam can be constructed with hydraulic sandfill. The length of this dam will be 40 km.

6.8 Shiplocks

Two shiplocks are required. One shiplock will connect the tidal basin with the sea, this shiplock provides entrance to the basin for repair and maintenance vessels. This shiplock also provides access to the Bhavnagar New Port. The second shiplock will provides access to the large industries near Dahej and possible new harbors. The shiplocks are not designed in this report.

7 Final gap

7.1 Introduction

The final gap is the deep section in the alignment that remains after construction of the Tidal power facility, the inlet sluices, the Narmada spillway, the islands and the secondary damsections. The final gap has a width of 10000 m. The design for the final gap is made in Part C.

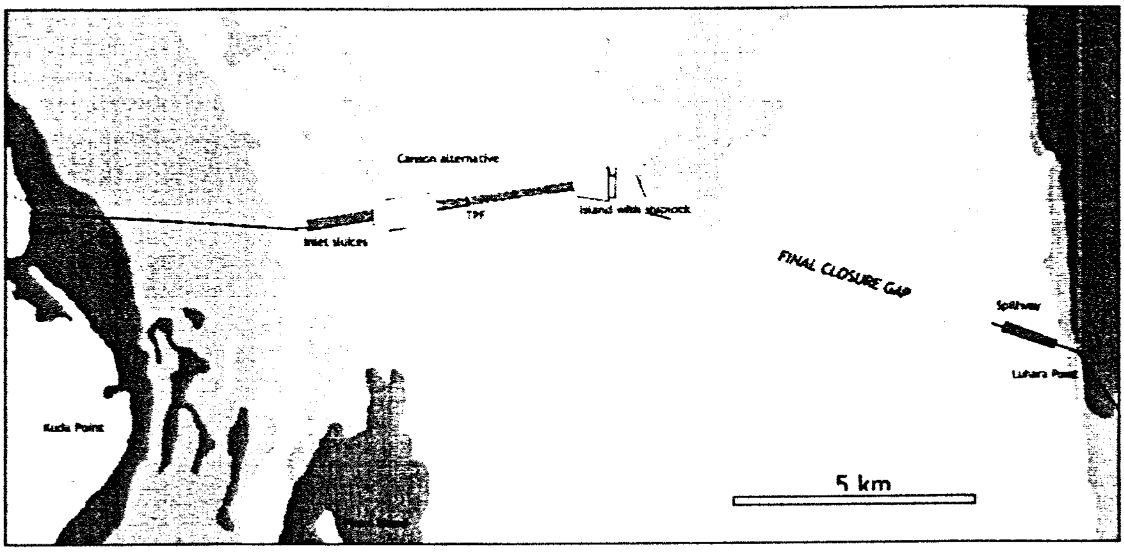


Figure 7.1 Location of the final gap

Because the final gap is the last section, the current velocities in this section will be higher than in any other section. Therefore care has to be taken in considering the closure technique. The different closure principles are described in this paragraph. In Chapter 8 the model is described and in Chapter 9 the results of the model are discussed. After this description the design for the final closure is made (in Part C).

7.2 Possible closure techniques

When building a dam with purpose to close a tidal basin, a problem arises. In the beginning the velocities in the entrance to the basin will usually be low; therefore, relatively cheap materials can initially be used to narrow the gap - if possible sand, gravel or clay.
By narrowing the entrance, the velocity increase, the higher these velocities are, the heavier the material has to be to be stable in these currents.

There are four basic closure techniques. They are:

- Horizontal closure;
- Vertical closure;
- Combined closure;
- Sudden closure.

These techniques are described in the next paragraphs, use has been made of 'The closure of tidal basins'.

7.2.1 Gradual horizontal closure

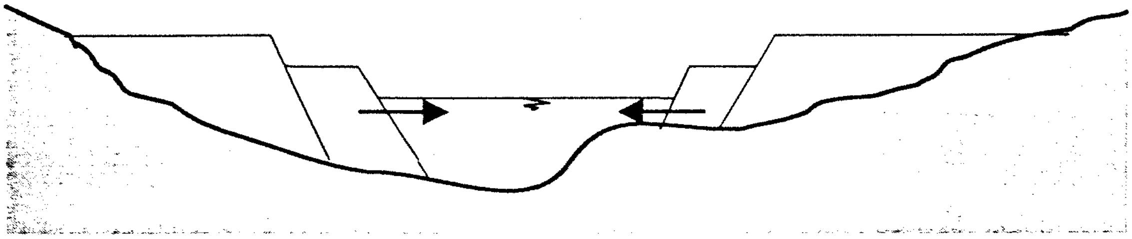


Figure 7.2 Horizontal closure

If a closure gap with a low sill, is horizontally, from the two dam heads, constricted (see figure 7.2) the current velocities increase in as the cross-section area decreases. Very heavy material will therefore be needed to close the last gap. The flow through this gap attacks the bottom as a very large jet. This jet flow requires very heavy bottom protection. Material is usually placed in position by dump trucks.

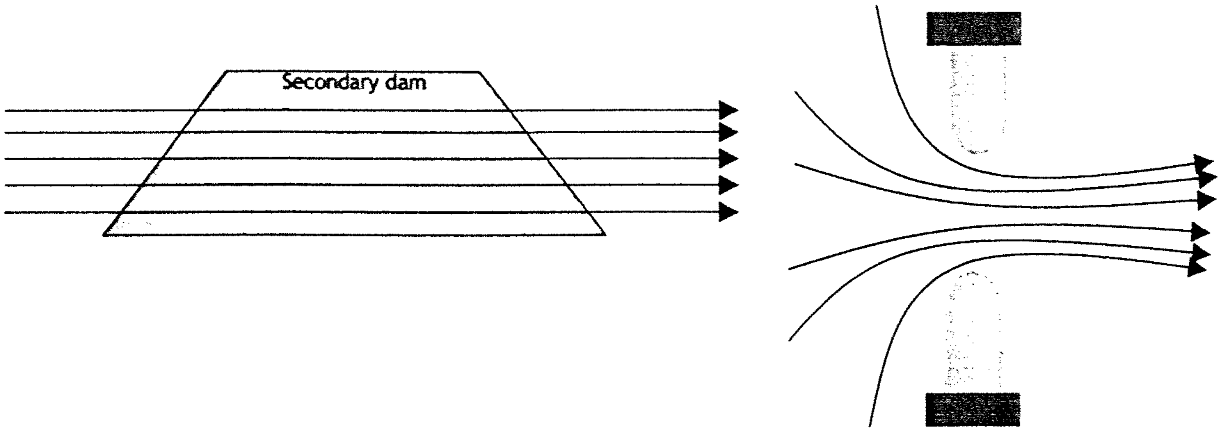


Figure 7.3 Principle of flow through horizontally closed gap

7.2.2 Gradual vertical closure

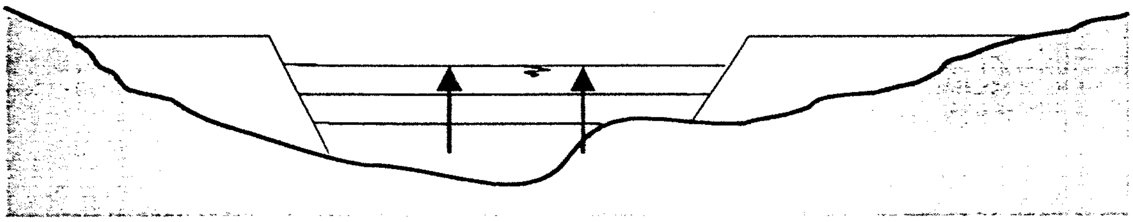


Figure 7.4 Vertical closure

The closure gap is built in horizontal layers (see figure 7.4). Consequently, the velocities increase when the sill height increases and heavier materials have to be used. There are various methods of dumping material, such as by floating equipment, a cableway, a (temporary) bridge or helicopters. Because the flowing water uses the whole width of the final gap, the attack on the bottom protection will be gentler than in a horizontal closure (see figure 7.5).

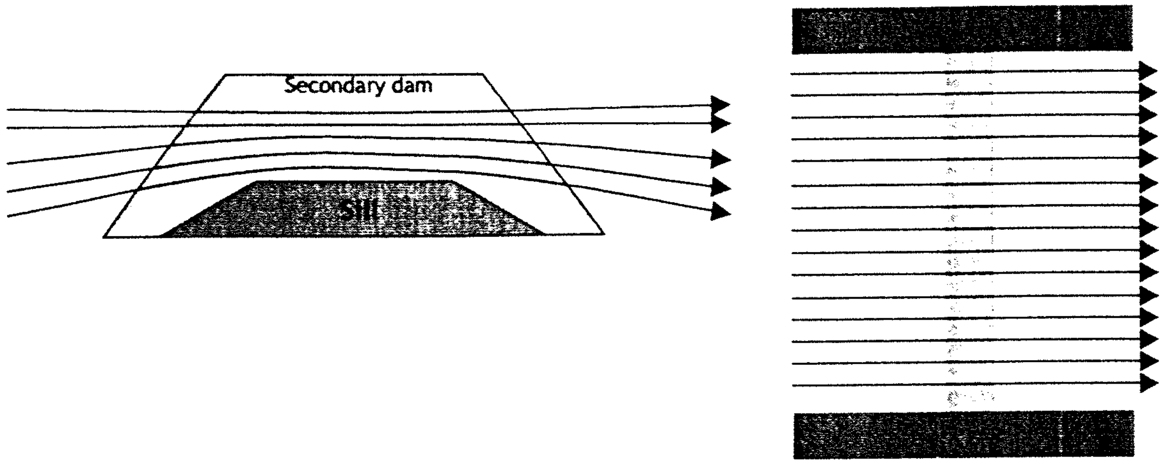


Figure 7.5 Principle of flow through vertically closed gap

7.2.3 Combined closure

These two methods are often combined in a so-called *combined closure*. First a sill is constructed, mostly waterborne. The construction of the sill is stopped at the moment that the ships are no longer able to work in the gap. Then landborne equipment closes the gap horizontally (see figure 7.6).

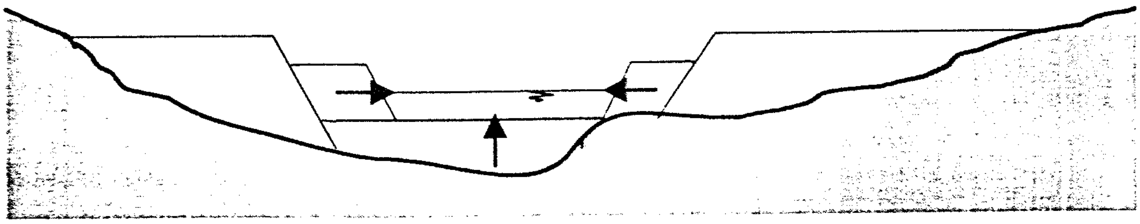


Figure 7.6 Principle of combined closure

7.2.4 Sudden closures

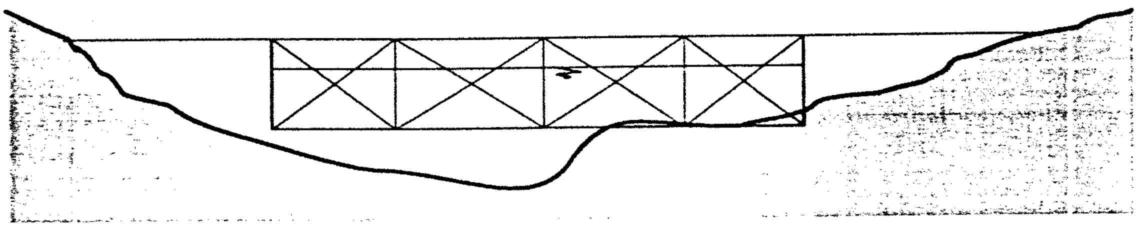


Figure 7.7 Sudden closure

This method deals with structures that allow the whole gap to be closed rapidly. The classical way is to place caissons (mostly concrete or steel boxes) one by one on a dumped sill in the gap during successive slack-water periods. When the last caisson is placed the dam is closed. Because current velocities increase a lot, the placing possibilities decrease. If it is impossible to place several caissons, the caissons have to be provided with sluice gates. In these caissons large orifices are created, after placing of the caisson on the sill, temporary walls are removed and the water is able pass through, reducing the velocity in the remaining part of the gap. After the last caisson is placed, al the caissons are closed with gates at the same moment, the dam is closed.

8 Description of storage area approach model

8.1 Introduction

For the design of the closure methods computations have to be made to predict the current velocities through and head differences over the various gaps in the closure dam and the water levels at sea and in the basin. The results from these computations will be used during this study as a guideline to verify whether a solution is possible or not. Therefore it is allowed to use a rough model. After choosing the method to close the Gulf of Khambhat, it is obvious that new calculations (2- or even 3-dimensional) have to be made.

8.2 The theory of the storage area approach

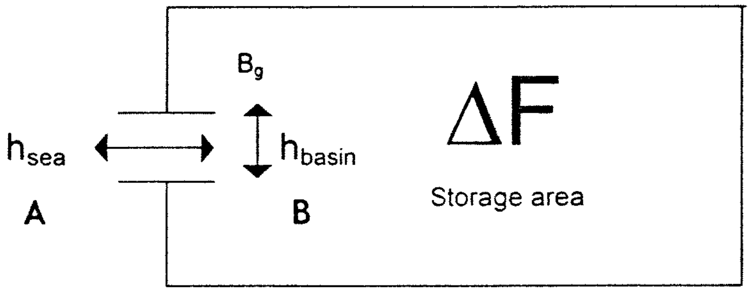


Figure 8.1 Schematization of a storage area

A tidal area with a narrow entrance, drawn in figure 8.1, can be schematized reliably by neglecting the resistance and the inertia. The current velocities are computed between A and B. The energy height at A is determined by the tidal movement represented as the water level at sea $h_{SEA}(t)$. The water level at B represented as $h_{BASIN}(t)$ follows the level at sea but is not the same as $h_{SEA}(t)$ due to the narrow entrance, and therefore unknown. $h_{BASIN}(t)$ represents the water level of the total basin area. The flow through the gap can be described by the formulas for a weir.

This model is strictly spoken not valid for use in the Gulf of Khambhat, the basin is too large and the gully system is not taken into account in the model. However, results from this approach are quite satisfying and are accurate enough to predict the current velocities occurring at the proposed closure techniques in the closure gaps.
The main rule is: the smaller the gap, the better the results. One general prediction can be done towards a more sophisticated model: velocities calculated with the storage area approach are quite conservative, by using a more detailed model the velocities will be a bit lower than calculated with the storage area approach. A thorough analysis can be found in chapter 15 (Risks).

8.3 The formulas of the storage area approach:

Discharge through gap:

$$Q_{GAP}(t) = \mu \times A_{GAP} \times \sqrt{2 \times g \times (h_{SEA}(t) - h_{GAP}(t))} \qquad (8.1a)$$

With: $h_{GAP}(t) = h_{BASIN}(t)$ for $h_{BASIN} > \frac{2}{3} h_{SEA}$ (submerged weir) (8.1b)

Or: $h_{GAP}(t) = \frac{2}{3} h_{SEA}(t)$ for $h_{BASIN} < \frac{2}{3} h_{SEA}$ (free-flow weir) (8.1c)

Storage area:

$$Q_{GAP} = \Delta F \times \frac{dh_{BASIN}}{dt} \tag{8.2}$$

The following symbols are used in the formulas above:

Q_{GAP}	=	discharge through the closure gap (m^3/s)
A_{GAP}	=	cross-sectional profile at the location of the gap; $B_g{}^*h_{gap}$ (m^2)
g	=	acceleration of gravity (m^2/s)
h_{SEA}	=	energy height upstream of the gap with respect to sill height d' (m)
h_{BASIN}	=	water level downstream of closure gap with respect to sill height d' (m)
h_{GAP}	=	water level in closure gap with respect to sill height d' (m)
d'	=	sill level in closure gap with respect to reference level
Δ F	=	surface of basin;
μ	=	discharge coefficient for submerged weir, m for free-flow

The model offers the following input parameters:

Timestep t =	300	Seconds
Gap width =	10000	Meters
Sill level below CD =	10	Meters
Extra gap (in tidal power facility) =	54000	m2
Gap width above CD (in tidal power facility) =	4400	Meters
Maximum storage area =	2187	Km2

The following figure (8.2) represents the dam in the model, the different gaps are not to scale.

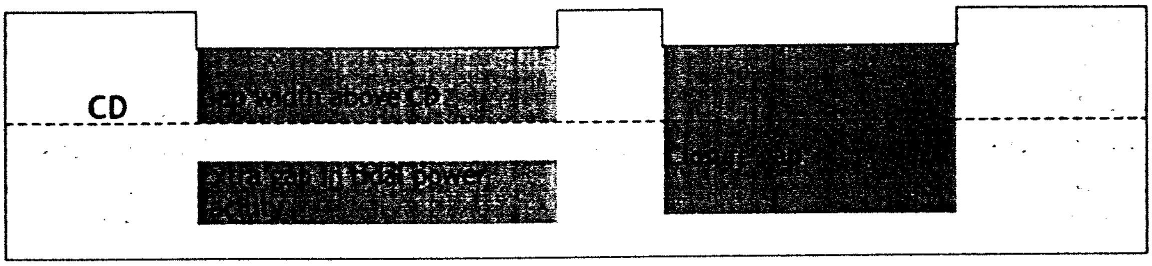


Figure 8.2 View of the gaps in the dam

8.4 Calculation schedule

With these input values the following calculations, according to the Heun numerical method, are made:

Method of Heun:

$$y^{*}_{n+1} = y^H_n + f(t_n, y^H_n) \quad (predictor:\ n \geq 0) \tag{8.3a}$$

and

$$y^H_{n+1} = y^H_n + \frac{1}{2}(f(t_n, y^H_n) + f(t_{n+1}, y^{*}_{n+1}) \quad (n \geq 0) \tag{8.3b}$$

Applying this schedule on the formulas stated above gives the following calculation schedule

Step 1

$$h_{SEA}(t+\Delta t) - h_{BASIN}(t) = \Delta Head(t) \qquad (8.4a)$$

$$Q(t) = A(t) \times \sqrt{2 \times g \times \Delta Head(t)} \qquad (8.4b)$$

$$A(t) = gap\ width \times h_{BASIN}(t) \qquad (8.4c)$$

$$\Delta h_{BASIN}(t) = \frac{Q(t) \times \Delta t}{\Delta F(t)} \qquad (8.4d)$$

$$\Delta F(t) = 526.2 + \frac{2187 - 630}{15} \times h_{BASIN}(t) \qquad (8.4e)$$

This is a linear relation based on the planimetric values as presented in the Kalpasar study. Above CD only two values are known, at CD the surface is 630 km^2 and at CD +15 m the surface is 2187 km^2. These two values are linearly interpolated.

$$h^{\bullet}_{BASIN}(t+\Delta t) = h_{BASIN}(t) + \Delta h_{BASIN}(t) \qquad (8.5)$$

This is the predictor value according to the Heun schedule.

Step2

$$h_{SEA}(t+\Delta t) - h^{\bullet}_{BASIN}(t+\Delta t) = \Delta Head^{\bullet}(t+\Delta t) \qquad (8.6a)$$

$$Q^{\bullet}(t+\Delta t) = A^{\bullet}(t+\Delta t) \times \sqrt{2 \times g \times \Delta Head^{\bullet}(t+\Delta t)} \qquad (8.6b)$$

$$A^{\bullet}(t+\Delta t) = gap\ width \times h^{\bullet}_{BASIN}(t+\Delta t) \qquad (8.6c)$$

$$\Delta h^{\bullet}_{BASIN}(t+\Delta t) = \frac{Q^{\bullet}(t+\Delta t) \times \Delta t}{\Delta F^{\bullet}(t+\Delta t)} \qquad (8.6d)$$

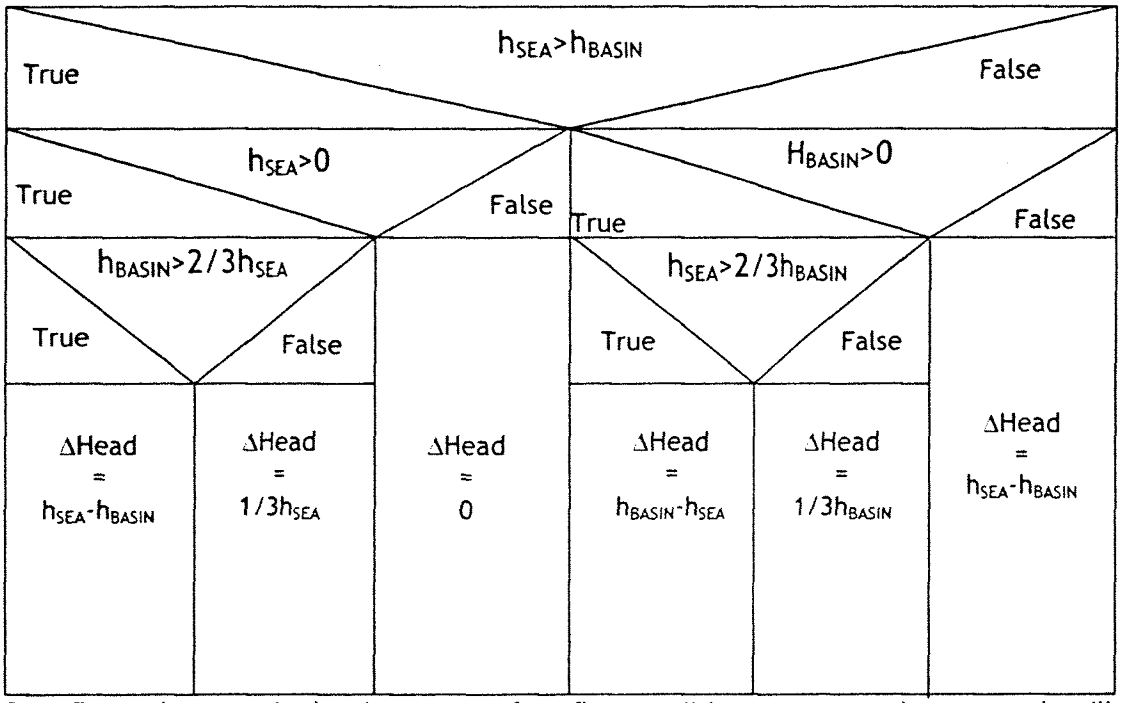
$$\Delta F^{\bullet}(t+\Delta t) = 526.2 + \frac{2187 - 630}{15} \times h^{\bullet}_{BASIN}(t+\Delta t) \qquad (8.6e)$$

$$h_{BASIN}(t+\Delta t) = h_{BASIN}(t) + \frac{1}{2}\left((\Delta h_{BASIN}(t) + \Delta h^{\bullet}_{BASIN}(t+\Delta t)\right) \qquad (8.7)$$

This is the requested value.

8.5 *Modeling of free-flow situation*

This calculation is repeated for every time step. The criteria for free-flow are brought in the model according the following logical scheme:



Free-flow only occurs in the closure gap, free-flow conditions start somewhere around a sill level of CD -5 m. Since the floor of the temporary sluice gates (on top of the Tidal power facility) is located at CD -5 m, there will not be any free-flow through these gaps.

8.6 *Embedding of tidal wave*

In the calculations the following tidal components have been incorporated (table 8.1). This is done to prevent unnecessary safe (only calculating with the maximum amplitude) or risky calculations (only calculating with the average amplitude). By incorporating four components, a two monthly cycle of the tide is constructed. This is necessary for some proposed construction techniques and certainly for planning of critical waterborne activities. The tidal wave is plotted in figure 8.3.

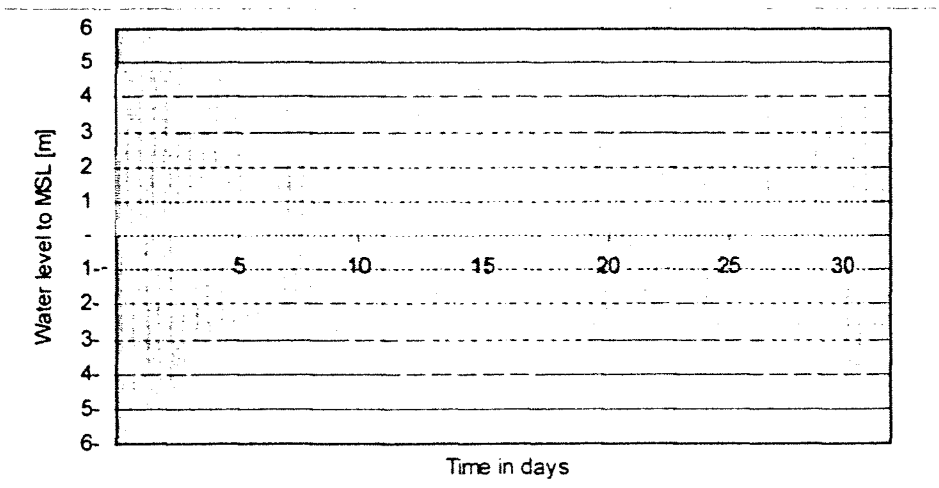


Figure 8.3 Tidal wave during a month

Table 8.1 Tidal components

Tidal component	Amplitude in m (Bhavnagar)	Period
M2	3,14	12h25min
S2	0,96	12h
K1	0,76	23h56min
O1	0,34	25h50min

With the calculated water levels at sea and in the basin, it is possible to determine the other parameters needed to predict the closure process. Therefore the orifice in the dam (A_{TOTAL}) is divided in two separate components (see figure 8.2):

The orifice of the TPF = A_{TPF} = extra gap (in TPF) + extra width (in TPF)*water depth in gap
The orifice of the closure gap = A_{GAP} = gap width * water depth in gap

After the split up, the following parameters can be determined:

The velocity through the closure gap $= U_{GAP}(t) = \sqrt{2 \times g \times \Delta Head_{GAP}(t)}$ (8.8a)

The velocity through the TPF and extra gap $= U_{TPF}(t) = \sqrt{2 \times g \times \Delta Head_{TPF}(t)}$ (8.8b)

The discharge through the closure gap $= Q_{GAP}(t) = A_{GAP}(t) \times U_{GAP}(t)$ (8.8c)

The discharge through the TPF and extra gap $= Q_{TPF}(t) = A_{TPF}(t) \times U_{TPF}(t)$ (8.8d)

After this definition it is possible to draw the velocities in the final closure gap, these velocities are the most important parameters. As can be seen in formulas 8.8, the values for m and μ are set at 1. This is done because the final shape strongly determines the size of these factors. Their influence is biggest in very small gaps.

9 Results of flow calculations

9.1 Introduction

This chapter discusses the results gathered with the model. These results serve as basis for the choice between the different closure scenarios, splitting the basin or not (see §3.3), and the closure techniques, vertical or horizontal. There are two other closure techniques discussed in Chapter 7 the combined and the sudden closure. The combined closure is in fact an optimization between the vertical and the horizontal closure techniques (waterborne vertical and landborne horizontal). The effect of combination is shown after all the alternatives are discussed. The sudden closure is not treated in this chapter because this is no gradual closure of a certain gap but the remaining section is closed of at once.

The following closures are compared in this paragraph:

- The vertical closure of a final gap with a width of 10000 m;
- The horizontal closure of a final gap of 10000 m wide;
- The vertical closure of the split basin, two gaps of 5000 m wide;
- The horizontal closure of the split basin, two gaps of 5000 m wide.

For each closure three tidal power facility scenarios are calculated:

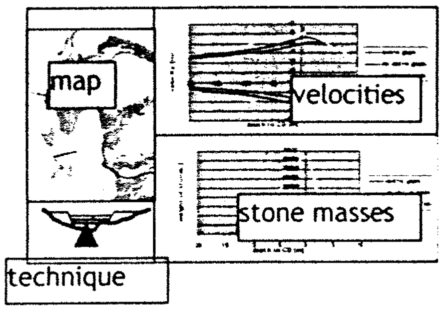
- Extra gaps = Tidal power facility, spillway and extra gaps on top of these structures;
- No extra gaps = only Tidal power facility and spillway without temporary gaps on top;
- No structures = this would be the case when both the Tidal power facility and the spillway are build in a special pit which is opened after the closure of the Gulf.

For the closures without split, the maximum surface of the basin is 2187 km^2, this is at the level of CD +15 m. For the split-up the basin is divided in 1/3 and 2/3 of these 2187 km^2. So for dam A & B this becomes 1458 km^2, and for dam C 729 km^2. Dam A is at the proposed Tidal power facility location, dam B is located north in the basin at the end of the separation dam (this dam will separate the tidal basin from the freshwater basin). Dam C is located at the same place as the final gap in the non-split basin.

All gaps have been calculated with a timestep of 300 seconds.

9.2 Layout of the results

For every closure, a map that shows the location of the gap, a chart shows the maximum velocities in the gap during ebb and flood, and a second chart translates these velocities in stone masses, this is done to show clearly the impact of a certain velocity. A small window at the left bottom corner shows whether it is a horizontal or a vertical closure.



The stone masses are determined by using the Shields criterion for beginning of motion:

$$d = \frac{u^2}{\psi \Delta C^2} \tag{9.1}$$

In which:

d = stone diameter [m];
u = the maximum occurring velocity (mostly flood) [m/s];
ψ = the Shields parameter, the standard save assumption is 0,03, this is used;
Δ = the relative density of the stones, for rock a density of 2700 kg/m^3 is used and for the water a density of 1035 kg/m^3, this results in a values for Δ of 1,62;
C = Chezy coefficient, value that determines the roughness of the gap:

$$C = 18\log\left(\frac{12h}{r}\right) \tag{9.2}$$

in which:

h = the average water depth in the gap [m];

- r = the roughness parameter, three times the stone diameter (3d) this indeed implicates an iteration process.

The Chezy factor should not be smaller than 30. If a calculated value is less than 30, the value is set at 30. This is done because smaller values than 30 are extremely rough.

Calculating the mass is done according the following relation:
$M = d^3 \times \text{density} = d^3 \times 2700$ [kg]

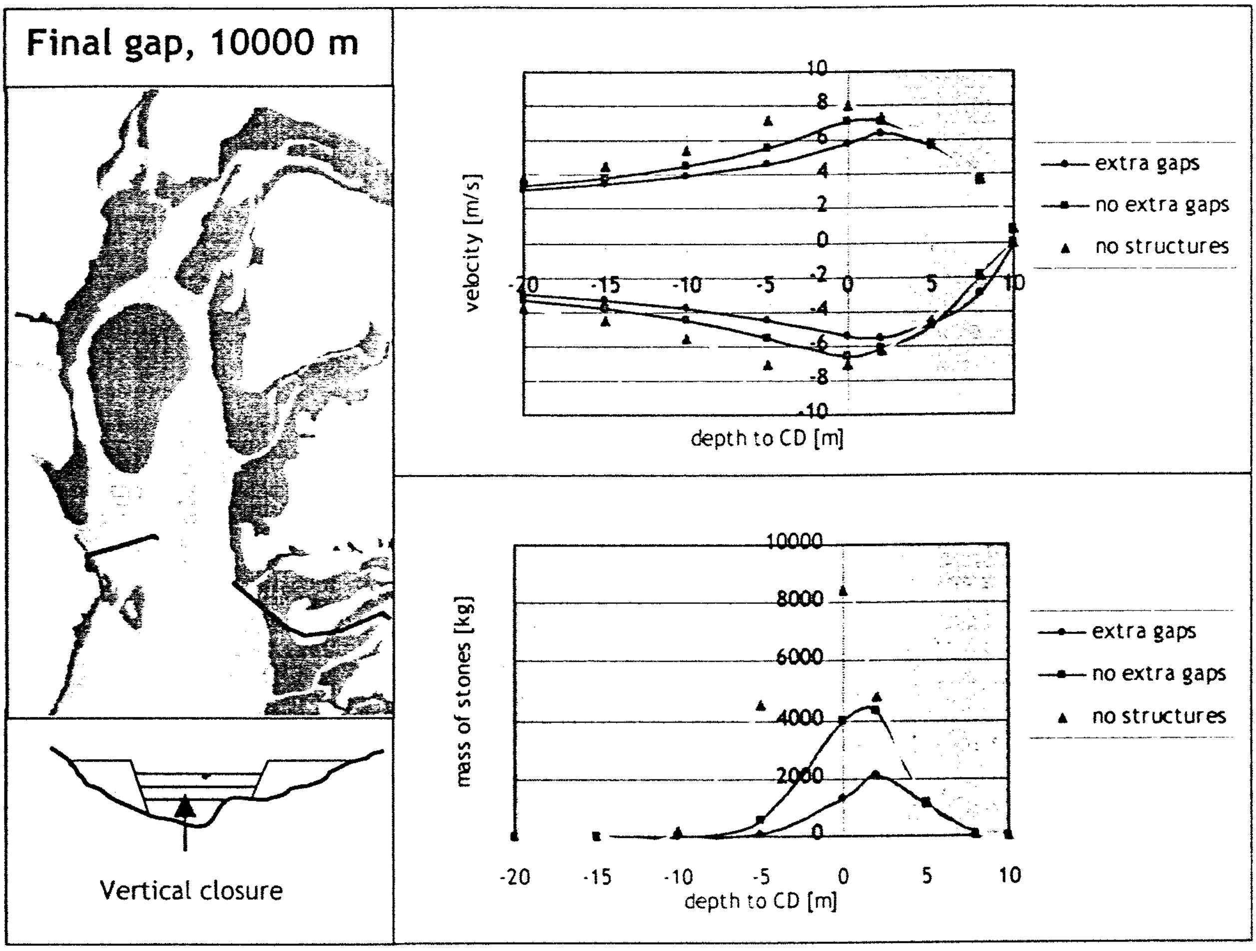


Figure 9.1 Vertical closure final gap 10000 m

This picture shows the impact of a vertical closure of the final gap on the velocities in the gap. The three tidal power facility scenarios, as described above, have the following orifices:

Extra gaps:
- Below CD = 54000 m^2, these are the turbines, the part of inlet sluices below CD and the part of the spillway below CD;
- Width above CD of the extra gap = 4400 m. These are the parts of the spillway and inlet sluices above CD and the extra gaps constructed above the turbines.

No extra gaps:
- 36000 m^2 gap, the turbines, sluices and part spillway below CD;
- 2000 m width, part of spillway above CD.

No structures: 0 m^2 and 0 m.

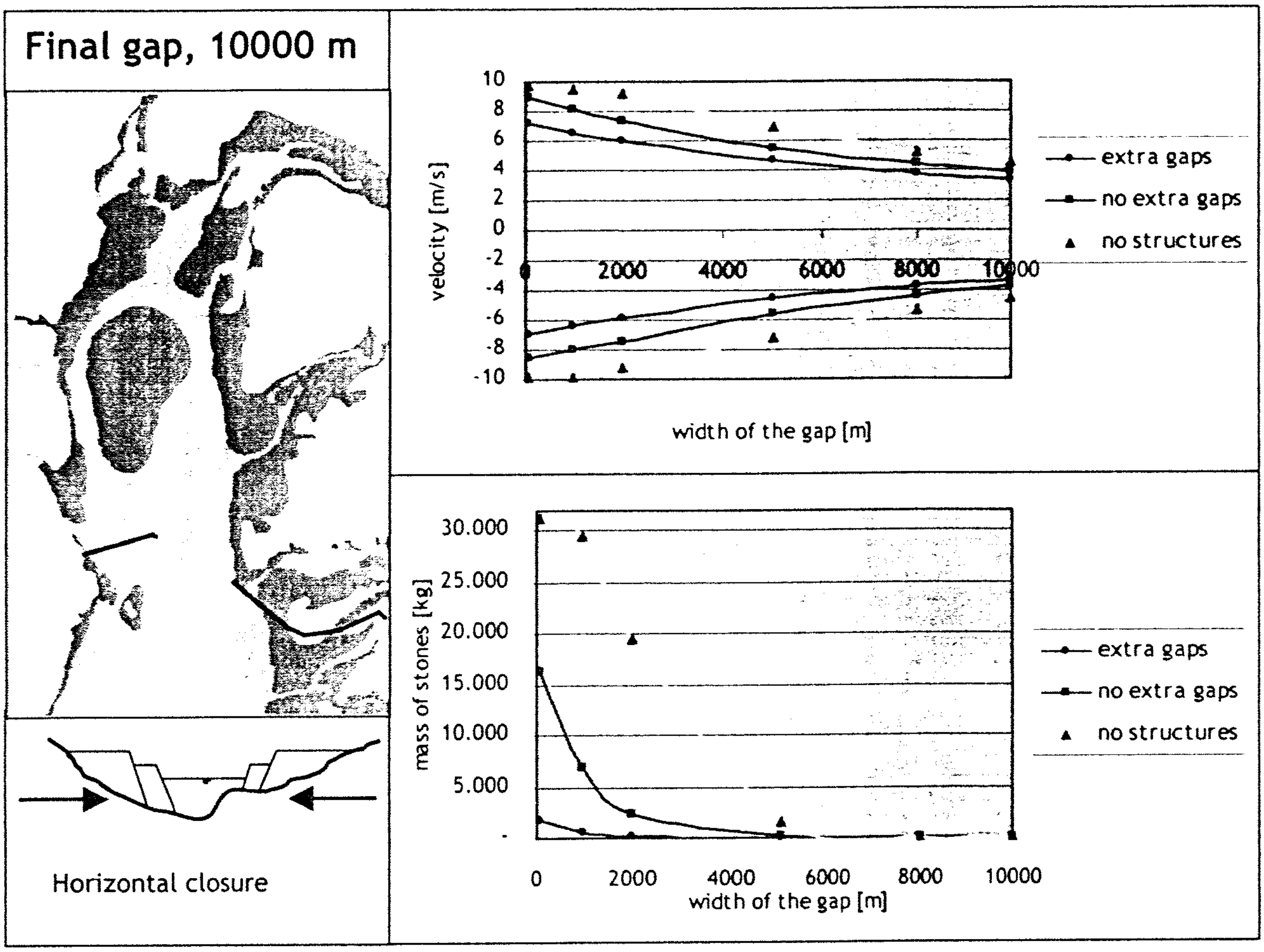


Figure 9.2 Horizontal closure final gap 10000 m

This picture shows the impact of a horizontal closure of the final gap on the velocities in the gap. The three tidal power facility scenarios, as described above, have the following orifices:

Extra gaps:
- Below CD = 54000 m^2, these are the turbines, the part of inlet sluices below CD and the part of the spillway below CD;
- Width above CD of the extra gap = 4400 m. These are the parts of the spillway and inlet sluices above CD and the extra gaps constructed above the turbines, sluices and spillway.

No extra gaps:
- 36000 m^2 gap, the turbines, sluices and part spillway below CD;
- 2000 m width, part of spillway above CD.

No structures: 0 m^2 and 0 m.

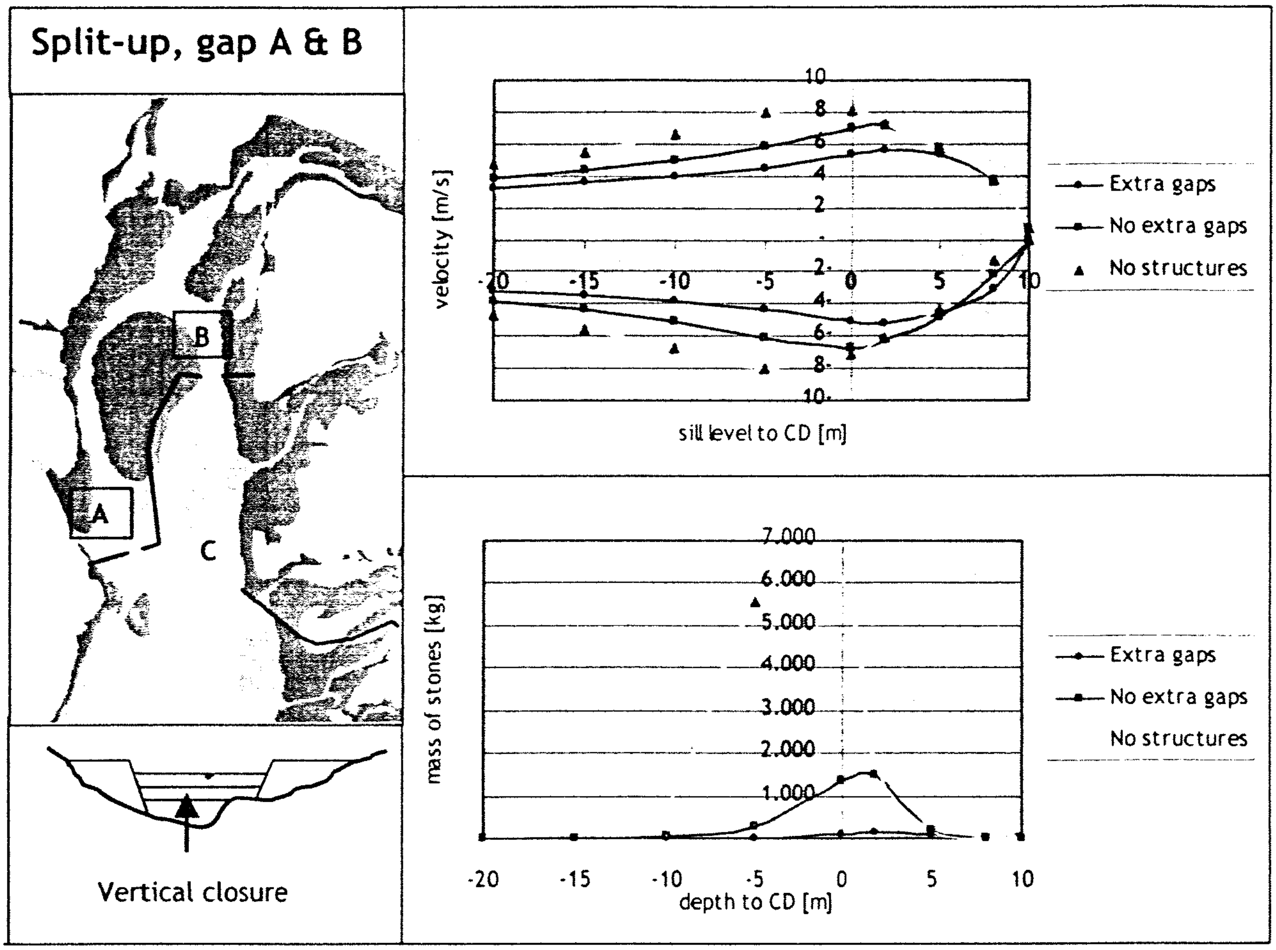


Figure 9.3 Vertical closure gap A&B, 5000 m, split

This picture shows the impact of a vertical closure of gap A & B on the velocities in the gaps. The three tidal power facility scenarios, as described above, have the following orifices:

Extra gaps:
- Below CD = 50000 m^2, these are the turbines, the part of inlet sluices below CD;
- Width above CD of the extra gap = 3600 m. These are the parts of the inlet sluices above CD and the extra gaps constructed above the turbines.

No extra gaps:
- 32000 m^2 gap, the turbines and sluices below CD;
- 1200 m width, part of sluice above CD.

No structures: 0 m^2 and 0 m.

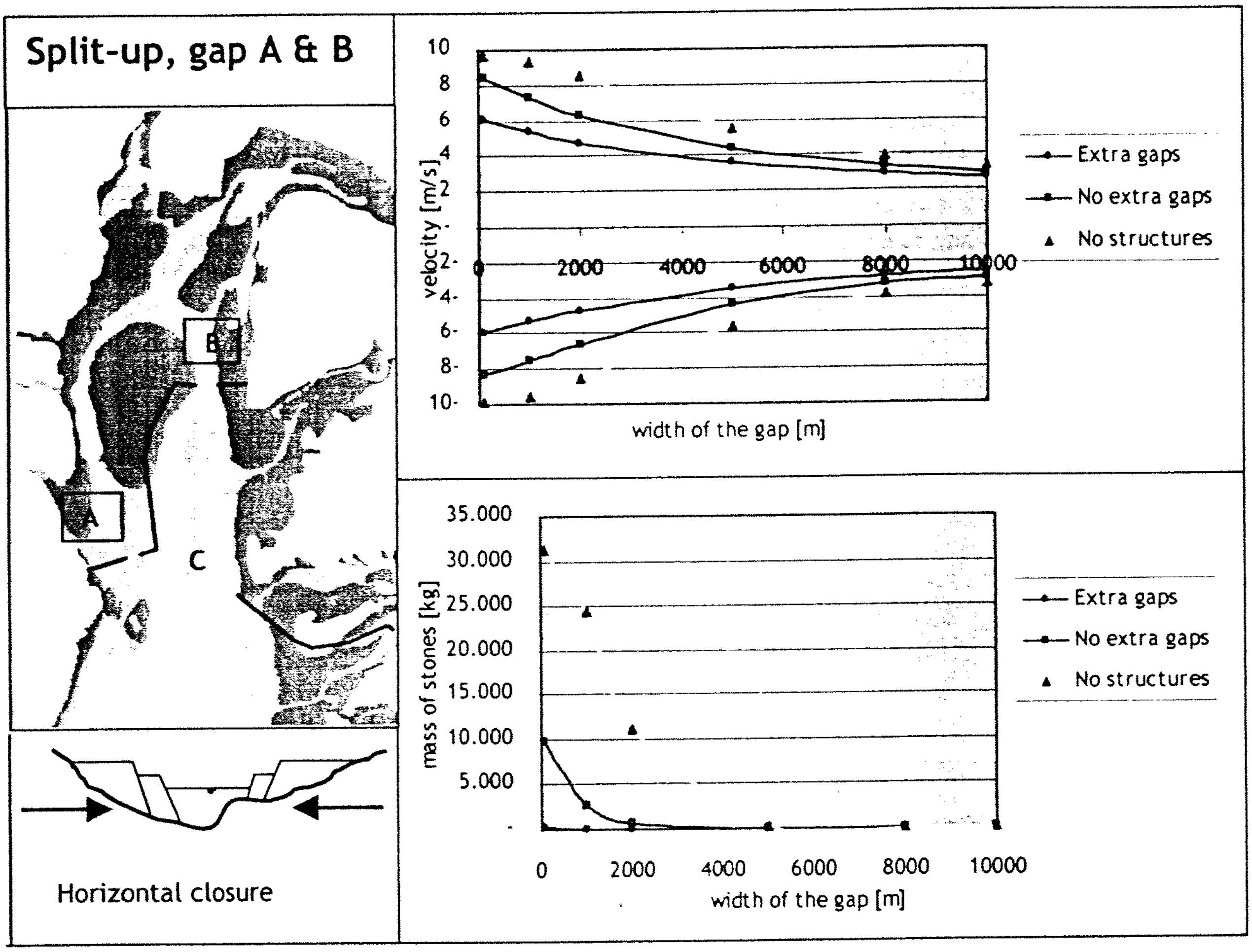


Figure 9.4 Horizontal closure gap A&B, 5000 m, split-up

This picture shows the impact of a horizontal closure of gap A & B on the velocities in the gaps. The three tidal power facility scenarios, as described above, have the following orifices:

Extra gaps:

- Below CD = 50000 m^2, these are the turbines, the part of inlet sluices below CD;
- Width above CD of the extra gap = 3600 m. These are the parts of the inlet sluices above CD and the extra gaps constructed above the turbines.

No extra gaps:

- 32000 m^2 gap, the turbines and sluices below CD;
- 1200 m width, part of sluices above CD.

No structures: 0 m^2 and 0 m.

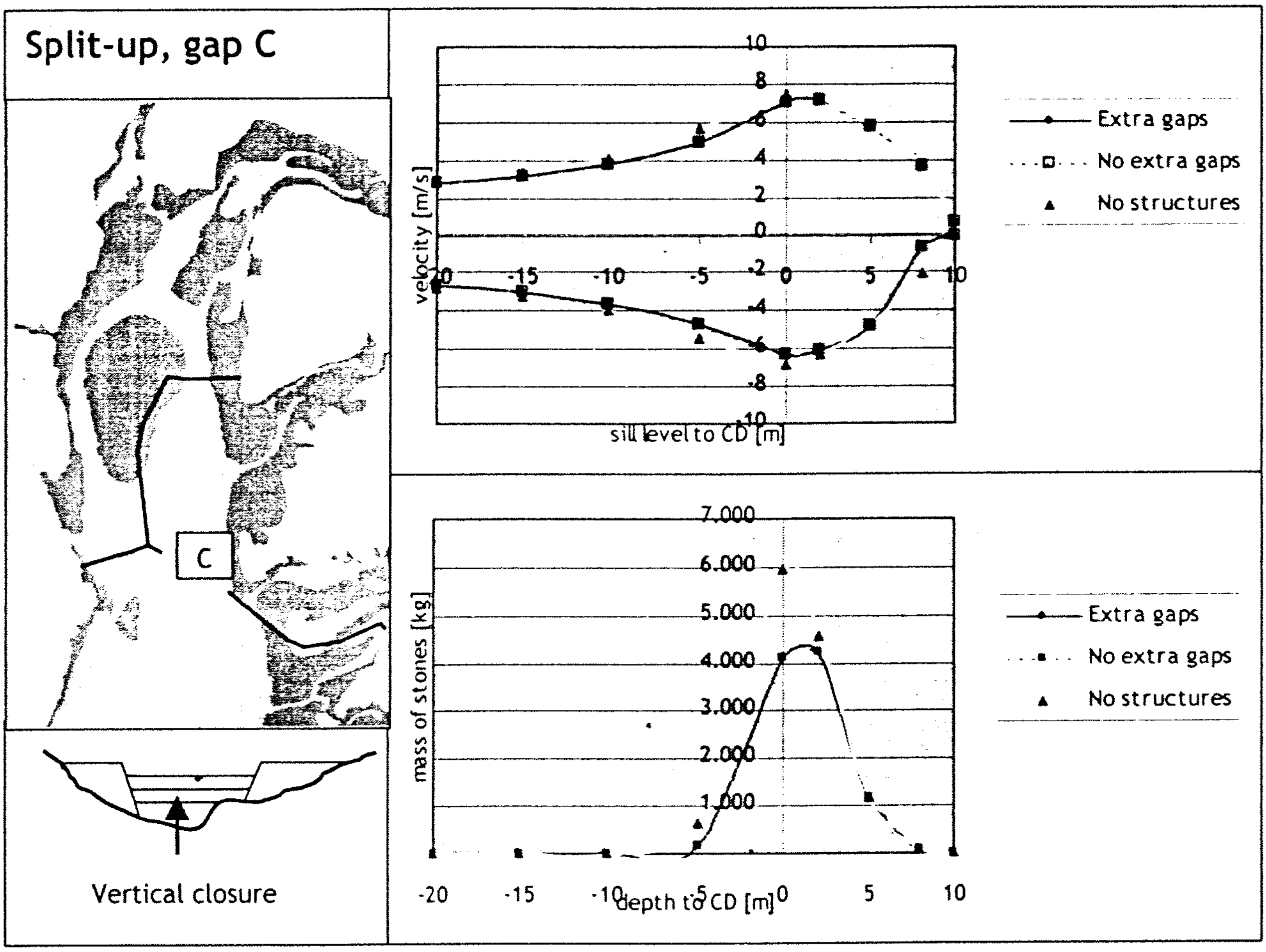


Figure 9.5 Vertical closure gap C, 5000m, split-up

This picture shows the impact of a vertical closure of gap C on the velocities in the gaps. The three tidal power facility scenarios, as described above, have the following orifices:

Extra gaps:
- Below CD = 4000 m^2, this is the part of spillway below CD;
- Width above CD of the extra gap = 800 m. This is the part of the spillway above CD.

No extra gaps:
- 4000 m^2, part of spillway below CD;
- 800 m width, part of spillway above CD (notice: these values are the same as for the extra gap option, there is no tidal power facility in this basin section possible).

No structures: 0 m^2 and 0 m.

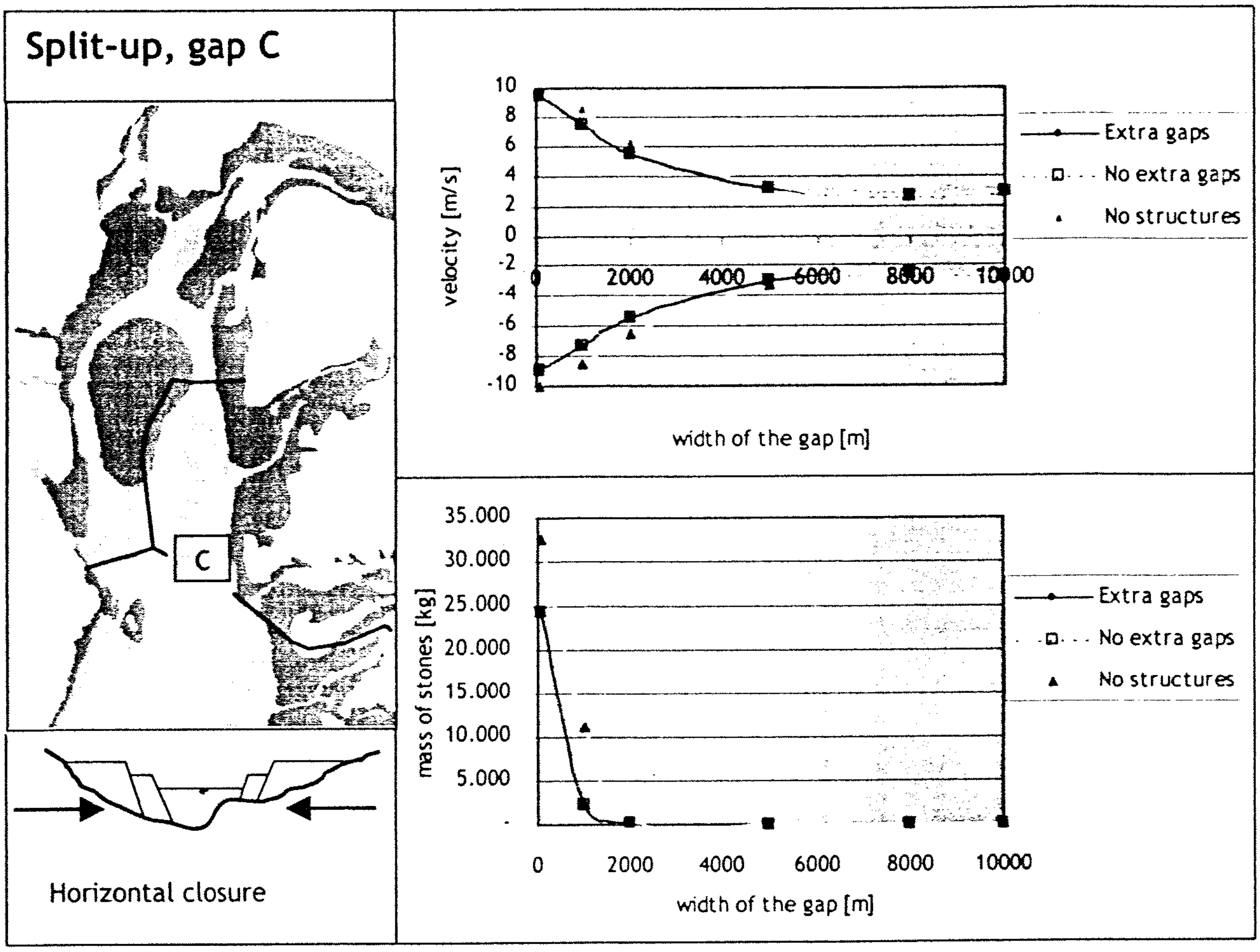


Figure 9.6 Horizontal closure gap, C 5000 m, split-up

This picture shows the impact of a horizontal closure of gap C on the velocities in the gaps. The three tidal power facility scenarios, as described above, have the following orifices:

Extra gaps:
- Below CD = 4000 m^2, this the part of spillway below CD
- Width above CD of the extra gap = 800 m. This is the part of the spillway above CD.

No extra gaps:
- 4000 m^2, part of spillway below CD;
- 800 m width, part of spillway above CD (notice: these values are the same as for the extra gap option, there is no Tidal power facility in this basin section possible).

No structures: 0 m^2 and 0 m.

Combined closure

As already mentioned, a combined closure is a combination of the horizontal and the vertically construction method. First the stones are dumped by vessels creating a vertically constructed dam body, when the velocities in the gap are to high to operate the vessels or the draught from the vessels is to large for the gap, the process is switched from waterborne to landborne. Landborne construction is mostly done by dump trucks. The following assumptions have been done about the workability of the dumping vessels:
maximum velocity occurring: 5 m/s, maximum operation velocity: 2 m/s, leaving time windows of around 1 hour 15 minutes; required minimum draft 7 meters

When all extra gaps are used, no split up is considered, the following comparison can be made (see figure 9.7):

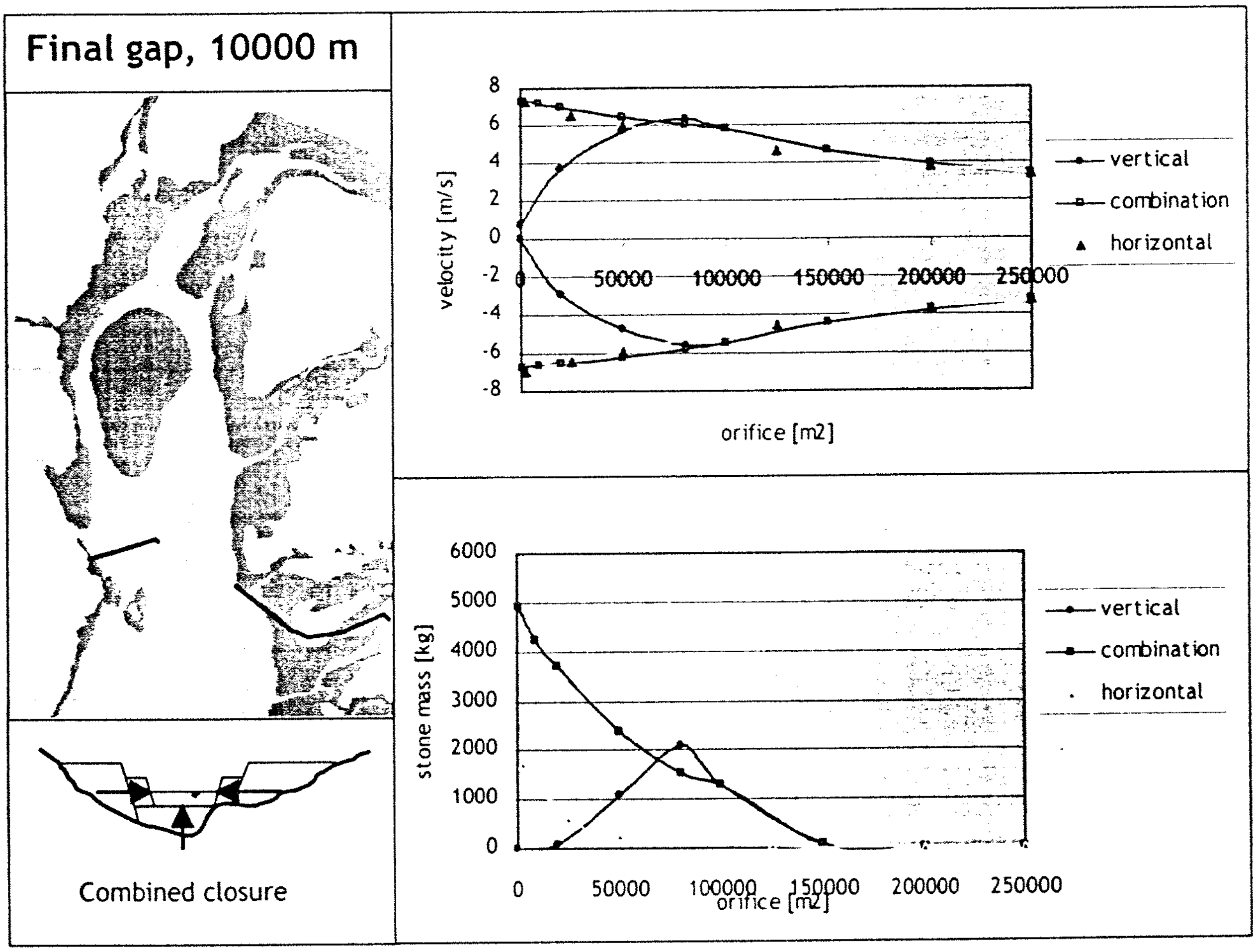


Figure 9.7 Comparison between vertical, horizontal and combined closure

From the figure clearly can be seen that closing with the combined technique offers no advantage in reducing the velocities. The vertical technique results in lower velocities all the time and the horizontal technique requires smaller stones due to the fact that the depth where the high velocities occur is bigger than at the vertical closure (deep water results in a higher Chezy coefficient).

The combined closure therefore is not further evaluated in this chapter. (The big advantage of the combined technique is the availability to use ships for the vertical layers, since these ships can also be used near the dam heads in the horizontal closure, and in a vertical closure, this advantage is also valid for the other two techniques.)

9.3 *Comparing the different closure alternatives*

To compare all the alternatives, they are plotted against the theoretic quarry output. To do so, it is necessary to translate the required volumes to percentages, schematizing the dam as presented in figure 9.8 does this. For the split alternatives two sections of 5000 m are joined, to compare with a no split length of 10000 m.

The stone dam that will be constructed for this experiment has the following dimensions:

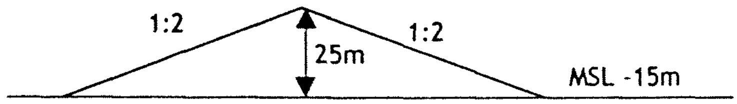


Figure 9.8 Cross-section used to construct percentages

The length of this dam will be 10000 meters, so the total volume of the dam is 12.500.000 m^3. With a porosity of around 35% this will result in a volume of rock in the quarry of 8125000 m^3. It is now possible to determine the amount of required stones of a specific diameter, and compare these percentages with the theoretic percentages.

By comparing the required quarry outputs with a theoretic quarry output (see figure 9.9), it is possible to predict whether the blasting of rock will be economically or that a large overproduction can be expected. This theoretic quarry output is derived partly from Hafkamp (1996) and partly from experience of the Dutch Ministry of TPW&W.
The reality about quarry yield curves is that they always have to be predicted after thorough field investigations by experts. The real quarry yield curve will depend on the parameters of the rock, and the experience of the blasting engineer.

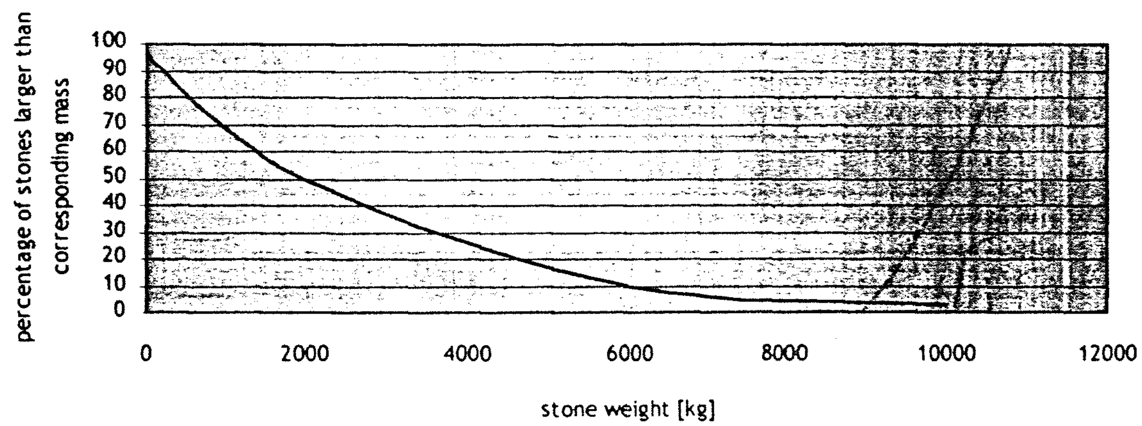


Figure 9.9 Assumed quarry yield curve

Figures 9.10 & 9.11 show the required quarry distribution for each closure concept as presented in paragraph 9.2. It can be seen that some closure scenarios require quarry distributions that can not be delivered. The quarries can deliver all the lines below the assumed quarry yield curve, and all the lines above the assumed yield curve require extra production. These lines represent only the volume of stones required for the gaps, not for the other components in the dam alignment.

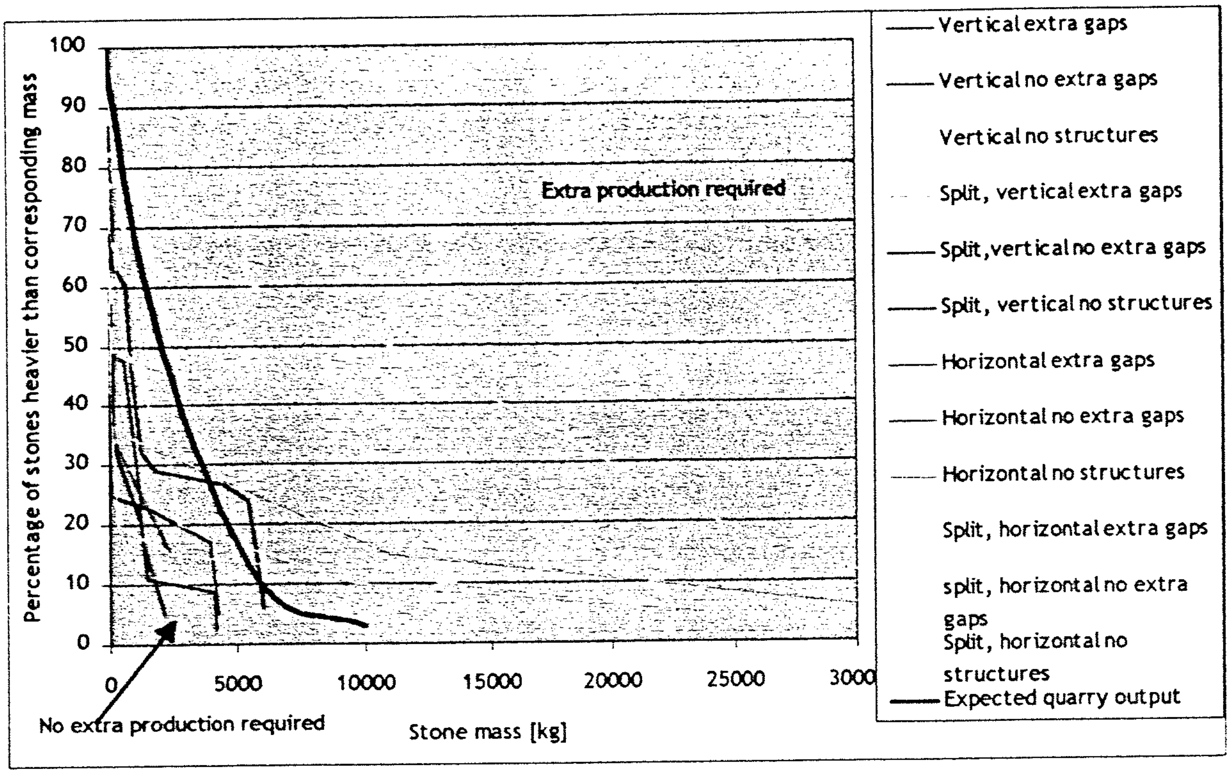


Figure 9.10 Feasibility of closure scenarios with respect to quarry outputs

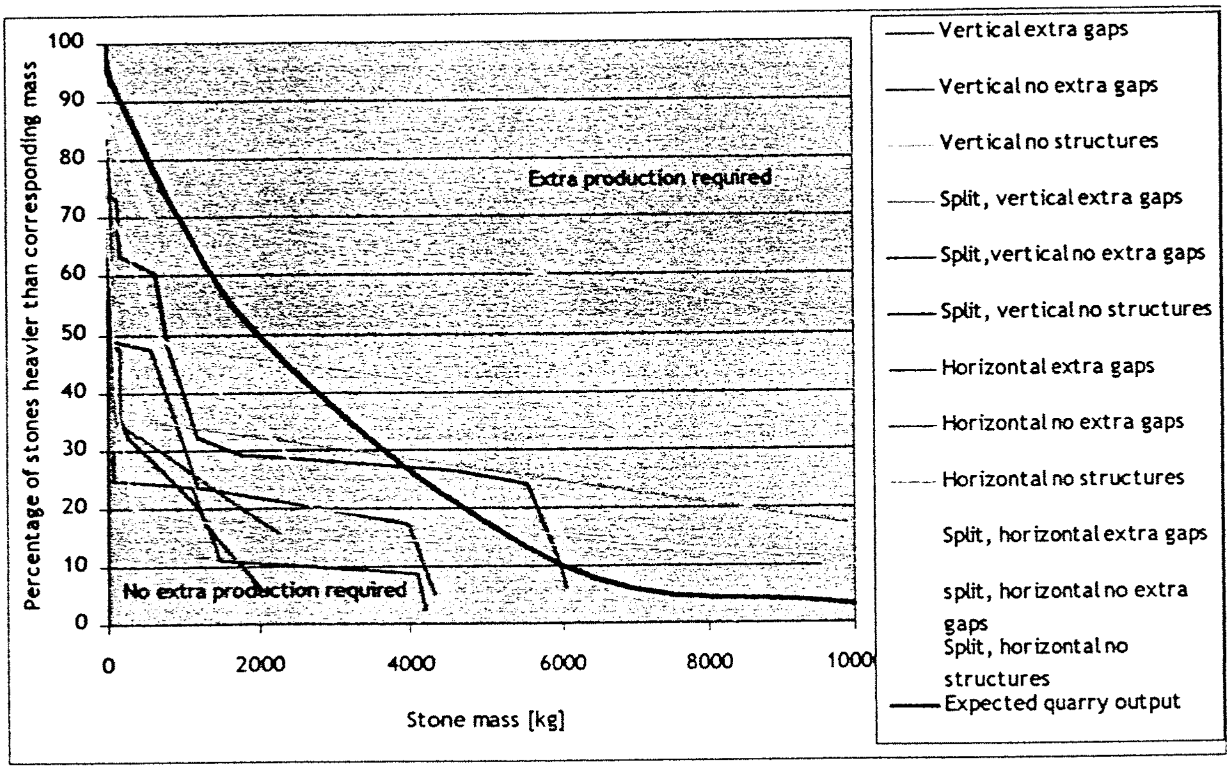
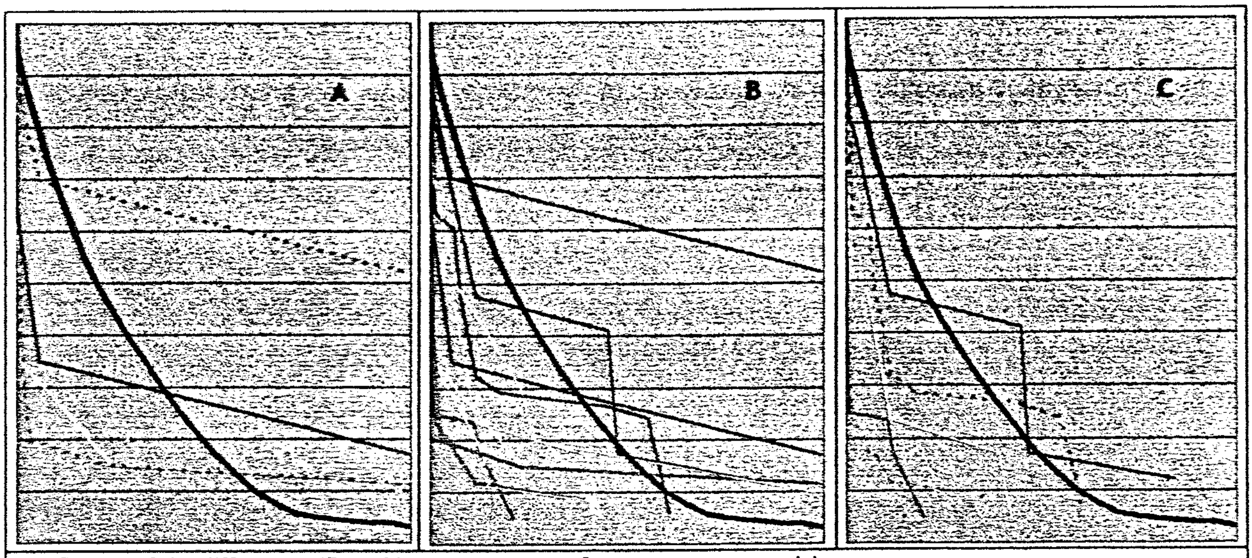


Figure 9.11 Feasibility of closure scenario's with respect to quarry outputs, 0-10000 kg

Figure 9.12 shows the individual influence of the parameters in the closure process. The first and most left picture shows the influence of the different extra gaps in the dam. It is clearly visible that the larger the gaps the better (less overproduction) the required yield curve is. The second picture shows the influence between horizontal and vertical closure. It can be seen that all but one horizontal closure requires heavier stones than the vertical closures. The third picture shows the difference between splitting-up the basin or not. This picture shows that the cheapest option is a closure without a split.

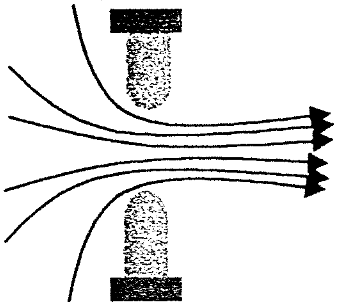


A, influence of the different orifices, blue=extra gaps, yellow=no extra gaps, pink=no gaps
B, hoizontal closures, difference between no split (solid lines) and split (dashed lines), orifices according A
C, Vertical closures, difference between no split (solid lines) and split (dashed lines), orifices according A

Figure 9.12 Influences of different parameters

9.4 Discussion of results

The closure alternatives are calculated with the use of a storage area approach. The orifice discharge factor m (for free-flow) and μ (for submerged weir) have been set at 1. This is not a correct assumption for very small (thus horizontally closed) gaps. These factors are lower than 1 for small gaps because of friction and irregularities in the gap. This results in smaller orifices (as shown in figure 6.3). These smaller orifices result in lower discharges, which results in larger head differences over the gap, which finally results in higher velocities through the closure gap.



The adding of the factors m and μ would result in higher velocities in each gap than calculated, although from the picture can be derived that the larger the gap, the smaller the influence of these factors will be. In the very small gaps, less than 1000 meters the influence of these factors will result in higher velocities, this would require slightly more heavy stones than predicted at this moment.

9.5 Conclusions

The following (remarkable) conclusions can be drawn from the above calculations:

- The incorporation of the tidal power facility in the closure process results in a substantial decrease of the velocities. The scenario with all extra gaps will be used;
- Splitting the basin will not require smaller stones, and is therefore not recommended;
- The horizontal closures require larger stones than the vertical closures. There is however one exclusion, the horizontal closure with all extra gaps results in the most economical stone demands. However the influence of local turbulence factors is not calculated in these results;
- Most closures require a stone gradation that can easily be delivered by the assumed quarry distribution curve.

10 Bottom protection

10.1 Introduction

A described earlier at the different dam parts, the bottom will have to be protected against the (some times very heavy) current velocities to prevent erosion. This is necessary because the bottom materials in the Gulf of Khambhat are sand and clay, sensitive to erosion. Erosion will be visible as deep pits behind the dam sections. These pits can endanger these structures and ultimately lead to collapse of the dam section.

The way erosion develops is described in paragraph 10.2. With the model (chapter 8) the maximum velocities behind each structure/component can be predicted, and a bottom protection plan for the Gulf of Khambhat is developed.

10.2 Theory of local scour

Flow over a sill will always decelerate at the end of the sill, see the figure below. Right behind the sill an eddy occurs and at a certain point the main flow reaches the bottom again, from this point on the two flows start to mix up to a smooth current. At the end of the bottom protection (if placed) this current will erode the sediment (see figure 10.1). It is from the utmost importance to know the development of this scour in time and in depth, because these two parameters will determine the required length and strength of the bottom protection. The development in time is necessary to determine how much protection each phase during closing needs. For the permanent flow and therefore permanent scour from the tidal power facility only the final depth is interesting.

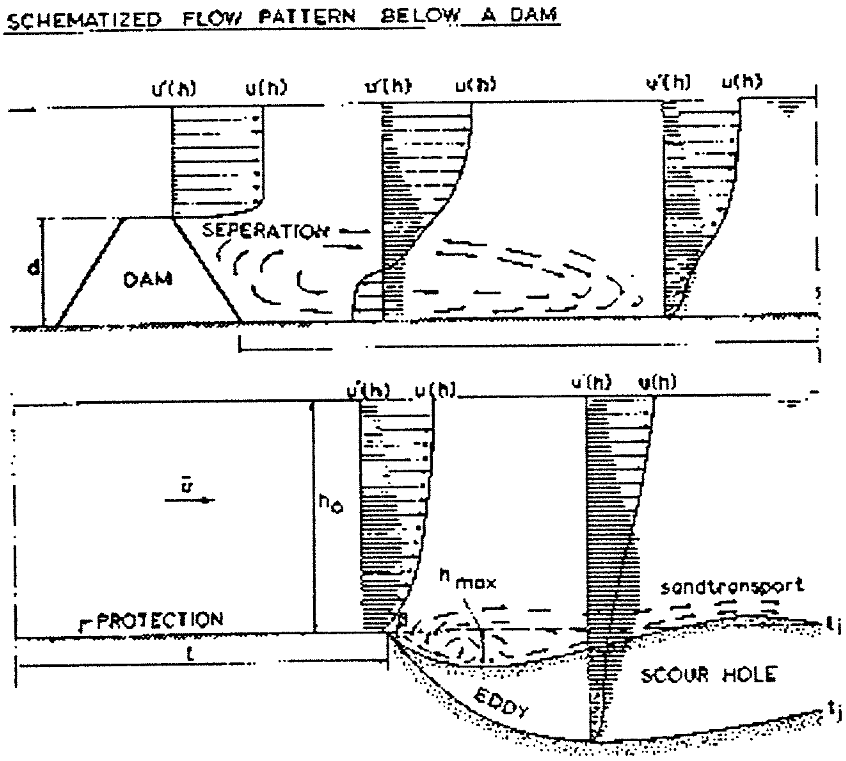


Figure 10.1 Flow pattern behind a sill, causing erosion

Much research has been done during the Delta works to investigate the erosion that would occur at the end of a bottom protection. Note that there will always be some erosion when something is placed in flowing water with an erodable bottom.

In the evolution of a scour hole there are four phases determined during laboratory research (see figure 10.2).

1st phase: The initial phase is only very short and can be described by existing formulas for sediment transport. This phase is of little practical interest.

2nd phase: The development phase. The upstream slope of the scour hole, β, becomes constant. The scour hole is deep enough for flow separation. An eddy exists, having a lower velocity than the main flow and being in the opposite direction at the bottom. This eddy takes the sediment in the lower part of the scour hole. This phase generally lasts much longer than the first phase and is therefore of great interest during closure operations.

3rd phase: The stabilization phase. In this phase the scour hole is so large that the velocity near the bottom nearly reaches the critical velocity at the beginning of motion of bottom sediment. Low velocities mean that the sediment transport also will be low and therefore scour generally takes place very slowly. β Remains constant.

4th phase: The velocity is nearly equal to the critical velocity; there may be some transport near the bottom but the stream is not able to carry the sediment out the scour hole. A certain dynamic stabilization of the scour hole is then achieved. The maximum depth of the scour hole in this phase is defined as the equilibrium depth.

For practical purpose only phase 2 and 4 are interesting. Phase 2 to predict how fast an erosion hole grows, and therefore what kind of precautions is necessary during construction. Phase 4 is the final erosion depth and is very important for construction and use of the structure.

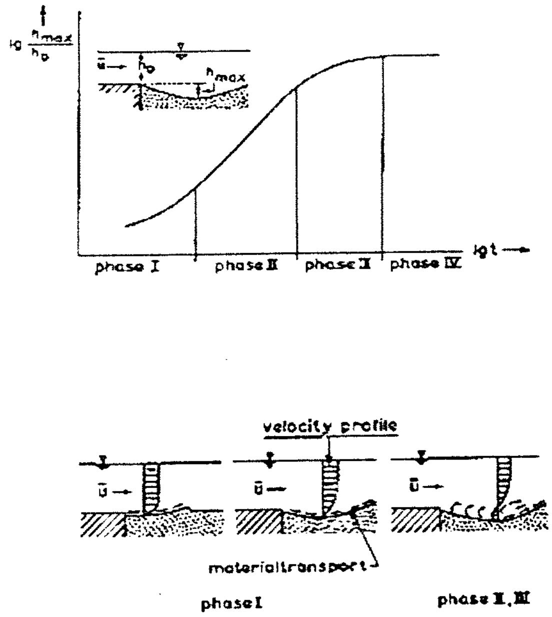


Figure 10.2 Different erosion stages

10.2.1 Phase 2 in scour process

From experimental observations it appeared that during the second phase the scouring depth ($h_s(t)$) behind a bottom protection increases exponentially with time,

$$h_s(t) = \frac{(\alpha\bar{u} - \bar{u}_{cr})^{1.7} h_0^{0.2}}{10\Delta^{0.7}} t^{0.4} \qquad (10.1)$$

In which:

$h_s(t)$ = maximum scour depth in (m) as a function of time;
h_0 = water depth behind the sill (above bottom protection) in (m);
t = time in hours;
α = dimensionless scour factor involving velocity distribution and the influence of the turbulence due to geometry of the structure.
α can be estimated by the following relation:

$$\alpha = 1.5 + 5r \text{ for } \alpha > 1.8 \qquad (10.2)$$

In which

r = the roughness parameter. Roughly can be said that r varies linear from 0 for a non-disturbed flow (no sill, only bottom protection) to 0,5 for a sill just below the water level.

α Will decrease by using long bottom protections L/h_0 >5. And α will increase by using smooth bottom protection material like asphalt, only for C>40. These two factors are not important for the first estimates.

$\bar{u}$ = Mean velocity in (m/s) = Q/A, Q is discharge in m^3/s, A is the wet cross-section at the end of the bottom protection = ($B * h_0$) in m^2;

u_{cr} = Critical velocity for initiation of motion in (m/s). u_{cr} can be determined with the formula:

$$u_{cr} = 1{,}7\sqrt{\Delta g d} \quad \text{(Isbash)} \tag{10.3}$$

Δ = Relative density of bottom material under water = $\dfrac{\rho_s - \rho_w}{\rho_w}$ (10.4)

10 = Constant but not dimensionless.

Formula 10.1 is only valid for phase 2; $(\alpha\bar{u} - u_{cr})$ should therefore be not too small (> ≈ 0.1 m/s). Otherwise the process is in fact in phase 3.

The exponent 0,4 in the equation is derived from two-dimensional model investigation. For 3-dimensional flows, this factor starts a little higher and ends a little lower than 0,4. The value of 0,4 will be used in this report. This is also recommended by author of F4 [lit 17].

10.2.2 Influence of the tide on erosion time

For scour in a tidal flow, formula 10.1 can be used with the tide taken into account as a succession of stationary flow situations, replacing $(\alpha\bar{u} - u_{cr})^{1.7}$ for each flow direction of the structure, by:

$$\frac{1}{T}\int_0^T (\alpha\bar{u} - u_{cr})^{1.7}\,dt \tag{10.5}$$

It's obvious that a scour hole in a tidal flow will not grow as fast as a scour hole in a stationary flow.

10.2.3 Phase 4 in scour process

From experiments a rough but simple relation has been found for the equilibrium depth in phase 4:

$$h_{se} = \left(\frac{0{,}5\alpha\bar{u} - u_{cr}}{\bar{u}_{cr}}\right)h_0 \tag{10.6}$$

The parameters in this formula are the same as in formula 10.1.

10.2.4 Checking the validity of the theory

An example with some Khambhat values to predict the final scour depth gives a quite unexpected result. The sand on the bottom has a diameter of 200 μm (0,2 mm).
For this sand the critical velocity is determined using formula 10.3

$$u_{cr} = 1{,}7\sqrt{\Delta g d} \quad = 1{,}7\sqrt{1.6 \times 9.8 \times 0{,}0002} = 0.1\,m/s$$

Taking the minimum value for α (= 1,8), and implementing u_{cr} and α in formula 10.6 gives the following relation:

$$h_{se} = \left(\frac{0{,}5\alpha\bar{u} - u_{cr}}{\bar{u}_{cr}}\right)h_0 = \left(\frac{0.5 \times 1.8 \times 6 - 0.1}{0{,}1}\right)h_0 = 53h_0.$$

This is unrealistic high (the depth in the gullies is over 25 m), therefore another relation for the scour holes is needed.

The new relation is based on field measurements during the Delta Works. From these measurements appeared that a scour hole never reaches a depth larger than three or four times the undisturbed water depth. For the Gulf of Khambhat this would result in maximum scour holes of 120 m deep, still very much.

10.2.5 Strategy

As shown, the standard erosion formulas are not valid here. In the Kalpasar study great effort has been made to predict the scour holes that develop at the end of the bottom protection. However, calculations indicate that hole depths up to 150 m (6 to 7 times the water depth) can be expected. Calculations with the formulas described earlier show even greater depths. This is due to the extreme difference between the actual and critical velocity. These depths have never been recorded and seem very unrealistic.
Experience during in the Delta-works showed scour depths of 2 to 3 times the water depth.

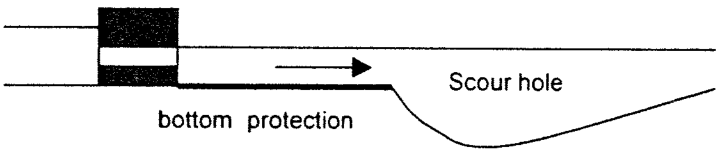
It is outside the scope of this study to make extensive calculations, as it will be very difficult to predict these depths without extensive modeling. For the scour holes an erosion depth of 2,5 times the average water depth is expected maximum to occur.

10.3 Stability of a construction

The question is now, how can a scour hole destroy a structure. This paragraph deals with the question what happens when a scour hole develops.

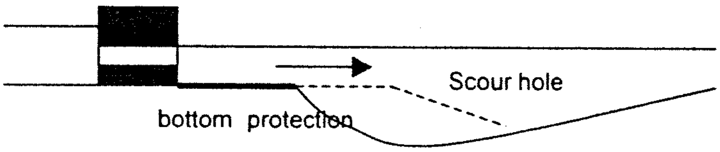
Scour behind a bottom protection as mentioned in the above paragraph is not a problem as long as the structure doesn't collapse. There are tree situations that can occur after being exposed for a long time to a heavy current.

1.



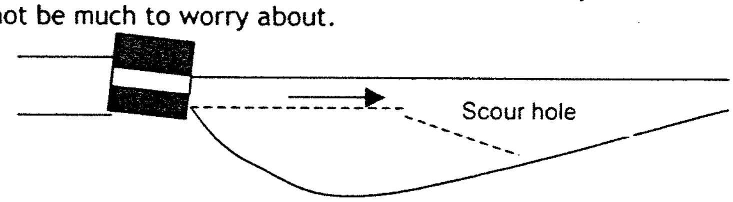
In this situation a scour hole has developed, but the bottom protection is still intact, so there are no negative consequences for the construction.

2.



In this situation the bottom protection has partly collapsed, this does not directly endangers the construction although repair work should be done as fast as possible, by dumping stones or other material. The danger from a collapsed bottom protection depends on the material where it is made from, the collapse of a gradual filter could be a great threat for the structure because there is a severe risk that it will collapse further. If on the other hand the protection is coherent for instance by the use of a strong geotextile, there will not be much to worry about.

3.



If the bottom protection is not repaired in time and collapses further, there is a severe risk that the scour hole will grow until it reaches the structure and that the structure itself will collapse.

10.3.1 The slope angel β of the scour hole

The way that the bottom protection will collapse is always the same (assuming that the protection is strong enough to withstand the attacking current). The upstream slope β becomes to steep and collapses.

From systematic scour research Hoffmans, 1993, derived the relation for β:

$$\beta = \arcsin\left[3 * 10^{-4}\frac{u_0^2}{\Delta g d_{50}} + (0.11 + 0.75 r_0) f_c\right] \quad (f_c = \frac{C}{40}, \quad f_c = 1\ for \quad C \leq 40) \quad (10.7)$$

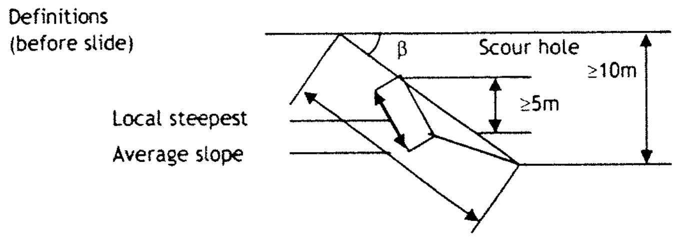
The velocity, turbulence, a smooth protection and fine sand, cause steep slopes. Experiments resulted in higher values for β than the natural angle of response φ.

10.3.2 Stability and slides

If a slope becomes to steep over a certain height, it is not able to withstand gravity forces and slides. Soil moves side and downward causing a flatter slope. For first approximation it is assumed that this will happen with cotβ>2 with h_s (unprotected) >5m. The two possible collapses are drawn in figure 10.3:

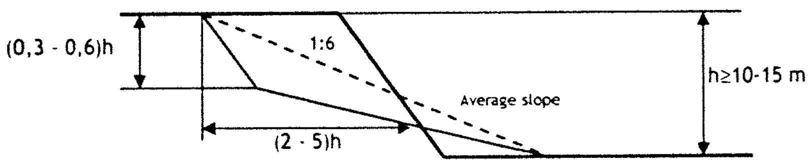
Permissible scour hole/ empirical criteria

- Max. scour depth, h_{max}
- Max upper slope, β



Criterion occurance	Before slide				After slide
	Steepest		Average slope		
	Slope ≥	*Depth ≥	Slope ≥	Depth ≥	Average slope
Flow slide	1 : 4	5 m	1 : 7	10 m	1 : 15
Normal slide	1 : 2	5 m	1 : 4	10 - 15 m	1 : 6

Normal slide



Flow slide

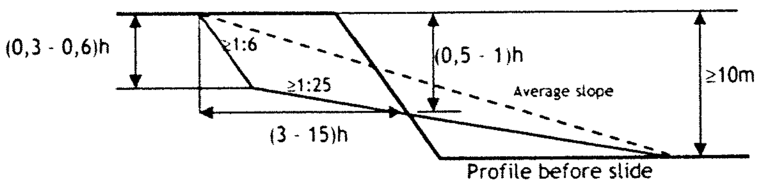


Figure 10.3 Stability and slides

Sand slides to a gentler slope than the angle of repose ϕ. For an average slope for densely packed sand 1:6 can be used. Loosely packed sand temporary becomes a thick fluid when it collapses (liquefaction). The slope will behave like quicksand, resulting in a much flatter slope than with a normal slide. An average value for the slope of 1:15 is recommended.

Assuming that the sand is not sensitive to liquefaction, the first type of sliding resulting in a slope of 1:6 determines the required free space behind the structures.

10.4 Structures to be protected

A division can be made regarding the structures in the Khambhat dam between structures that have to be protected during both construction and operation and structures that only need bottom protection in the few months or years of construction.

The tidal power facility and the Narmada spillway need to be protected for the design period of 100 years. However, the highest loads occur during closure operations when the gates are used to reduce the current velocities in the final gap. The maximum current velocities of 8,0 m/s occur at spring HW when the final gap is almost closed.
During normal operation (producing electricity) the maximum velocity will be lower because the storage area is reduced (570 instead of 2187 km^2). When the sluice gates (inlet structure) and the turbines are used for filling the reservoir, the maximum velocity will be 6,9 m/s.
It is necessary to use this (conservative) value of 8 m/s during closure. Because during use of the tidal power facility the accepted risk is much lower than during the construction, it is not possible to accept a smaller determining velocity. During construction large amount of stones and equipment are available to repair a failure immediately. In these 8 m/s, occurring during closure of the final gap, local turbulence is not incorporated. This turbulence would require even heavier stones than 8 m/s plain. Because the 8 m/s situation occurs only for a short time, 8 m/s without turbulence is regarded to be safe.

The design head difference value for the Narmada spillway is not determined. When the spillway should be able operate under all circumstance, the design head is 10 m, resulting in a current velocity of 14 m/s. This is a very high value. This current however occurs not over the full depth, and will rapidly spreads over the entire depth, reducing to 8 m/s.

The final closure gap has only to be protected during closure operations. The maximum velocities are calculated at 6,5 m/s. This velocity is also expected for the island section between the turbines and the inlet sluices of the Tidal power facility.

The sections between the Tidal power facility and the final closure gap and the closure dam of the Narmada are opposed to maximum velocities of 3,5 m/s. The section between the Tidal power facility and the final gap has to be protected. The section in the Narmada mouth will be constructed out the local bottom material clay. The clay is able to withstand the 3,5 m/s current and therefore this section needs no bottom protection.

The minor dam sections on the shoals near Ghogha, the work islands and harbors have to be protected against flow with a maximum of 2 m/s.

These structures and their lengths and design velocities are shown in figure 10.4.

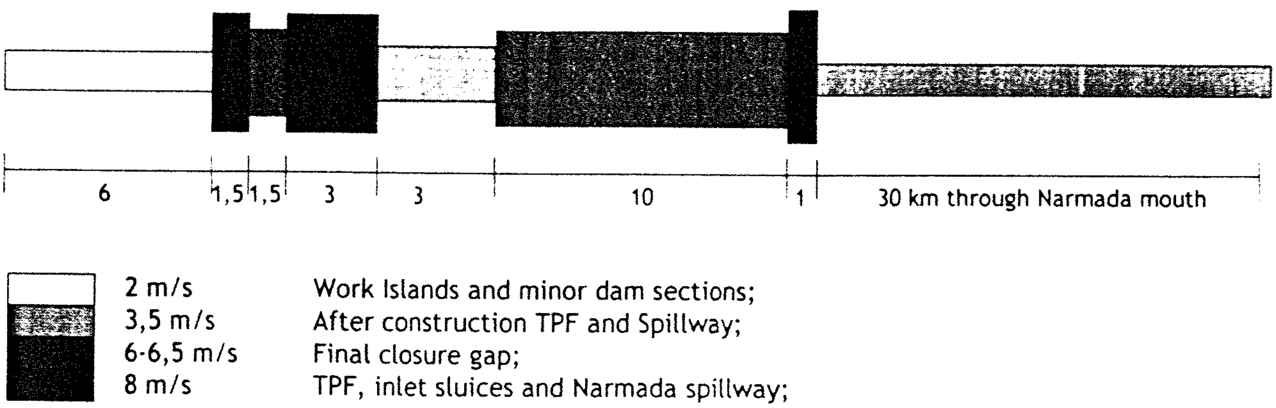


Figure 10.4 Different zones to be protected

10.5 Bottom protection behind the tidal power facility and spillway

The bottom protection behind these structures has to withstand high current velocities for the design period. As the inlets are positioned close to each other, the flow behind the structure will not spread horizontally easily. In fact, both the sluices and the turbine section work as very wide jets. When the bottom protection is assumed to be horizontal, the flow can not spread in vertical direction either. The bottom protection should then be very long and, due to the still high current velocities, deep scour holes will develop.

It might be better to place this bottom protection on a slope. To do this, trenches have do be dredged on both side of the structure, starting at about 50 to 100 m from the structure. These slopes will be gentle (about 1:7) to prevent the break free from the current off the slope. With a length of 200 m, the 'hole' depth will be 30 m, which is about half the expected scour depth. An unprotected bottom that slides has an expected steepness of 1:6 (see paragraph 10.3). When the slope is protected the bottom material and the outflow of water (porous flow) determine the steepness. The seepage is assumed to be almost horizontal, which leads to a maximum steepness of $\varphi/2$ = 15 degrees (1:3,7). This means that the slope will be stable (no slides).
This principle is shown in figure 10.5, compared to a horizontal protection.

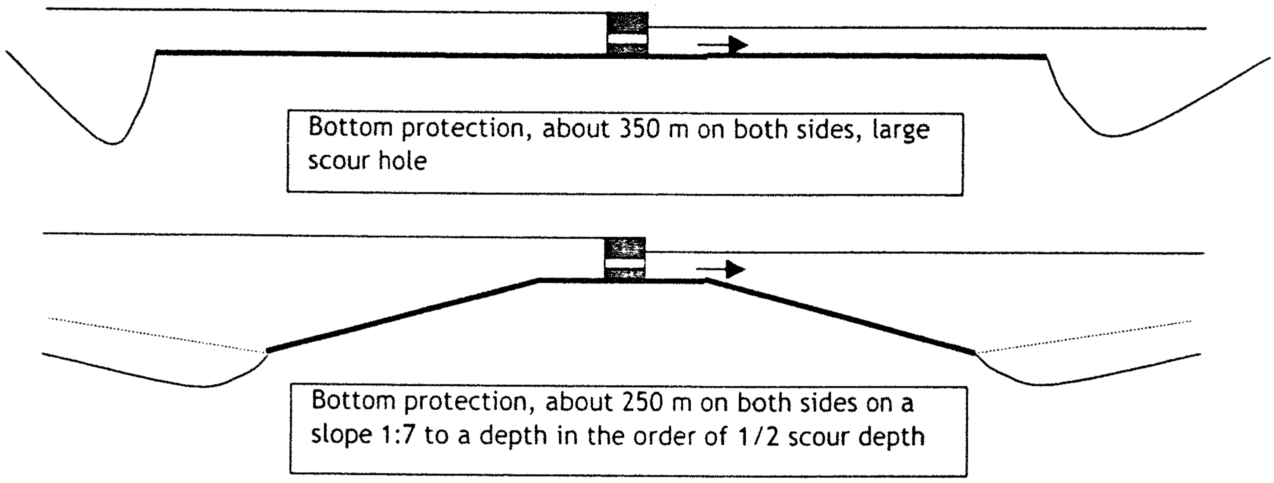


Figure 10.5 Difference between horizontal and bottom protection on a slope

The greatest advantage is that the current velocity is reduced considerably. When the water depth is increased from 30 to 60 m the velocity decreases by a factor 0,5. At a distance of 300 m downstream of the structure the flow is spread horizontally which also decreases the velocity. The sluice inlets have an effective flow width of 1200 m in a total width of 1500 m. Assuming a horizontal diversion of 1:8, the flow width at a point 300 m downstream is 1500 + 75 m. This reduces the velocity by a factor 0,72.

The design velocity is 8 m/s. At a point 300 m downstream the velocities are 5,7 m/s for the 'normal' protection and 2,8 m/s for the protection on slopes. The scour hole will develop anyway, but it is smaller behind the protection and it is more controlled. Therefore this dredging and protecting on a slope is recommended.

Still the bottom protection will have to cover a large area, but it will definitely be smaller. Dredging can be done until the expected scour depth is reached, but also to a lower depth of, say 2/3 of the expected scour depth. The end of the bottom protection can be covered with a large amount of stones that cover the slope of the scour hole (falling apron). This is often applied in Indian rivers. This is indicated in figure 10.6.

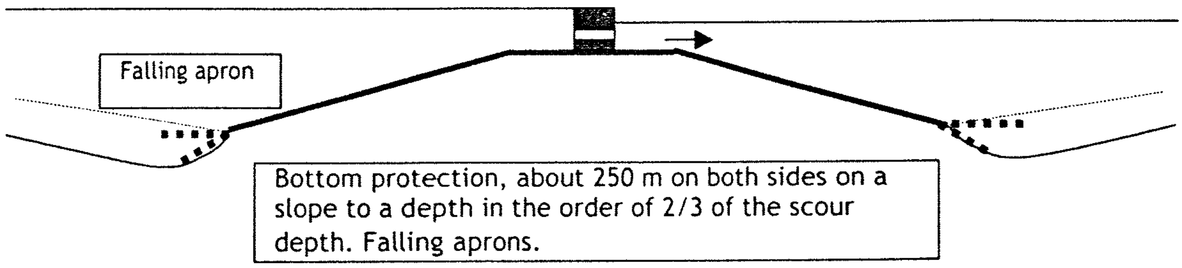


Figure 10.6 Falling aprons

10.6 Types of bottom protection

Several types of bottom protection exist. Which is best for the given situation depends on the expected maximum current velocity. This determines the top layer. Underneath this layer a filter should be constructed. The function of the latter is to prevent sand from eroding through the bottom protection.
Dutch Ministry of Transport, Public Works and Watermanagement designed for the Haskoning study four different principles, which are shown in figure 10.7. A classification in 'loose', 'light', 'heavy' and 'very heavy' is detailed enough in a pre-feasibility study. These designs are applied in this study. The bottom protection behind the tidal power facility will be placed on a slope as suggested. The last type in figure 10.7 is a so-called 'sand mattress', which is described in paragraph 10.9. The sand mattress is designed for possible use in the final gap.

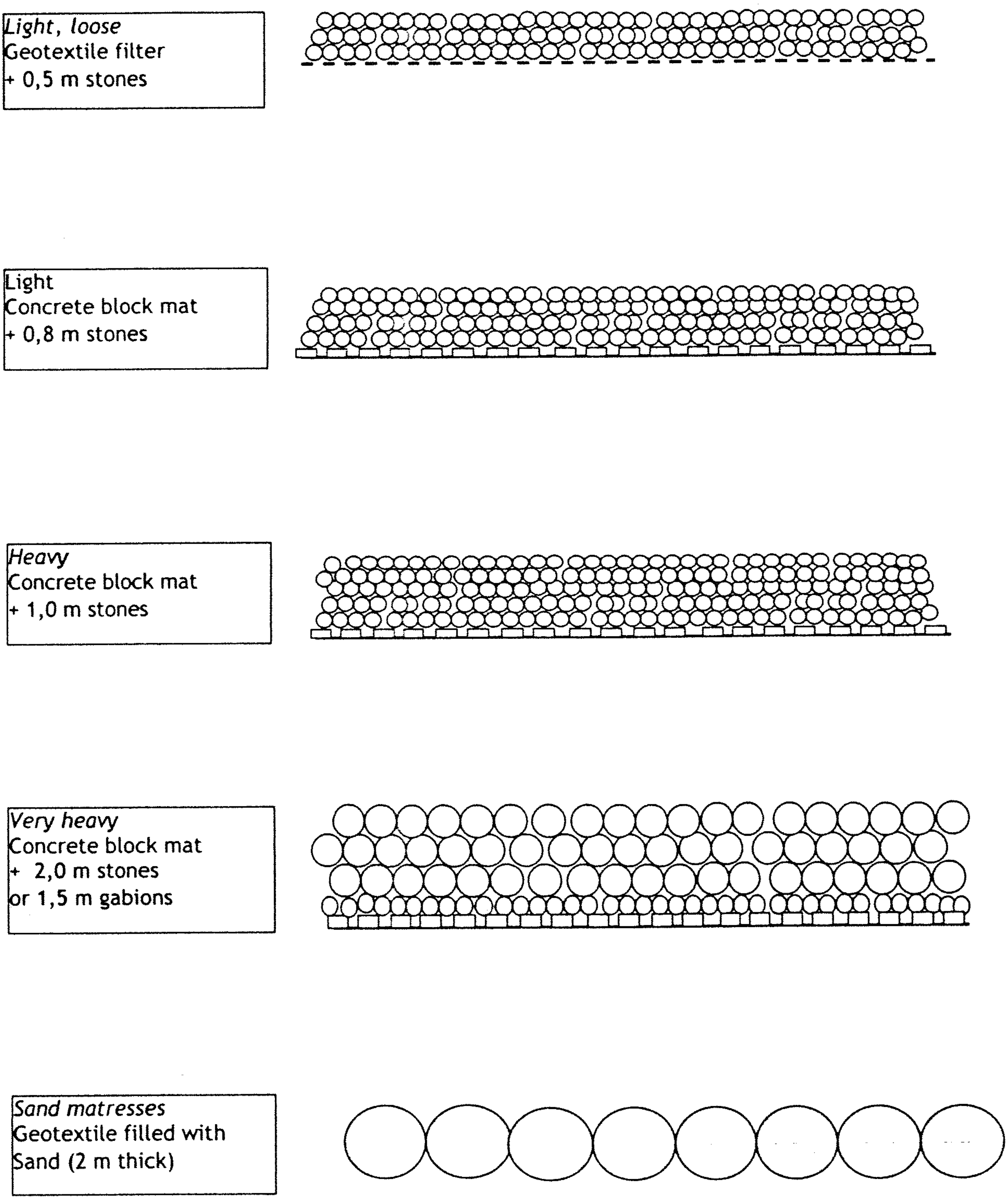


Figure 10.7 Types of bottom protection

10.7 Required amount of bottom protection

When the figures with the length and type of bottom protection are combined, the total amount of bottom protection can be determined (table 10.1).

Table 10.1 Overview bottom protections

	Length of section in m	Average depth in m	Length of bottom protection	Design velocity in m/s	D_{50n} In m	Type of bottom protection	Total amount in m2
Tidal power facility	3000	30	550	8	1,05	Very heavy	1650000
Inlet sluices	1500	30	550	8	1,05	Very heavy	825000
Spillway	1000	25	550	8	1,05	Very heavy	550000
Work island and final gap	14500	15-20	400	6,5	0,5	Light	5800000
Secondary damsections	6000	10-15	250	2	0,2	Loose	1500000

This results in a total amount of bottom protection with a surface of 10 million m^2.

The protection behind the tidal power facility turbine section should be very heavy during closure when the temporary gates are used. During normal operation the high velocities occur only inside the turbines, and not outside the structure. So, the bottom protection could have a decreasing flow resistance after several years. This could be done by gabions filled with stones. The gabion should be stable during extreme closure conditions, and the individual stones during operational conditions. When the gabion material rusts the transition between gabions and individual stones takes place after several years. This could fulfill both wishes.

The dam in the Narmada mouth will be constructed with clay. This dam will need no bottom protection. It might be possible that the last part (the gully of the Narmada) will need some protection. If necessary, this will be a 'light' protection with a small surface.

For the final gap the type of closure technique mainly determines the actual amount of bottom protection. At the start of the closure the maximum occurring velocity is 3,5 m/s, the highest possible velocity is 6,5 m/s. If the higher values only occur during a very short period (sudden closure), the demand to the bottom protection are different than when the high velocities occur longer.

The (heavy) protection mattresses should be as large as possible. Dimensions of 250 * 50 m are suggested.

10.8 Sand Mattresses

10.8.1 Introduction

During the delta works experience was built up with several methods of bottom protection, other than traditional dumping of stones. For instance mattresses of geotextile with concrete blocks on it, or prefabricated granular filters in mattresses, and mattresses of geotextile filled with sand.

This last idea seems to be very suitable for the enormous amount of bottom protection needed in the Gulf. There are two main options for the mattresses.

- Fabricating of the mattresses on shore, and transportation to the site;
- Waterborne fabricating of the mattresses on the site by a machine that also places the mattresses on the bottom of the Gulf.

10.8.2 Required thickness of mattresses

First thing to know is the needed thickness of the mattresses. There are two separate working forces on the mattresses, a lifting force caused by the flowing water, and an uplifting force resulting from a pressure build-up caused by the head difference over the dam. The influence of the pressure will not be very high by the following reasons:

- The mattresses will not be watertight;
- The head difference will be very small during most of the construction phases, only when the Gulf is closed the head difference will be much higher, but at that time the need for bottom protection is absent.

Therefore only the lifting force is taken into account. The lifting force can be described by the following formula:

$$F_L = \tfrac{1}{2} C_L \times \rho_w \times u^2 \times A_L$$

In which:

C_L	=	the lift coefficient;
A_L	=	the surface being opposed to the lifting force (m^2);
ρ_w	=	density of water (1035 kg/m^3);
u	=	water velocity (m/s).

For C_L a value of 0,5 is estimated, based on several handbooks in flow mechanics. The force is calculated for one square meter so A_L=1. Therefore the lifting force will be 0,25 * 1035 * u^2 (N/m^2). There can be made a remark on the impact of the lifting force because the mattresses are not watertight like the objects where this formula is made for. For a first attempt the lifting force is estimated at:

u = 6m/s, F_L = 0,25 * 1035 * 36 = 10 kN/m^2 This requires a weight of the mattress of at least 10 kN, which is the submerged weight of one cubic meter sand. For safety and other uncertainties a thickness of 1,5 meters is recommended.

10.8.3 Size of the mattresses

Because the area that has to be covered is very large, it is necessary that the mattresses are quite large. A first estimate for the basic dimensions is length 300 meters, width 50 meters.

10.8.3.1 On shore production

The production on shore can be done in two separated ways, dry-fill and wet-fill. The latter should be done with dredged sand. The production will not give any problems and therefore is not of direct importance. The transportation of such mattress will be the bottleneck.

There are two main options for transportation:

1 Winding the mattresses around big cylinders and tugging them to the place where the mattresses are needed. Placing on the bottom will require a special vessel. The cylinder in the mattress-roll will have to carry at least 17700 m^3 sand. This requires a cylinder diameter of 24 meters. When the mattress is added, the draught is over 30

m. The depth makes it impossible to sail with these cylinders. The only option will be using smaller mattresses, which is not recommended for this project.

2 Producing of the mattresses directly on a barge or ship that sails to the location where the mattress is needed and where it will be placed directly from the barge. The mattress will be the same 17700 m^3. This requires a barge of 100 * 50 * 3.5 meters. In that case the mattresses will be lying in three layers on the barges. It is even possible to use existing pontoonships, which have almost the same dimensions as the requested barges.

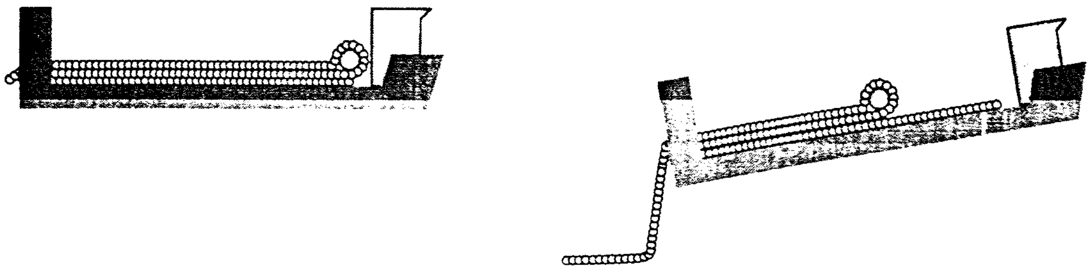


Figure 10.8 Mattress laying vessel

10.8.3.2 Off-shore production

The basic idea is to design a machine that is able to produce a continuous sand mattress. This machine will have to do the stitching and the filling of the mattresses on the location itself (figure 10.9). The advantage from this method versus the on-shore method is the possibility to fabricate mattresses with very big lengths (up to 1000 meters).

Main problem during construction will be the current velocity. Although the velocities are relatively low during the fabrication of the mattresses, it will be impossible to fabricate mattresses perpendicular to the flow direction.

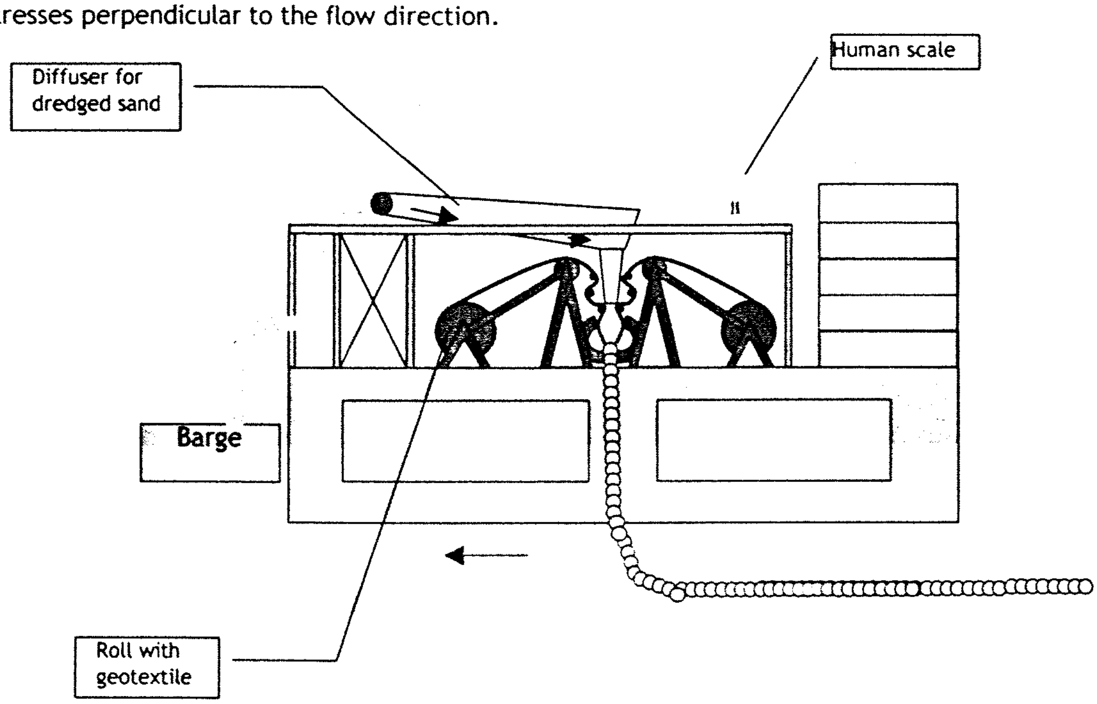


Figure 10.9 Mattress producing and laying vessel

10.8.3.3 Very severe current attack

In the final closure gap it might be possible that the mattresses will not be heavy enough to withstand the flow, the shape of the mattresses allows the dumping of large stones directly on the mattresses, only a fill up layer of medium stones has to be dumped in between.

10.8.3.4 Strength of the sand mattresses

The main problem with these sand mattresses is internal stability. It will be very difficult or almost impossible to create 100% filled tubes. That means that inside the tubes enough space is left for the grains to move through these tubes. With single 'sand sausages' this phenomena leads to movement of the bag. It is very unlikely that a mattress will roll away, on the other hand the sand will absolutely move through the tubes. This internal transport leads to densely packed sand on some locations and no sand on others. The empty part might flutter into pieces in the heavy current, inducing the collapse of a whole mattress.
It is assumed that this will not happen or can be prevented by adapting the design.

The heaviest load on the mattresses will occur during the placing operations. During placing the mattresses have to carry part of their own weight.

The longtime behavior of geotextiles is uncertain, therefore applying geotextiles at locations that are closed off permanently, like the sill under the tidal power facility, is not allowed.

10.8.4 Conclusion

Because the long-term behavior of geotextile is uncertain, it is not allowed to place the sand mattresses under the concrete structures. For the final gap it seems to be a promising solution.

Study to the behavior of sand in mattresses is necessary (already some experience exist, but not on this scale), it is unwise to recommend using these mattresses as permanent bottom protection. For use in the final gap, the closure technique has to be determined. Part C contains a special paragraph about the bottom protection in the final gap.

Offshore production seems to be the best, because it avoids the transportation of the very heavy mattresses.

Part C Overview of closure methods for final gap

This chapter deals with the closure of the final gap. This final gap is located in the eastern part of the Gulf of Khambhat. After construction of the shiplocks, the Narmada spillway and the dam through the Narmada mouth, there remains an open area defined as the final gap. The width of the gap is 10000 m or 10 km (see figure 11.1). This section has an average depth of CD -15 m (depth varies between CD -13 m and CD -17 m).

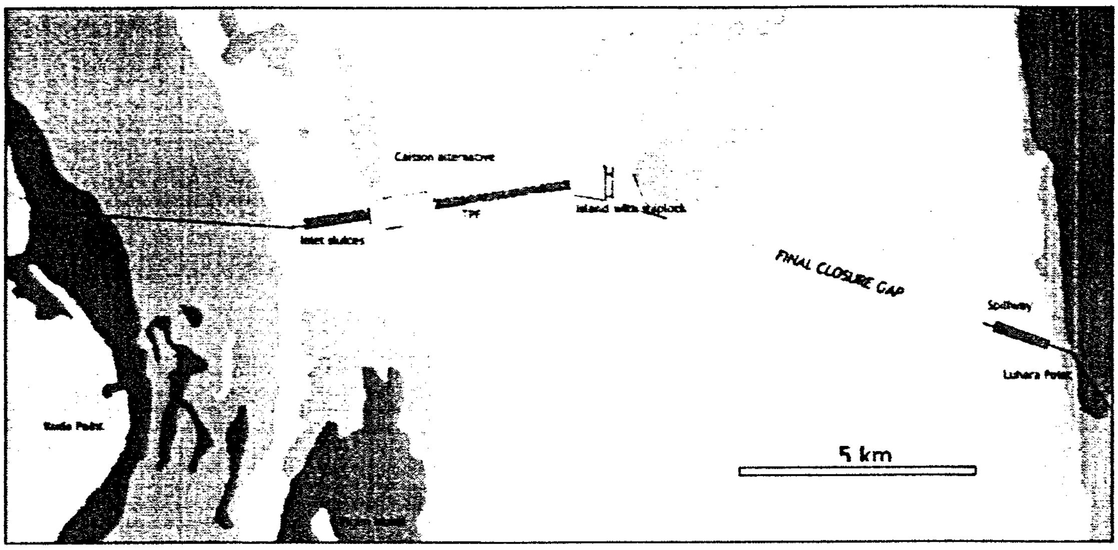


Figure 11.1 Dam alignment

Chapter 11 describes the various gradual options to close the final gap and makes a choice between the various strategies. Chapter 12 describes the overkill closure, a new gradual closure concept and chapter 13 the non-rock closures.

11 Rock closures

11.1 Introduction

This chapter describes the possible methods to close the final gap with quarry stones. For the use of stones two techniques have been considered, the 'traditional' and the 'overkill' closure.

Traditional closure
The classic way of stone constructing is the avoidance of loss; most stones are stable. This way of using rock is referred to as a traditional closure. Stability of stones is determined by the Shields relation (velocity) and by the relation for head difference.

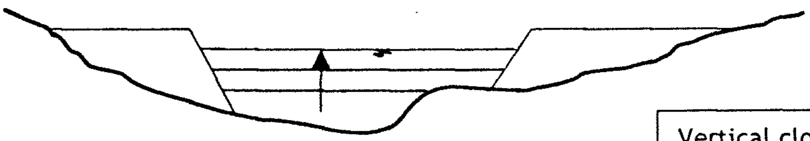
Overkill closure
The principles of the overkill closure are easy: by accepting a certain percentage of loss out the dumped stones it will be possible to construct the dam with smaller stones. This is favorable for two reasons:

- Smaller stones are easier to handle;
- Smaller stones form a bigger part of the quarry output and are therefore cheaper.

Accepting a certain amount of loss implicates that for the same progress a higher capacity is required, which is a disadvantage.

The effects of the overkill technique are calculated later on in chapter 12. The possible ways to dump the stones are discussed in this chapter.

11.2 The gradual vertical closure



Vertical closure

11.2.1 Bridge

One of the options to close the final gap vertically is using a large temporary bridge. This bridge can be used either for a traditional closure or the proposed 'overkill stone closure', which will be studied later.

11.2.1.1 Basic design requirements

The basic form will be a double railway track with a central construction. Trains are preferred above dump trucks because a higher capacity is possible with trains, and it is possible to use standard equipment for the complete route from the quarry to the dumping location.
The scale of this project is far too large for ordinary dump trucks, only new very big trucks would offer enough capacity, but also would create tremendous problems for the necessary strength of the bridge (high axial loads). Therefore the use of a train bridge is recommended. The length of the final gap and the necessity of a high capacity demand a double way railway track.

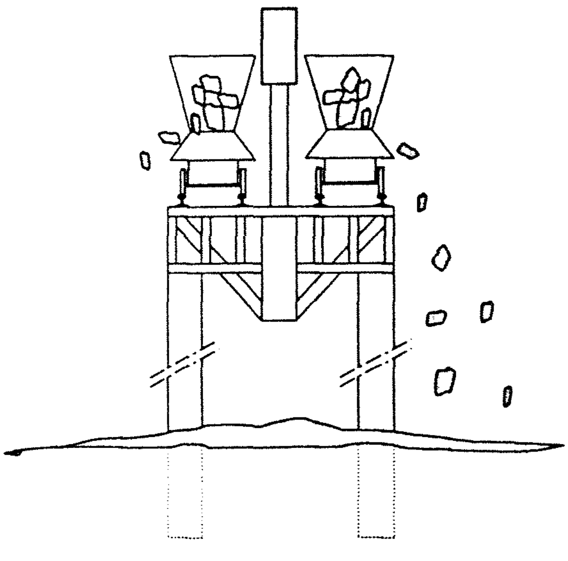


Figure 11.2 Temporary bridge

The trains will be loaded in the quarry and will dump their loads when traveling across the bridge, this is the main reason why the bridge construction has to be in the center of the bridge. This bridge requires at least two quarries, one on each side of the Gulf.
The length of this bridge will be 10 km. This is the distance from the island with the shiplock to the spillway near Luhara Point (final gap).

11.2.1.2 Dimensions of the dumping bridge

With the help of the Dutch standard for railway bridges design (Voorschriften Ontwerp Stalen Bruggen (VOSB 1963)) and some basic formulas the following first design was made:

- Span: 50 meters;
- Central construction: tubular structure with dimensions 2 * 1 * 0,05 (height * width * thickness of the walls) are able to withstand the load of the trains;
- Heart to heart distance of these tubes: 5 m;
- Foundation: double pile foundation.

11.2.1.3 The advantages and disadvantages of the dumping bridge

The most important advantages of the railway bridge are:

- Very high capacities possible;
- The train can unload while running across the Gulf, so without stopping;
- Existing technique (bridge & trains);
- Very flexible, every place in the gap can be reached from the beginning of the closure;
- Independent from climate;
- Also suitable for horizontal closure.

Main disadvantages are:

- The rest of the dam body should be finished before the final gap is closed;
- The bridge will be expensive;

11.2.1.4 Costs

The first cost estimate for this bridge is based on the surface of the bridge. A price estimate for such a bridge is NLG 1500 per m^2. The price of the piles then has to be added.
The width of the bridge will be 10 meters, the length is 10000 meters. This assumption results in a cost of NLG 150 million. The hammering of the piles is estimated at NLG 100.000 per pile. With 2 piles each 50 meters, 402 piles are needed, this will cost NLG 40,2 million.

These three cost estimates, combined with the length of the bridge give the following costs: NLG 190 million.

11.2.2 Cableway

11.2.2.1 Introduction

Another option to close the gap vertically is using a temporary cableway. A big advantage of a cableway is the possibility to lower the stones before they are dropped. This reduces the possible damage to the stones. The disadvantage is the flexibility of the structure. Just dropping the stones would set the cablecar in a dangerous oscillation.

11.2.2.2 Basic design requirements of the cableway

The basic form will be a two-way cableway. Two main cables running across the Gulf. There are two options for the cablecars, self-moving cablecars, and cablecars fixed on the cable that circulates like the little tourist-gondolas in the Alps.

The length of the cableway will be the same as the length of the bridge, 10 km. This length makes the second option, a rotating cable, not very suitable because the force required for this rotating movement will be very high, and the cable will have an incredible length (20 km). The static cable does not require a continuous cable.

The cables will be stretched between towers, and placed on concrete caissons. These caissons are placed on the bottom of the Gulf (see figure 11.3) and will have to be constructed in a building pit, preferably on the east side of the Gulf. (The gap is located in the east.)

More than two cablecars will operate, but the number of cablecars is limited by the number of towers. Only one cablecar per section is allowed. This means that the total number of cablecars is the double of the number of towers (double way).

Loading stations will be at each end. The stones will be delivered at this location by train or truck. This process requires an extra width of the dam in the middle of the Gulf.

11.2.2.3 History of cableway in The Netherlands

During the Delta Works cableways have been used three times, these cable ways had the following dimensions:

Table 11.1History of cableways in the Netherlands

Location	Length of cableway	Material	Capacity
Grevelingen, 1963	630 + 630 m	Rubble 60-300 kg In loading nets	120 tons/hour
Haringvliet, 1970	565 + 580 m	Telpher of 20 tons with 4 concrete blocks of 2,5 tons each	300 tons/hour
Brouwershavense Gat, 1972	380 + 395 +380 m	6 concrete blocks of 2,5 tons with telpher of 17 tons	1000 tons/hour

The main cables of the Haringvliet cableway had a diameter of 92 mm.
A telpher is a special unloading unit.

11.2.2.4 Dimensions of the cableway

With the historic table above and with some basic formulas the main dimension of the cableway can be determined.

- For the main span 500 meters is assumed;
- For the main cables a diameter of 120 mm;
- For the mass of the telpher + stones 100 tons;
- The strength of the cable is set on 1000 N/mm^2;
- The pre-tension in the cable is set on 15 % of the maximum force in the cable.

The result of this assumption is a lowering of the cable of 7,4 meters, which seems to be acceptable. Dynamic effects and the stretch of the cable will increase this lowering. Besides the lowering of the cable there should also be space for ships sailing through the gap. Therefore the height of the towers is estimated 25 meters above the maximum water level: CD +36 m.

11.2.2.5 Shape of the towers and caissons

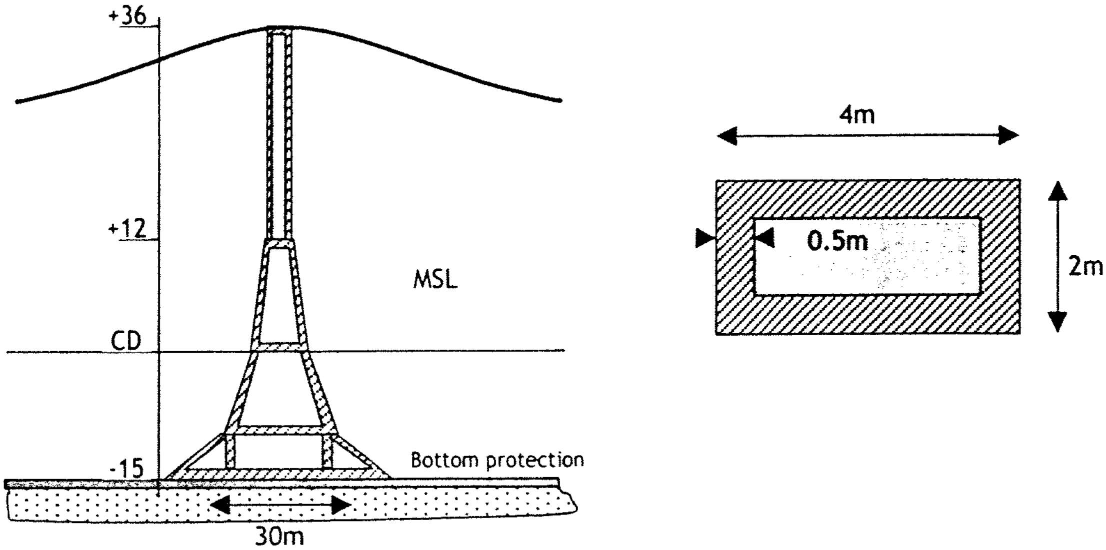


Figure 11.3 Cableway tower

A first estimate of the load on the towers will be the static load: two cablecars and the cable weigh about 300 tons = 3000 kN. With a concrete strength of 25 N/mm^2, this requires a minimum surface of 0,12 m^2, which seems to be quite small for such a tower. Because of the dynamical effects a hollow structure of 2 * 4 m suggested (figure 11.3). This tower is placed on a cylindrical shaped caisson with a diameter of 10 m at CD and 30 at the bottom (see drawing).

11.2.2.6 Advantages and disadvantages of the cableway

The main advantages are:

- The cableway is an existing technique;
- Very flexible method, every place in the gap can be reached from the beginning of the closure;
- Also suitable to for dumping other materials than rock.

The main disadvantages are:

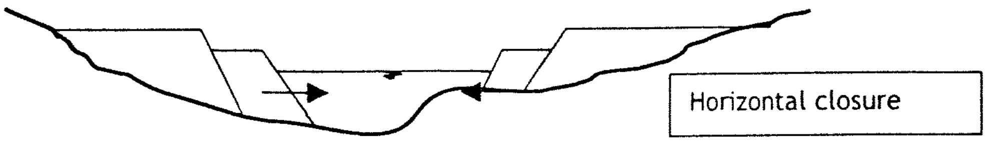
- Low capacity, the telpher has to stop to lower its load, just dumping the load would set the telpher in a dangerous oscillation. Placing the load placed gently prevents this;
- A cableway is not independent from the weather, especially the wind;
- A cableway requires extra handling of the stones at the loading stations.

11.2.2.7 Costs

The main components determining the costs will be:

- The cable;
- The towers;
- The caissons;
- The telphers;
- The two loading stations.

11.3 Gradual horizontal closure



Dumping trucks
Instead of dumping stones from a bridge or cableway as suggested before, it is also possible to dump stones from the dam heads. In this way the final gap is closed horizontally. Dumping should be done by special, very big, off-road trucks. This closure method is investigated earlier (Hafkamp, 1996) and resulted in the use of very big trucks with a load capacity of 165 tons. This is the maximum size for civil engineering use.

The big advantage of this method is its simplicity and the absence of a structure that crosses the closure gap. There are however a few disadvantages:
- The trucks turn around at the end of the dam. This, and passing each other on the dam crest requires a (locally) wide dam body;
- The only place to dump stones is at the dam head; this results in relative low production rates. The only way to improve this is by using more closure gaps. If not, the low capacity will result in a long construction time and thus heavier attacks on the bottom (protection). Horizontal closure results in higher velocities than vertical closure (see chapter 8);
- The trucks are very heavy (load = 165 tons, own mass will be around 80 tons, total at least 250 tons), the high axial loads will give problems elsewhere in the alignment, especially near the tidal power facility;
- Horizontal closure will result in larger stone diameters compared with vertical closure due to the higher velocities;
- The horizontal technique is not suitable for the overkill technique because high capacities are needed.

The costs of this closure method are determined by the operation of the trucks. The major design problem is more a logistic than a technical problem. By using ships the capacity can be raised, but weather conditions and currents then restrict the operation.

11.4 Selection of gradual closure alternatives

As can be imagined, there are many possible closure techniques. The most promising techniques are described in this chapter the others are not. As stated in the introduction of this chapter, the next chapter (12) investigates the possibilities of a closure using the overkill technique. For this technique the most suitable dumping technique has to be chosen. But there are also other criteria that determine the suitability of a certain closure method. To prevent unnecessary work, the closure methods suggested in this chapter are evaluated before the overkill technique is investigated. The three alternatives are:

The gradual closure alternatives are:
1 vertical closure by railway bridge;
2 vertical closure by cableway;
3 horizontal closure by using dump trucks.

Some of these techniques can be used for a 'traditional' and the overkill closure.

11.4.1 Criteria

The following criteria will be used to determine the best stone closure.

Stone handling actions
There are significant differences between the three methods. The railway option offers the possibility to transport the stones directly from the quarries to the closure gap. The stones

dumped by cableway and dump trucks should first be transported from the quarry to the final closure gap. This requires a stockpile on the dam body itself. (It is off-course possible to drive with the trucks from the quarry to the final gap, but the high axial loads from the trucks will request reinforcement off the tidal power facility and other components. This reinforcement reduces the orifice and is therefore not desirable.)

Size of stones required during closure method
The horizontal closure requires bigger stones than a vertical one because of the higher occurring velocities. However, the difference will be small due to the high tidal range resulting in a large head difference.
The overkill technique is expected to result in the use of smaller stones than the 'traditional' technique.

Volume of stones needed.
The horizontal closure requires a wider crest and thus the largest volume of stones.

Flexibility
The vertical methods offer the possibility to repair a part of the dam instantaneously. When needed it is also possible to close the gap horizontally with the dumping bridge. The bridge can be used under almost all circumstances, the operation of the cableway is determined by wind. But the cableway is capable in gently placing stones.

Extra structures needed
The vertical methods require extra structures (a bridge or a cableway). The horizontal method needs no extra structures.

Capacity
The highest capacities can be achieved with the railway bridge. The difference between the cableway and the dump trucks will not be very big, this because the stones are delivered at the final gap in the same way. Another problem with trucks, is the fact that they cannot unload all at the same time since the free space on the dam head is limited, also bulldozers have to flatten the sill after each truck. A high capacity is necessary for the overkill technique.

Costs
The bridge is expected to be more expensive than the cableway. However by using standard bridge elements, re-use of the bridge sections elsewhere in India after finishing the project could be possible. However this is not expected to generate much extra money. Indian Railways can use the trains, required for the closure, after finishing project. The telphers are useless.
The bigger the stones are, the more expensive. The larger the volume of stones, the higher the costs. With equal prices but different construction times, the shortest method will be cheaper due to the fact that less interest has to be paid.
More handling operations increase costs.

Risks
All methods use existing technologies. Only the horizontal closure requires (existing) special trucks. The size of trucks however will cause problems with the tires. The cable for the cableway cannot be produced as one piece, therefore extra problems will occur at the connections at the towers. The technique of the dumping bridge seems to have the smallest uncertainties.

11.4.2 Selection

Summarizing all the criteria results in the following matrix. The alternatives are ranked with + and -, or +/-. + Being the best alternative, - the worst. For equal suitability the same score is assigned. +/- Is assigned to the alternative that is either good or bad.

Table 11.2 Selection of rock dumping method

	Bridge	Cableway	Horizontal
Stone handling	+	-	-
Stone size	+	+	-
Rock volume	+	+	-
Flexibility	+	+/-	-
Extra structures	-	-	+
Capacity	+	-	+/-
Cost	-	-	+
Risk	+	+/-	+/-
Total	**+5**	**-2**	**-2**

From this table it is clear that the bridge offers the best possibilities and will be used in the comparison between the 'traditional' and the overkill technique.

12 Overkill stone closure

12.1 Introduction

The storage area model as described in chapter 8 will be the base to examine the possibility for the overkill closure. To predict this closure method the model has been extended.

The idea behind the overkill stone closure is based on the principle of a sand closure. For a sand closure a large capacity of dredging material is needed which pumps more sand in the gap than the flow is able to erode. This procedure can only be carried out with a maximum velocity of around 3 m/s. The Gulf of Khambhat has not exactly this conditions. Therefore it is impossible to apply a sand closure, however it might be possible do to something similar with rock: **dumping more rock in a given timestep than the flow is able to erode.**

This becomes more attractive noticing that many heavy stones are needed elsewhere, fro example behind the tidal power facility, serving as bottom protection. As only a very small fraction of the quarry yield curve consists of those, far more rock than necessary is produced while only the biggest stones are used. It can therefore be profitable to use the smaller parts in the curve, even when this means that a certain amount is lost.

Two criteria
The stones in the closure gap have to withstand two different but related forces:

- The maximum occurring current velocity;
- The maximum occurring head difference.

These are two different criteria that are treated and calculated separately and combined afterwards. This paragraph first describes some general properties of the model (§12.2), secondly the way the velocity criterion is implemented (§12.3), then the way the head difference criterion is implemented (§12.4), the results (§12.5) and finally the recommended strategy (§12.5 -§12.13).

12.2 Model properties for gradual (overkill) closure

Dumping
The stones will be dumped, as stated above, by using a train bridge. This allows the use of a constant hourly input. It is assumed that the dumping process is a constant process, 24 hours a day, 7 days a week all year round, since it is impossible to have all the works going on forever, three hours a days are considered to be lost. This effectively results in 147 hours a week.

Parameters
For this overkill rock closure there are a few important parameters:

- The grain size, D_{50};
- The density of the rock;
- The density of the water;
- **The velocity distribution in the closure gap, in time;**
- **The head difference over the closure gap in time;**
- The maximum capacity to dump the rock;
- The cotangent of the profile.

Layout of the dam
In the **calculation** the dam body is built in layers of 1m thick (figure 12.1a). Each layer requires a separate calculation run.

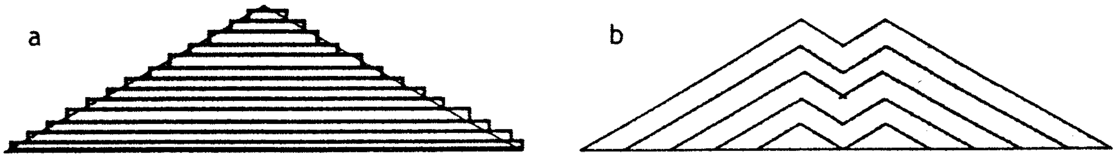


Figure 12.1 Layer principle of dam used in model (a), and real profile (b)

The dam or final gap is defined from CD -15 m up to CD +12 m. CD -15 m is the average depth in this section. CD +12 m is a crest with a minimum freeboard of 1,5-2 meter. The width of a certain layer in the profile is therefore: (12-(level of the sill to CD))*cotangent*2. When the cotangent is 4, this means a width on the bottom of 12-(-15)*4*2 = 216 meters.

The result of dumping with a bridge is not the layer shaped profile used in the model, but the triangular shaped profile drawn in figure 12.1b. This shape is not used in the model for two reasons:

- *The flow is assumed to flatten theses triangles, since a certain percentage of loss = erosion, is accepted;*
- *The box shaped layers are less complicated in modeling.*

Cotangent
The dam profile is determined for a cotangent of 4 and a cotangent of 2, both with a 25% and a 0% loss situation. This is done because the difference in cotangent is expected to give considerable differences in stone diameters. The value of 2 is not randomly chosen but based on the fact that dumping stones and accepting loss will lead to less steep slopes than when a dam is constructed with heavy stones and without loss. Cotangent 4 is just the double value.
Since the closure will be done using rock, it is almost impossible to construct very steep slopes. With special concrete blocks, placed gently by a cableway, a slope steeper than 1:1 is possible. But accepting some loss makes it impossible to create steep slopes, the steepness of the slope will be influenced by the permitted loss.

Loss
The main principle of this type of closure is the acceptance of a certain percentage of loss. For comparison, the situations with 0% and 25% loss have been calculated. This shows a reduction in stone diameter, while the total volume increases by 33%.

Runtime
The filling of each layer is calculated during the period of one month. This is done to incorporate the whole tidal wave. In that way the predicted stone diameters are not too large or too small. Too large diameters will result from calculations with the maximum amplitude only; too small values will result from calculations with the average amplitude.

Because the amount of stones needed for each layer differs from the others, it is necessary to choose a different input capacity for each layer. This capacity has to result in a construction time per layer of one month. When a real train capacity is determined, the real building time per layer can be predicted.

The use of the whole tidal range also prevents a few unfavorable situations. Like when the works stagnate for one or some reason. The layers are able to withstand the flow for a certain period (off-course not to long, erosion goes on, but one spring tide will not destroy the layer). It also prevents the use of too heavy stones, compared to when only the maximum tidal range is taken into account.

Timestep
The timestep used for the calculations is 300 seconds. This timestep is used for reasons of calculation velocity (one month is about 9000 timesteps, smaller timesteps would increase the calculation length too much).

The following input box is available, the white rows have to be filled in by the user of the model, the gray rows are calculated values.

Input box of model

Stone diameter	0,5000	m
Water depth	7,32	m
Maximum velocity	5,58	m/s
Slope 1 to	2,00	
Porosity of dumped rock	0,40	
Mass of stone	337,50	kg
Density rock	2700	kg/m3
Density water	1035	kg/m3
Delta (Δ)	1,61	
Gravity acceleration	9,80	m/s2
Fall velocity	3,24	m/s
Kinematic viscosity	0,000001	
d*	11753,60	m
Psi according Van Rijn	0,055	*read from figure*
Bottom roughness (in D50)	1,50	m
Chezy value	41,30	m^0.5/s
Critical velocity	8,69	m/s
Width of closure gap	8000	m
Sill level below CD	0	m
Capacity per hour	1100	ton
Capacity per timestep	92	ton

Total loss	**0,09**	%

12.3 Calculation schedule for velocity criterion

The following input and calculation steps are made:
1. Time (input);
2. Velocity (input);
3. Depth on the sill (input) h_S;

4. Psi

After calculation of velocity and waterdepth the actual value of ψ is determined using the Shields relation with the uniform flow relations. ψ then becomes:

$$\psi = \frac{\overline{u}_C^2}{d\Delta C^2} \qquad (12.1)$$

The Chezy value should be more than 30 (see equitation 8.2)

5. Paintal, erosion of the stones
With this ψ-value it is possible to determine the transport per meter width using the following relation (Paintal, 1971):

$$\dot{q_S} = 6.56 \cdot 10^{18} \psi^{16} \quad (for\ \psi < 0.05)$$
$$\dot{q_S} = 13\psi^{2.5} \quad (for\ \psi > 0.05) \qquad q_S = \frac{\dot{q_S}}{\sqrt{\Delta g d^3}} \qquad (12.2)$$

This is the erosion per time step per meter gap.

The Paintal relation is highly exponential especially in the area for small ψ-values, but it is the only formula found in literature. It is based on field investigations in small mountain rivers. All the other formulas are only valid fro sand.

6. Fall velocity, falling inside or outside the profile
The fall velocity of the particles and the current velocity in the gap determine whether the stones fall inside or outside the profile. The fall velocity is determined as follows:

$$w = \sqrt{\frac{4}{3} \times g \times \Delta \times D_{50}} \text{ (m/s)} \qquad (12.3)$$

This fall velocity divided by the depth (h_S) on the sill is the fall time. This fall time multiplied with the actual current velocity gives the distance a particle is moving through the water. When this distance is smaller than the required profile the stone contributes to the growth of the dam otherwise the stone is assumed to be lost (at least for the closure of the gap).

$$\frac{w}{h_S} = t_{FALL} \quad \& \quad t_{FALL} \times u_{GAP} = x_{FALL} \text{ (m)} \qquad (12.4)$$

The distance that a stone is allowed to fall is determined by the depth of the layer and by the cotangent of the slope. Figure 12.2 shows the principle described here.

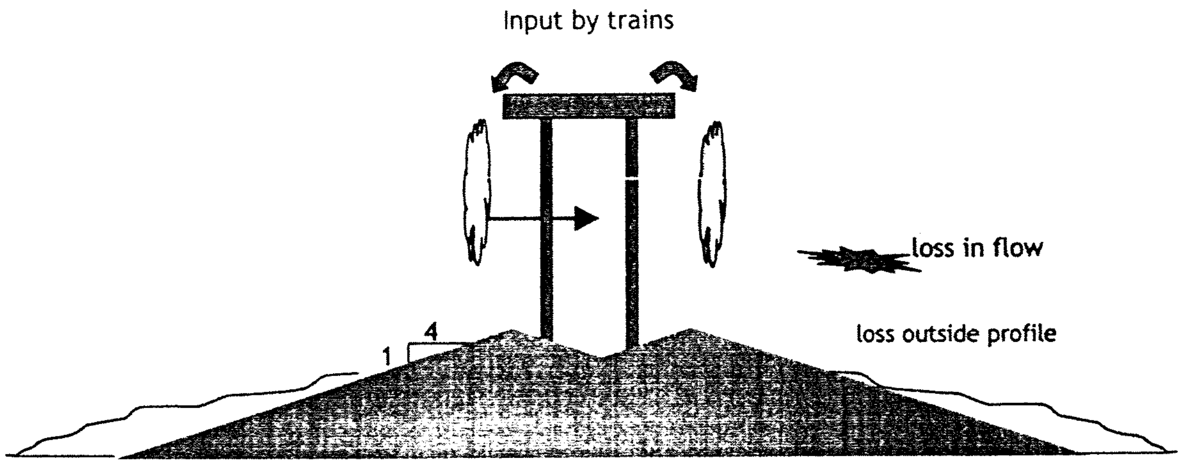


Figure 12.2 Principle of overkill closure

7. The growth of the dam
The growth of the dam per timestep can now easily be determined:
The input minus erosion and minus the loss by falling outside the profile = the growth of the dam profile.

This is done under the assumption that the stones that fall inside the desired profile are distributed uniformly over the width of the dam. This results in a uniform growth. The layer is assumed to be completed when the box shaped volume is filled with rock.

8. Loss
There are four different calculations made for the final closure gap, two calculations with a cotangent of four, and two calculations with a cotangent of two. For each cotangent a calculation is made with a loss percentage of around 25% and with the absence of loss (loss smaller than 1%).

9. Turbulence
After completion of the calculations, the turbulence was added to the velocities as a value of 15% of the velocity. With these values all the 4 scenarios are calculated again, resulting off-course in higher stone diameters.
The turbulence factor is needed because the flow over the sill is not uniform, due to high roughness (big stones) and sharp bending of the sill (especially when the slopes are steep). Besides that there will be two large bridge piles every fifty meters.

The used relations (Shields & Paintal) are derived from experiments, Paintal even from field measurements. The input velocities of these relations where also turbulent. But the predicted values from the used model (chapter 8) are not turbulent but represent uniform flow velocities. Therefore 15% turbulence is added.

From running the model, the fall velocity appears to have the biggest influence on the loss of stones at low sill levels (small stones) and Paintal (erosion) at high sill levels (big stones).

12.4 Head difference criterion calculation

The second representative force on the stones in the growing dam body is the head difference over the closure gap. The head difference is often said to be the most important parameter in the closure gap, the main reason for this assumption, made in the past, is:
It is very difficult to predict the exact velocities in the closure gap, this is mainly because in the situation with the highest velocities the stone diameters are very big and therefore parameters like the roughness of the gap are very hard to predict exactly.

Therefore investigations were done during the Delta Works in the Netherlands. The main results from these investigations are two relations (Closure of Tidal Basins). The first is presented in figure 12.3. This relation determines whether a stone is stable or not under a certain head difference with a relation to the height of the sill.

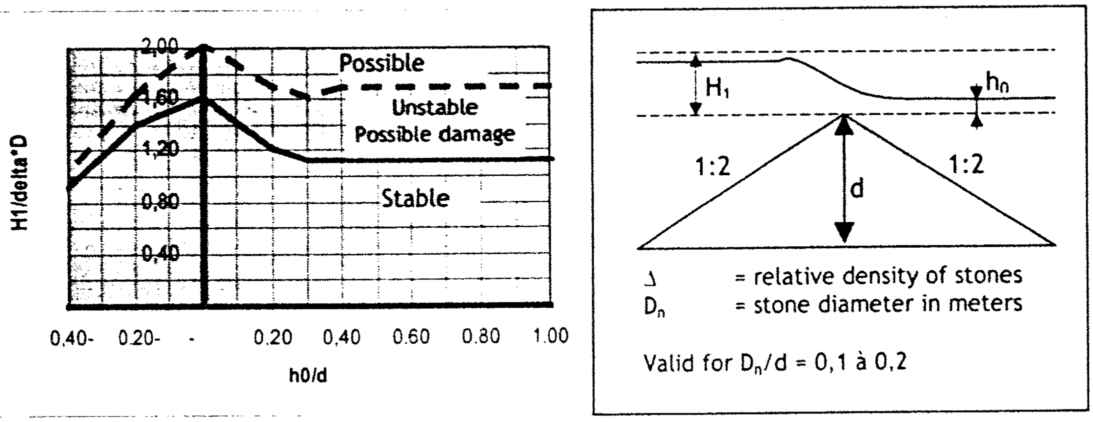


Figure 12.3 Critical head difference in relation to relative water depth

The second relation (also from the Closure of Tidal Basins) is presented in figure 12.4 This relation is independent of the height of the sill, but a transition zone between 'stable dam' and 'unstable dam' as in the first relation, is absent.

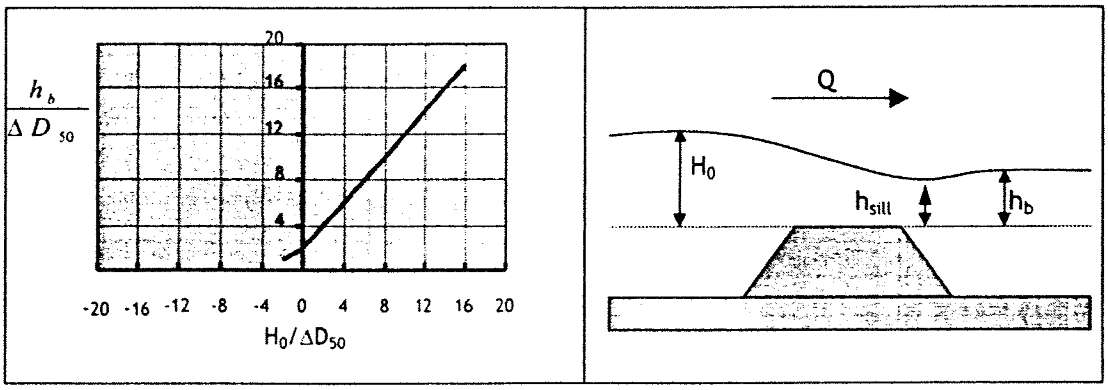


Figure 12.4 Critical overtopping height against $H_{DOWN}/\Delta D_{50n}$.

Since the first criterion contains the desired transition zone this criterion is used first. This relation is combined with the storage area approach. For every timestep, the head difference and the water levels in the sea and the basin are known and the following relation was incorporated in the model.

$U_{GAP}>0$: True			$U_{GAP}>0$: False		
	$h_{BASIN}>0$: True	$h_{BASIN}>0$: False		$h_{SEA}>0$: True	$h_{SEA}>0$: False
$\frac{h_0}{d} = \frac{h_{BASIN}}{d}$	$\frac{H_1 - h_0}{\Delta D_n} = \frac{h_{SEA} - h_{BASIN}}{\Delta D_n}$	$\frac{H_1}{\Delta D_n} = \frac{h_{SEA}}{\Delta D_n}$	$\frac{h_0}{d} = \frac{h_{SEA}}{d}$	$\frac{H_1 - h_0}{\Delta D_n} = \frac{h_{BASIN} - h_{SEA}}{\Delta D_n}$	$\frac{H_1}{\Delta D_n} = \frac{h_{BASIN}}{\Delta D_n}$

By adding this scheme to the storage area approach, for every time step the actual values related to the parameters h_0 and H_1 are known and can be plotted in figure 12.3. This plot is then evaluated. The stones are considered to be heavy enough when only a small part of the values is located in the unstable zone (between the two lines).

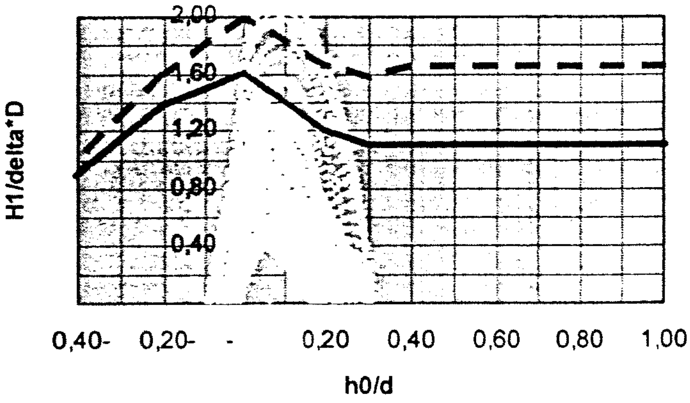
The overkill principle is can not be used in this criterion. The effects from points in the transition zone are uncertain. It is therefore impossible to accept a certain percentage of loss.

For a good and a bad relation see figure 12.5.

- 5a contains too light stones (unstable);
- 5b too heavy stones (unnecessary safe);
- 5c contains the right stones;
- Figure 5d shows all the h_0 & H_1 related values. In this figure can be seen that only a small percentage of the values is located in the unstable zone. This is allowed under the assumption that a little loss is acceptable (it is still an overkill closure!). Many other point are located in the double negative quadrant, the relation for the head difference is not valid for these points (both water levels are below the sill level), but they are shown to indicate that the amount of points in the critical zone forms a minor percentage.

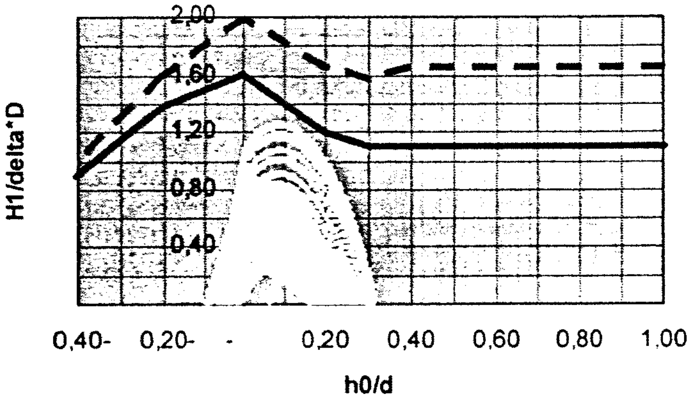
A

D50n = 0,70 m.



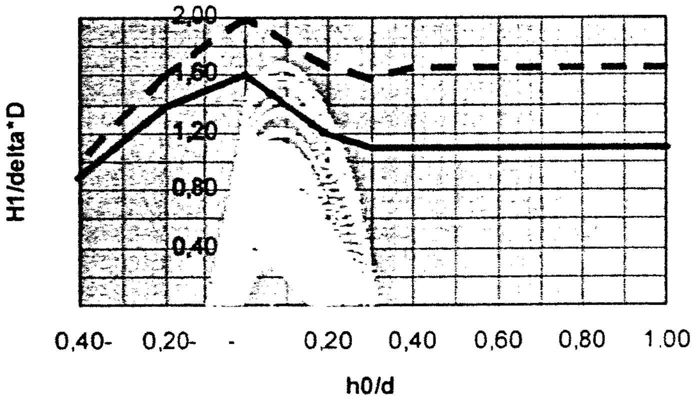
B

D50n = 1,20 m.



C

D50n = 0,90 m.



D

All values, D50n = 0,70 m.

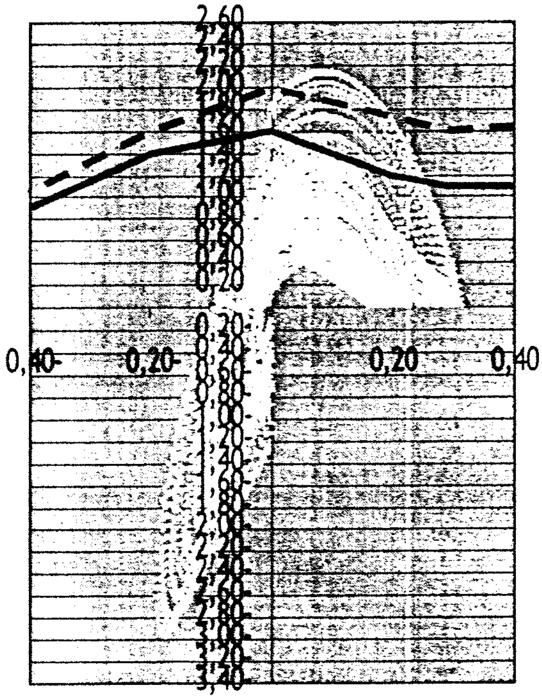


Figure 12.5

Data:

Sill at:	CD +4 m
Maximum velocity:	7,1 m/s
Maximum Δhead:	2,5 m

5a	Stone too small, points in possible failure area.
5b	Stone too big, always stable, not necessary.
5c	Good stone diameter, no points in the possible failure area, and a few in the unstable area.
5d	Complete range, most values in very stable.

Range of the calculations
The calculations for the head difference are not made over the entire depth. This might seem not logical, but it is. There will not be a real sill for the first meters, it is more a temporary higher bottom level. At the point of CD -2m, the calculations are started

Adding turbulence in this model is not necessary, because the head difference is not influenced by the turbulence in the gap, but by the waterlevel difference over the gap..

Validity
After the calculations one major problem occurred: The relation is not valid for this problem. D_{50n}/d is not 0,1 à 0,2 but always smaller, D_{50n}/d has a maximum value of 0,05. This might seem strange but this is the result of the tidal power facility with its extra gaps. The tidal power facility reduces the head difference and therefore also the stone diameter. Only for stones larger than D_{50n} = 2m this relation would be valid.
Since a second criterion exists, this is not a problem. This second criterion is now implemented in the storage area approach.

The second criterion (see figure 12.4), contains no influence of the sill height, which is an advantage. But this criterion has no defined transition zone between stable and unstable which is a disadvantage. This is therefore defined during the calculations. Many points in the unstable area are defined as unstable stones (see figure 12.6a). All points under the criterion line is defined as too stable (see figure 12.6c). All points located just below the line and against the line, are defined as the desirable stone diameter (see figure 12.6b). Small stones are declared stable when they are located around the criterion line, far outside the defined criterion area (see figure 12.7a). Zooming in around zero shows the relation for a high sill level (stable stones, see figure 12.7b).

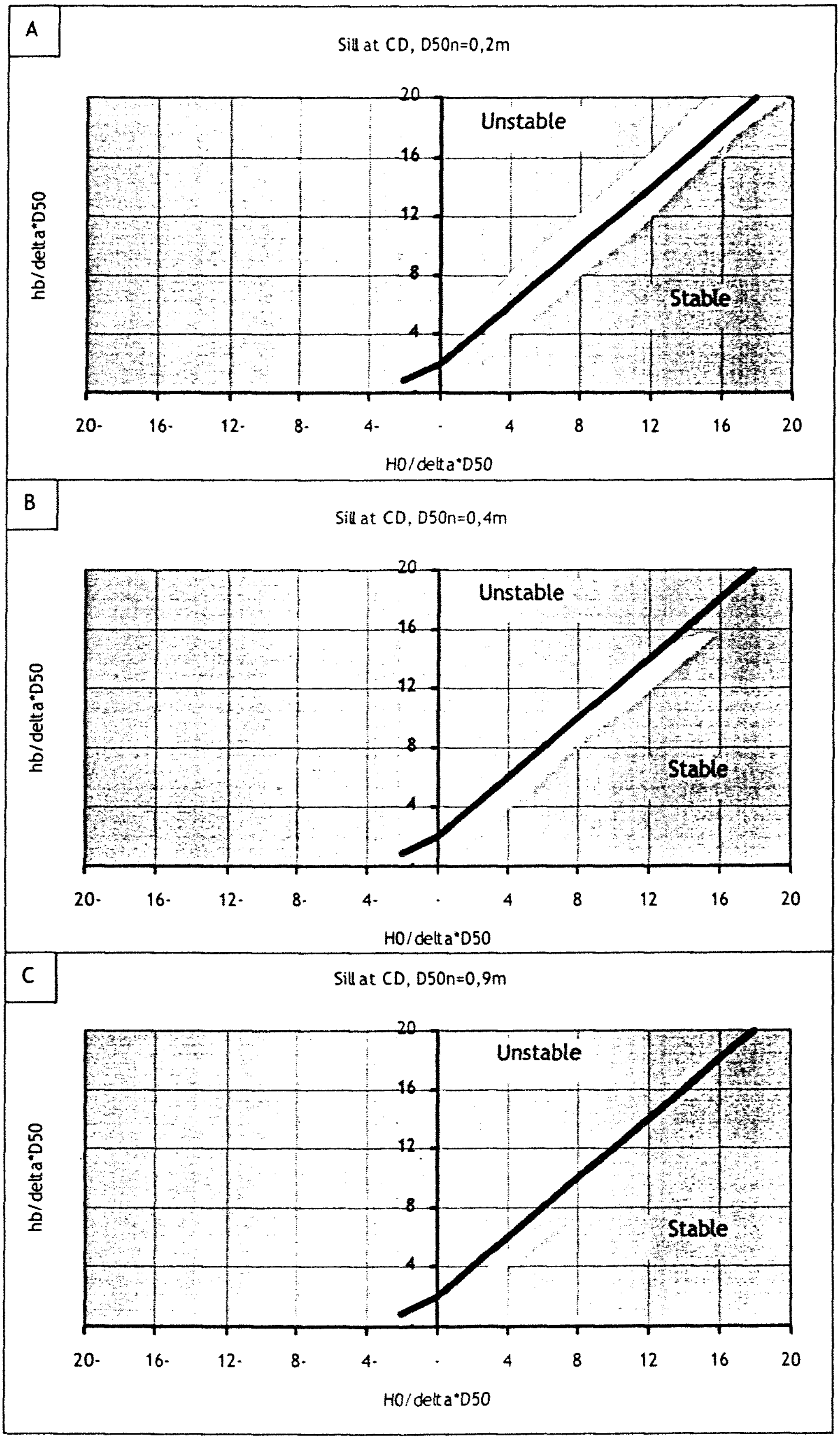


Figure 12.6 Difference between unstable stone diameter (A), assumed stone diameter (B) and unnecessary safe (stone diameter (C). (T=300s, A=54000m^2+4400m, sill at CD)

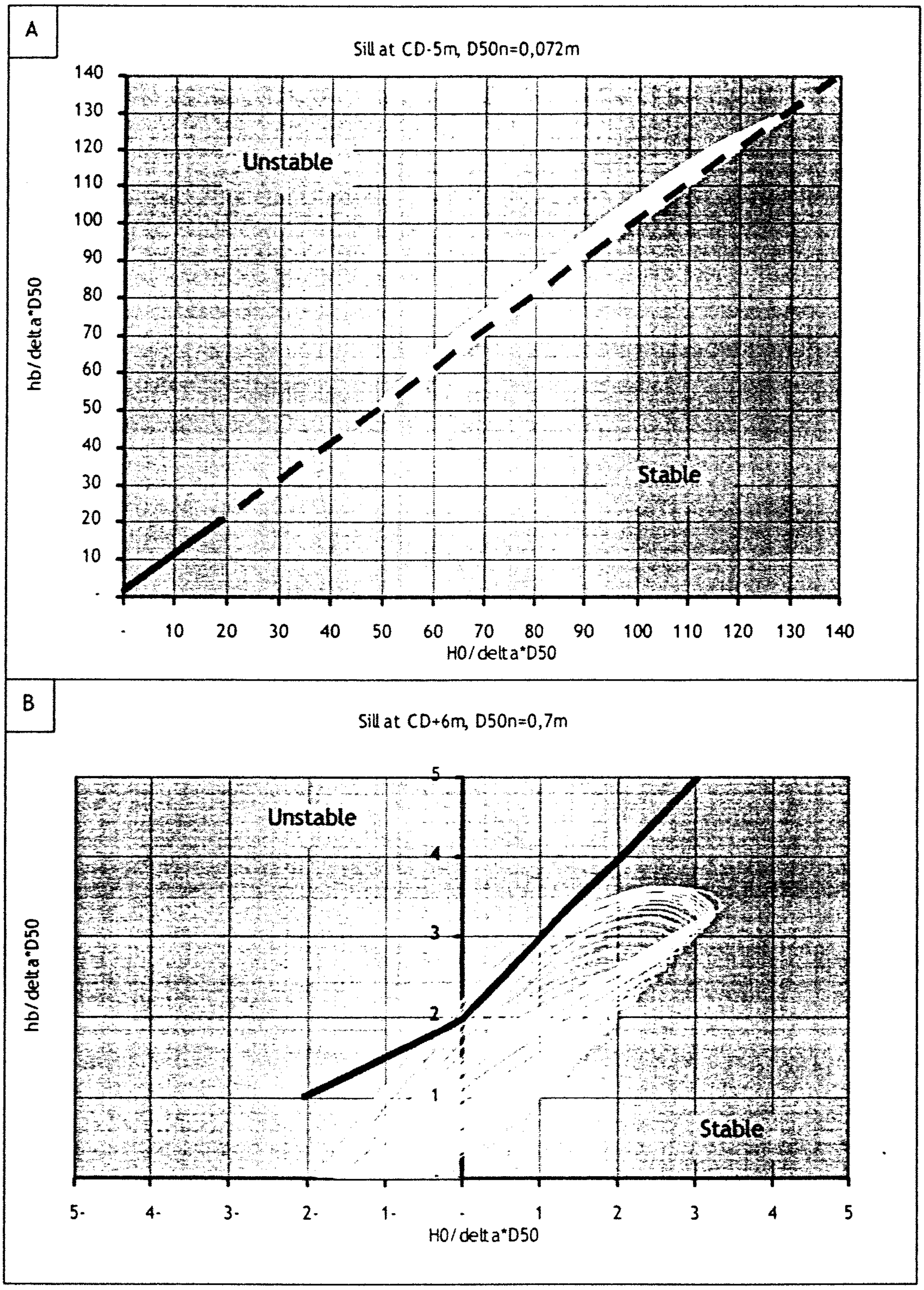


Figure 12.7 Problem at low crested sill (A), complete range of calculations is located outside defined area, dashed line is extrapolation of criterion.
Zoom of relation at CD+6m (B), stable stones.

12.5 Results of both criteria

After determining the required stone diameter matching the maximum velocity criterion and maximum head difference criterion, it is possible to compare the values and determine which stone distribution is needed to close the final gap. (The figures in this paragraph may differ a bit from the pictures with the final grading, presented in paragraph 12.8, since the calculations are not made for each layer but for layers of 2 meters thickness. In paragraph 12.8 the final closure is calculated in meters.)

First the diameters for velocities without turbulence are compared with the diameters for the head difference. In figure 12.8 it can be seen that the 25% loss criteria result in very small stones. The second remarkable result is the very small difference between cotangent 4 and 2. The third remarkable result is the fact that the 25% loss criterion can not be used because the head difference requires much larger stones than the velocity.

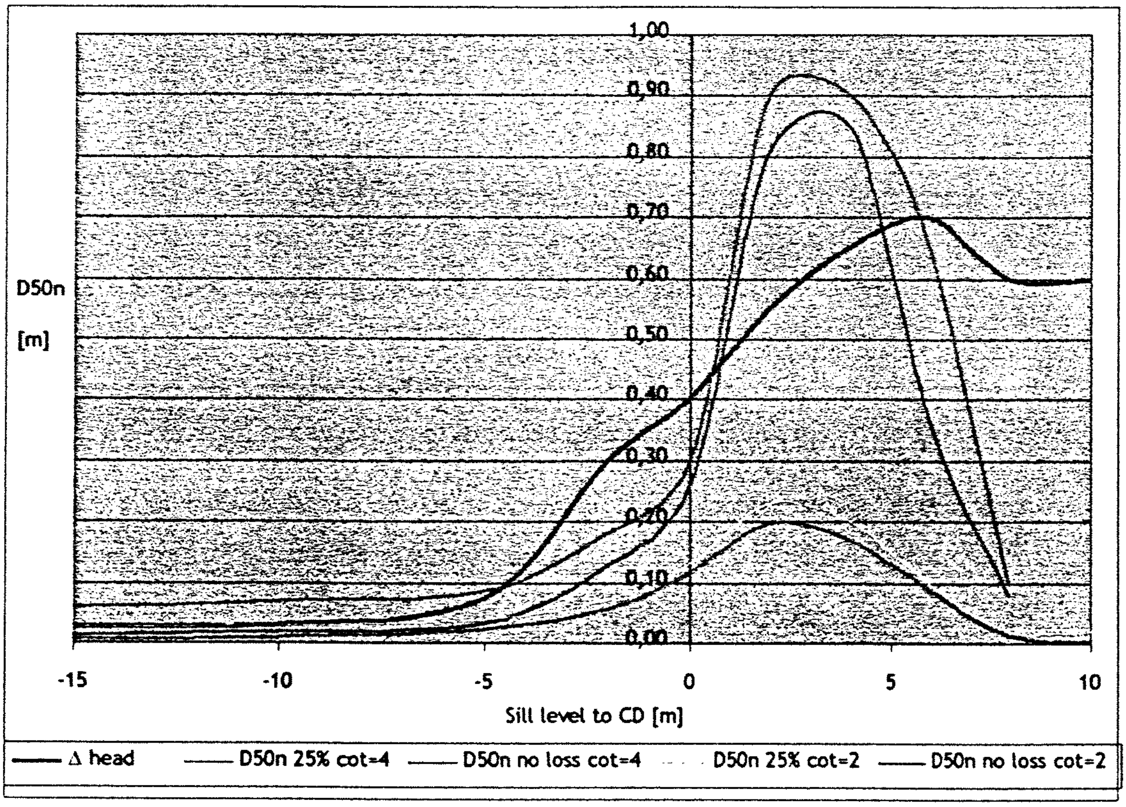


Figure 12.8 Diameters for velocities without turbulence and head difference

As stated earlier, it is necessary to ad a certain amount of turbulence (15%) to compensate all the irregularities in the profile (like the piles of the bridge) and the high roughness, caused by the large stone diameters. In figure 12.9 the consequences of this higher velocities on the stone diameter is clearly visible.

Since the head difference is not influenced by the velocities but only by the tidal difference, a compensation factor is not needed for this parameter.

The velocities and head difference, plotted in figure 12.9, are the parameters that have to be taken into account when the gap is closed. From figure 12.9 clearly can be seen that, by following only the head difference criterion, there might occur a problem since the 25% loss criterion requires heavier stones. At a sill level of 2 meters above CD the loss of the stones that match the head difference criterion is even 100% due to the large velocity.

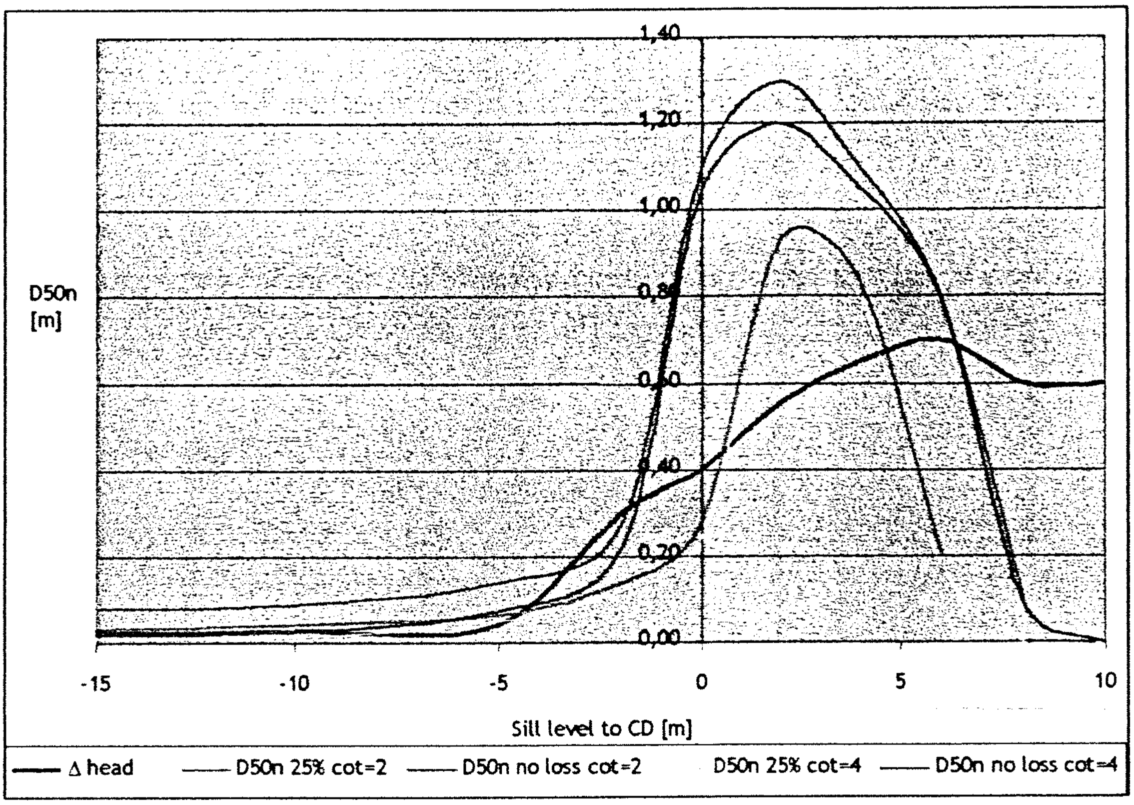


Figure 12.9 Diameters for velocities (15% turbulence) combined with head difference

From the figures 12.8 and 12.9 can be seen that the influence of the cotangent is not so large as expected. Because cotangent 4 requires twice as much stone as cotangent 2, in only a slightly larger diameter, using only a cotangent of 2 is recommended.

After exclusion of the cotangent 4, it is wise to look at the desired stone mass for the closure. In figure 12.10 can be seen that the no-loss closure will need very heavy stones, these stones will form a minor part of the quarry output and should be used as less as possible, the essence of the overkill closure.

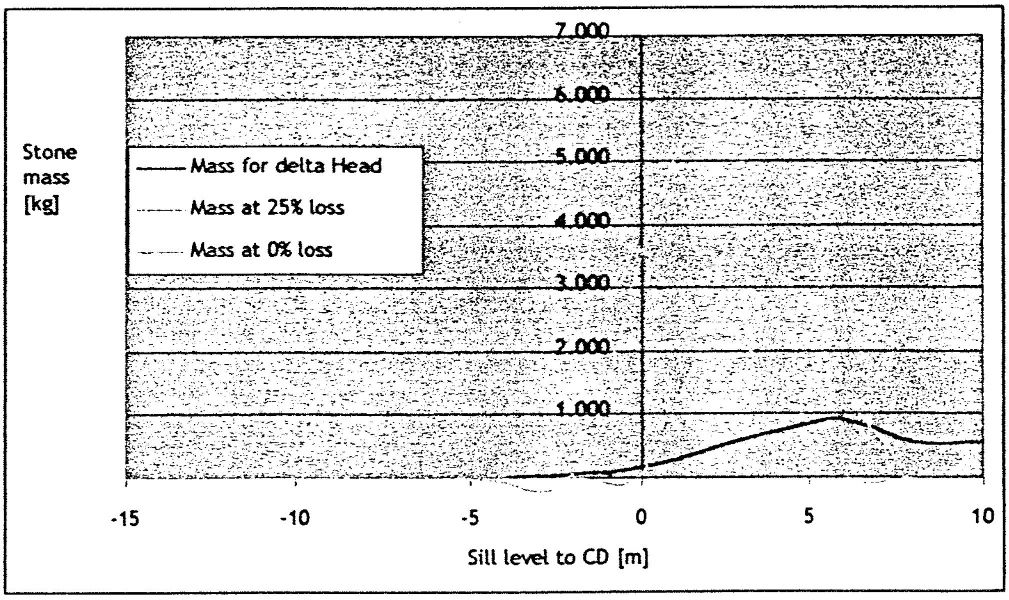


Figure 12.10 Stone masses for different criteria (15% turbulence).

The wish to use the overkill technique as much as possible is based on the experience that when loss is not accepted, a large amount of rare stones is needed. The choice to use the overkill technique seems to be not so difficult. At this point a line can be constructed (see figure 12.11) which meets the representative criterion at each sill level.

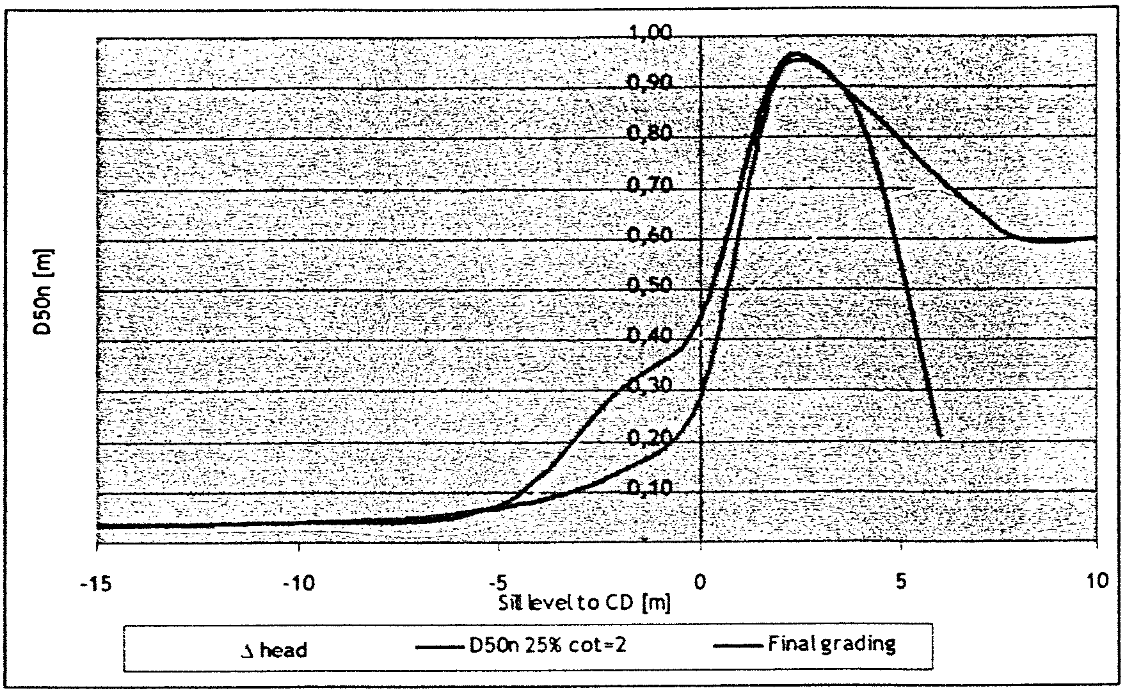


Figure 12.11 Final grading to close the final gap (rough calculation).

At this moment the overkill technique proved to be a satisfying option to close the gap. Whether it can be applied or not depends on a few other parameters such as the availability of stones, the required capacity and the influence of waves

12.6 Stone mining and dumping

12.6.1 The capacity of the quarries

The main problem of the closure as proposed will be the capacity of the quarries. The largest quarry in the world produced around 100.000 tons stones per week or 600 tons per hour. This high capacity requires an excellent logistic structure. When building directly out such a quarry, the building time is still very long, table 12.2 shows the building time in days and years for the slope with cotangent is 2 and 15% turbulence. The capacity per hour is the sum of two individual quarries. It can be seen that 1000 tons per hour result in a very large building time. In figure 12.12 (and table 12.1) the construction time in days of the dam body is plotted against the sill level for several hourly capacities (only for 25% loss and no-loss). The building times related to the capacities are for 147 hours a week.

Table 12.1 Construction time with cotangent 2 and 15% turbulence

	25% loss	0% loss	Final
1000 tons/hour	1379 days	1077 days	1340 days
	= 3 years, 9 months	= 3 years	= 3 years, 8 months
2000 tons/hour	690 days	539 days	670 days
	= 1 year, 11 months	= 1 year, 6 months	= 1 year, 11 months
4000 tons/hour	340 days	269 days	335 days
	= 11 months	= 9 months	= 11 months
5000 tons/hour	272 days	215 days	268 days
	= 9 months	= 7 months	= 9 months

The problem with a long construction time is mainly a safety problem. The monsoon is characterized by rougher conditions at sea. Especially the waves will be higher than in the dry season. This means that the dam has to be able to withstand heavier conditions than when build in one dry season.

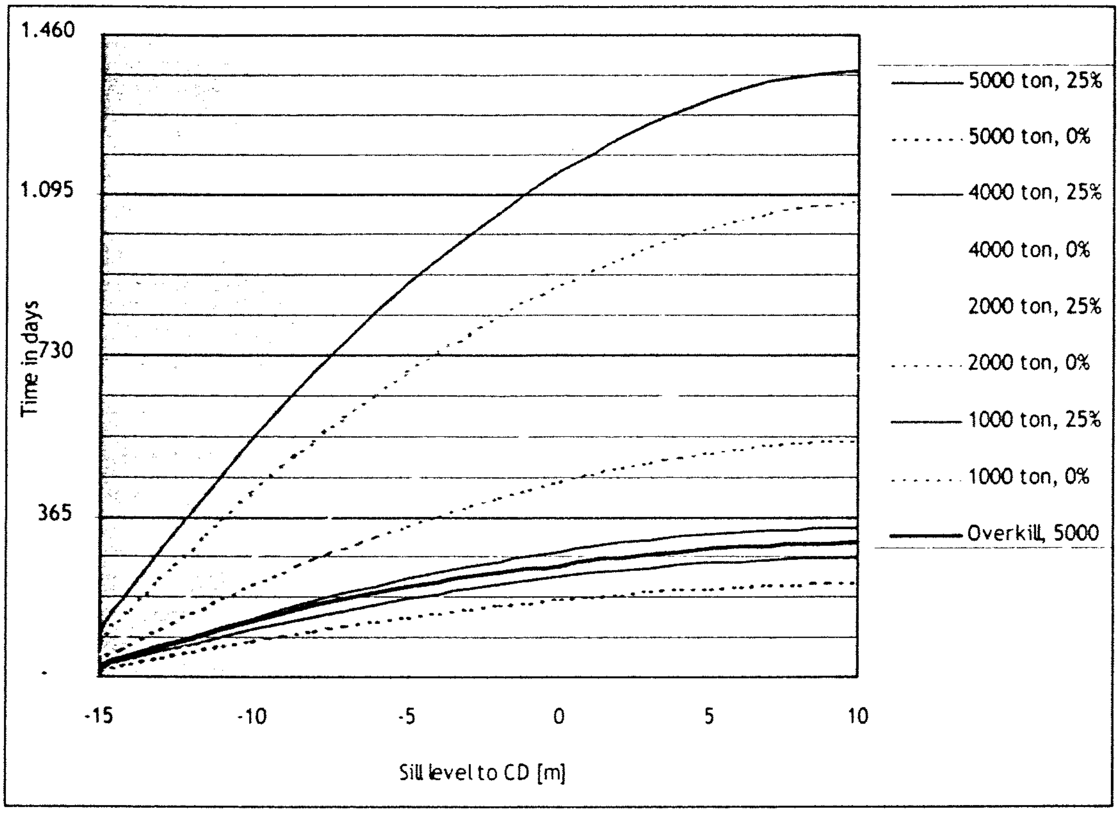


Figure 12.12 Construction time final gap for different capacities.

Even by operating two quarries with an hourly capacity of 1000 tons each, the construction lasts for over a year.
To prevent a long construction phase for the final gap, it is necessary to raise the quarry output. This can be done either by increasing the quarry production, by stockpiling of stones during a few years or by operation more quarries.

The capacity of the quarries can not be raised without consequences. A favorable quarry output would lie between 2000 and 2500 tons per hour. This would result in two(!) of the largest quarries in the world, even three or four times bigger than the current largest quarry ever exploited. Main risk of these quarries will be the continuity of the production. Accidents, bad weather, strikes, etc. may stop the production and therefore the construction of the dam. Because the construction of the dam is based on certain percentage of loss, there will be a problem: erosion will go on. To prevent this, the second option, stockpiling, would be more favorable.

Stockpiling offers a few major advantages compared too dumping of the actual quarry production:

- Stockpiling offers the possibility to continue the closure process when the mining of rock has stopped for some reason like bad weather, bad quality of rock (will happen with 100% certainty), etc.;
- Stockpiling can be done near the dam, resulting in a faster closure process (shorter distance between loading and dumping point);
- Stockpiling over a few years offers the possibility to operate smaller or more quarries, this might be favorable or even necessary because one location is not capable of providing the required amount of rock;

- Stockpiling makes it possible to close with a higher capacity than the quarry output (the capacity of the dumping equipment will be the bottleneck instead of the quarry capacity);
- Stockpiling in necessary anyway because the bottom protection behind the tidal power facility requires very heavy stones. This bottom protection has to be placed before the closure of the final gap has started. These heavy stones only form a minor part of the quarry yield and the mining of these heavy stones will already result in a large amount of minor rock which is needed in the first layers of the final dam and in the secondary dam sections;

The main disadvantages are:
- Stockpiling increases the number of handling with the stones, which causes damage to the stones (reduced diameter);
- Stockpiling requires much extra space somewhere;
- Extra handling is extra costs.

Former experiences of Van Oord ACZ suggest that for important works, like the closure of the final gap, it is unwise to start the process before all the stones have been mined.

The third option is to operate more than two quarries. This is far more realistic than operating two gigantic quarries. When, for instance, one quarry out ten shuts down for some reason, still 90% of the production is available.

Having more quarries also offers the possibility to use a part of the production directly in the closure if the total quarry output matches the required capacity. In that case only the biggest stones have to be divided from the quarry output.

Taking the above remarks and accepting the extra handling of the stones, it seems wise to advise stockpiling combined with smaller quarries, instead of two very high capacity quarries.

The proposed quarry locations west of the Gulf are located close to the beginning of the dam (25-30 km). At the east side the locations are further away (around 100 km.). Therefore the following stockpiling/mining schedule is suggested:
- At Saurashtra stockpiling is done near or in the different quarries. The quarries are connected to the railway over the dam at one central junction. It is now possible to use large trains from the different quarries, or connecting smaller trains together at the junction before they run across the dam.
- At the east side of the alignment it is suggested to create a large stockpile at Alia Bet (the island in the Narmada mouth). Since the quarries are far removed from the dam it seems to be most favorable to stockpile the stones as close to the dam as possible.

Assuming that stockpiling capacity is not a problem, it is possible to determine a required closure capacity independent of the quarry output. The available time gap between the two monsoon periods is around 9 to 10 months. It is out of the question that the last part of the closure will be done during a monsoon, for the lower layers this might not be such a problem. But avoiding the monsoon is the boundary for determining the required capacity. With a 24 hours a day, 7 days a week schedule the required dumping capacity will be around 5000 tons per hour. The exact dumping time according to the model is 268 days. As stated before, it is not possible to work 168 hours per week, it is assumed that only 147 hours per week are used, still using 5000 tons/hour would result in (168/147)*268=306 days. This line is also drawn in figure 12.12 (the thick line).

12.6.2 Dumping capacity of the trains

The next step in the process is determining the dumping capacity of the trains. This capacity is determined by the following parameters:
- The load of one train (tons or volume of rock);
- The velocity of the train;
- The length of the train (determines the free space on the bridge);

- The distance of the supplying quarries and stockpiles;
- The capacity of the quarries (should not be the bottleneck);
- The number of trains available.

Unloading is done without stopping on the bridge. The length of the bridge will be around 10 km. The length of the closure dam is 30 kilometers; the distance of the quarries (at Saurashtra) is estimated at 25 km, the distance from the dam to the stockpile at Alia Bet this gives the following circle (figure 12.13):

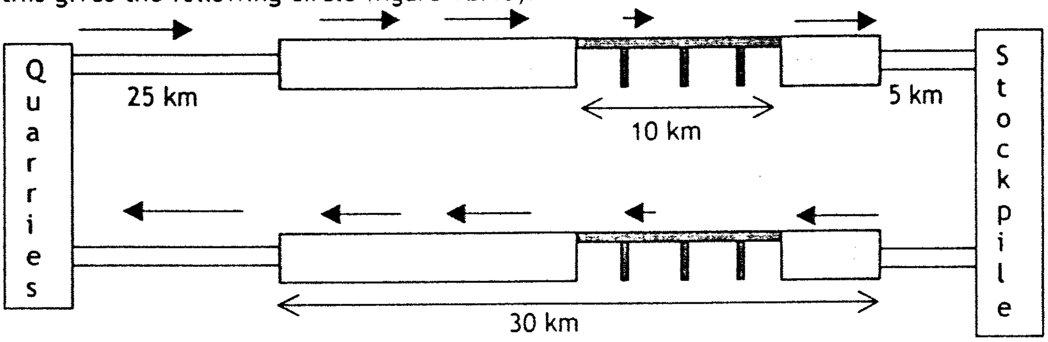


Figure 12.13 The circle of the trains

The estimated velocity of the train during this cycle is 30 km/h. The estimated turn around time is 15 minutes (filling of all the wagons at the same time or picking up a new set of filled wagons). The time needed for one circle then becomes 4,5 hours. During this period stones are dumped twice in the closure gap.

Now it is possible to determine the amount of trains needed to close the final gap. When each train carries 1000 ton, and the required capacity is 5000 tons per hour this means 2,5 trains in both directions each hour. With an estimated cycle time of 4,5 hours, the number of trains needed is 12. For reasons of safety and maintenance it is desirable to have 13 or 14 trains (locomotives) available.

Assuming that a wagon is a bit longer than 20 meters and able to carry 100 tons (60 m^3 including porosity) and a locomotive has a length of around 20 meters the length of these trains will be around 250 meters.

12.7 Influence of the quarry yield curve

The most important factor to choose a certain strategy (percentage of loss), is the prevention of over production of rock. This can only be done when the major volumes of stones with their diameters are known. The requested amount of stones can be derived from the chapter dealing with the secondary dam sections (chapter 5) and dealing with the bottom protection (chapter 9). Table 12.2 shows the total amount of stones needed for these objects.

The volumes of stones are divided in to standard categories, this is done because these standard gradings are well known in hydraulic engineering. When the rock is really blasted, the stones have to be divided in special Khambhat closure gradings.

Table 12.2 Stones in other structures than final gap

Secondary dam	m3 required mountain volume in quarry					
sections	10-60	60-300	300-1000	1000-3000	3000-6000	>6000
Dam Ghogha - TPF	1418300	-	130000	-	167700	-
Dam TPF - final gap	1311440	-	130000	-	83850	-
Bottom protections						
Loose	487500	-	-	-	-	-
Light	50000	50000	-	-	-	-
Heavy	-	1260000	2510000	-	-	-
Very heavy	-	-	-	1966250	1966250	-
Totals	3267240	1310000	2770000	1966250	2217800	-

The amount of stones needed to close the final gap should be added to the quantities as stated above. The total sum of required rock then has to be compared with the standard quarry yield as described in chapter 8, the percentages of stones required for the sections in table 12.2 are plotted against the theoretical quarry output (see also figure 8.9).

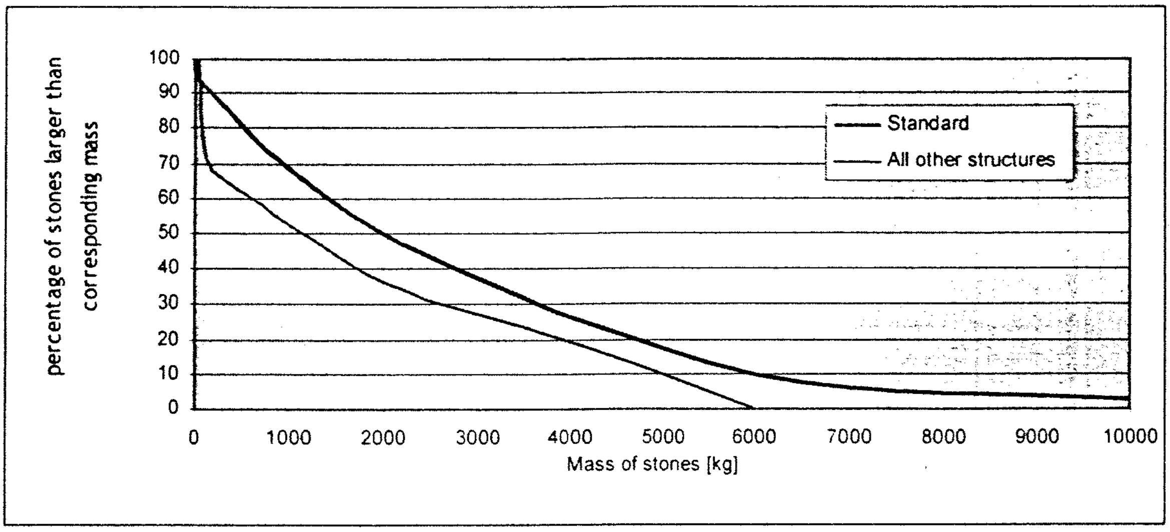


Figure 12.14 Theoretical quarry output against required output.

Selection of the stones

The selection of the stones in the quarries has to be such that are minimal. However, the fines should be absent in the rock that will be dumped in the water, as these fraction will erode without a doubt. These fractions can be used in the concrete structures.

The other side of the quarry yield curve contains the very big stones. These big stones are necessary as bottom protection behind the tidal power facility.

12.8 Final gradation (25% loss)

With all the parameters determined and their influence calculated, it is possible to choose the most favorable strategy. This strategy is already shown in figure 12.11, and is the following:

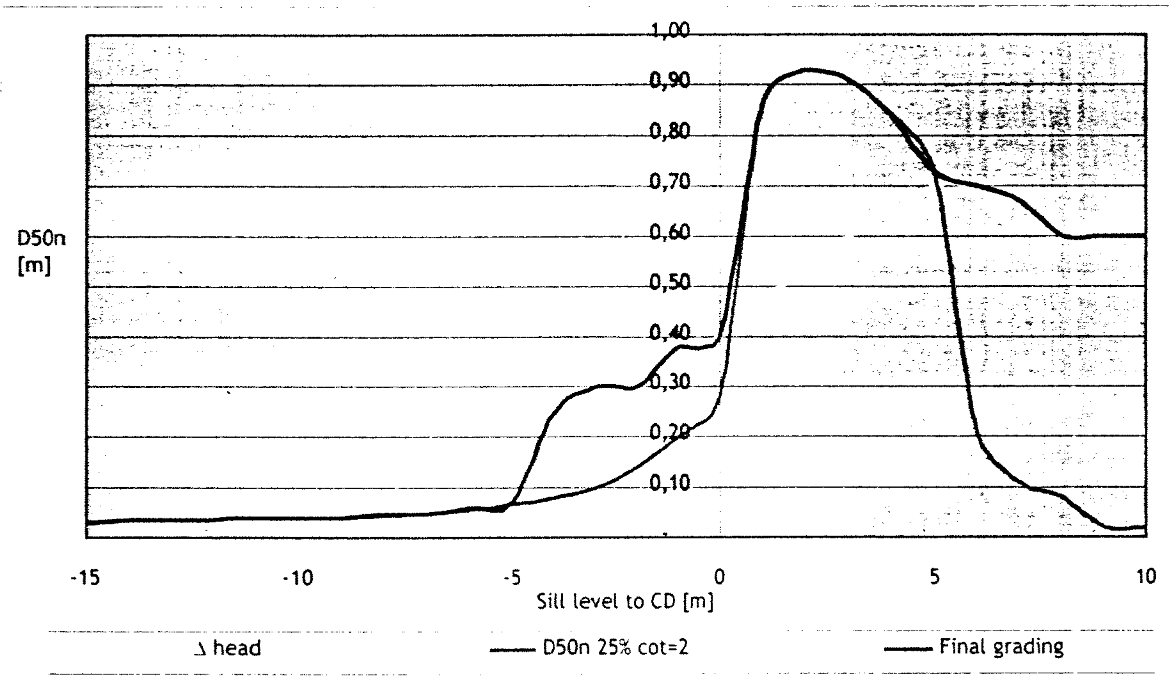


Figure 12.15 Final stone gradation

- First the line determined by the velocity, at 25% loss, is followed, the head difference criterion is not assumed to be valid for this section;
- At CD -5 m, the stone diameter required by the head difference becomes larger than the diameter required by the velocity. The head difference is now valid;
- Around CD the velocity requires larger stones than the head difference;
- Around CD +6 m, the velocity decreases and the head difference is the only factor of importance.

This gradation has the following properties (table 12.3, figure 12.15 & 12.16):

Table 12.3 Properties of final dam

sill level to CD	D50n	Mass	actual loss	
-15	0,03	0,1	23,39	VELOCITY
-14	0,04	0,1	21,32	
-13	0,04	0,1	23,76	
-12	0,04	0,1	24,13	
-11	0,04	0,1	22,60	
-10	0,04	0,2	20,45	
-9	0,04	0,2	22,94	
-8	0,05	0,2	24,59	
-7	0,05	0,3	22,40	
-6	**0,06**	0,6	17,07	**Δ HEAD**
-5	**0,07**	0,9	17,68	
-4	**0,25**	42	0,03	
-3	**0,30**	73	0,05	
-2	**0,30**	73	0,38	
-1	**0,38**	148	1,82	
0	**0,40**	173	10,02	
1	0,87	1.778	21,86	VELOCITY
2	0,93	2.172	19,19	
3	0,91	2.035	20,29	
4	0,84	1.600	22,16	
5	0,75	1.139	19,12	
6	**0,70**	926	3,87	**Δ HEAD**
7	**0,67**	812	0,16	
8	**0,60**	583	-	
9	**0,60**	583	-	
10	**0,60**	583	-	

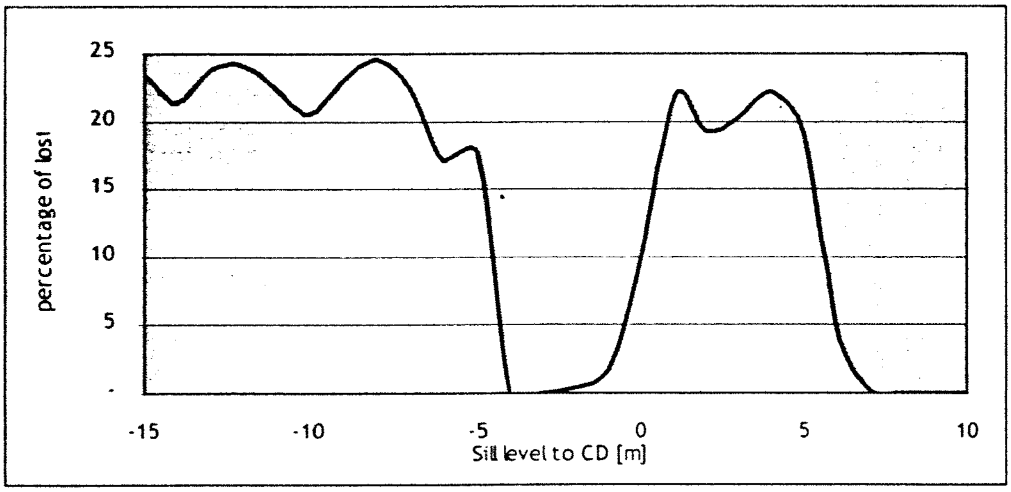


Figure 12.16 Loss during the closure process, induced by the velocity criterion

12.9 Possible failure of the dam by head difference or velocity

Since the proposed construction is a vertical closure, there are some risks involved with this closure. Theory always describes that a local deepening of the sill will result in a deep gap and transform the closure into a horizontal closure. The reason is simple: the local deepening of the sill will not enlarge the orifice. Since the size of the orifice determines the velocity at a certain sill level, the velocity will not decrease. So if this happens, the lower layers (smaller stones) in the sill are faced with higher velocities than the design velocity. That this will cause extra erosion is clear. Besides this first effect, a second important phenomenon exists, the local pit in the sill creates extra turbulence in this pit. This turbulence will also create extra erosion.

The used relation for head difference also shows that, if the head difference of a high sill level acts on the stone diameter belonging to a lower sill level, these stones will erode. This is shown in figure 12.17, the sill level is at CD +3 m, the corresponding diameter for CD +3 m is 0,70 m. A gap of 3 meters deep will uncover the stones with a diameter of 0,40 m.

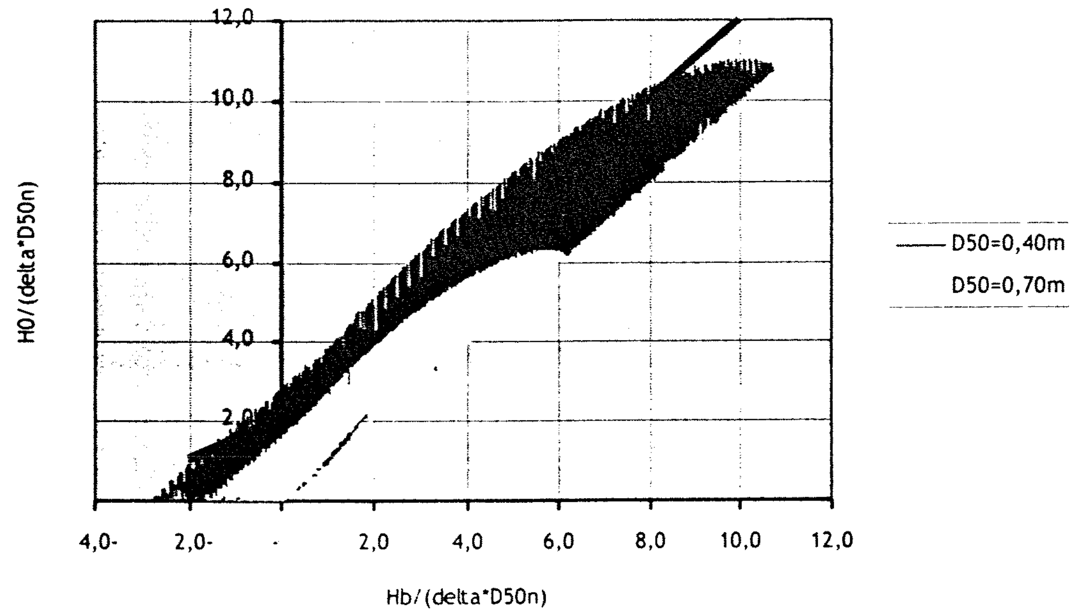


Figure 12.17 Head difference at CD +3 m acting on the required stones (0,70 m) and same head difference on the stones in a gap of 3m deep (0,40 m)

On the other hand can be said that if a hole has developed in a vertically constructed sill, the depth at the sill increases. Since the relation for the head difference depends directly on the water levels, it seems to be logical to calculate with the increased water level (decreased sill height). The result of such an assumption would be quite comfortable, because under this assumption the depth of the gap doesn't matter because the stones are always related to the right water depth (=head difference).
The chance that this assumption would be really occurring is quite small as long as the change in orifice by the deepening of the sill is small. As the orifice doesn't change, the velocities will not decrease and the erosion of such a deepening will go on.

The question is now, what are the risks involved with the proposed vertical overkill closure as proposed in this chapter since this closure is based on erosion. To investigate this, the amount of erosion caused by velocities occurring at sill levels one and two meters higher than the calculated layer will be plotted (see figure 12.18). The input values for the erosion process are determined with a capacity of 5000 tons/hour. From this figure appears that some problems are to be expected when for some reason a pit develops in the constructed

dam body. However, the erosion never reaches a critical value (larger than 100%). When a developing pit is discovered in the dam, action is needed. The growth of such a gap can be stopped by dumping a little heavier stones than calculated, too large stones will cause the problem of increased roughness and thus increased local turbulence, causing erosion somewhere else.
The layer from CD -6 m up to CD serves a stop for erosion of the dam when it has reached higher levels (this layer requires heavier stones due to the head difference criterion, see table 12.3).

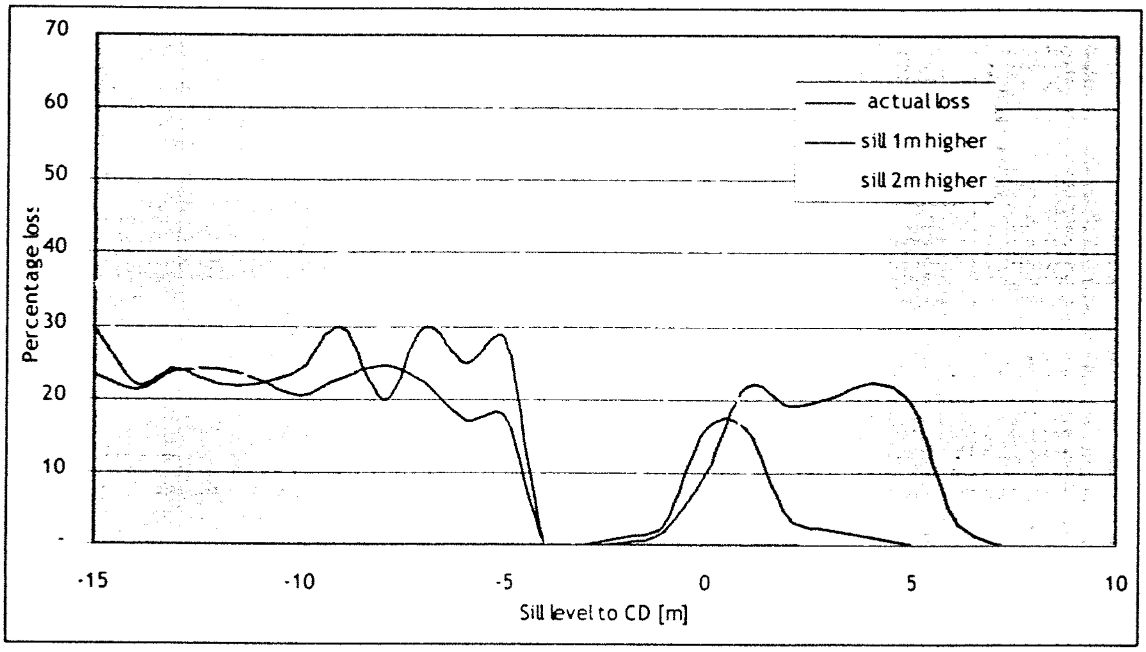


Figure 12.18 Erosion rates occurring in local lowering of the sill

As long as dumping capacity is available there are no major risk with closing vertically since the erosion rate does not grow larger than 100%. By continuing the dumping of stones required for the sill level in which the pit has developed, the pit can be filled again. The stone diameter of these stones is larger than the required stone diameter in the gap, so the erosion rate in the gap will be lower. The head difference influence is also positive for these stones, see figure 12.19 where the stones belonging to CD +3 m are dumped in a pit with a depth of CD.

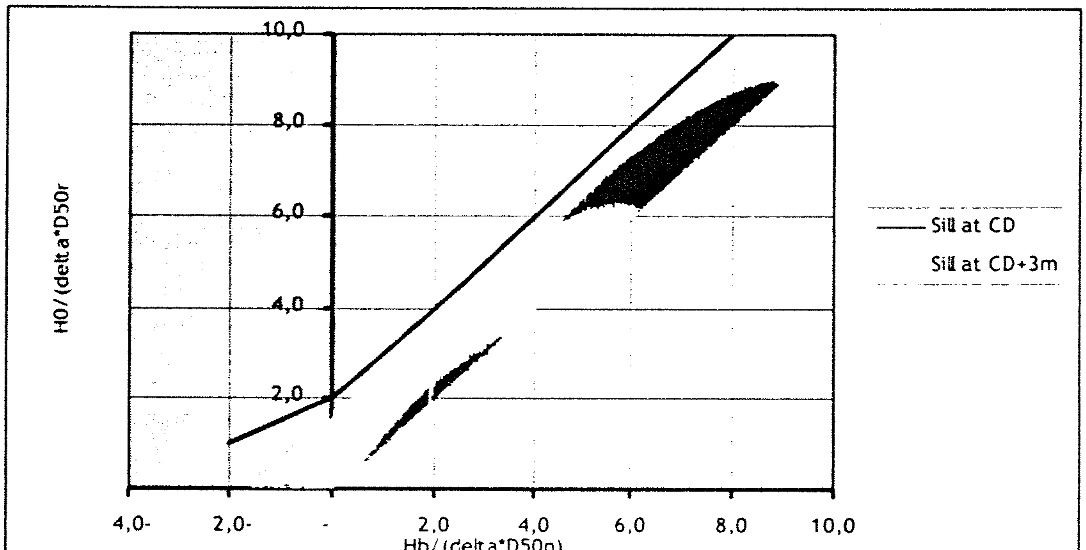


Figure 12.19 Influence of different head differences on same stone diameter

Resuming can be stated that the vertical closure of the final gap in the Gulf of Khambhat will not cause severe problems as long as the dumping of stones will go on. The necessity of the heavier layer between CD -6 m and CD will be the safety stop in the closure process.
A very practical fact will also reduce the risk in erosion of the lower parts. The required stone diameter is 0,05m, with a mass below the kilogram; these stones form the smallest fraction in the desired quarry output. They are even smaller than so-called quarry run (10-60 kg). Since it is not economically feasible or necessary to select these stones out, the quarry production that will be dumped in the first ten meters will contain much heavier stones and thus form an extra protection against erosion.

12.10 Wave forces on the dam in the final gap

Although current velocities are the main problem of the closure, the influence of the waves has to be checked on the final dam body. Waves will occur during the monsoon in July and August. The wind direction is southwest and the fetch length is large. During the short winter monsoon, the wind direction is north and therefore the fetch length is very limited.

From Jansen and Vreeburg (1992) the following significant wave is determined with a chance of exceedance of 2%.

- Significant wave height: H_{sig} = 3,5 meter;
- Significant wave period T_{sig} = 7 seconds;

The deep-water wavelength of such a wave is 76 meters. By using the Van der Meer relations for stability of stones on breakwaters the required stone diameter can be determined. During construction the allowable damage level is higher than after completion of the dam (during construction stones and equipment are available). The following input parameters are used:

Significant wave height	Hs	3,5	m
wave period	T	7	s
cotangent of dam profile	cotα	2	
permeability of the dam	P	0,5	
Density of the stones	ρ(stone)	2700	kg/m^3
Density of the water	ρ(water)	1035	kg/m^3

For the construction phase the number of waves is set at 3000 and the damage level at 10, for the final dam the number of waves is set at 7000 (maximum) and the damage level at 2. This result in the following stone demands:
Construction: Stone diameter 0,79 m, stone mass 1,3 tons.
Final dam: Stone diameter 1,19 m, stone mass 4,5 tons.

The section of the dam where these stones are needed in construction phase is located between CD and the top of the dam. The final dam is much higher and the area that has to be protected is larger than during construction phase. The dimensions of this area are determined by the final dam design, which is not the subject of this study. It is assumed that the amount of armor needed is 20% higher than during construction.

Conclusions
From this calculations can be concluded that the stone diameters used during construction cannot be relied upon completely during the monsoon season. Special attention is therefore needed during the first months after the closure and it is recommended to reinforce the section around CD as soon as possible after closure.

The final dam needs heavier armor stones than the stones that are used during the closure. These stones should be separated and stockpiled in the quarries since they form a minor part of the quarry output.

12.11 Required quarry output for all components

As stated earlier, the amount of overproduction in the quarries should be minimized. The total required quarry yield curve can be compared to the theoretic quarry yield curve. By translating the calculated stone diameters to standard diameters, they can be compared to the theoretic quarry yield curve. This is done for the 25% loss criterion and for the no-loss criterion (see figure 12.20).

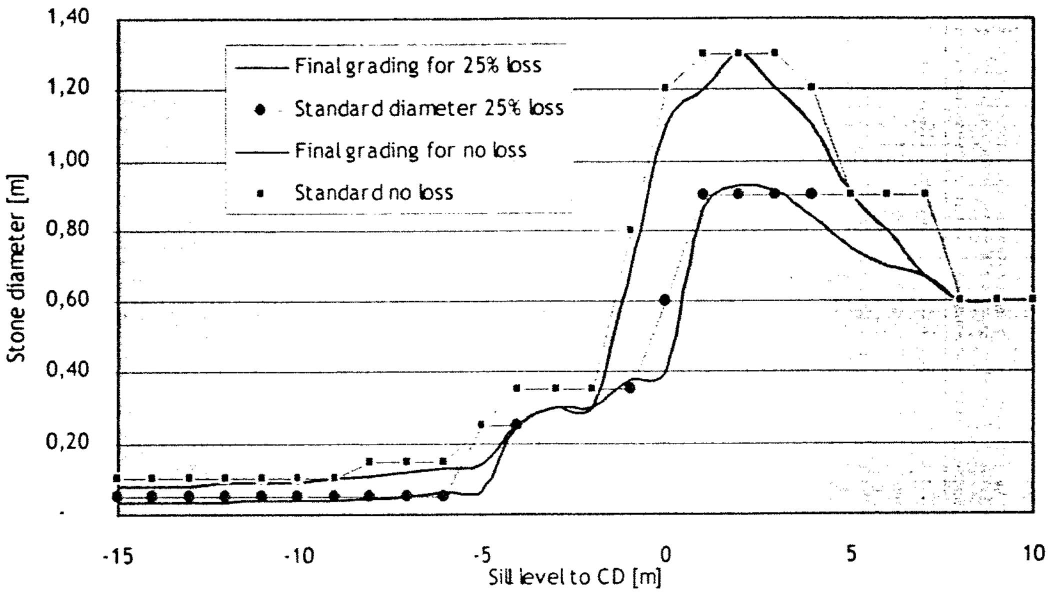


Figure 12.20 Assumed standard grading (25% and 0% loss)

In figure 12.21 the amounts of rock required to close the final gap are added to the amounts of rock required for the other components in the dam alignment, as determined in §12.7.

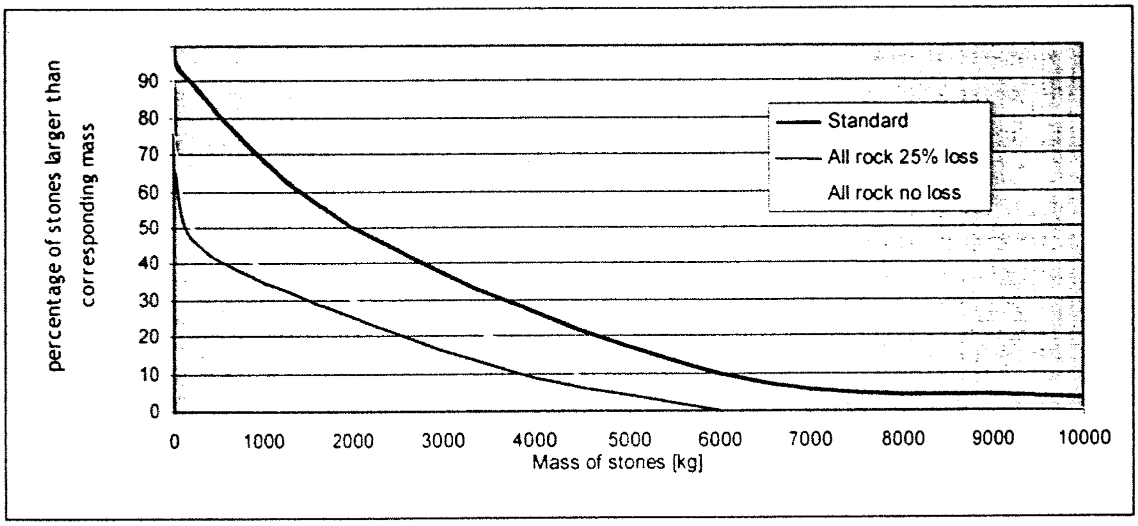


Figure 12.21 Required and theoretic yield curves

As can be seen in figure 12.21, both required quarry yield curves can be provided by the theoretic quarry yield curve. This means that it is possible to close the final gap without accepting loss. This reduces the required amount of stones with 6 million tons, but increases the maximum required stone mass from 2 to 6 tons.

12.12 Rejection of the overkill technique for the Gulf of Khambhat

When both the 25% loss and the 0% loss alternative are directly compared, the following conclusions can be drawn.

- No loss requires 29 million tons of stone, 25% loss requires 35 million tons of stone;
- Without loss, the building time is 8 weeks shorter;
- Accepting a loss of 25% is only advantageous in the layers CD -1 m to CD +6 m. In the lowest layers a slightly larger diameter results in a stable layer;
- The required stone diameter for wave force matches the required no loss diameter in the wave attack zone. This means that no protecting layer has to be constructed;
- The (larger) required stones of the no loss alternative will be more difficult to handle, and thus be more expensive. The different stone handling costs are not calculated. But the handling of both 2 and 6 ton stones requires special equipment anyway;
- This model demonstrates that it is useful to use the overkill technique to construct the rockfill dam when the quarries are not capable of producing the required gradation, and stones are inexpensive. This is not the case with the assumed quarry yield curve.

If the proposed theoretic quarry yield curve can be realized, it is best to use the stone dumping without loss. If the proposed quarry yield curve can not be realized, accepting a certain amount of loss is a good alternative.

The overkill technique is rejected for closing the Gulf of Khambhat. The no-loss gradual vertical closure will compete with the non-rock closures (chapter 13).

The final shape of the dam can be achieved in the same way as the secondary damsections are finished (see chapter 6).

12.13 Discussion of the overkill closure

This chapter describes a very interesting gradual closure technique, the overkill closure. Since the tidal power facility reduces the velocities considerably the overkill technique is not suitable for this specific closure. The quarries can also deliver the required no-loss gradation. There might however be other proposed closures in the world where the use of a tidal power facility is not possible, or where the quarries can not produce the required gradation. In those cases the overkill technique might be a good alternative. There are however a few facts that have to be specified before the technique can be used. They are:

Shortcomings in the model

- The *Paintal relation* is developed in Switzerland, where Paintal studied transportation of stones in little mountain streams. This is not the same situation as water flowing over the sill in the final gap and eroding stones. But it is the only formula that describes the transport of stones. Before the closure is done in this way, there should be more clearness about the validity of the Paintal relation for this problem. From observation of the results appears that fall distance is the determining factor at lower sill levels (small stones), and the erosion (Paintal) at the high sill levels (large stones);
- The *relation for head difference* is based on laboratory experiments which where stopped at the moment the sill started collapsing. Since the amount of damage is very interesting, this relation should also be extended after the start of collapsing;
- The stone diameters are determined by *manual iteration* with the model. There are two parameters that could be changed, the dumping capacity, and the stone diameter, they both influence the amount of loss. These parameters are optimized in such a way that the closure period is always around 700 hours (=one month) long. This implicates that the required stone diameters could change a bit when using the final dumping capacity (5000 tons/hour);
- The *turbulence factor* is based on the contraction in the whole gap, but local circumstances might heighten the turbulence factor even more. This would result in higher erosion than predicted in this paragraph.

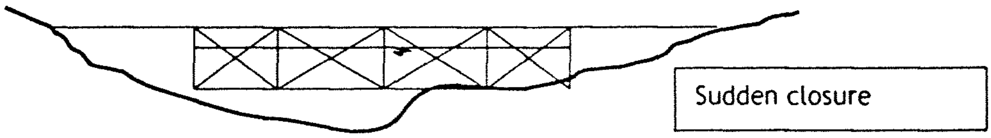
Shortcomings in the definition of the dam body:

- *Density of rock* 2700 kg/m^3. This value can be higher or lower, most rock in this area (and elsewhere on the world) has a density around this 2700 kg/m^3. It is however possible to select only heavy rock (ρ=2900 kg/m^3) for the upper layers resulting in smaller stones;
- *Cotangent of the slope*. The values of 2 and 4 are arbitrarily chosen; a steep slope needs less stone than a gentle slope. There is no great difference in required stone diameter. The exact slope of the dam has to be determined by optimization with the help of scale tests;
- *25% loss*. This factor has to be determined with specific demands in stone grading for other elements in the closure dam or bottom protection compared with the quarry output. This to prevent overproduction of stones;
- The layers are assumed to be filled *box shaped* as suggested in figure 12.1a. However the stones are dropped in two lines, this suggests the forming of two triangular crests as drawn in figure 12.1b, this always happened in the past. By using the overkill technique there will be significant transport of stones, it is therefore assumed that these pyramids are flattened out by the flow. For the no-loss this is questionable.

The proposed technique

- The *capacity* of the dumping trains;
- The basic principle of this closure is that dumping has to be done continuously, otherwise the dam will erode;
- Horizontal overkill closure is not investigated. Using trucks seems not to be suitable for overkill closing, but using a train bridge offers the possibility to close horizontally as well. Closing horizontally however requires heavier stones than closing vertically.

13 Non-rock closures or sudden closures



13.1 Introduction

The alternative for rock dumping is using large units that are able to block the flow. This paragraph deals with these large units. First the well-known sluice caissons are described and thereafter a new concept based on new materials and the old technique of sluice-less caissons.

Caisson closures have been used for many times. The idea is simple: On a flat sill placed, with barges, large floating units (mostly prefabricated caissons) are placed and sunk on the sill. When the last caisson is placed, the gap is closed. This method however is only useful if the velocities just before the last caisson is placed, are not too high, otherwise it is impossible to place the last caisson(s). The answer to that problem is the sluice caisson. The same strategy is followed but after sinking on the sill there, temporary walls are removed and a large orifice results. When all the caissons are placed, special doors are placed in the caissons and the dam is closed.

13.2 Concrete sluice caissons

The first option to close the final closure gap is by using concrete sluice caissons. The design made by the Dutch Ministry of Transport, Public Works and Watermanagement for Haskoning uses this type of caissons. This design consists of two sections of five kilometers sluice caissons and a five kilometers wide closure gap to be closed off with stones. This option will not be redesigned, for details the reader is referred to the Kalpasar study. The dimensions are drawn in the drawings in §13.2.2.

There is however one big difference with in the proposed closure procedure between this report and the Kalpasar study: In the Kalpasar study the tidal power facility is not incorporated in the closure process, but build in a separated building pit and finished at the same moment as the final gap.

13.2.1 Risks

The risks involved with this design are relatively low because use is made of existing techniques, although on a bigger scale.

13.2.2 Dimensions

The dimensions of the caissons are 108*40*30 (length*width*heigth). The cross section and a top view of the caissons are drawn below (figure 13.1 and 13.2)

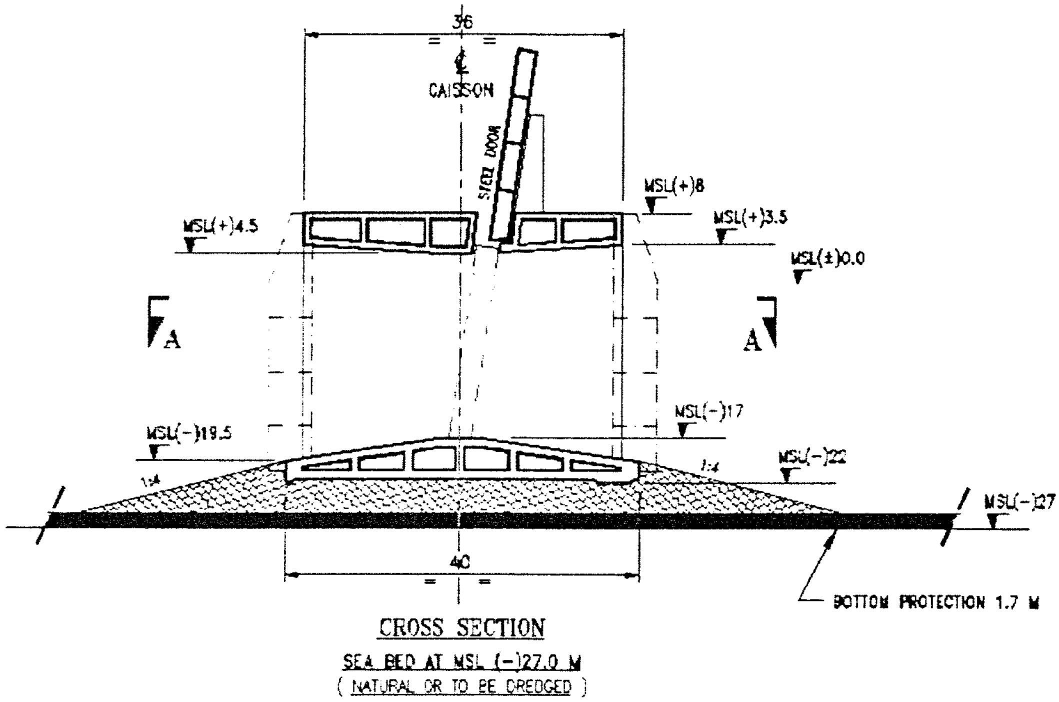


Figure 13.1 Cross section of sluice caisson

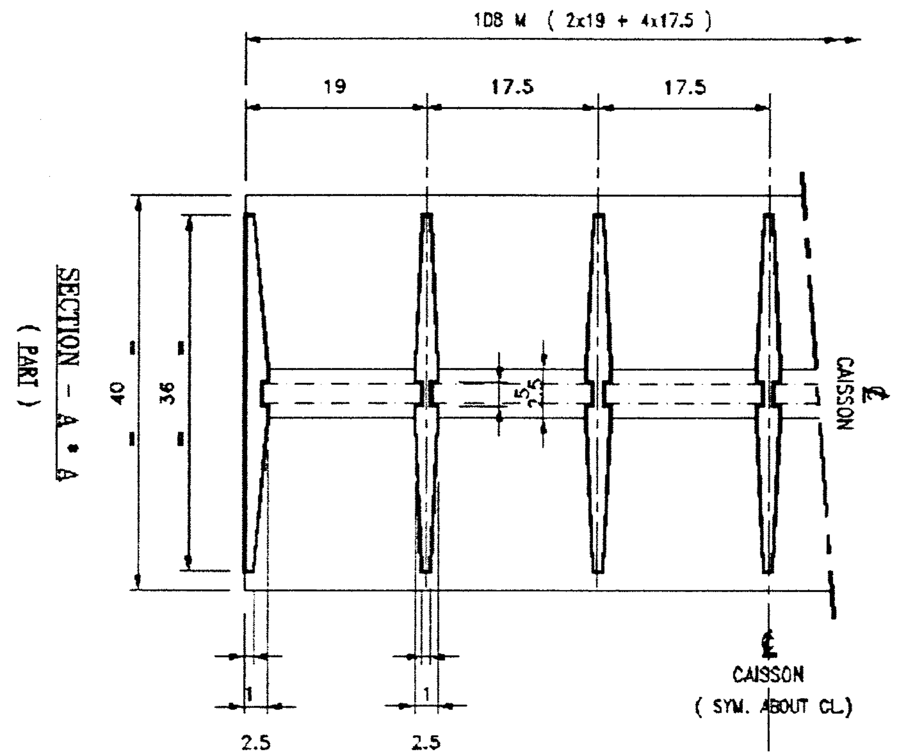


Figure 13.2 Top view of sluice caisson

13.3 Geotextile structures

13.3.1 Origin of the idea

This paragraph deals with a totally new concept. The idea is developed using the railway bridge, this bridge will need two piles each 50 m as a foundation. Could it be possible to place something between those piles? Or could a combination of piles and textile provide a solution?

The second basic assumption is the availability of strong aramide fibers (2900 kN/m^2), such as Kevlar, Twaron and Dyneema. These fibers are much stronger than existing geotextiles. These fibers are still very expensive but prices decrease continuously.

The third, is the wish is to use sand instead of rock, locally dredged sand is cheaper than rock, and dredging equipment is ever improving.

Geotextile brainstorm: development of a new idea.

Introduction

Modern geotextiles are used in a variety of structures. Most common application is as a filter to block transport of grains, but not of water. In this brainstorm this principle is used in a closure method, where grains (sand) are captured in the geotextile. A kind of 'sandbags', which are used for centuries, on a very large scale. With sandbags it is possible to withstand the large head differences as long as the dam, build out these sandbags, is wide enough. This depends on the construction type, the strength and filter capacities of the textile and the sand (grain size-piping length).

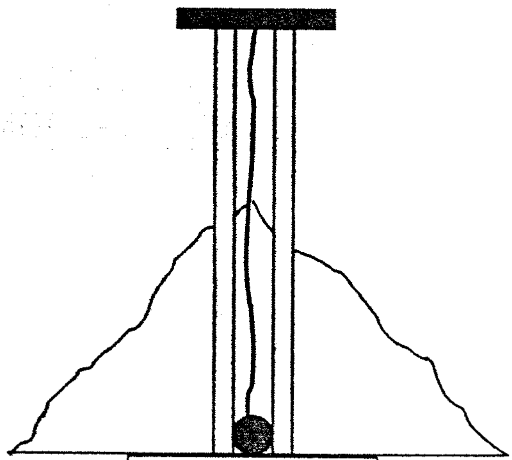
Single geotextile between piles

The start of this geotextile brainstorm is the bridge as described earlier is this report. The first alternative is constructing a kind of netting between the two piles of the railway bridge. This net should be held in place by a heavy beam on the bottom of the gulf. The dam is then partially closed and it is impossible for stones to move through this net. The partial closure results in reduced current velocities. The dam itself is constructed by dumping stones from the bridge.

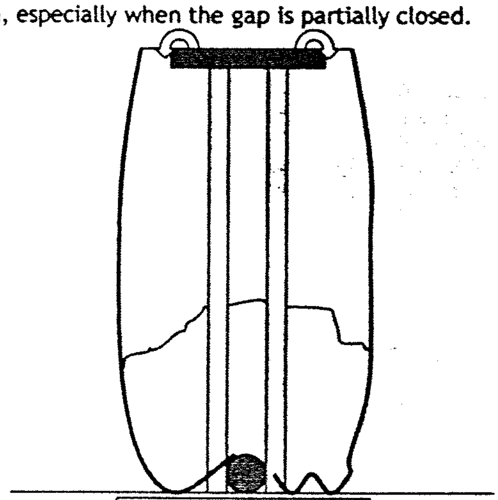
A flat bottom is required for the beam; otherwise seepage causes erosion. The advantage of this net should be the possibility to use smaller stones than would be needed without a net.

Risks involved with this method and others are:

- A flat bottom is required for the beam, otherwise there would be seepage of sediment;
- The current forces on the netting will be very high, especially when the gap is partially closed.



Single geotextile



Double geotextile

Double geotextile

The second idea is to lower two nets on both sides of the bridge, offering the possibility to create a sort of cofferdam. An example is given above, with an oversized geotextile that is fixed on the outer ends of the bridge.

Using sand

The next step in the thinking process is to find away to improve this 'geotextile-bag'. Most favorable would be a solution with sand, because sand seems to be the cheapest material as it is found directly near the dam. Modern technology makes it easy to produce and transport very large quantities. The largest trailer suction hopper dredgers have capacities of around 20.000 m^3. And 30.000 m^3 vessels are already on the drawing board.

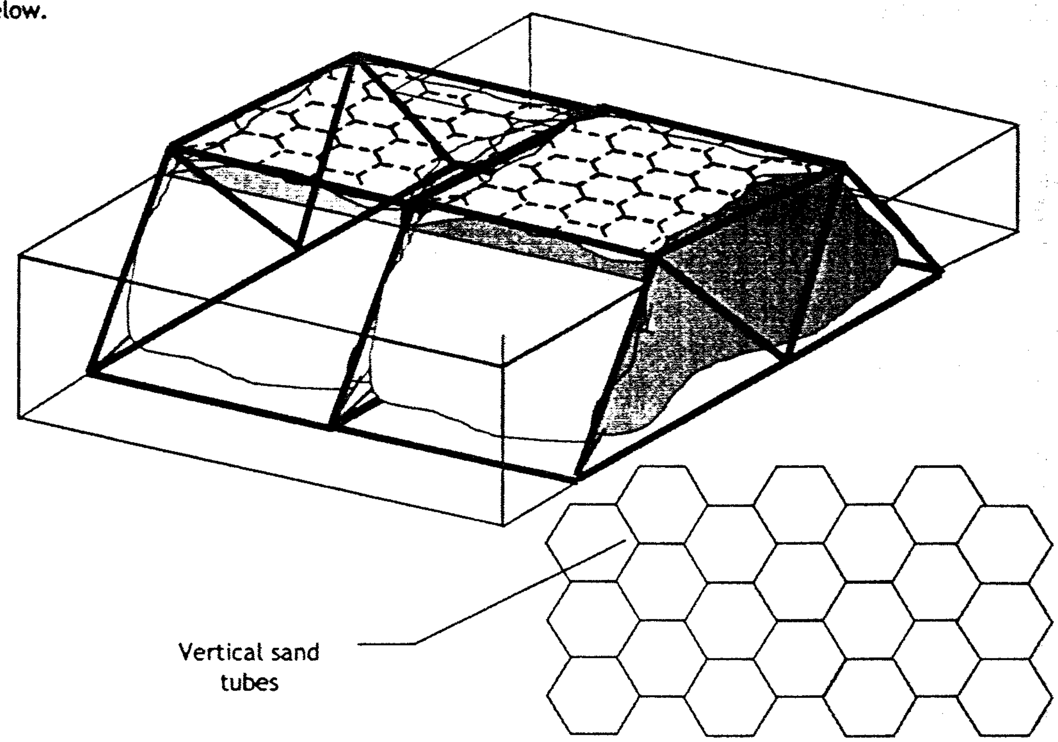
The usage of sand makes a bridge not longer necessary, since filling of the geotextile can be done waterborne (directly out the dredging vessels). However, the forces in the bag still require a support structure for the textile at least during the filling procedure *(compare the man who holds a bag in position, while another is filling it).*
The next problem is the shape in which the geotextile has to be formed to be able to withstand the forces of the sand. Main problem of the dredged sand is the fact that is has no direct internal strength when placed in the water. A reasonable option for this problem is to create a kind of framework to support the sand. When filled, the sandbag is not different from a very large 'stone' with a relative density of about 1. If a framework is needed anyway, the units can be placed independent of the bridge. This means that drilling of piles can be avoided.

Support structure
The framework needed for this operation could be a framework made of steel beams. As a first guess a box of 25 * 50 * 22 m (width * length * height) is taken. The height is taken from CD -10 m to CD +12 m. This means that this is a sort of caisson, which reduces the gap width by 25 m.
As a first assumption of the size of the beams, a diameter of 2 m can be taken. The geotextile will be fitted in this box. This box will be pre-fabricated on land and placed by a floating crane. There are two options for the closing procedure:

- Placing each box with the help of the floating crane directly on the bottom of the sea during slack water. After placing, the box can be filled with dredged sand.
- Alternatively, all the boxes could be placed on piles, and the lowered all together at once with the help of winches. This requires guiding of the boxes; at least two piles per unit. After lowering, the flow is blocked completely and the boxes will have to be able to withstand the head difference on the geotextile only. Filling should be done as fast as possible, which requires a tremendous dredging capacity.

The absence of piles and the manageability of the first closing process result in a preference for the first option.

For internal stability of the sand inside the geotextile bag, compartments are needed. In this way a bundle of vertical 'sausages' is created. Experience with horizontal geotextile sausages exists, the main difference is the fact that they are rotated 90 degrees upward. The basics of this box are drawn below.



Each compartment will consist of a hexagonal vertical tube. A round tube would be more favorable, but between round tubes some free space remains. The proposed structure is shown in the figure above:

13.3.2 Temporary design of the geotextile bag

After a brainstorm the following construction unit was developed. It is a steel-framed box, covered with geotextile and filled with a tubular geotextile structure. See figure 12.3.

These structures will be placed one by one in the final closure gap with the use of a large floating crane. After placement they will be filled with sand.

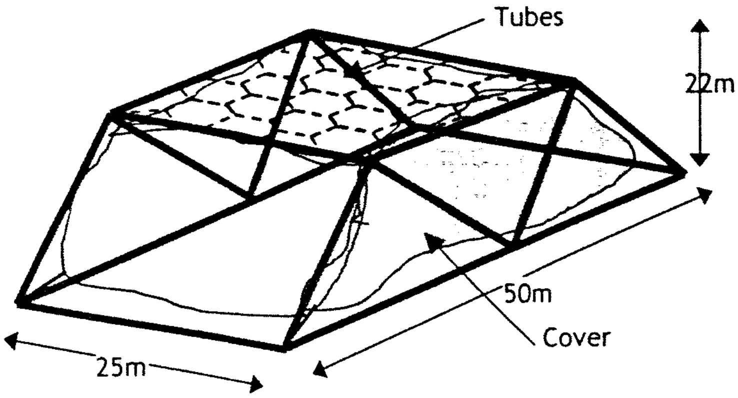


Figure 13.3 Steel framed box with geotextile

The construction of this closure will be the following:

1. Creating bottom protection;
2. Creating TPF with large extra openings;
3. Creating a sill (maximum current velocity will be high; around 6,5 m/s at the final gap);
4. Closing secondary dam sections (through shallow water);
5. Placing the prefabricated boxes, these are built on-shore. They are transported to the placing location by barges and are then lifted by a crane and positioned above the desired location. At slack water it is lowered to the seabed and filled by the dredging vessels. This procedure is repeated every slack water.
6. When the last box is placed the dredging vessels will create a dam body behind the bags just as shown at the secondary dam section. At the seaside a stone defense is necessary.

13.3.3 Developing the final design of the geotextile bags

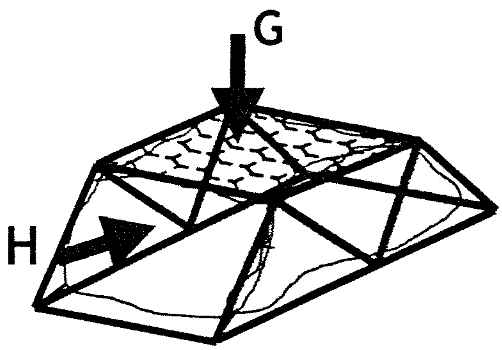
First has to be determined whether such a bag will be stable or not. Secondly has to be determined if it is possible to construct these boxes of geotextile. The third step is to reconsider the steel frame and the possibility to remove the frame. The fourth design step is the filling procedure. The last step is the execution of the closure. Now the final shape of the bags can be determined.

13.3.3.1 Stability of the boxes

There are three main loads on a box placed in the closure gap, first the hydraulic forces and secondly the forces of the sand inside the box. Furthermore, wave forces should be taken into account.
The force of the current depends mainly on the current velocity. In other words, the placing conditions of these structures determine the current velocity and thus the forces the structure has to withstand. In the sections of placing of these structures the maximum velocity is chosen at 6,5 m/s.

Static stability

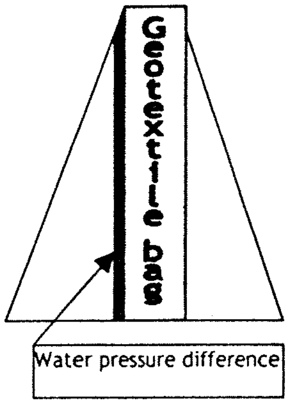
The first question is whether these geotextile boxes will be stable when used in a closure. This can be checked easily. The question is simple: will the box be pushed aside by the water? The following relation is valid for this problem:



$$n \times H \leq G \times \tan \varphi$$

In which:

n = safety factor, normally stated at 1,5
H = horizontal force by water pressure.



- H is maximum head difference. This maximum head difference is 2,7 meters (dam is closed, tidal power facility and all extra gaps still open). If the orifices are closed directly after the last geotextile box is placed, the maximum head will be 5m. is:

$$((27)\times 2{,}7\times \tfrac{1}{2}+19{,}3\times 27)\times 25=13939kN$$

G = The vertical force resulting from the own weight. The own weight of one bag is roughly:
Volume * density, the volume is estimated at 35 x 25 x 22 = 19250m^3. The density of the wet sand is estimated ate 1900 kg/m^3. A worst case scenario occurs when most of the sand is under the water level, the situation that occurs will be 2,5m wet sand and 19,5m sand under water. In that situation G will be:

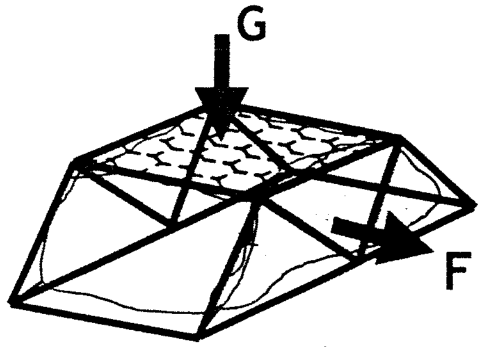
$$G=V_{wet}\times\rho_{wet}+V_{underwater}\times\rho_{underwater}$$
$$=(2{,}5\times 35\times 25)\times 19+(19{,}5\times 35\times 25)\times 9=195125kN$$

φ = the angle of internal friction between the bottom and the geotextile bag

This results in the following required minimal value for φ of 6,1°. This will be critical when the boxes are placed on the proposed sand mattresses (§10.8) because the commonly used value for φ for geotextile placed on geotextile is 0,2 φ_{sand} (φ_{sand} is 30°, so $\varphi_{geotextile}$ = 6°). Using a thin layer between the two geotextiles (coconut fibers for instance) can solve this. (If the tidal power facility and spillway are closed immediately after the last geotextile bag is placed, the required $\varphi_{geotextile}$ is 10,6°. This value seems to be too high to be easily compensated.) If the boxes are placed on a dumped (rock) sill, there occurs no problem, φ for geotextile on sand or rock, is 0,6-0,9 $\varphi_{sand/rock}$. These values are about 30° or more.

Flow
The flowing forces result in two main attacks on the boxes: An active 'suction' pressure caused by the water:

$$F=\frac{1}{2}*C*\rho*u^2*A$$

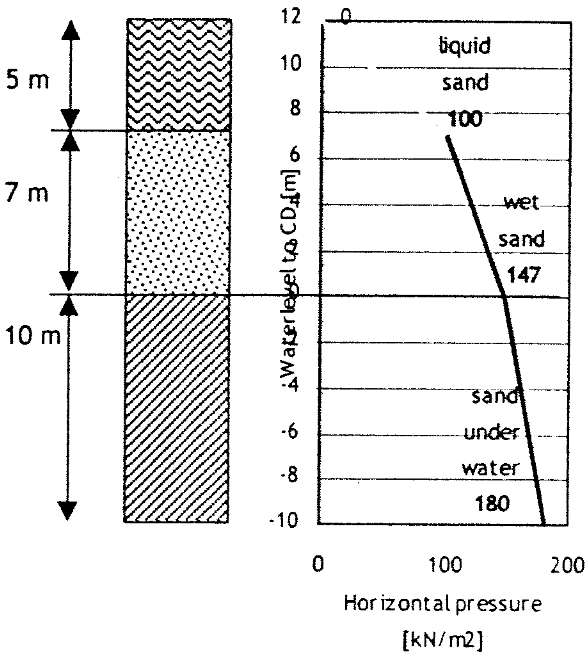


This results in an estimated force of 11000 kN (u = 6,5 m/s, C=1, A=15 x 35) on a box. The same static relation as above is valid resulting in a required φ of 4,8°.

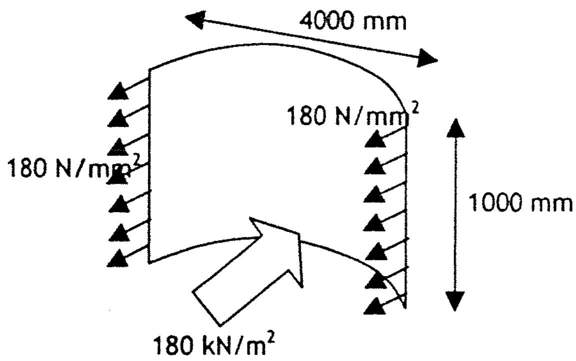
The water flowing through the closure gap will also induce a severe dynamic suction force on the boxes resulting in fluttering of the geotextile. This dynamic force will cause degeneration of the geotextile as long as the boxes are not filled. Because the boxes are filled within one hour, this fluttering will not be very severe (right after slack water the velocities are low) it is assumed that the geotextile will be able to withstand these forces.

13.3.3.2 The forces on the geotextile

It is very difficult to estimate the exact forces acting on the geotextile. Two situations are determined.



The first force occurs inside the tubes. The tubes will be filled with hydraulic sandfill. This hydraulic filling will cause some temporary fluidized sand in the tubes. When the tubes are filled with wet sand, the water outside is low (Low Water Spring) and the upper five meters of the tubes are filled with liquid sand, the worst possible situation occurs.
The forces have their maximum near the bottom. The first five meters (liquid sand) will create a horizontal force of 20 kN/m^2. In the rest of the tube the horizontal grain pressures will produce the stress in the fibers. The horizontal pressure is 1/3 off the vertical grain pressures. From the bottom upward to CD the underwater weight of the sand is 10 kN/m^3, and from CD to CD +12 m upward the weight of the wet sand is 20 kN/m^3. All this results in a horizontal grain pressure of 180 kN/m^2.



This pressure has to be taken by the geotextile tube. For a tube with a diameter of 4 meters, this results in a force of 180 N/mm^2 for a 2mm thick geotextile at the bottom. This is a rather high value. To avoid an extreme thickness, the geotextile should be reinforced.

Using a formula for composite structures:

$$\sigma_C = V_A \times \sigma_A + V_B \times \sigma_B$$

(In this relation, σ is the tension in MPa and V is the relative volume.)
With 80 MPa for normal geotextile and 2900 MPa for the aramide fibers, this leads to an aramide percentage of 10.

The weakest points in the tubes will be the connection seams. It is however possible to weave these tubes directly (seamless) just like a pantyhose. The tubes still have to be connected to each other but these seams between the tubes are less critical than the seams in the tubes. Besides that there will be a covering geotextile which will support the tubes in standing straight up and therefore reduce the forces on the connection seams.

The force on this covering geotextile is assumed to be much larger. It is assumed that for placing safety is wise to have the possibility to lift a partly filled bag. Since no existing crane vessel can lift a completely filled bag, is useless to design the geotextile for a completely filled bag. The largest crane vessel in the world can lift 14000 tons, the frame will have a certain weight (1000-4000 ton force), so decided is that the amount of sand maximum allowed in the bag is 10000 tons being underwater this also means 10.000m^3.

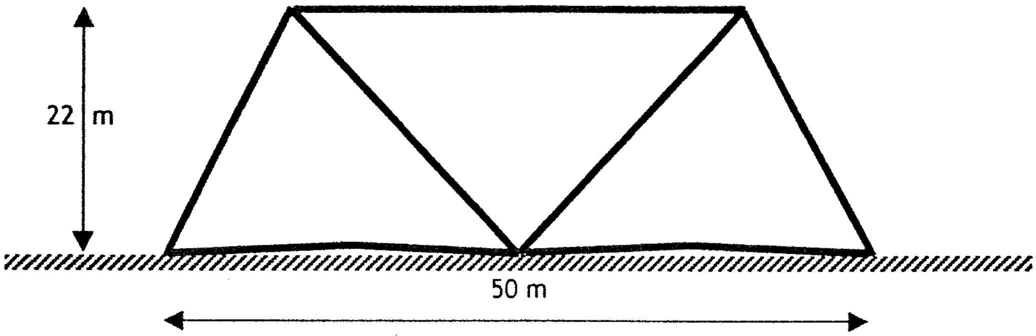
This second representative force (100.000 kN) will be carried by the covering geotextile. Only at the frame the geotextile is hold. This connection will have a total length of around

100 m. The force per meter is 1000 kN or 1000 N/mm. This is about three times the maximum force in the tubes, which requires a higher aramide fiber content (about 15 percent for 2 mm thick geotextile).

13.3.4 Removal of the steel framework

The steel support frame will partly take the loads of the water and sand. When filling the tubes, the frame provides stability. When the tubes are being filled, it provides general stability; it prevents movement of the filled bags, and prevents movement of the whole structure.

The framework is schematized in the following figure (side view):



The height is 22 m, the width at the bottom is 50 m and at the top 30 m. The length of one section is 25 m. These are first estimates.

Other framework?
The question rises whether it is possible to reuse the steel frame or not. The steel frame forms an expensive part of the structure with main objective to 'spread the bag' when filling and providing stability in the initial phase after filling. The steel frame has a certain risk: It requires a smooth bottom to prevent peak-loads on the steel tubes which could lead to collapse of the steel tube and damage the geotextile. Also, if it is too strong, the bag might tear off the frame and the frame.
The second problem of the frameworks are the side walls. These serve the stability of the framework, but they also hinder the geotextile bag in its settling against the other sandbags. This settling is absolutely needed to prevent flow between two neighboring bags. See figure 13.4.

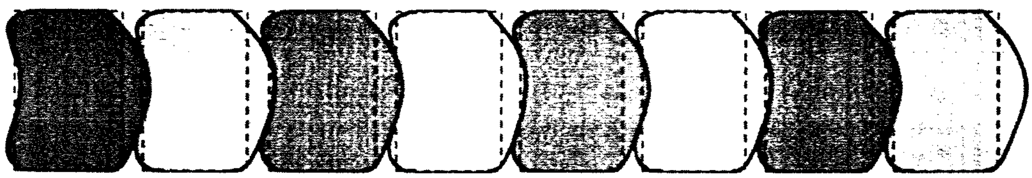


Figure 13.4 Settling of the geotextile bags against each other.

The solution for this problem would be a temporary framework. This framework should keep the bags on their places during the placing and filling of the bags, and should be removed after stabilization of the sandbags. Such a framework will be more complicated (=expensive) than the earlier proposed frameworks. But reducing the amount of frameworks will reduce the costs, and the absence of frameworks in the dam will provide a closed dam body.

The main question for this alternative framework is: are the geotextile sandbags stable without the support of the steel framework or not? This should be investigated by model tests. There are no investigations done on this problem. Ordinary sandbags have been studied but cannot be compared to these 'bags'. (*Ordinary sandbags can be lifted by one man!*)

Taking the size (and especially the width) of the bags into account, it can be assumed that the bags will be stable when filled with sand, without a framework, as long as they will not have to survive a severe storm.

The question now rises whether it is possible to combine the functions of the framework (initial stability and 'spreading the bag') with the filling procedure.

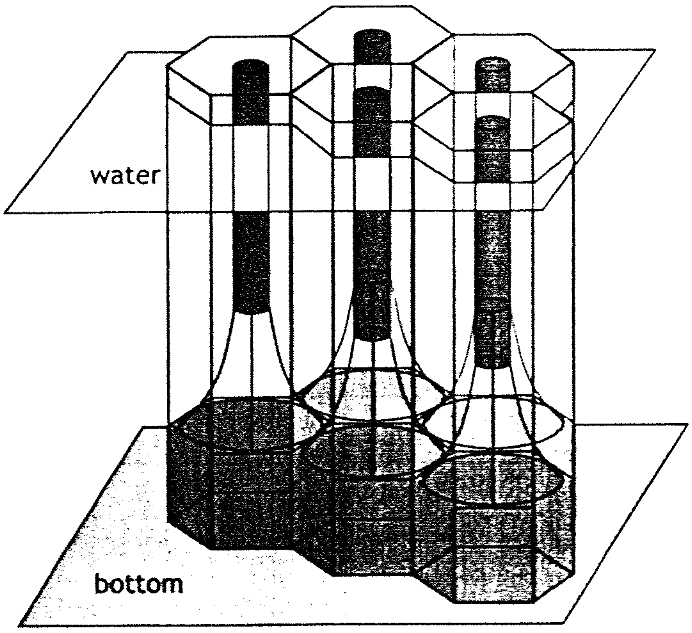
13.3.5 Filling procedure for the geotextile bags

The bags are placed in the gap by use of a crane, either floating or standing and walking on the sea bottom. Directly after placing the bags have to be filled with the sand. This filling has to be done very fast to create enough stability in the increasing current velocities. The filling will be done by large trailer suction hopper dredgers, capable in pumping 10000m^3 sand out their hopper per hour. The sand will be in a suspension of 1 to 4, so for each m^3 sand 4m^3 water is added. This water has to leave the bag without destroying the bag or eroding enormous amounts of sand. To realize this a special loading construction is needed to which the dredgers are connected. This filling construction should preferably be connected to the frame and be handled by the crane. The following design is made to fill the tubes.

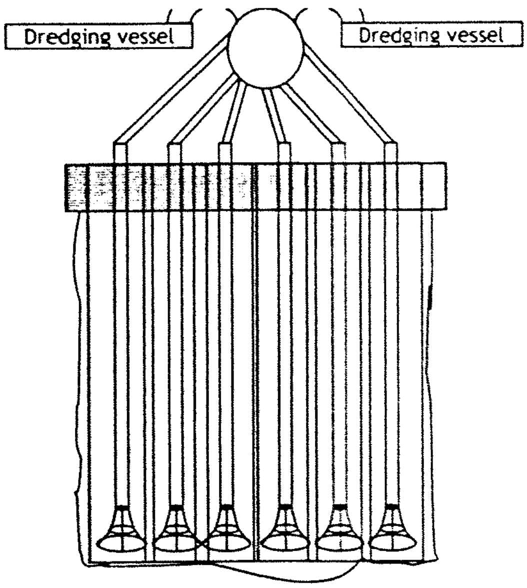
To a large steel frame, steel pipes (the same number as there are geotextile tubes) are connected. These pipes have a diameter of 0,8 m. At the end a large diffuser cone is placed. These cones have a diameter of around 3,5 m. (see figure 13.5).

At the top, all the pipes are connected to a diffuser. This diffuser can be connected to two trailer suction hopper dredgers. A gigantic crane can lift the whole construction. The bags are placed around the pipes, like fingers in a glove. Since the pipes almost reach the bottom of the bag, they can replace the steel frame, earlier designed to spread the bag and to provide initial stability. An impression how this works is given in figure 13.5.

The filling of the bags is schematized in figure 13.6. The slurry (sand water mixture) is pumped through the pipes, in the cones the flow spreads over the whole tube. The sand sinks to the bottom of the tube and the water flows upwards, out the of tubes. It is possible to hang a vibrating cone below the filling cone to improve the density of the settled sand. During the filling process the whole steel structure is slowly lifted out the Superbag preventing it from getting stuck in the sand and the bag.



Detail: steel pipes in tubes



Cross section of filling frame with bag

Figure 13.5 Tube filling system

The placing of a Superbag has the following phases:

Placing the Superbag around the steel pipe system;
The whole system is lifted up by the crane and positioned above the proposed location;
The dredgers are connected to the central diffuser. Most suitable option seems to be a connection at the backside of the lifting vessel and a flexible connection across the lifting vessel;

1. At slack tide the whole system is lowered to the sea bottom and the filling process starts immediately;
2. During the filling the steel frame is slowly lifted upwards, leaving the Superbag standing in the water;
3. When the frame is free from the Superbag, the dredgers are disconnected from the crane vessel and will fill their hopper again;
4. The crane is moved for the next placing operation and the cycle starts again.

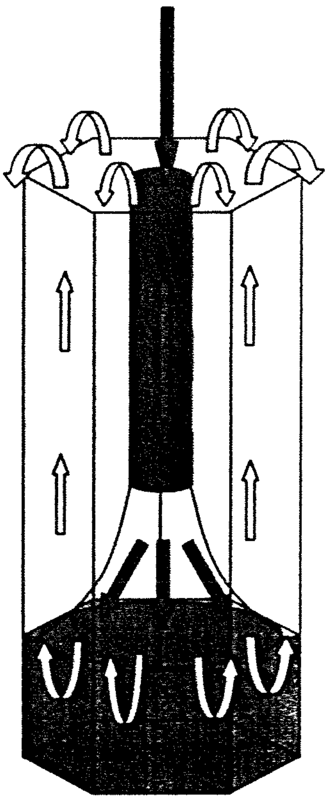


Figure 13.6 Flow during filling of geotextile tubes

The slurry flows downwards through the central (dark) pipe. In the cone the flow spreads, the sand sinks to the bottom of the tube, the water flows back through the tube. At the top the water is spilled in the sea.

13.3.6 Placing of geotextile units

13.3.6.1 Introduction

The geotextile caissons will be placed in the 10000 m wide closure gap that remains after the tidal power facility and the secondary dams have been finished. This section has an average depth of CD -15 m. The height of the closure dam will be CD +12 m (MSL +7 m).
Since the framework is removed from the design, a sill with a continuous height is not necessary any longer. The bags can have different heights, and will be placed directly on the bottom protection. Since the average depth in the final gap is CD -15 m, the height of the bags will be about 25 m. (Bottom protection of (sand) mattresses, two meters thick and bags up to CD +12 m.)

The placing of the bags continues until the current velocities have increased to the maximum value that the caissons can withstand. This value is (arbitrarily) estimated at 6,5 m/s. The first time this value is reached will be at Spring tide. The corresponding gap has a width of 2500 m.

According to the model results in chapter 8 the width at which this velocity occurs is much smaller (1800 m). This closure however is a horizontal closure, so the contraction of the flow has to be taken into account. Also the fact that the calculations in chapter 8 are made for a sill level of CD -15 m has to be compensated. The last parameter to incorporate is the fact that the tidal range can be a bit higher than the predicted tidal wave. The maximum amplitude is set at 5,2 m. All this results in a maximum velocity of 6,5 m/s at a gap with of 2500 m. The total reduction factor is ≈ 0,7 (=1800/2500).

The development of the velocities plotted in figure 13.7. It can be seen that under maximal amplitude conditions the value of 6,5 m/s is reached at a gap width of 2500 m.

At a width of 2500m the placing of the bags is stopped temporarily.

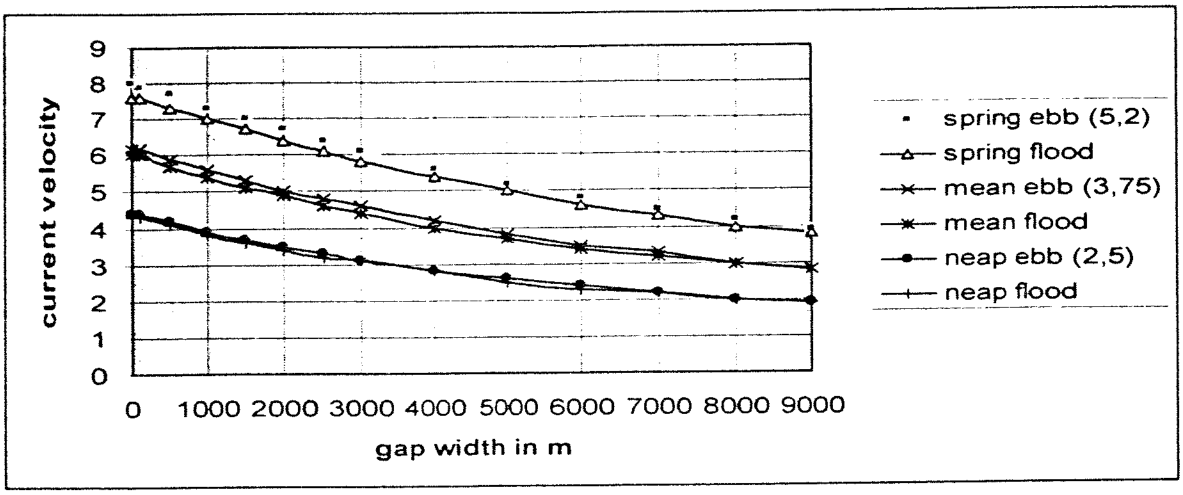


Figure 13.7 Development of the (absolute) maximum velocities related to the gap width

13.3.6.2 Strategy of geotextile bag placing

Closure can proceed as described above until the current velocities have reached the value of 6,5 m/s at spring tide. After spring tide the tidal difference reduces until Neap tide is reached a week later. Then the difference increases again.
When the closure of the final gap of 2500m starts, use can be made of the decrease in tidal difference. This will compensate the increase of current velocity due to the gap reduction.
A time window of 9 days exists until mean tide (7,5 m). After this, the tidal difference causes unacceptably high velocities. This is made visible in figure 13.8.

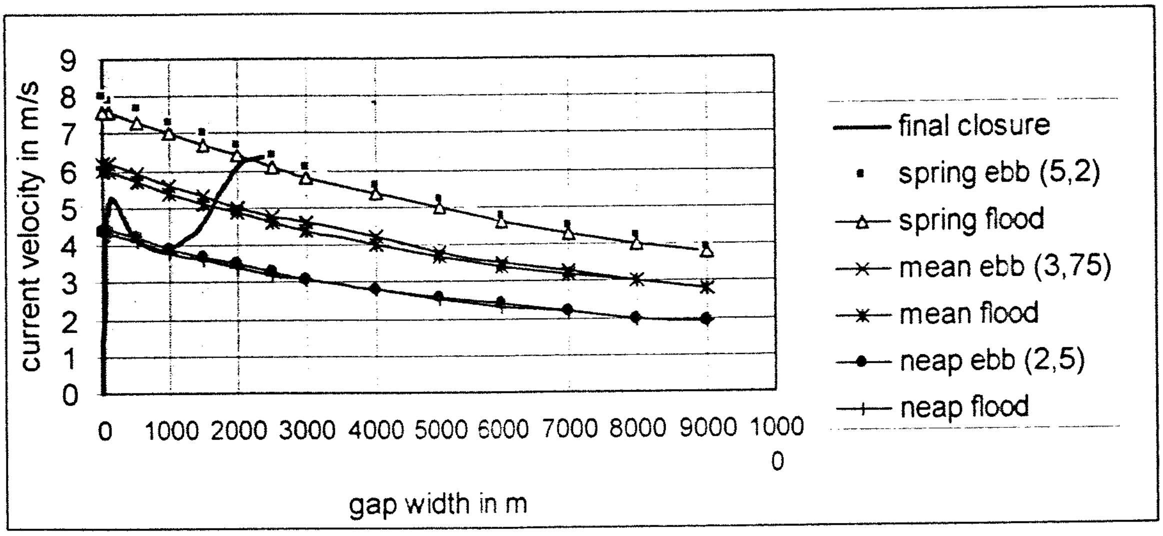


Figure 13.8 Closing of the final 2500m

In this figure (12.8) it is shown that, although the gap is reduced, the current velocities even decrease as a result of the much lower tidal range.
So, the gap of 2500 m has to be closed within these 9 days. In these 9 days 35 slack water periods are available for placing. With continuous placing (each slack water period one structure and working from both sides) a unit width of at least 35 m is required. When the width becomes greater the planning is easier, as not every slack water period has to be used. When the width is 50 m, ten periods do no have to be used. When the width is 25 m, as proposed earlier in this study, two units have to be placed per slack water period. Since two placing operations at the same gap during slack water seems to be impossible, the required width (in gap direction) of the geotextile bags is changed in 40 meters (final closure 8 days).

The units are filled by hydraulic sandfill. Each unit has dimensions of 40*30*25 m. This corresponds with a volume of 30000 m^3. These 30000 m^3 have to be filled with sand by trailer suction hopper dredgers. Since the largest dredgers have a hopper capacity of 20000 m^3 it is possible to fill the bag using two.

It should be very clear that when the final 2500 m will be closed, a point of no return is reached. If for any reason the process is stopped when the gap is (for example) only 1000 m, the currents may increase to a point that the structure will collapse. This can be avoided by increasing the strength of the unit (which increases the costs). The exact possibilities for increasing strength and stability (increase of weight) have to be investigated. It is necessary to learn of the problems encountered in the other 7500 to ensure a problem-free final closure.

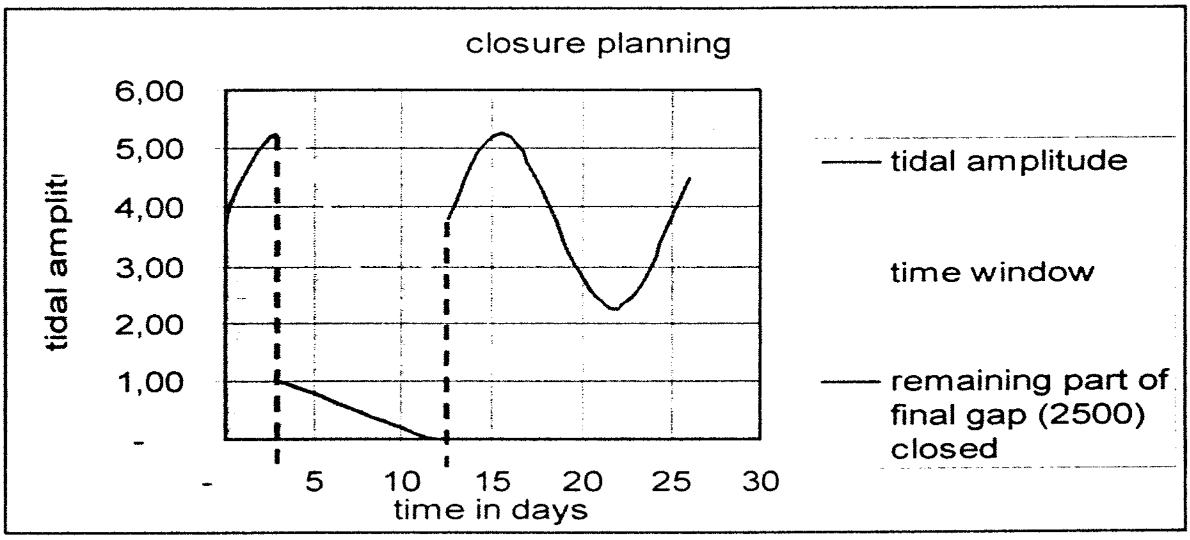


Figure 13.9 Development of tidal amplitude and width of final gap during closure

In figure 13.9 the tidal amplitude (varying from mean to spring and neap), the time and the proceeding of the closure (related to the whole 2500 m gap) are presented. It should be clear that use can be made of the tide to make the final closure easier. However, the advantage of the lower velocity is not everlasting. Finishing before the deadline is reached is necessary otherwise the process fails.

13.3.7 Wave forces

Wave forces can be considerably, but storms mostly occur in monsoon periods. As this type of closure can be executed very quickly, wave forces during the closure process will not be that much of a problem. When the closure dam is finished but the final sea defense not, a different situation occurs. It should be investigated by model tests whether the filled unit acts like a sand dike with a geotextile coating, or as a (very large) stone with relative density 1

In this study this is not engineered further. It is assumed that the strength of the textile and the internal forces let the sandbag act as a solid unit. This approach causes no stability problems. If during model investigations problems occur, the planning of this closure has to be made very safe, starting right after the monsoon, leaving the rest of the year for reinforcement.

13.3.8 Final shape of the Superbags

The shape of the original geotextile bags is changed. Stability of these new shape will be the same as the temporary design. So the final dimensions of the geotextile bags, further referred to as Superbags, are 40*30*25 m (see figure 12.10). The height of the bags can be adapted to the exact depth.

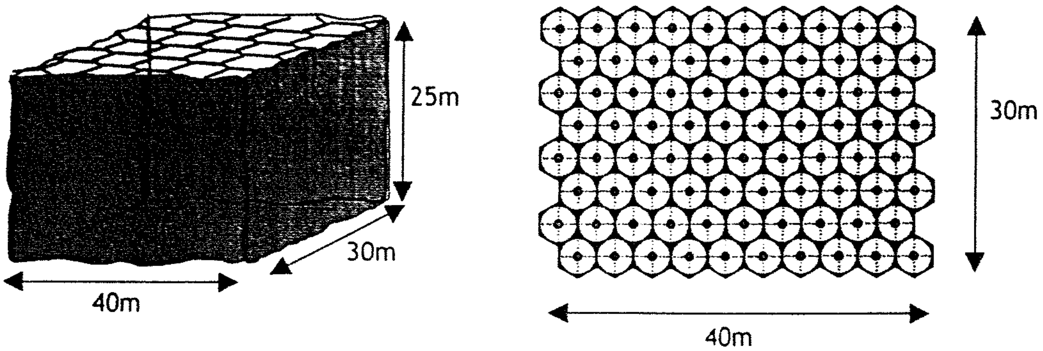


Figure 13.10 Final shape of the Superbags

When the tubes have a diameter of 4 meters, 80 tubes per Superbag are needed. The total amount of Superbags needed is 10000/40 = 250.

One tube needs $2*\pi*r*h = 2*3{,}14*2*25 = 314\ m^2$ geotextile per tube. For 80 tubes this results in 25000 m^2. For the covering geotextile $2*40*25+2*30*25+40*30 = 4700\ m^2$ is needed.

The steel filling frame will need 80 steel pipes, when these pipes have a thickness of 25 mm, the mass of these 80 tubes will be: $(2*\pi*r*d*h)*80*7800 = (2*3{,}14*0{,}4*0{,}025*25)*80*7800 = 1000$ tons. Since these tubes have to be connected, and the filling installation has to be added, the total mass of the structure is estimated at 2000 to 2500 tons. This is very heavy, but can be lifted by existing crane vessels (the largest floating crane in the world can lift 14000 tons). The biggest problem for a crane vessel will be the depth, the required working space and the current velocities near the dam head. These problems might require a lifting platform. This problem should be subject to further studying.

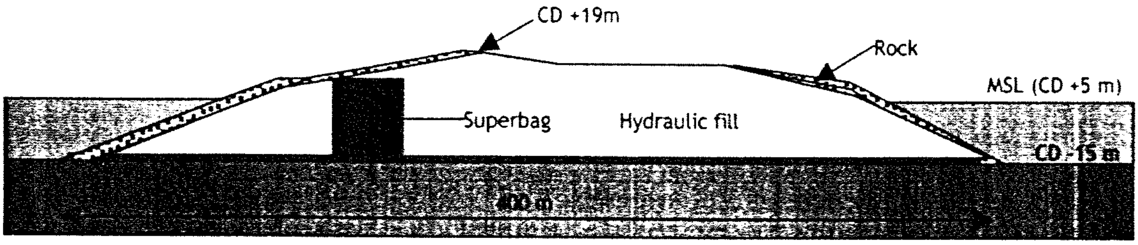


Figure 13.11 Final dam body behind the Superbag dam

The final dam can be created by hydraulic fill. The same procedure as for the secondary dam sections will be followed. The difference is that the initial dam is not a rockfill dam that can be used, but the Superbag dam. Therefore hydraulic fill is also needed at the seaside of the dam. The whole dam has to be protected with rock against waves.

13.3.9 Costs

Amount of geotextile

This alternative is based on aramide fibers, these fibers are very expensive, so the question rises whether the price of these fibers will influence the cost of this alternative dramatically.

The price of Aramide fiber is based on the price given by Akzo-Nobel about their aramide fiber Twaron®. The minimum price at this moment (1998) is stated at NLG 30 per kg (or NLG 30.000 per ton). Their annual production is around 10.000 tons; their main competitor is Dupont. Dupont fabricates Kevlar®, estimated annual production 20.000 tons, their price for use in civil engineering is around NLG 40.000 per ton. Since the real price will be a matter of negotiating, both offers are averaged.

The total amount of geotextile needed to close the final gap will be:
250 Superbags, containing 25000 m^2 geotextile for the tubes and 4700 m^2 for covering.
When these geotextiles are 2 mm thick the required aramide fiber content is 10 and 15 %.
The density of aramide fibers is 1440 kg/m^3 (the density of the other fibers is 1200 kg/m^3).
The total amount of aramide needed is:

Type of textile	m2 of geotextile	Tons per Superbag
Tubes, 10% Aramide fiber	25000	7,2
Covering 15% Aramide fiber	4700	2,2
Total for one Superbag	29700	9,4

250 Superbags will require 2350 tons of aramide fibers. With a kilogram price of NLG 35.000 the material costs will be 82 million NLG. Although very expensive, the quantity of aramide fibers is low enough to have a minor influence on the total project costs.

The price of aramide fibers has been decreasing for many years, it can be expected that the price will decrease even further. The used prices are only an indication given by the two companies.

13.3.10 Conclusions about the Superbags

The possibilities of this method seems promising, extensive research should be done to obtain more insight on the behavior of geotextile and sand in combination with a framework in heavy current situations.
The main advantages are:

- The closure and the final sea defense can be done by hydraulic sandfill;
- The closure can be carried out very quickly (last gap of 2500 m in 8 days).

Disadvantages are:

- The costs of high-tech materials are high, but the amount of high-tech material needed is small;
- The risks in placing are high, due to the tight schedule. These risk however decrease when insight in behavior and possibilities of the Superbags increases;
- The method is new, and currently very little insight exists in the hydraulic behavior;
- The filling equipment will be very heavy and therefore difficult to handle.

13.4 Bottom protection for final gap

The Superbags consist of sand, just like the proposed sand mattresses, the combination of these two geotextile structures leads to a better structure than a Superbag on a traditional rock bottom protection. By combining the geotextile structures the whole structure is homogeneous and undesirable flow (piping via the rock bottom protection) is absent.

The same can be said of the rock closure, dumping stones on stones will give a homogeneous dam construction. A strange situation in the dam body occurs, small stones are dumped on a bottom constructed with much larger stones. When all the small stones are covered with a layer that contains the same stone diameter as the bottom protection, this is nor problem

Summarizing can be said that the bottom protection should be of the same material as the closure dam. There are however many uncertainties about the sand mattresses.

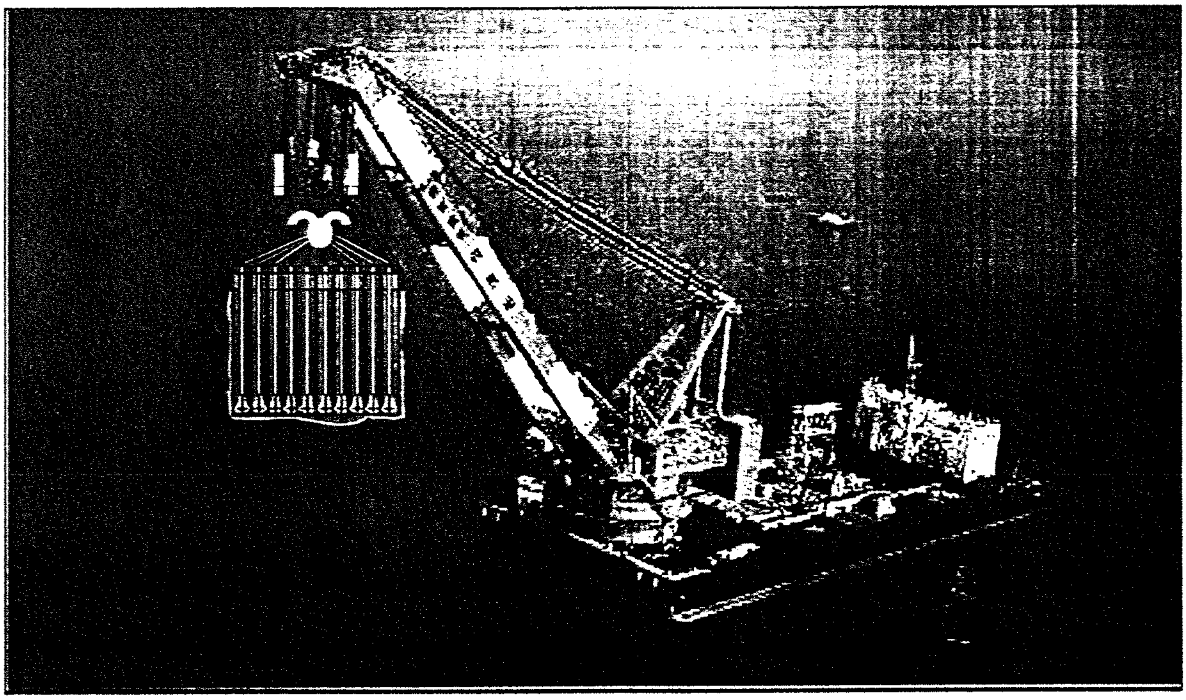


Figure 13.12 Impression of size Superbag lifted by a crane vessel with 2500 tons lifting capacity

Part D Construction time, costs and risks

This Part describes the costs, building time and risks involved with two final designs. The basics for these designs are (see figure 14.1):

- tidal power facility (including sluices) constructed as caissons with maximum extra orifices, as described in chapter 4;
- The Narmada Spillway constructed as caissons, described in chapter 4;
- The secondary dam sections will be constructed of rock and enlarged with hydraulic sandfill as described in chapter 6;
- The islands and diversion dam will be constructed with hydraulic sandfill and some protection works as described in chapter 6;
- The shiplocks will be constructed in the island east of the tidal power facility;
- For the final gap there are two design considered, the rock closure with a train bridge (chapter 12) and the closure with (geotextile) Superbags (chapter 13).

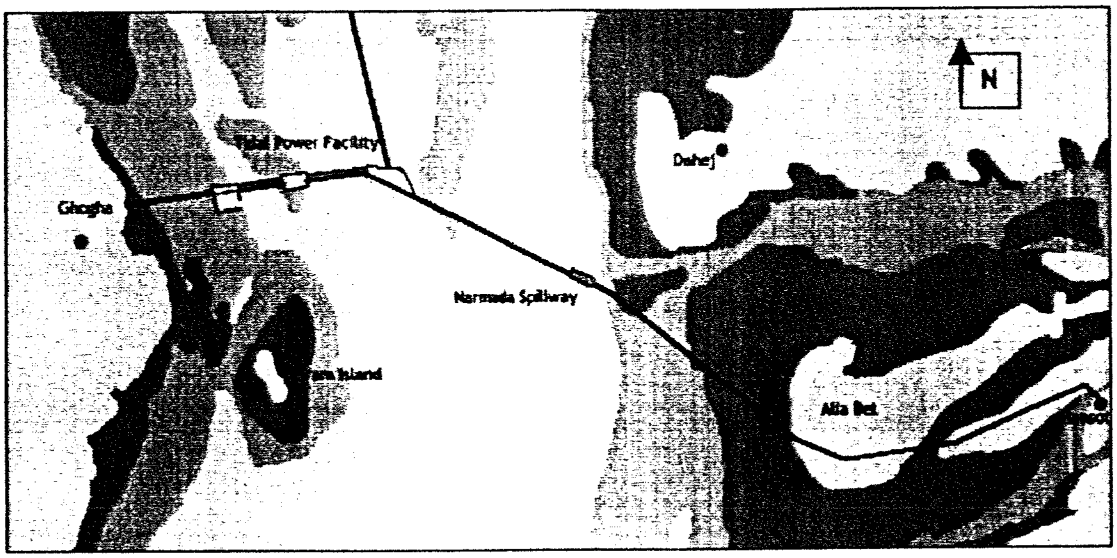


Figure 14.1 Overview of dam alignment

Chapter 14 compares the costs of the rock and Superbag closure with concrete sluice caissons. It also compares the building time of the rock and the Superbag closure.
Chapter 15 is a qualitative risk analysis.

14 Construction time and cost estimate

14.1 Introduction

In this part a planning of all the works will be made. From the previous parts is has become clear that the two most promising alternatives are the stone closure and the geotextile structures. The best option for the tidal power facility is the caisson alternative, as described in part two.
With these alternatives two combinations are made. For these two the construction time and the costs will be calculated, the risks will be discussed in chapter 15. These three parameters are compared to the existing design with closure caissons.

These are estimates for the closure dam only. The following items that are necessary for the Kalpasar project and have not been calculated in this study are:

- Finishing of the tidal power facility including turbines and powerhouse installations;
- Narmada spillway installations;
- Irrigation scheme.

Included are:

- Final dam profile;
- Separation dam between tidal and fresh basin;
- Roads and railways on the dam and from the quarries to the dam.

14.2 Setup of cost and building time calculations

All calculations have been made in one calculation scheme, which has been built up using the following procedure.

Component	These are the components that have been defined earlier, such as 'Secondary damsections' and 'final gap'.
↓ Subcomponent	Subcomponents are parts of the components, such as 'section Ghogha - tidal power facility'.
↓ Item	Items are parts of the subcomponents with uniform material or construction demands.
↓ Material	The material of which an item is constructed.
↓ Construction method	How the item is constructed, and what kind of equipment is required.
↓ Amount and unit	The total amount of material that an item requires, and the unit in which it is expressed.
↓ Unit rate	The price of one unit in Indian rupees. One NLG equals 20 rupees (one US$ equals 38 rupees).
↓ Cost	The amount of material multiplied by the unit rate.
↓ Building capacity	The building speed of the equipment per unit of time.
↓ Construction time	The amount of material divided by the building capacity. The construction time is expressed in months.

With the results of these calculations (the construction time per item in months) the planning of the total design can be made. There are however some considerations in the planning. As some items have a very strong influence on the boundaries, some relations between items are mandatory.
This is for example the case with the placing of the caissons. These have a serious impact on the current velocities in the vicinity. When the bottom around and near the caissons is not protected, erosion can cause serious problems.
The closure dam in the final gap is of course the last part that has to be constructed.

Below the price table is given with the costs per unit as has been used in this study.

Table 14.1 Unit price list

price code	description	unit	unit rate IRS	unit rate NLG
A	trailer dredger + dumping	m3	42	2,1
B	trailer dredger + pumping	m3	77	3,9
C	trailer dredger 15 km + pumping	m3	96	4,8
D	profiling activities 5000 m3/hour	m3	4	0,2
E	earth, excavating, 3km transport and profilin	m3	90	4,5
F	profiling sand	m3	9	0,5
G	cutter suction dredge + 3km transport	m3	72	3,6
H	cutter suction dredge + 3km transport (clay)	m3	101	5,1
J	producing stones	ton	100	5,0
K	load on dump truck	ton	10	0,5
V	unload	ton	43	2,2
L	transport (average)	ton	90	4,5
M	stocking	ton	21	1,1
N	load and dump with splice barge	ton	90	4,5
O	geotextile mat including equipment	m2	3.000	150,0
P	reinforced concrete all-in	m3	8.000	400,0
Q	bridge	m2	30.000	1.500,0
R	construction steel	ton	60.000	3.000,0
S	placing caisson	st	50.000.000	2.500.000,0
T	harbor facilities	st	20.000.000	1.000.000,0
U	train dumping	ton	200	10,0
X	geotextile 15%	m2	500	25,0
Y	geotextile 10%	m2	200	10,0
Z	placing of geotextile unit	st	10.000.000	500.000,0
AB	road	km	17.280.000	864.000,0
AC	railway	km	200.000.000	10.000.000,0
AD	town 20000 people	st	400.000.000	20.000.000,0
AE	electro/mechanical for caisson	st	15.000.000	750.000,0
AF	slope protection seaside	m	60.000	3.000,0
AG	slope protection basinside	m	30.000	1.500,0

14.3 Schedules

The following A3 pages contain the cost estimate and planning of both the stone closure and the Superbags (both including the tidal power facility).
The planning is in months. Each year a monsoon period of two months is expected.

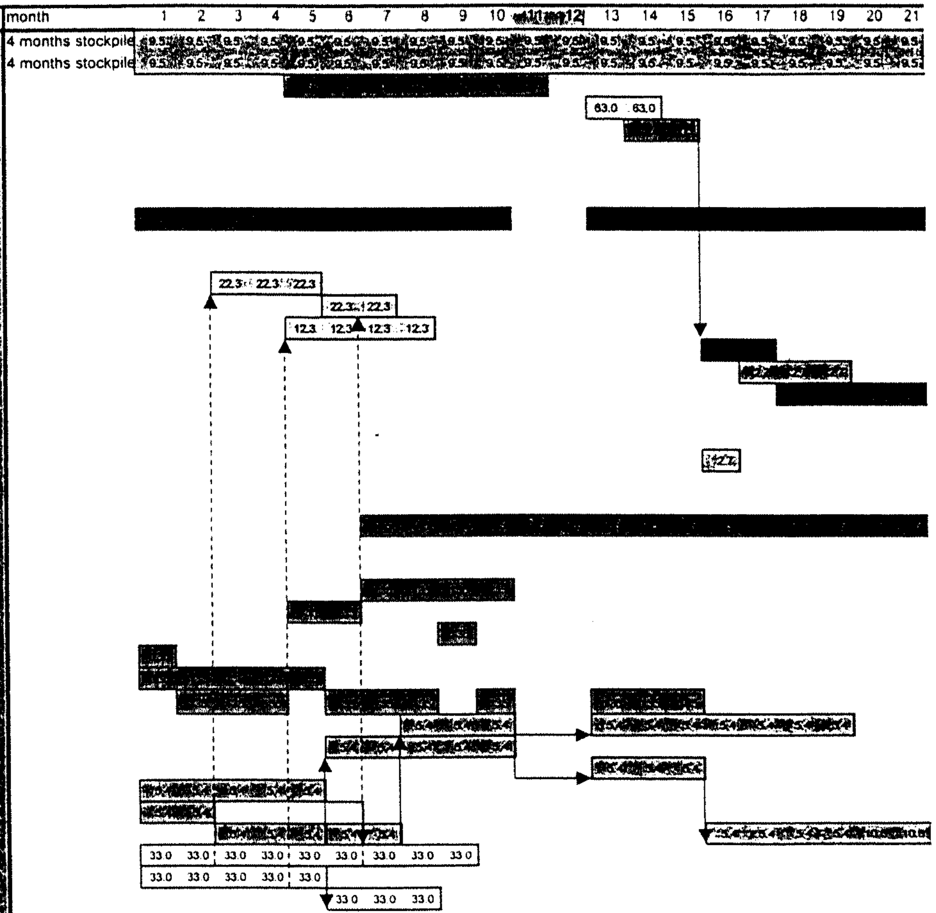
Planning of the works: TPF caisson rock closure

TOTAL in crores F 13,055

Component	subcomponent	item	material	construction method	amount	unit	comment	unit rate in rs	code	cost in crores rupees	building capacity	building time (days)	month		cost per month
quarries	east side (Bharuch area)		rock	blasting, sorting	12365256.6	ton	100000 tons per week	221	J K L M	273.27	14286	866	28.9	A	9.5
	west side (Bhavnagar site		rock	blasting, sorting	49461026.4	ton	100000 tons per week	221	J K L M	1,093.09	14286	3,462	115.4	B	9.5
secondary damsections	Ghogha - TPF	top level CD+19													
		closure dam	rock	landborne	4180410	ton	trucks, 500 ton/hour	43	V	17.98	10000	418	13.9	C	1.3
		CD-3,5 (6000 m)	sand	hydraulic fill	10800000	m3	2 jumbos, 4 hour cycle	105	C F	113.40	200000	54	1.8	D	63.0
		Armour layer (6000 m)	rock 1,2m	landborne	452790	ton	crane 250 ton/hour, finishing dambody	86	VV	3.89	6000	75	2.5	E	1.5
	TPF - Final gap	top level CD+19													
		closure dam	rock	water- and landborne	3891888	ton	trucks, 500 ton/hour	43	V	16.74	10000	389	13.0	F	1.3
		CD-10 (3000 m)	sand	hydraulic fill	9000000	m3	2 jumbos, 4 hour cycle	105	C F	94.50	200000	45	1.5	G	63.0
		Armour layer (3000 m)	rock 1,2m	landborne	226395	ton	crane 250 ton/hour	86	VV	1.95	6000	38	1.3	I	1.5
	Spillway - Hansot	top level CD+16													
		CD (10500 m)	clay	landborne	14112000	m3	crane, trucks, 1000 ton/hour	90	E	127.01	24000	588	19.6	J	6.5
		CD +10 (20000 m)	clay	landborne	5280000	m3	crane, trucks, 1000 ton/hour	90	E	47.52	24000	220	7.3	K	6.5
		CD +5 (4000 m)	clay	landborne	2816000	m3	crane, trucks, 1000 ton/hour	90	E	25.34	24000	117	3.9	L	6.5
work islands	TPF (Center)	top level CD+16													
		CD -6 (1500*300)	sand	hydraulic fill	9900000	m3	2 jumbos, 4 hour cycle	59.75	3/4 A, 1/4 B, F	59.15	200000	50	2.7	M	[illegible]
		slope protection seaside			1500	m		60000	AF	9.00	100	15			
		slope protection basinside			1500	m		30000	AG	4.50	100	15			
	TPF (East/shiplock)	top level CD+16													
		CD -6 (1500*300)	sand	hydraulic fill	9900000	m3	2 jumbos, 4 hour cycle	59.75	3/4 A, 1/4 B, F	59.15	200000	50	2.7	N	[illegible]
		slope protection seaside			1500	m		60000	AF	9.00	100	15			
		slope protection basinside			1500	m		30000	AG	4.50	100	15			
	Construction dock for caissons	top level CD+16													
		dike CD-5 (4000 m)	sand	hydraulic fill	8400000	m3	2 jumbos, 4 hour cycle	59.75	3/4 A, 1/4 B, F	50.19	200000	42	4.1	O	[illegible]
		slope protection seaside			4000	m		60000	AF	24.00	100	40			
		slope protection basinside			4000	m		30000	AG	12.00	100	40			
Tidal Power Facility		caisson factory	-			st		20000000		2.00			2.0	P	[illegible]
		construction of sill	rock	waterborne	1474200	ton	2 conventional stone dumping vessels	90	N	13.27	10000	147	4.9	Q	[illegible]
	30 turbine caissons	62000 m3	concrete	construction dock	1860000	m3	2 concrete production plants, 60 m3/hour	8000	P	1,488.00	3840	484	16.1	R	[illegible]
	15 sluice caissons	50000 m3	concrete	construction dock	750000	m3	2 concrete production plants, 60 m3/hour	8000	P	600.00	3840	195	6.5	S	[illegible]
		placing of caissons		floating	45			50000000	S	225.00	3 per 14 days (only neap tide)		7.0	T	[illegible]
Spillway	10 sluicespillway caissons	construction of sill	rock	waterborne	327600	ton	2 conventional stone dumping vessels	90	N	2.95	10000	33	1.1	U	2.7
	same dock as TPF	40000 m3	concrete	construction dock	400000	m3	2 concrete production plants, 60 m3/hour	8000	P	320.00	3840	104	3.5	V	92.2
		placing of caissons		floating	10			50000000	S	50.00	3 per 14 days (only neap tide)	56	1.9	W	[illegible]
Final gap	piles for bridge	322 piles	steel	offshore hammering											
10000m	bridge structure construction	160 bridge elements	steel	welding											
	bridge width 10 m		-	floating crane	100000	m2		30000	Q	300.00			40.0	X	[illegible]
	dumping	29160000 tons	rock	train	29160000		5000 ton/hour	200	U	583.20	100000	292	9.7	Y	[illegible]
	dambody	4000 m3/m	sand	hydraulic fill	40000000	m3	2 jumbos, 4 hour cycle	77	A	308.00	200000	200	6.7	Z	[illegible]
		slope protection seaside			10000	m		60000	AF	60.00					
		slope protection basinside			10000	m		30000	AG	30.00					
Bottom protection															
	mattresses:				m2										
	TPF			mattress layer	1650000	m2	one vessel, one mattress (300*50) per day	3000	O	495.00	15000	110	3.7	AA	[illegible]
	Inlet sluices			mattress layer	825000	m2	one vessel, one mattress (300*50) per day	3000	O	247.50	15000	55	1.8	AB	[illegible]
	Spillway			mattress layer	550000	m2	one vessel, one mattress (300*50) per day	3000	O	165.00	15000	37	1.2	AC	[illegible]
	Work islands			mattress layer	1800000	m2	one vessel, one mattress (300*50) per day	3000	O	540.00	15000	120	4.0	AD	[illegible]
	Dam in Narmada mouth														
	Secondary damsections			mattress layer	1500000	m2	one vessel, one mattress (300*50) per day	3000	O	450.00	15000	100	3.3	AE	[illegible]
	Final gap, rock			mattress layer	4000000	m2	one vessel, one mattress (300*50) per day	3000	O	1,200.00	15000	267	8.9	AF	[illegible]
	covering:														
	TPF	very heavy 1650000 m2, 2m	rock	waterborne	5791500	ton	2 large stone dumping vessels	90	N	52.12	20000	290	9.7	AG	[illegible]
	Inlet sluices	very heavy 825000m2, 2m	rock	waterborne	2895750	ton	2 large stone dumping vessels	90	N	26.06	20000	145	4.8	AH	[illegible]
	Spillway	very heavy 550000m2, 2m	rock	waterborne	1930500	ton	2 large stone dumping vessels	90	N	17.37	20000	97	3.2	AI	[illegible]
	Work islands	heavy 1800000m2, 1m	rock	waterborne	3159000	ton	2 large stone dumping vessels	90	N	28.43	20000	158	5.3	AJ	[illegible]
	Dam in Narmada mouth	none													
	Secondary damsections	loose 1500000m2, 0,5m	rock	waterborne	1316250	ton	2 large stone dumping vessels	90	N	11.85	20000	66	2.2	AK	[illegible]
	Final gap, rock	heavy 4000000m2, 1m	rock	waterborne	7020000	ton	2 large stone dumping vessels	90	N	63.18	20000	351	11.7	AL	[illegible]
Dredging	Behind TPF	pit 500 wide, 35 deep, 3000 long	sand	waterborne	54000000	m3	2 jumbos, cycle 4 hours	55	A D F	297.00	200000	270	9.0	AM	[illegible]
	Behind sluices	pit 500 wide, 35 deep, 1500 long	sand	waterborne	27000000	m3	2 jumbos, cycle 4 hours	55	A D F	148.50	200000	135	4.5	AN	[illegible]
	Behind spillway	pit 500 wide, 35 deep, 1000 long	sand/mud	waterborne	18000000	m3	2 jumbos, cycle 4 hours	55	A D F	99.00	200000	90	3.0	AO	[illegible]
	Connection channel Narmada	gully 3000*500*5 m	mud	waterborne	7500000	m3	cutter	101	H	75.75	70000	107	3.6	AP	[illegible]
Other structures	railway	80 km double track			80	km		2.00E+06	AC	1,600.00					
	roads	100 km 20 m wide			100	km		17280000	AB	172.80					
	shiplock (fresh)	220 * 35 m	concrete	construction pit	234930	m3	assumption	8000	P Q	187.94					
	shiplock (tidal)	150 * 25 m	concrete	construction pit	99285	m3	assumption	8000	P Q	79.43					
	towns	20000 people			1	st		400000000	AD	40.00					
	diversion dams	40 km, 20 m high, 10 m crest, 1:3	sand	hydraulic fill	56000000	m3		77		431.20					
	impermeable damcover	slope protection basinside			40000	m3		30000	AG	120.00					
Other costs	electro/mechanical				6000	ton		60000		36.00					
	steel for caissons				60000	ton		60000	R	360.00					
	electro/mechanical for caissons				55	st		15000000	AE	82.50					
									total rs	13,055.23					

Planning of the works: TPF caisson rock closure

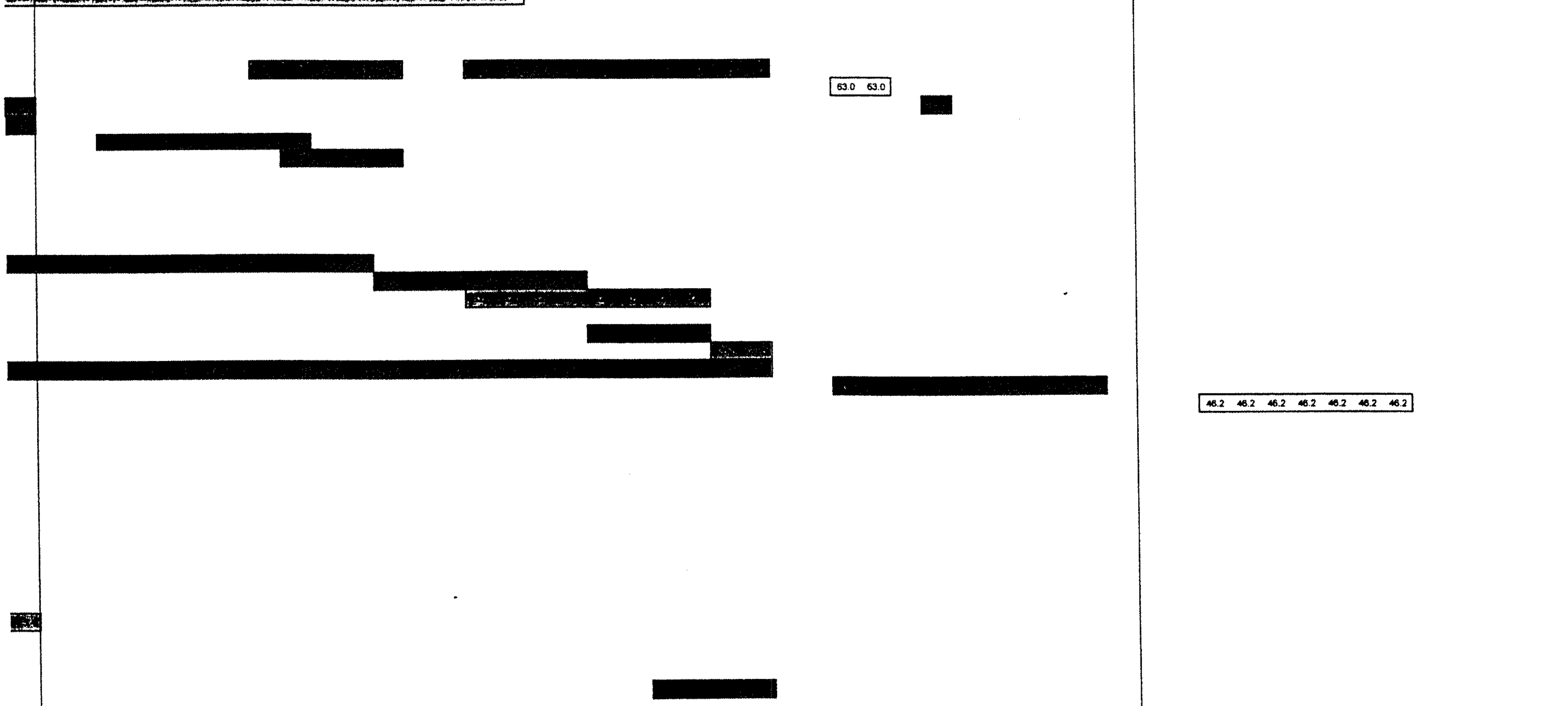
Component	subcomponent	item		time in months	cost per month
quarries	east side (Bharuch area)		A	28.9	9.5
	west side (Bhavnagar site		B	15.4	9.5
secondary damsections	Ghogha - TPF	closure dam	C	13.9	13
		CD-3,5 (6000 m)	D	4.8	63.0
		Armour layer (6000 m)	E	2.5	15
	TPF - Final gap	closure dam	F	13.0	13
		CD-10 (3000 m)	G	[illegible]	63.0
		Armour layer (3000 m)	I	[illegible]	15
	Spillway - Hansot	CD (10500 m)	J	19.6	6.5
		CD +10 (20000 m)	K	[illegible]	6.5
		CD +5 (4000 m)	L	[illegible]	6.5
work islands	TPF (Center)	CD -6 (1500*300)	M	[illegible]	22.3
	TPF (East/shiplock)	CD -6 (1500*300)	N	[illegible]	22.3
	Construction dock for caissons	dike CD-5 (4000 m)	O	[illegible]	12.3
Tidal Power Facility		caisson factory	P	[illegible]	[illegible]
		construction of sill	Q	[illegible]	[illegible]
	30 turbine caissons	62000 m3	R	[illegible]	[illegible]
	15 sluice caissons	50000 m3	S	[illegible]	[illegible]
		placing of caissons	T	[illegible]	[illegible]
Spillway	10 sluicespillway caissons	construction of sill	U	[illegible]	[illegible]
	same dock as TPF	40000 m3	V	[illegible]	[illegible]
		placing of caissons	W	[illegible]	[illegible]
Final gap	bridge width 20 m		X	[illegible]	[illegible]
10000m	dumping	29160000 tons	Y	[illegible]	[illegible]
	dambody	4000 m3/m	Z	[illegible]	[illegible]
	TPF		AA	[illegible]	[illegible]
	Inlet sluices		AB	[illegible]	[illegible]
	Spillway		AC	[illegible]	[illegible]
	Work island (TPF center)		AD	[illegible]	[illegible]
	Secondary damsections		AE	[illegible]	[illegible]
	Final gap, rock		AF	[illegible]	[illegible]
	TPF	very heavy 1650000 m2, 2m	AG	[illegible]	[illegible]
	Inlet sluices	very heavy 825000m2, 2m	AH	[illegible]	[illegible]
	Spillway	very heavy 550000m2, 2m	AI	[illegible]	[illegible]
	Work islands	light 1800000m2, 1m	AJ	[illegible]	[illegible]
	Secondary damsections	loose 1500000m2, 0,5m	AK	[illegible]	[illegible]
	Final gap, rock	heavy 4000000m2, 1m	AL	[illegible]	[illegible]
Dredging	Behind TPF	pit 500 wide, 35 deep, 3000 long	AM	[illegible]	[illegible]
	Behind sluices	pit 500 wide, 35 deep, 1500 long	AN	[illegible]	[illegible]
	Behind spillway	pit 500 wide, 35 deep, 1000 long	AO	[illegible]	[illegible]
	Connection channel Narmada	gully 3000*500*5 m	AP	[illegible]	[illegible]
Other structures	railway	80 km double track			
	roads	100 km 20 m wide			
	shiplock (fresh)	220 * 35 m			
	shiplock (tidal)	150 * 25 m			
	towns	20000 people			
Other costs	electro/mechanical				
	steel for caissons				
	electro/mechanical for caissons				



month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
cost	372	372	395	395	408	408	416	393	348	315	28	26	242	243	180	47	47	139	139	136	136

22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72

[redacted]



22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72
132	119	119	125	125	125	125	125	126	133	126	126	126	119	119	152	152	133	133	133	133	154	154	57	57	-	-	123	123	60	62	60	60	60	60	60	-	-	-	46	46	46	46	46	46	46	-	-	-	-	-

Planning of the works: TPF caisson Superbags

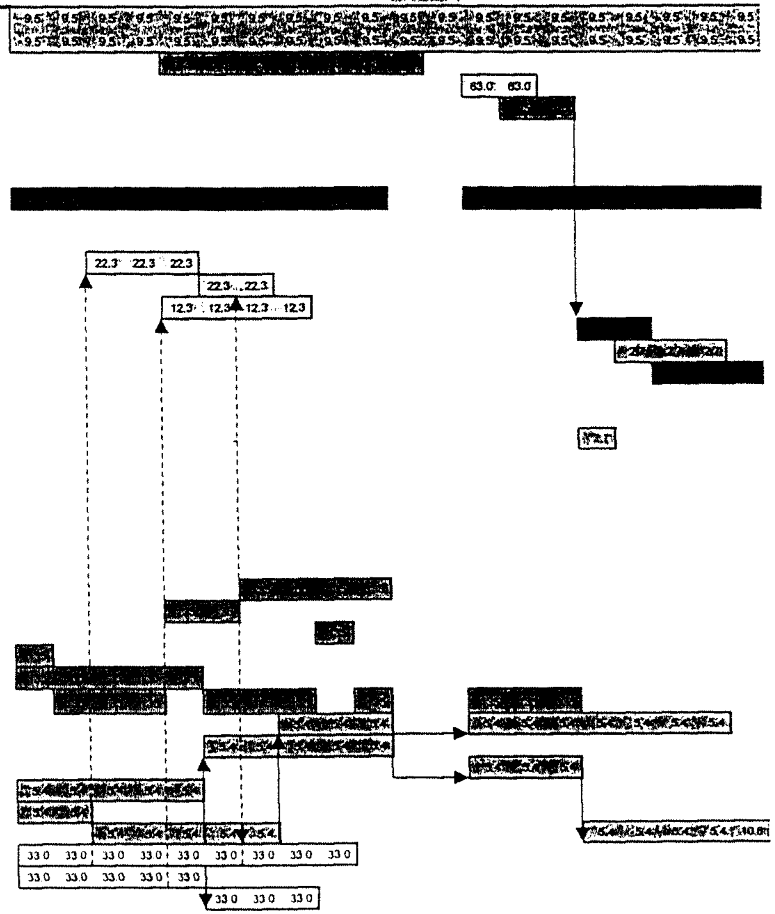
TOTAL in crores F 11,798

Component	subcomponent	item	material	construction method	amount	unit	comment	unit rate in rs	code	cost in crores rupees	building capacity per day	building time (days)	month		cost per month
quarries	east side (Bharuch area)		rock	blasting, sorting	5129256.6	ton	100000 tons per week	221	J K L M	113.36	14286	359	12.0	A	
	west side (Bhavnagar site		rock	blasting, sorting	20517026.4	ton	100000 tons per week	221	J K L M	453.43	14286	1,436	47.9	B	
secondary damsections	Ghogha - TPF	top level CD+19													
		closure dam	rock	landborne	4180410	ton	trucks, 500 ton/hour	43	V	17.98	10000	418	13.9	C	
		CD-3,5 (6000 m)	sand	hydraulic fill	10800000	m3	2 jumbos, 4 hour cycle	105	C F	113.40	200000	54	1.8	D	
		Armour layer (6000 m)	rock 1,2m	landborne	452790	ton	crane 250 ton/hour, finishing dambody	86	VV	3.89	6000	75	2.5	E	
	TPF - Final gap	top level CD+19													
		closure dam	rock	water- and landborne	3891888	ton	trucks, 500 ton/hour	43	V	16.74	10000	389	13.0	F	
		CD-10 (3000 m)	sand	hydraulic fill	9000000	m3	2 jumbos, 4 hour cycle	105	C F	94.50	200000	45	1.5	G	
		Armour layer (3000 m)	rock 1,2m	landborne	226395	ton	crane 250 ton/hour	86	VV	1.95	6000	38	1.3	I	
	Spillway - Hansot	top level CD+16													
		CD (10500 m)	clay	landborne	14112000	m3	crane, trucks, 1000 ton/hour	90	E	127.01	24000	588	19.6	J	
		CD +10 (20000 m)	clay	landborne	5280000	m3	crane, trucks, 1000 ton/hour	90	E	47.52	24000	220	7.3	K	
		CD +5 (4000 m)	clay	landborne	2816000	m3	crane, trucks, 1000 ton/hour	90	E	25.34	24000	117	3.9	L	
work islands	TPF (Center)	top level CD+16													
		CD -6 (1500*300)	sand	hydraulic fill	9900000	m3	2 jumbos, 4 hour cycle	59.75	3/4 A, 1/4 B, F	59.15	200000	50	2.7	M	
		slope protection seaside			1500	m		60000	AF	9.00	100	15			
		slope protection basinside			1500	m		30000	AG	4.50	100	15			
	TPF (East/shiplock)	top level CD+16													
		CD -6 (1500*300)	sand	hydraulic fill	9900000	m3	2 jumbos, 4 hour cycle	59.75	3/4 A, 1/4 B, F	59.15	200000	50	2.7	N	
		slope protection seaside			1500	m		60000	AF	9.00	100	15			
		slope protection basinside			1500	m		30000	AG	4.50	100	15			
	Construction dock for caissons	top level CD+16													
		dike CD-5 (4000 m)	sand	hydraulic fill	8400000	m3	2 jumbos, 4 hour cycle	59.75	3/4 A, 1/4 B, F	50.19	200000	42	4.1	O	
		slope protection seaside			4000	m		60000	AF	24.00	100	40			
		slope protection basinside			4000	m		30000	AG	12.00	100	40			
Tidal Power Facility		caisson factory				st		20000000		2.00			2.0	P	
		construction of sill	rock	waterborne	1474200	ton	2 conventional stone dumping vessels	90	N	13.27	10000	147	4.9	Q	
	30 turbine caissons	62000 m3	concrete	construction dock	1860000	m3	2 concrete production plants, 60 m3/hour	8000	P	1,488.00	3840	484	16.1	R	
	15 sluice caissons	50000 m3	concrete	construction dock	750000	m3	2 concrete production plants, 60 m3/hour	8000	P	600.00	3840	195	6.5	S	
		placing of caissons		floating	45			50000000	S	225.00	3 per 14 days (only neap tide)		7.0	T	
Spillway	10 sluice/spillway caissons	construction of sill	rock	waterborne	327600	ton	2 conventional stone dumping vessels	90	N	2.95	10000	33	1.1	U	
	same dock as TPF	40000 m3	concrete	construction dock	400000	m3	2 concrete production plants, 60 m3/hour	8000	P	320.00	3840	104	3.5	V	
		placing of caissons		floating	10			5000000	S	50.00	3 per 14 days (only neap tide)	56	1.9	W	
Final gap 10000m	250 geotextile units	tubes (10% aramide)	geotextile	special placing	6250000	m2		200	X	125.00					
		outer (15% aramide)	geotextile	special placing	1175000	m2		500	Y	58.75					
		placing			250			10000000	Z	250.00	4 per day	63	2.1	X	
		filling	sand	hydraulic fill	7200000	m3	jumbo, connected to special unit	77	B	55.44					
	dambody	4000 m3/m	sand	hydraulic fill	40000000	m	2 jumbos, 4 hour cycle	77	A	308.00	200000	260	8.7	Y	
		slope protection seaside			10000	m		60000	AF	60.00					
		slope protection basinside			10000	m		30000	AG	30.00					
Bottom protection					m2										
	mattresses:														
	TPF			mattress layer	1650000	m2	one vessel, one mattress (300*50) per day	3000	O	495.00	15000	110	3.7	AA	
	Inlet sluices			mattress layer	825000	m2	one vessel, one mattress (300*50) per day	3000	O	247.50	15000	55	1.8	AB	
	Spillway			mattress layer	550000	m2	one vessel, one mattress (300*50) per day	3000	O	165.00	15000	37	1.2	AC	
	Work islands			mattress layer	1800000	m2	one vessel, one mattress (300*50) per day	3000	O	540.00	15000	120	4.0	AD	
	Dam in Narmada mouth														
	Secondary damsections			mattress layer	1500000	m2	one vessel, one mattress (300*50) per day	3000	O	450.00	15000	100	3.3	AE	
	Final gap			mattress layer	4000000	m2	one vessel, one mattress (300*50) per day	3000	O	1,200.00	15000	267	8.9	AF	
	covering:														
	TPF	very heavy 1650000 m2, 2m	rock	waterborne	5791500	ton	2 large stone dumping vessels	90	N	52.12	20000	290	9.7	AG	
	Inlet sluices	very heavy 825000m2, 2m	rock	waterborne	2895750	ton	2 large stone dumping vessels	90	N	26.06	20000	145	4.8	AH	
	Spillway	very heavy 550000m2, 2m	rock	waterborne	1930500	ton	2 large stone dumping vessels	90	N	17.37	20000	97	3.2	AI	
	Work islands	heavy 1800000m2, 1m	rock	waterborne	3159000	ton	2 large stone dumping vessels	90	N	28.43	20000	158	5.3	AJ	
	Dam in Narmada mouth	none													
	Secondary damsections	loose 1500000m2, 0,5m	rock	waterborne	1316250	ton	2 large stone dumping vessels	90	N	11.85	20000	66	2.2	AK	
Dredging	Behind TPF	pit 500 wide, 35 deep, 3000 long	sand	waterborne	54000000	m3	2 jumbos, cycle 4 hours	55	A D F	297.00	200000	270	9.0	AM	
	Behind sluices	pit 500 wide, 35 deep, 1500 long	sand	waterborne	27000000	m3	2 jumbos, cycle 4 hours	55	A D F	148.50	200000	135	4.5	AN	
	Behind spillway	pit 500 wide, 35 deep, 1000 long	sand/mud	waterborne	18000000	m3	2 jumbos, cycle 4 hours	55	A D F	99.00	200000	90	3.0	AO	
	Connection channel Narmada	gully 3000*500*5 m	mud	waterborne	7500000	m3	cutter	101	H	75.75	70000	107	3.6	AP	
Other structures	railway	80 km double track			80	km		2.00E+08	AC	1,600.00					
	roads	100 km 20 m wide			100	km		17280000	AB	172.80					
	shiplock (fresh)	220 * 35 m	concrete	construction pit	234930	m3	assumption	8000	P Q	187.94					
	shiplock (tidal)	150 * 25 m	concrete	construction pit	99285	m3	assumption	8000	P Q	79.43					
	towns	20000 people			1	st		400000000	AD	40.00					
	diversion dams	40 km, 20 m high, 10 m crest, 1:3	sand	hydraulic fill	56000000	m3		77		431.20					
	impermeable damcover	slope protection basinside			40000	m3		30000	AG	120.00					
Other costs	electro/mechanical				6000	ton		60000		36.00					
	steel for caissons				60000	ton		60000	R	360.00					
	electro/mechanical for caissons				55	st		15000000	AE	82.50					
									total rs	11,798.47					

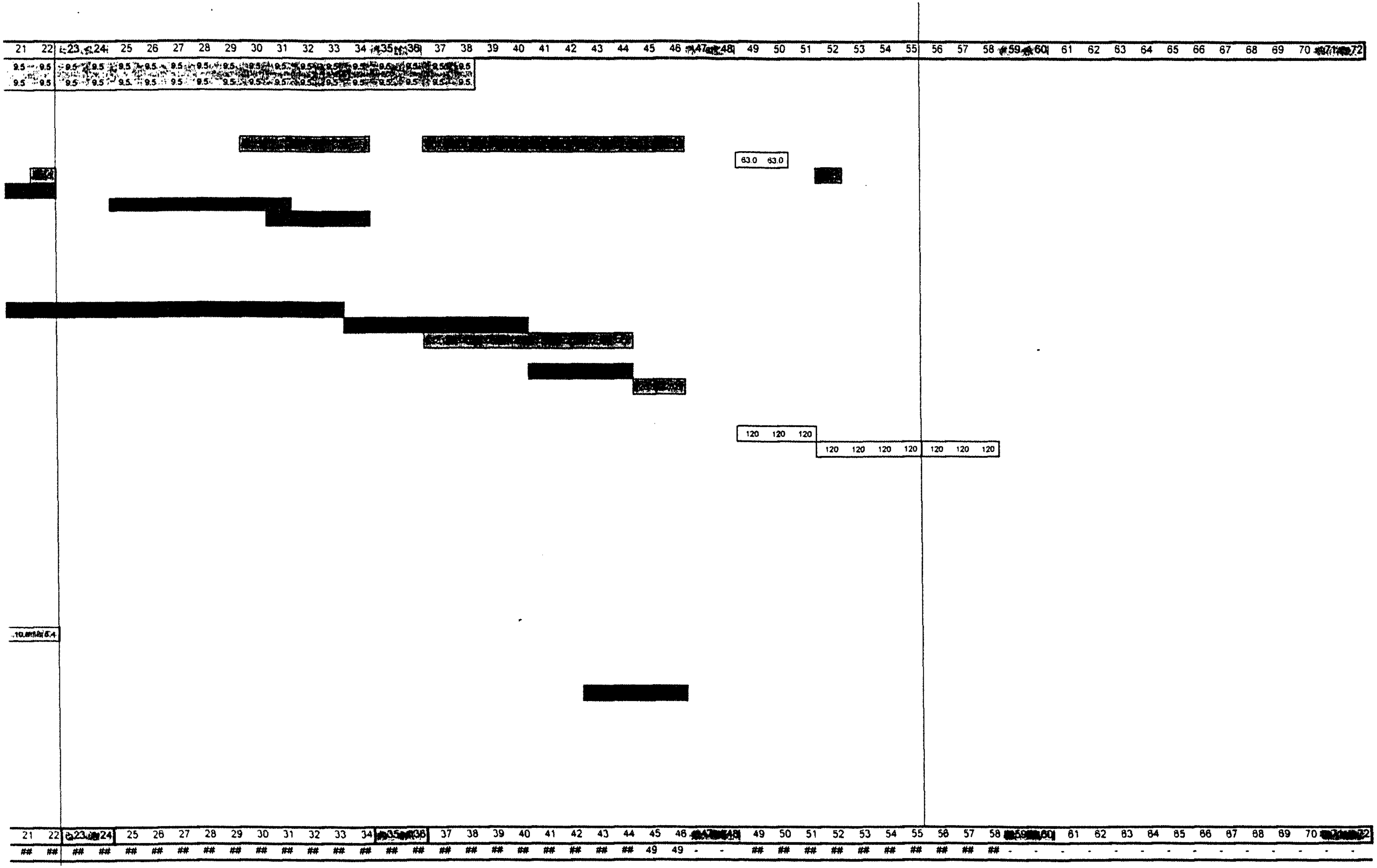
Planning of the works: TPF caisson Superbags

Component	subcomponent	item		time in months	cost per month
quarries	east side (Bharuch area)		A	28.9	9.5
	west side (Bhavnagar site		B	15.4	9.5
secondary damsections	Ghogha - TPF	closure dam	C	13.9	4.3
		CD-3.5 (6000 m)	D	[illegible]	63.0
		Armour layer (6000 m)	E	[illegible]	1.5
	TPF - Final gap	closure dam	F	13.0	4.3
		CD-10 (3000 m)	G	[illegible]	63.0
		Armour layer (3000 m)	I	[illegible]	1.5
	Spillway - Hansot	CD (10500 m)	J	19.6	6.5
		CD +10 (20000 m)	K	[illegible]	6.5
		CD +5 (4000 m)	L	[illegible]	6.5
work islands	TPF (Center)	CD -6 (1500*300)	M	[illegible]	22.3
	TPF (East/shiplock)	CD -6 (1500*300)	N	2.7	22.3
	Construction dock for caissons	dike CD-5 (4000 m)	O	[illegible]	12.3
Tidal Power Facility		caisson factory	P	2.0	1.0
		construction of sill	Q	[illegible]	27
	30 turbine caissons	62000 m3	R	[illegible]	[illegible]
	15 sluice caissons	50000 m3	S	[illegible]	[illegible]
		placing of caissons	T	[illegible]	[illegible]
Spillway	10 sluicespillway caissons	construction of sill	U	[illegible]	[illegible]
	same dock as TPF	40000 m3	V	[illegible]	[illegible]
		placing of caissons	W	[illegible]	[illegible]
Final gap 10000m	250 geotextile units	tubes (10% aramide)			
		outer (15% aramide)			
		placing	X	[illegible]	21.0
	dambody	4000 m3/m	Y	[illegible]	[illegible]
Bottom protection	TPF		AA	[illegible]	[illegible]
	Inlet sluices		AB	[illegible]	135.0
	Spillway		AC	[illegible]	[illegible]
	Work island (TPF center)		AD	[illegible]	[illegible]
	Secondary damsections		AE	[illegible]	[illegible]
	Final gap, overkill		AF	[illegible]	[illegible]
	TPF	very heavy 1650000 m2, 2m	AG	[illegible]	[illegible]
	Inlet sluices	very heavy 825000m2, 2m	AH	[illegible]	[illegible]
	Spillway	very heavy 550000m2, 2m	AI	[illegible]	[illegible]
	Work islands	light 1800000m2, 1m	AJ	[illegible]	[illegible]
	Secondary damsections	loose 1500000m2, 0.5m	AK	[illegible]	[illegible]
	Final gap, overkill	heavy 4000000m2, 1m	AL	[illegible]	[illegible]
Dredging	Behind TPF	pit 500 wide, 35 deep, 3000 long	AM	[illegible]	33.0
	Behind sluices	pit 500 wide, 35 deep, 1500 long	AN	[illegible]	[illegible]
	Behind spillway	pit 500 wide, 35 deep, 1000 long	AO	[illegible]	33.0
	Connection channel Narmada	gully 3000*500*5 m	AP	[illegible]	[illegible]
Other structures	railway	80 km double track			
	roads	100 km 20 m wide			
	shiplock (fresh)	220 * 35 m			
	shiplock (tidal)	150 * 25 m			
	towns	20000 people			
Other costs	electro/mechanical				
	steel for caissons				
	electro/mechanical for caissons				

month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
4 months stockpile																				
4 months stockpile																				



month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
cost geotextiles	##	##	##	##	##	##	##	##	##	##	20	19	##	##	##	40	40	##	##	##


21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72
63.0 63.0
120 120 120
120 120 120 120 120 120 120
21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72
49 49 - - ## ## ## ## ## ## ## ## ## ## - - - - - - - - - - - - - -

The planning shows that the difference between both alternatives is very small. In fact the closing speed of the final gap determines the difference as all other activities can be planned parallel to others that are the same for both alternatives. The construction time is 67 months for the flexible stone closure and 58 months for the geotextile structures. This is without the time that the quarries have to be opened in advance (approximately a year) to produce a considerably amount of large stones required for bottom protecting works.

14.4 Costs

The costs are so-called *direct costs*. These are costs of material, installations, infrastructure and equipment that are directly related to one part of the project. In fact this is mostly a multiplication of the unit price.
Besides these, *indirect costs* will have to be made: costs that concern the project as a whole such as start-up costs.

Additional costs are not included in any of the specifications, such as costs for land purchase, damages and compensation, engineering and survey.

Then, a percentage is added for miscellaneous, contingencies (costs that are not specified but are expected by experience) and unforeseen.

In the diagram (figure 14.2) these types of costs are schematized. In figure 14.3 (total costs) the percentages for each type of costs are given. Although VAT is named in the diagram, it has not been taken into account in the cost estimate.

The results of the direct cost calculations increased with the all additional percentage described are shown in the following table.
The costs of the two alternatives from this study are compared to the caisson closure from the study by Haskoning. In that study the tidal power facility is designed separately and in the cost calculation the construction dock is taken into account, but the concrete structure and the powerhouse installations are not. This is the same for the Narmada Spillway. In this study for both the tidal power facility and the Spillway the concrete structure is incorporated, but the powerhouse installations are not. These installations are quite comparable in costs.

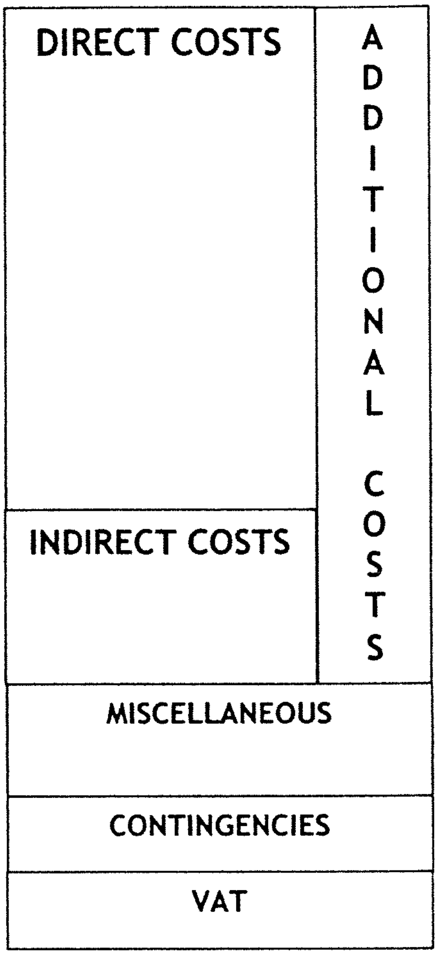


Figure 14.4 Types of costs

The concrete structure has two functions, as a temporary orifice during closure and later as a tidal power facility (TPF). The reconstruction into a powerhouse is expensive, but still expected to be considerably cheaper than the construction of a separate tidal power facility.
Thus it can be concluded that the difference in costs of the two alternatives in this study and the caisson closure will increase even more.

Costs are given in crores Indian rupees. One crore is ten million (10.000.000).

Cost Estimate			Caisson closure TPF in construction dock 2 * 5000m caissons	BW TPF caisson Superbags	BW TPF caisson stone closure
Direct costs in crores (10e7) rupees			13.407,85	11.798,47	13.055,23
	additional costs	railway	1.600,00	included	included
		roads	172,80	included	included
		shiplock (fresh)	187,94	included	included
		shiplock (tidal)	79,43	included	included
		towns	40,00	included	included
		diversion dams 40 km	551,20	included	included
			15.488,02	11.798,47	13.055,23
Indirect costs			%	%	%
	insurance, taxes, guarantees		2	2	2
	interest during construction		1	1	1
	mobilization, demobilization, investments		18	18	18
	office charges		2	2	2
	profit		8	8	8
	risk		4	4	4
	total indirect cost		35	35	35
Miscellaneous and unforseen					
	miscellaneous		1	5	5
	unforseen				
		earth works	2	2	2
		stones	2	1	3
		bottom protection	9	9	9
		construction works	9	9	9
		others	2	3	2
	total miscellaneous and unforseen		25	29	30
Contingencies			5	5	5
		total	65	69	70
Total cost in crores rupees			25.555,24	19.939,41	22.193,90
Total cost in NLG			F 12.777.620.468	F 9.969.704.553	F 11.096.949.688

Figure 14.5 Total costs of three alternatives

14.5 Conclusions

It can be concluded that the alternative where Superbags close the final gap is by far the cheapest. This is only partly (5%) because of the cheaper bottom protection (sand mattresses), but mostly because the used material is cheaper itself. The closure with Superbags is also expected to be considerably faster (58 to 67 months).

15 Risk analysis

15.1 Introduction

In order to make a good comparison between the two alternatives, it is necessary to look not only at the cost and the construction time and the advantages during construction or operation. But also to analyze the risks involved with the design, during all phases.
Therefore a risk analysis has been made.

Most of the calculations (and certainly all hydraulic calculations) that are made in this study use the storage area approach that is described in chapter 8. Although this method is accepted to be accurate enough in these circumstances, this can hardly said of the boundaries. Many data are obtained from the Admiralty Chart, which shows a complicated gully system. This gully system is a priori neglected by a storage area approach. The current velocities that are used in the overkill closure are also directly derived from this model.
So, to make a risk analysis, it is necessary to start with a study to the sensitivities of the calculations that have been used.

Of the *whole closure dam* with all the components a fault tree is made (§15.3). The purpose of this is to discover the mechanisms that can lead to failure. These failure mechanisms are discussed. No chances of failure are give, as it is too early to make satisfying calculations. Also methods of avoiding risks and possible solutions to solve problems are given. When this leads to a difference between alternatives, the best option is discussed.

Finally, the difference in planning and the sensitivity to delays between the alternatives is discussed. Is a simple adaptation or redesign possible, does it have impact on the **planning** and what are the costs of such? In these parts the reader is referred to the planning sheets that are give in chapter 14.

Finally a conclusion and final choice is made.

15.2 Sensitivity of the calculations

As a storage area approach is a quite rough approach, it should be discussed whether the results of such an approach are reliable. This paragraph will give an indication about this. Calculation examples are made for a gap with a width of 10000 meters, and an extra gap of 54000 m^2 below CD and an extra gap above CD with a width of 4400 meters. The used timestep is 300 seconds.

The following parameters have more or less influence at the results:

15.2.1 The influence of the tidal wave

The tidal wave is used with its main components (M2, S2, K1, O1). This is done because a single sine would result in the absence of the two-monthly oscillation of the tidal wave. Calculations with one tidal component, either the maximum resulting in (too) high values, or an average resulting in too low values, will not give satisfying results. Figure 15.1 shows the difference in tide between a single sine tidal wave and the wave with the four main components. The upper sine has an amplitude of 5 m, the lower sine has an amplitude of 3,5 m and the central sine is contains four components (maximum amplitude = 4,9 m). The timestep is 300 seconds. It is clear that the tidal range of 10 meters results in too high current velocities, while using only the average amplitude results in too low maximum values.

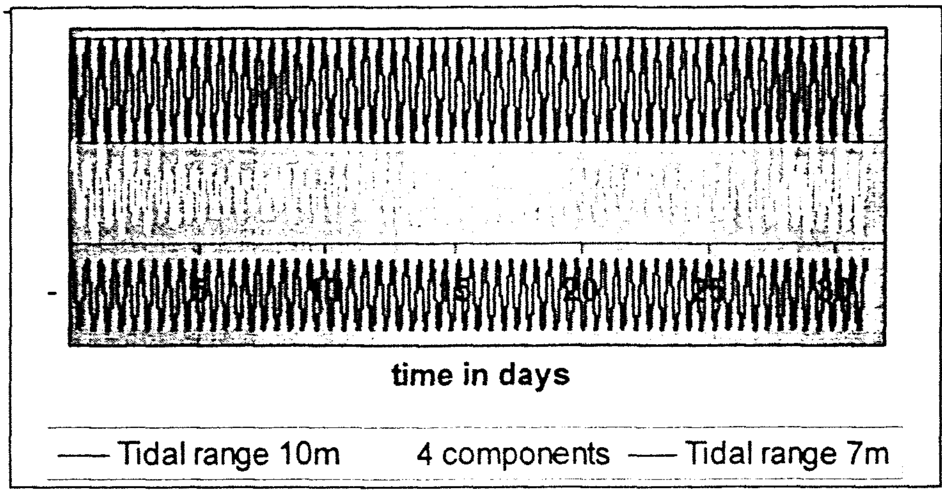


Figure 15.1 Difference between single sine and a tidal wave with four components

Second problem with the tide is the irregularity across the Gulf. The tidal range at Bhavnagar is almost one meter larger than the tidal range at Dahej. Near Khambhat (north in the Gulf) the tidal ranges are even bigger than at Bhavnagar. These irregularities are caused by the earth rotation (Coriolis Force) and by the shape of the basin. The model uses the values at Bhavnagar to calculate the water level at sea. So the assumption of the Bhavnagar values for the entire dam alignment is a safe one.

Not all the tidal components are taken into account. More components might heighten the tidal range a bit once in a while. An increase of the tidal range will also lead to an increase on the velocities. Before the final design is made a thorough tidal study is required. The effects of the other tidal components are expected not to influence the chosen techniques, but they might have influence on the planning. (For instance the canceling of a caisson placing operation.)

Resuming can be stated that the tidal wave as used in this study is a fairly safe assumption.

15.2.2 The geometry of the different gaps

The shape of the orifices is determined as the width of the gap multiplied by the water level in the gap. Since this water level is not known, an assumption is done in the model about the value of this water level (see figure 15.2). This assumption can result in a too low orifice and thus too high velocities.

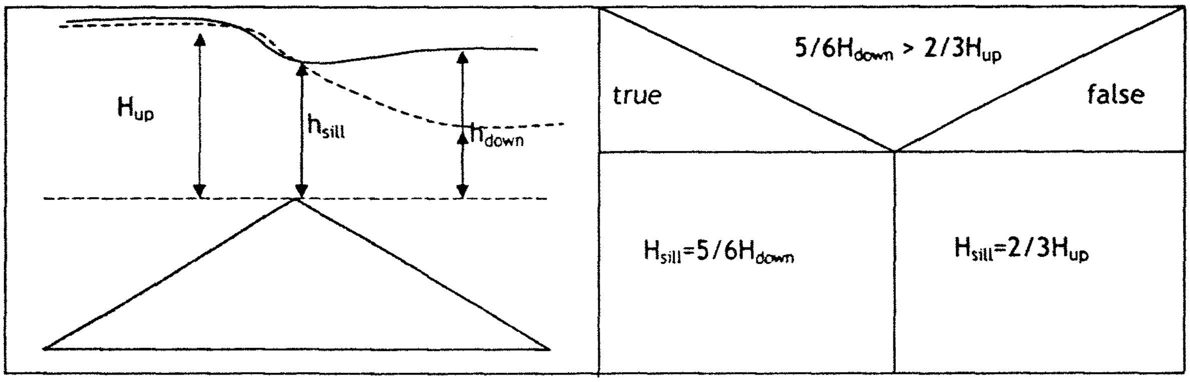


Figure 15.2 Schematic cross section of dam body

The other assumption is about the μ-factor (submerged weir) and the m-factor (free-flow): both are set at one. This assumption will not be true for very small gaps. In small gaps the real velocities will be higher than calculated. Assuming a certain percentage of turbulence as done in chapter 11 will compensate a part of the influence of these factors.

15.2.3 The shape of the basin (planimetric figure)

The planimetric relation is determined from the Kalpasar study (see figure 15.3). Comparing this with the Admiralty Chart shows that the chance that this is a correct assumption is very small. However the line between the two last points (CD, 630 km^2 & CD +15 m, 2187 km^2) has the direction that can be expected, but the real line is likely to be a sphere instead of a straight line. This would result in a bit smaller volume of the basin, which again would result in lower current velocities. So the assumption of a straight line is a safe assumption.

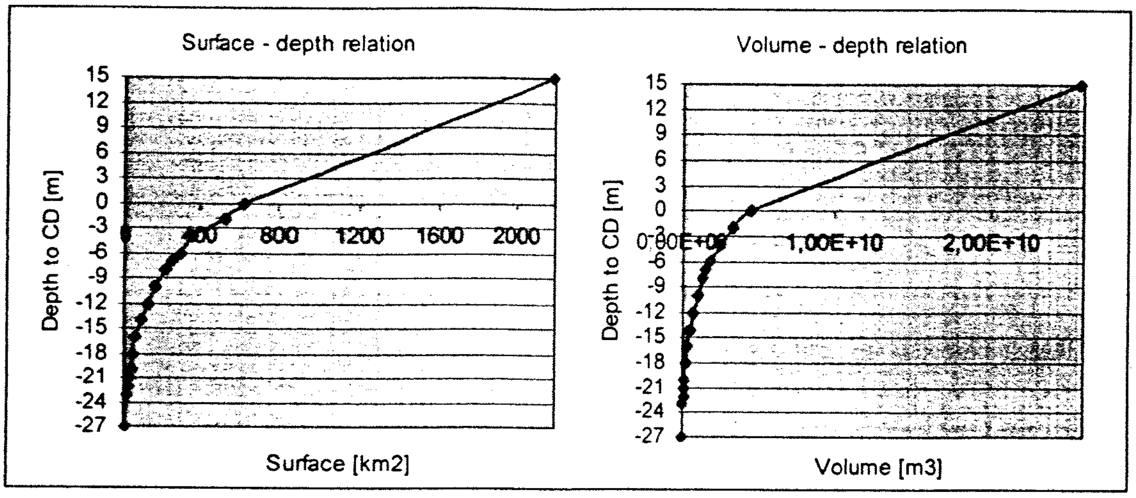


Figure 15.3 Surface and volume relation of the basin

Still the used bathymetric figures itself form the greatest risks. Since the bottom geometry in the Gulf changes yearly, the gully system that is shown by the Admiralty Chart is outdated since it is printed.

15.2.4 The used calculation schedule

The influence of the chosen calculation schedule (Heun) is very small. By choosing another schedule the results might be slightly better, but this will be an improvement of centimeters per second instead of meters, the influence of the timestep is much larger.

15.2.5 The used timestep

The model requires a certain timestep to calculate all parameters. This timestep should be as small as possible to be as precise as possible. However, the smaller the timestep, the longer the calculations will take. Most calculations are done with a time step of 300 seconds. However, using a timestep of 60 seconds results in a bit smaller values of the velocities (under extreme conditions the velocities can decrease with a few decimeters per second). Larger timesteps result in too much irregularities in the results, and above all: the bigger the timestep the higher the velocities. The used timestep (300 seconds) results consequently in a little too high values for the velocity, which contributes to the safety of the design. Figure 15.4 shows the influence of the different timesteps on the calculated maximum velocities. The maximum differences occur at low sill levels, at CD -15 m the maximum occurring velocities are:

T = 60 s : 3,12 m/s, T = 300 s : 3,40 m/s and for T = 600 s : 3,83 m/s.

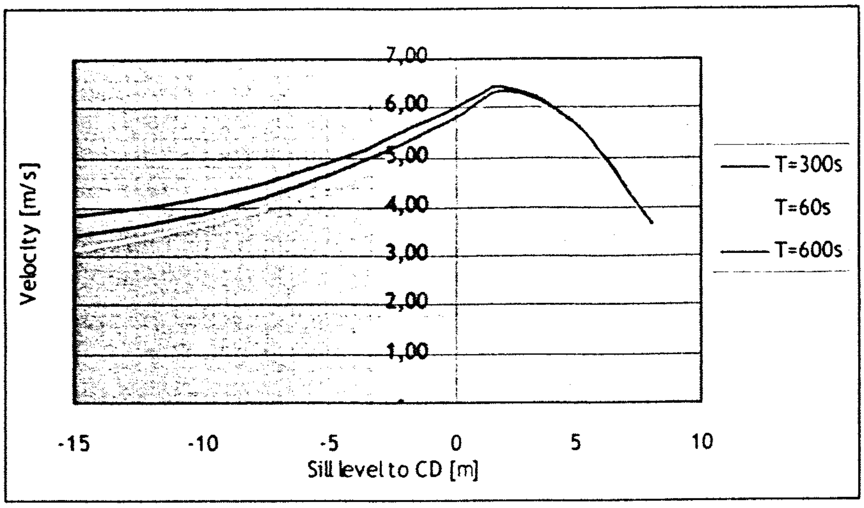


Figure 15.4 Influence of different timesteps on the maximum velocity

15.2.6 The size of the gaps

There is one main problem with the proposed storage area approach. The size of the 'entrance gap' of the basin is at the beginning of the closure works actually to big for the storage area approach. Phenomena like the tidal differences and geometric properties of the basin, especially the gully system, give the calculated velocities for very large gaps only an indicative status. Because these values are relatively small this is not such a problem.
The main design problems are the velocities and head differences over the final gap, these values are calculated accurate enough for this study.

15.2.7 General conclusions about the validity of the model

A stated above, some remarks could be made to the storage area approach. They are summarized in the following table, in which also is indicated what influence each parameter has on the main parameters velocity and head difference. '+' Marks a positive influence = predicted values are too high, a '-' marks a negative influence in the model =predicted values might be too low, too low factors are a danger to the chosen stone diameters.

	Velocity	Head difference
The tidal wave	-	-
The water level above the sill	+	+
The absence of μ- and m-factor	-	-
The shape of the basin (planimetric figure)	+	+
The timestep used to calculate everything	+	+
The used calculation schedule	+/-	+/-
The size of the gaps	+/-	+/-

Besides the influence of the velocity, it is also good to know how often a certain velocity occurs during a month. Figure 15.5 shows which velocity occurs how often during a month. Horizontally is plotted the percentage of time in a month at which the corresponding velocity is lower than the specific value. From this picture can be seen that the higher the sill in the gap the higher the velocities, it is also visible that at high sill levels the velocity is longer at high values, as can be expected.

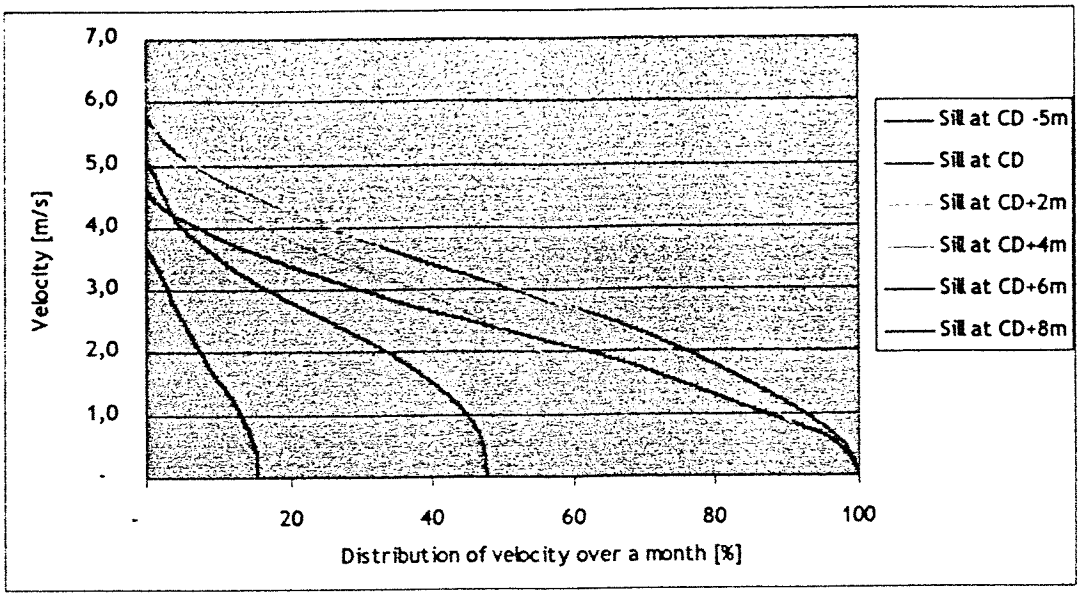


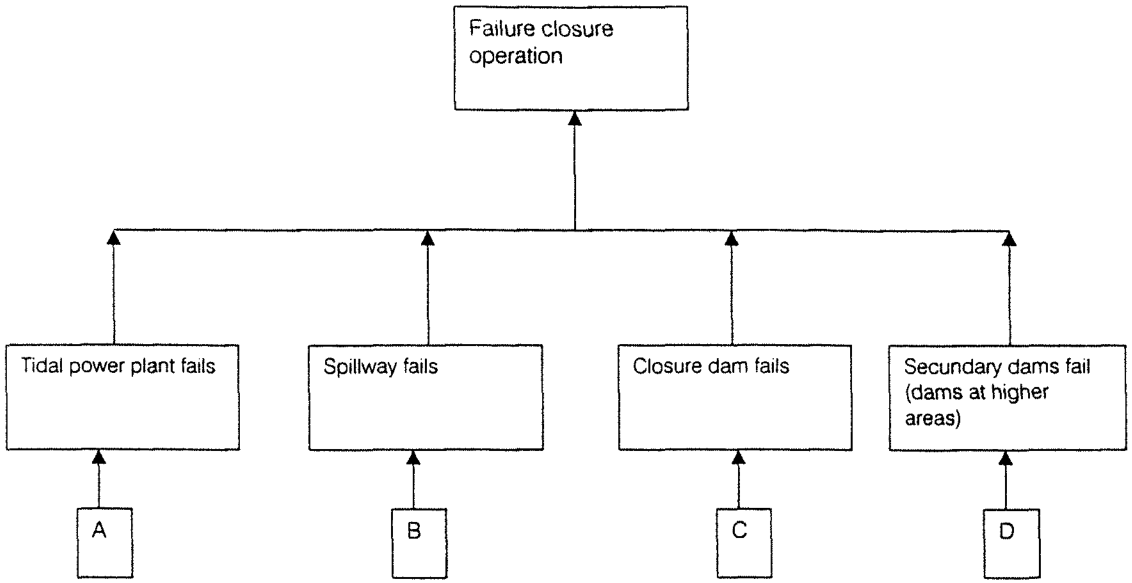
Figure 15.5 Velocity distribution over a month

Summarizing the influences of all the parameters in the storage area approach, there are two major influences, the timestep and the absence of the factors μ and m. Since the gradual horizontal closure is not a option, the absence of the μ- and m-factors is assumed not to be a very big problem. Assuming that the closure will be vertically the following conclusion can be drawn from the model: the model is conservative and results in a little too high velocities. For the placing of the Superbags the μ- and m-factors are compensated.

15.3 Fault trees of closure

Next step in the risk analyze is to predict what might go wrong during the construction of the Kalpasar dam. Therefore a fault tree is drawn for the whole closure dam. This fault tree is presented in the following A3 pages.

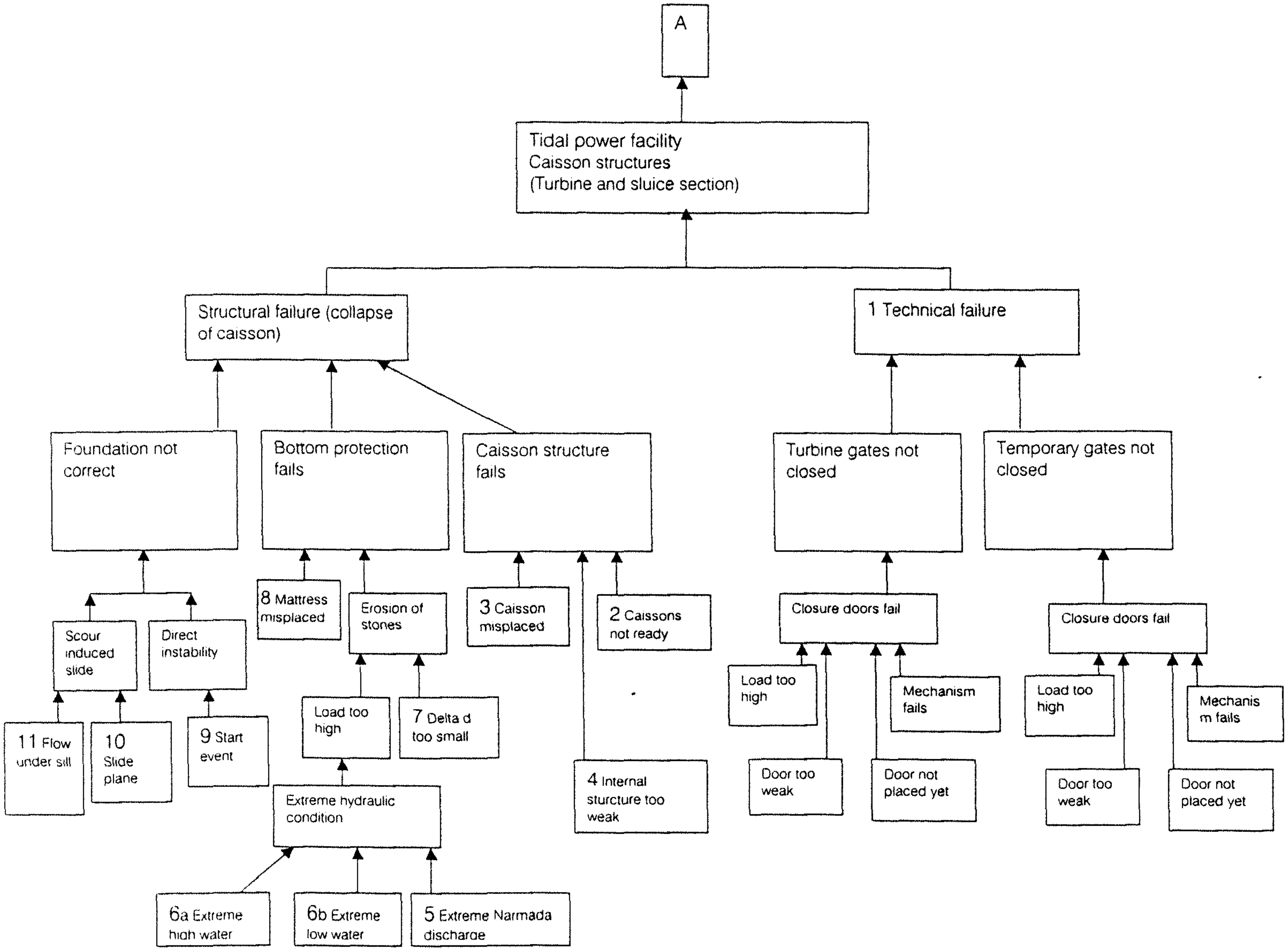
The events are numbered in the schedule and explained in the column right on each page.



Fault tree of closure dam

In this parts the fault tree of the closure process is described. On the left side of these pages the failure mechanisms are drawn schematically. On the right side the comments to the events are given, either with a direct explanation or solution, either with a reference to a paragraph elsewhere in this study.

As the Tidal Power Facility, the sluices and the Narmada spillway all have a caisson construction method in common, and thus their fault trees are alike, they are discussed together.



Remarks

1. Technical failure
   Right branch has not been investigated. This is a mechanical design problem. For the total of 55 caissons, 440 openings have to be closed. Letting the doors fail should close the openings.

2. Caissons not ready
   The caissons are built in a dock with several compartments. Placing of caissons from one dock can only start when all caissons in that dock are finished. The others are still being built. A delay of placing will delay the dam construction. As the closure of some of the secondary damsections can only start when the caissons are placed, this will not cause failure of the dam, but a delay of constructing.

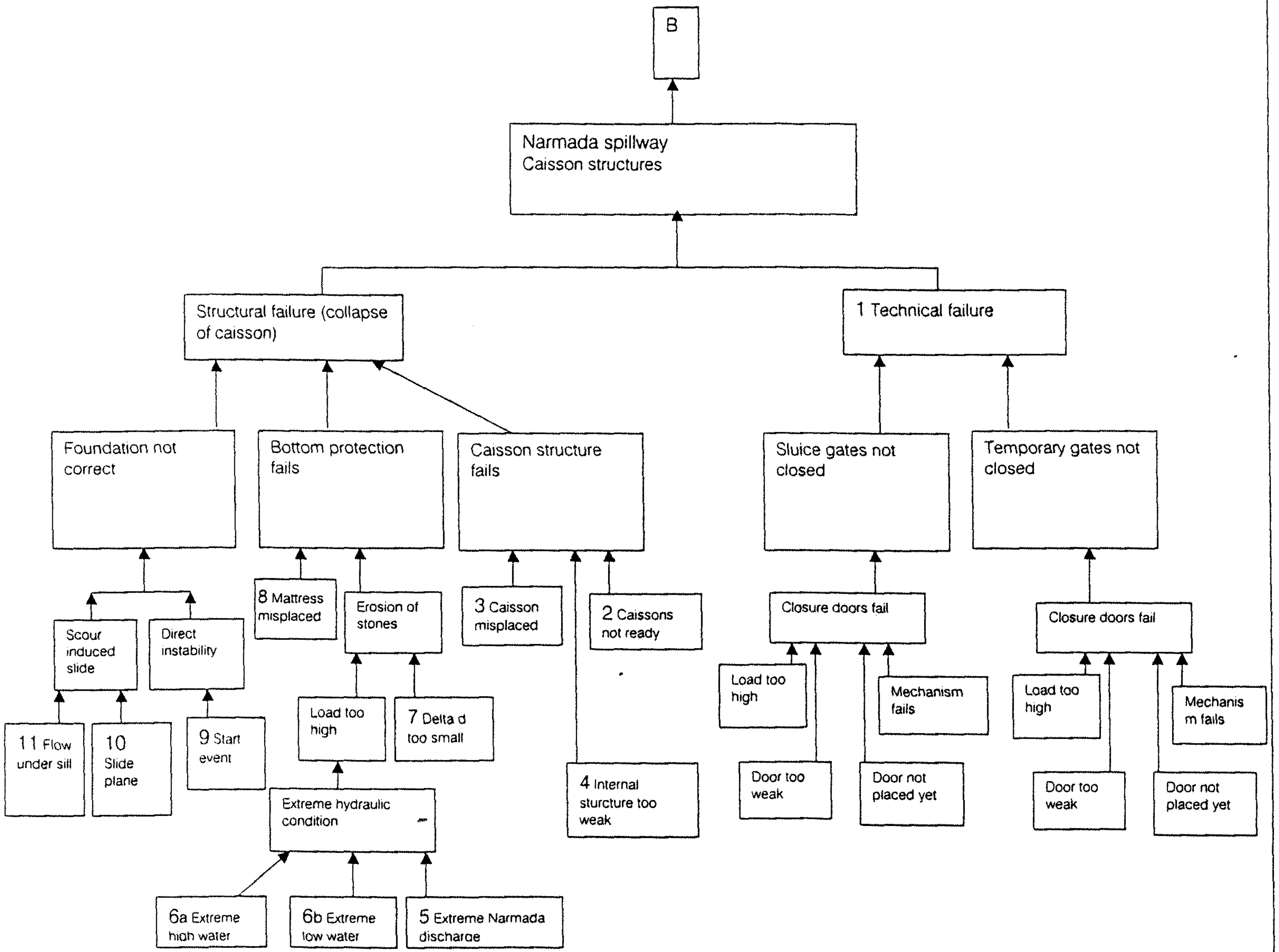
3. Caisson misplaced
   When one of the 55 caissons is not placed correctly, it should be re-floated, parked somewhere and re-placed when the sill has been corrected. If during placing the current becomes too strong, the caisson should be sailed away to a sheltered area. Note: placing is only done during neap tides, around HW slack. This reduces placing risk.

4. Internal structure too weak
   The internal structure of a caisson, especially the roof should be strong enough to construct a double track railway (and eventually also a 4-lane road).

5. Extreme Narmada discharge
   The effects of an extreme Narmada discharge are simulated in the paragraph 15.4.1.

6. Extreme high or low water
   The effects of extreme tides on the current velocities and the deviation to the calculations are discussed in the paragraph 'Sensitivity to calculations'. The effects of wind setup are neglected.

7. Delta d too small
   Stone diameters are calculated using Shields' formulas. Used velocities are conservative. During construction the velocities are higher than during operation. Temporary extra safety is ensured by steel netting over the stones.

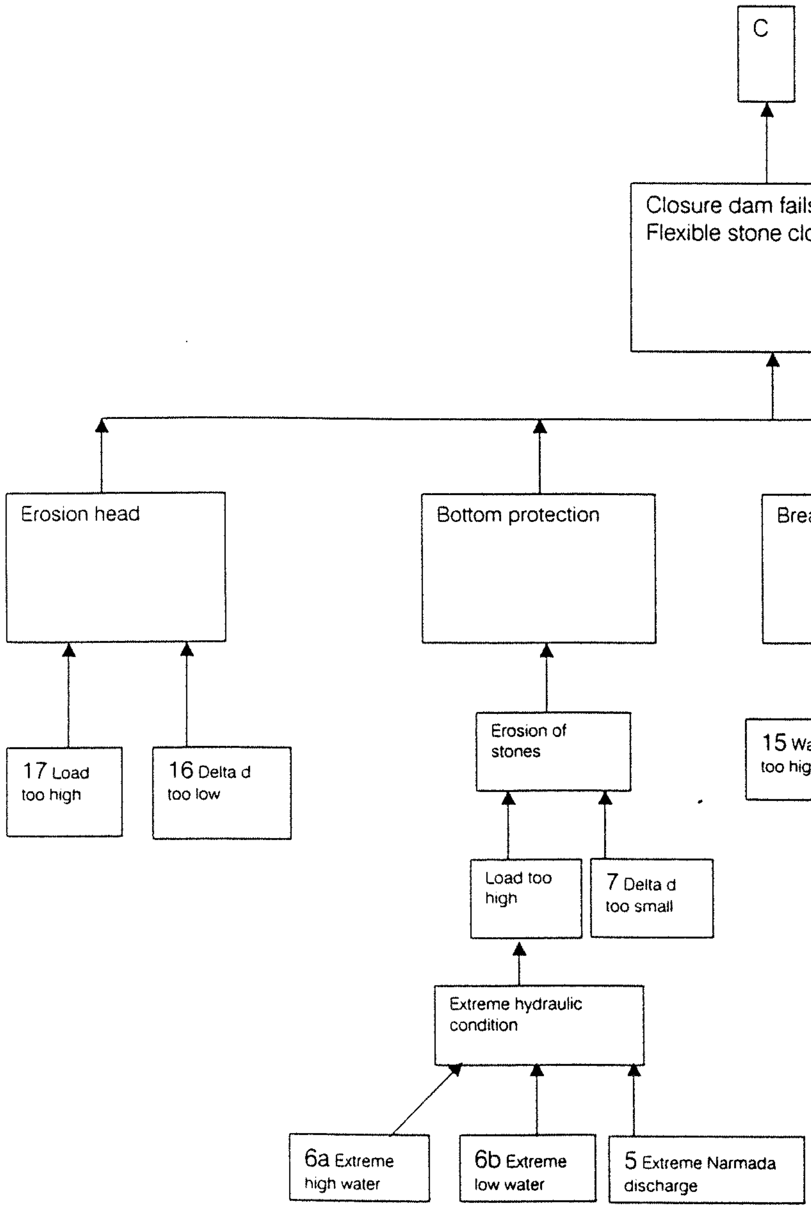


8. Mattress misplaced
   960 mattresses have to be placed. It is inevitable that mattresses will be misplaced. However, this only causes serious problems for the mattresses *under* constructions. These are only 20 to 30 mattresses. Only these have to be removed and replaced when something goes wrong.

9. Start event
   A possible start event is an earthquake. These have been reported rarely. A tropical cyclone caused great damage in 1998. This was however during monsoon season.

10. Subsoil too weak
    It should be investigated whether the subsoil is able to resist these loads.

11. Flow under sill
    Flow under the sill is not so much of a problem as a geometric closed stone filter is constructed under the sill.



12. Production too low
When using the flexible stone closure the capacity of material supply is very important. This is described in paragraph 15.4.3.

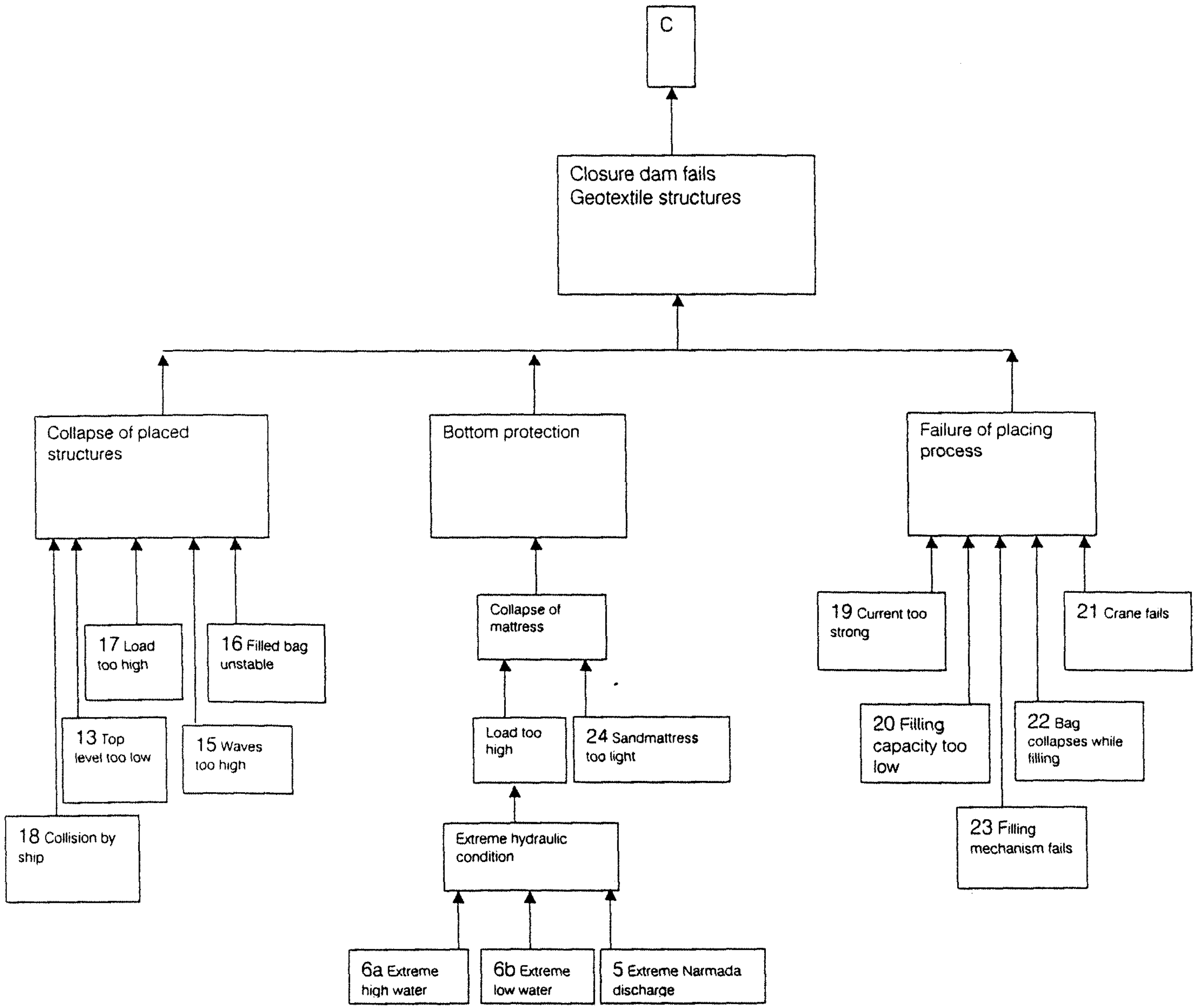
13. Top level too low
The closure dam is designed to CD + 12 m. This is two meters above the level of Spring tide, which seems reasonable. However, locally the dam can be lower due to incorrect construction. This should be avoided by measuring the level of the top of the dam.

14. Delta d too low
The effect of a local breach is simulated in Part C, paragraph 12.9

15. Waves too high
During the season in which the final closure dam is constructed (between two monsoon seasons) waves are much lower than the significant wave height. However, from calculations it is concluded that for the final dam profile a stronger revetment is required. Occurrence of such higher waves during construction can have a devastating result on the highest levels (the lowest are below the level where orbital movement is significant).
Damage however can easily be repaired by dumping extra stones.

16. Delta d too low
The stone diameters have been calculated with the computer model. Its results are a little conservative. The chance of a stone diameter that is too small is therefore expected to be very small.

17. Load too high
The load is based on tidal variations only. Wind setup and higher waves (swell) can cause greater head differences. This has not been calculated.



18. Collision by ship
Contrary to the overkill closure, ships are sailing in the vicinity of the dam. The consequences of a collision between for example a dredging vessel and a dam that consists of (unprotected) filled bags have to be investigated. This can be a major problem, as a partly damaged dam cannot easily be repaired.

19. Current too strong
In all calculations the maximum current velocity the bags can resist is set at 6,5 m/s. This critical velocity should be determined by *model test on a large scale*. Also the strength of the bottom protection is set at that velocity.

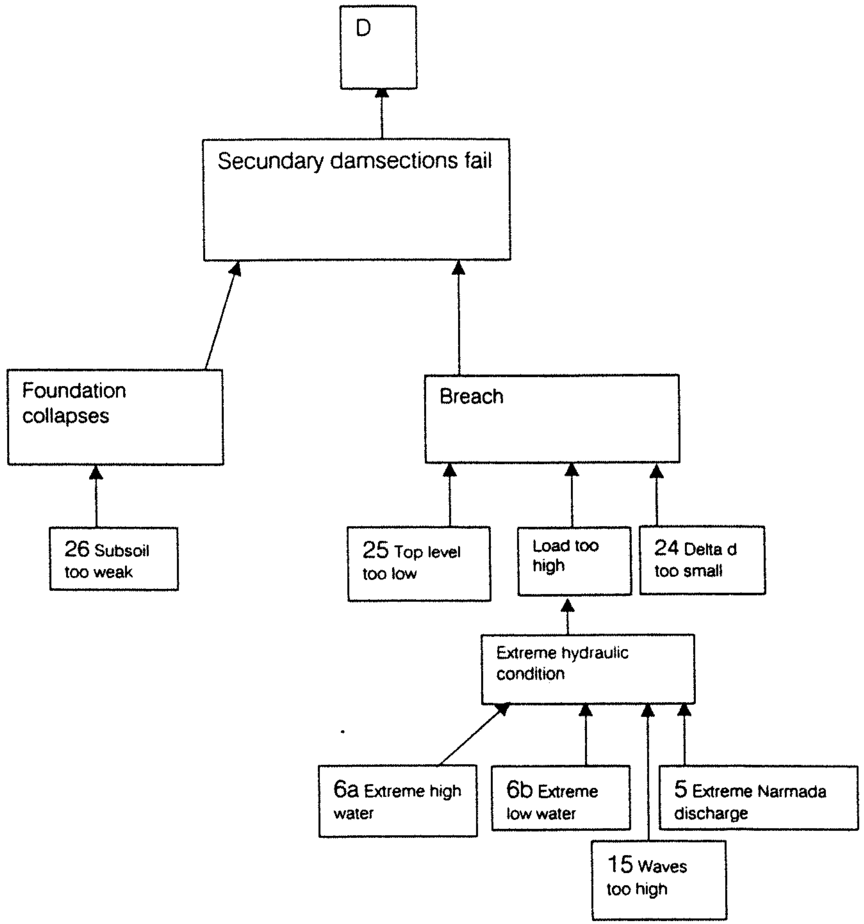
20. Filling capacity too low
The volume of the bag (18000 m^3) has to be filled in a very short period. This means that the capacity of all pumping material should be enough to guarantee this production, including a safety margin.

21. Crane fails
The bag itself is made of geotextile, with a relatively small weight at the bottom to ensure controlled sinking, is not very heavy. However, the filling mechanism that is connected to the crane that places the bag is very heavy. The effects of the crane arm in the water (although placed at slack water, there will still be some currents) have to be investigated.

22. Bag collapses while filling
When the bag is filled, it is possible that due to turbulence in the water the textile encounters problems, such as rupture of the cloth against filling pipes or getting tied up.

23. Filling mechanism fails
The filling mechanism with the pipes is very quite complex. No experience exists with this method. It is of greatest importance that experience is gained. This can for example be done by constructing part of the secondary dams with this method.

24. Sand mattresses too light
Although maximum current velocities are used for calculation of the required weight, chances exist that extreme flow conditions cause the mattress to lift.



25. Top level too low
The secondary damsections are constructed to the level of the final profile directly. This gives it the same safety against overtopping as the final dam.

26. Delta d too low
As the secondary damsections are constructed to last the lifetime of the dam, the stone diameter that is used will be high enough to withstand the forces during construction.

27. Subsoil too weak
The dam in the Narmada mouth will be constructed on clay subsoil. Great care should be taken regarding consolidation of this clay. The load of the dam should not be put on the clay at once, as the subsoil will inevitably collapse. Either drainage or a gradual increase of load will provide a safe construction.

15.4 *Effects of several scenarios on the two alternatives*

In the fault tree some events are named that might have a negative influence on the continuity of the closure operation. These events are discussed in this paragraph.

15.4.1 What happens when the Narmada has an extreme discharge?

The Narmada is neglected in the whole storage area approach. This is an accepted simplification for small rivers. The Narmada is not a small river but 150 km upstream on the Narmada a large dam is under construction (the Sardar Sarovar dam). This dam is designed for irrigation purposes and will catch most of the water flowing through the Narmada. The volume of water that flows through the Narmada is mostly very small (1000 - 2000m^3/s), however during monsoon the discharge can increase dramatically. During the monsoon severe spilling can be expected from the Sardar Sarovar dam, this spilling water is also necessary to fill the Kalpasar Lake every year.

The design value for the Narmada discharge is 67000 m^3 per second (or even more). The high water wave is assumed to have duration of 60 hours. With a special calculation, the influence of the design flood of 67000m^3/s is investigated (see figure 15.7). The calculation is made for a gap of 10000meters wide with several sill levels, an orifice of 54000m^2 under CD and an extra gap above CD of 4400m wide (these orifices are the concrete structures like the tidal power facility and the spillway).

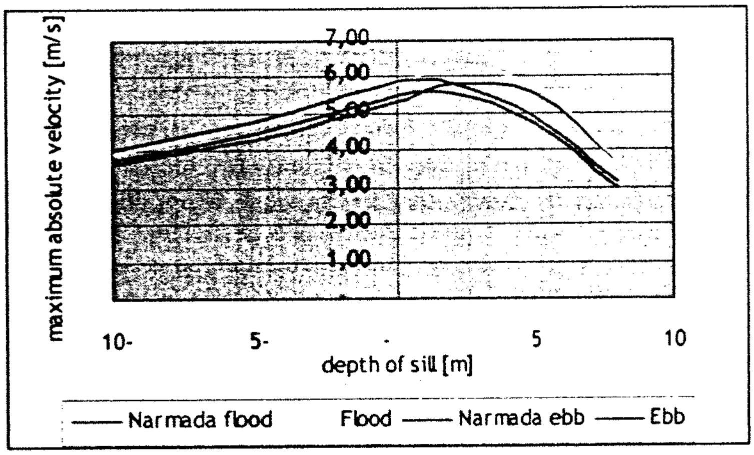


Figure 15.7 Maximum occurring (absolute) velocities during vertical closure

During normal conditions the maximum velocities are flood velocities. During extreme Narmada discharges the maximum current velocities are ebb-velocities for low sill levels and flood velocities for high sill levels. The (absolute) ebb-velocities are slightly higher than the maximum flood-velocities during normal conditions.
The maximum difference is 0,16 m/s. The influence of this increase in velocity can be calculated using the Shields relation (see formula 8.1) for the beginning of motion. The maximum calculated difference occurs at a sill level of CD -5 m, the maximum occurring flood velocity under normal conditions is 4,64 m/s, at extreme Narmada discharges this can become 4,80 m/s.
With a rock-density of 270 kg/m^3 and a Shields parameter of 0,03 the following stones are required:

	Maximum occurring velocity	Stone diameter	Stone weight
Normal flood	4,64 m/s	0,30 m	77 kg
Ebb with extreme Narmada	4,80 m/s	0,35 m	115 kg

However the difference in velocity is not that big, the influence of the Narmada discharge on the required stone diameter can be quite dramatically during monsoon on sill levels below CD.

The following events have to coincide before the dam is in danger:
- Spring tide (every 14 days);
- Peak in Narmada discharge (60 hours);
- Sill level below CD;
- No buffer capacity in Sardar Sarovar Lake.

The chance at a high Narmada discharge outside the monsoon period is very small. During monsoon the chance is one time in hundred years. The chance at these factor coincide is estimated at:

$$P = \frac{Narmada}{Springtide} \times Chance\ at\ Narmada\ \ peak\ = \frac{60}{14 \times 24} \times 0{,}01 = 0{,}002$$

This is not a very big chance, and this chance is still only valid when the sill level is still below CD and when there is no capacity left in the Sardar Sarovar Lake.

Therefore it can be stated that the Narmada will form a minor risk during construction of the dam.

The influence of this increase of current velocity on the geotextile structures is not calculated. However, when a bag is schematized as one unit of 18000 m^3 and a relative density of 1, it is not expected to cause problems. To make a fair comparison, behavior of these structures in high velocities should be thoroughly investigated.

15.4.2 What happens when the monsoon starts earlier or lasts longer?

In the planning of the works the monsoon is taken into account as a two-month period. During this period rainstorms occur frequently, wind velocities are generally higher and, most important of all, waves are much higher. The significant wave height is 3 m. Above all, tropical cyclones occur, although not frequently.
These waves make waterborne operations like placing of caissons or mattresses impossible. Even less complicated operations like dredging can not take place during this period.

In the planning effort has been made to schedule activities just in between two monsoon periods. To do so, sometimes the capacities have been adjusted.
There is a possibility that the monsoon starts earlier. In this situation it depends which activity is carried out. Most crucial is the placing of the caissons of the Narmada Spillway. These are scheduled in the last two months before the monsoon. In this period the chances of waves and heavy rain (that cause high Narmada discharges) increase. Stopping of caisson placing and then restarting after the monsoon is over causes problems in the next season. This differs for both final gap closures.
The rock closure requires 10 months for the final closure. In the remaining month it is not possible to place these last Spillway caissons and construct a railway on top for the temporary bridge.
The options that remain are:
- Speed up construction of caissons. This is possible, and the result is that the caissons will be available earlier. This requires higher concrete capacities.
- Speed up placing. This is not recommended as this implicates acceptance of higher risk!
- Construct more compartments in the constructing dock. This makes it possible to sail out caissons while others are still being built.
- Construct the spillway in a building pit. This is an alternative design that has not been considered in this study. It requires an enormous building pit, but will make construction independent of the other planning aspects.

The geotextile closure requires about 7 months, which leaves 3 workable months in the following season in which the last caissons can be placed.

When the monsoon lasts longer a serious problem arises in the schedule of the bottom protection. The covering of the mattresses scheduled just within two seasons. When the monsoon period in between lasts for example three instead of two months, this delay works through and delays the finishing of all bottom protection works by three (!) months. This is unwanted, because bottom protection is more and more necessary as the secondary damsections slowly reduce the flow area. The item that is constructed is the bottom protection of the final gap. Building speed can only be increased to a limited extend, as all stone dumping vessels are working on the final gap (months 20 and 21 have a doubled capacity).
This problem is the same for both alternatives. Using more dumping vessels in the second season when the first monsoon lasts longer than normal can solve it.
As stone dumping vessels are relatively cheap, the delay of only the covering of the bottom protection is not so much of a problem in terms of costs.

It can be concluded that the overkill closure is most sensitive to monsoon problems.

15.4.3 What happens when capacities are lower than expected?

To finish this large project in such a short time the production capacities have to be extremely high, especially the quarry capacities. For the overkill closure the total amount of stones is 56 million tons against 23 million for the geotextile structures. It is impossible to produce all these stones in advance and store them in a stockpile. However, to produce the heavy stones that are needed for the bottom protection it will be inevitable to stockpile at least part of the stones, say to work several months 'in advance'.

The overkill closure is a method where with a very high capacity and an acceptance of loss a dam is constructed. This process will take about one year and requires 24 million tons of stones in a variety of sizes. Neither stockpiling of this amount nor producing directly from the quarry (and thus being totally dependent on that capacity) is an attractive option. Stagnation of construction during closure should be avoided 'to all costs', as the method itself implicates that a certain part of the stones erodes.
A combination of high quality managing of the quarries and a limited stockpile of 1 or 2 months should be near the optimal solution for this problem. One stockpile is projected on Alia Bet, it should be investigated whether another (on the Saurashtra side) should be situated near the quarries or near Ghogha.

The closure using geotextile structures depends on two capacities: the production of hydraulic sandfill and the supply of geotextile bags. The bags can be produced in advance. The capacity of hydraulic sandfill is not expected to cause problems as sand can easily be obtained and dredging vessels will be available. The placing speed is not known yet, but estimates made are on the safe side.

Both alternatives have a crucial activity in common: the placing of the mattresses. To place this huge number of over 900 mattresses (300 * 50) in a short period two mattress laying vessels have to be used. Delay of mattress placing causes delay in construction of the bottom protection. Delay in bottom protection causes delay of construction of the secondary damsections. This means a delay of the total project. Mattress placing is a complicated procedure.

It can be concluded that the overkill closure is most sensitive to capacity problems.

15.5 Conclusions

Having discussed the major failure mechanisms, it can be concluded that the main risk is that the capacity during construction becomes too low. Both alternatives depend on the highest building speeds that are expected possible, just to avoid an interruption of the closure process by the monsoon.

Sensitivity analysis of the used calculations and boundaries points out that no great surprises are expected. A more detailed velocity-prediction 2D-computer model will provide more accurate results. It is clear that up-to-date bathymetric data are of the greatest importance. Without these data any model predicts results that are inaccurate for this closure.

When it comes to a comparison between the overkill closure and the geotextile structures, it can only be concluded that the overkill closure seems more affected by capacity and planning problems.
The word 'seems' is chosen because experience and insight in this alternative is only very preliminary. Further study to the advantages and disadvantages of the geotextile structures is absolutely necessary.

15.6 Choice of final closure method

With the all the presented results it can only be concluded that the closure with Superbags is cheaper and faster. However, further investigations are absolutely necessary, as such structures have never been used before. If all these investigations still confirm these results, the closure with the Superbags is recommended to close the Gulf of Khambhat.

Part E Conclusions and recommendations

16 Conclusions

Several conclusions can be drawn from this study. However, many uncertainties are still present.

- Integration of temporary and permanent functions of the tidal power facility into a design where the tidal power facility is used as a temporary orifice during construction is beneficial. The maximum current velocities in the final gap reduce from 8 m/s to 6 m/s (spring).

- This tidal power facility is built with caissons that are transported floating and sunk in position. As this is in an early stadium of closure, current velocities are low.

- For the closure of the final gap two alternatives are developed. The first is a temporary railway bridge from where rock is dumped. For this alternative two options exists. A conventional dumping process and a process where a certain amount of loss is accepted (overkill closure), to use smaller stones. The overkill principle is only applicable in the lower layers (below CD -6 m) and between CD +1 m and CD +5 m. In the lower layers the stones are very small (a slightly larger diameter, without extra cost per ton, reduces loss considerably) and in the upper layers wave attack demands bigger stones. The assumed quarry yield curve can produce both the no-loss and overkill demand. Therefore the overkill closure is rejected. The maximum stone diameter is 1,2 m.

- The principle of overkill closing can however be a solution in other projects where quarries can not produce the desired quarry yield curve, but only large amounts of cheap and low-quality rock.

- The other alternative for closing the final gap is using sand-filled geotextile Superbags. These bags, with dimensions of l * b * h of 40 * 30 * 25 m, work as a set of aramide-reinforced geotextile tubes filled with densely packed sand. This filling and placing is done in one special operation. This method is very fast and cheap. It uses local material (sand). Combined with a bottom protection of sand-filled mattresses a good connection between the original bottom and the dambody is evident.

- The stone closure takes 6 years and costs 22000 crores rupees. The Superbags take 5 years and cost 20000 crores rupees. These building times are without pre-producing of the quarries and all other startup works. It stops when the closure dam is finished and the hydraulic sandfill of the final dambody is constructed. All costs include the roads and railways and concrete works of the tidal power facility, but exclude the reconstruction costs of the tidal power facility, turbine costs and irrigation scheme. (Reconstruction involves closing of temporary gaps, placing of turbines and creating of turbine hall on location temporary gaps.)

17 Recommendations

17.1 Rock

As stated in Part C, the overkill closure seems to be a promising alternative when the quarry can not produce the required output for a no-loss situation and when the rock is very cheap. Since the proposed Khambhat quarries can produce the required stone gradation the overkill technique will not be used. But there are some other uncertainties about the overkill closure that have to be clarified before the overkill technique can be used safely.

The transportation formula for rock

Investigations should be done to derive a formula that describes the transport of large stones on a sill. The only existing formula is the Paintal relation (formula 12.2), this formula contains two parts, one part for a Shields value (ψ) below 0,05, and one for ψ above 0,05. The relation for ψ>0,05 has the same shape as other transport formulas for sediment (sand!), but the other formula (for ψ<0,05) is a highly exponential relation.
Because ψ-values larger than 0,05 implicate that all the stones will be transported, this situation will not occur in the model (the example described in chapter 12 uses a loss of 25%). The used equation, is the exponential relation for ψ<0,05. This relation leads to serious questions. First, the relation is derived for small stones. The validity for large stones should be investigated. The second problem is the high exponential relation. If loss is not accepted, the exponential relation leads to very small values, as can be expected. However, for the overkill closure the acceptance of loss leads sometimes to very high peak-values in the exponential relation. These are values that actually represent the movement of all the stones, corresponding with a ψ-value larger than 0,05. In fact the ψ-value is lower.

Especially the validity of the Paintal relation for high velocities (5 to 10 m/s) should be investigated in model tests.

Head difference or velocity?

There are two determining criteria: the maximum current velocity for erosion of stones (Paintal) and the relation(s) for the head difference. Both relations seem to be valid, but they differ in required stone diameter. Sometimes closures are only based on the head difference criterion. This leads to unfavorable situations in the Khambhat situation (head difference requires smaller stones than velocity).
Second problem (especially with the used relation) for the head difference criterion is the absence of a transition zone in which the proposed 25% loss can occur. In the used relation it is either stable or unstable (=collapse). The Paintal relation offers a certain transition zone, although this is questionable.

Since the validity of the Paintal relation is very questionable it is not possible to determine which relation really determines the required stone diameter.

The distribution of the stones

In the model, only the fall velocity determines the location of the stones. This seems to be a good approach, but the assumption in the model that the stones are dropped in box shaped layers in stead of triangular shaped hills, is questionable. Without loss, the layers will have a triangular shape, but the assumption is made that the flow will flatten the hills when smaller stones are used (thus loss is accepted). This should also be checked during model tests. The triangular shaped dam will require a larger amount of heavy stones than the box shaped dam.

Horizontal or vertical closure
The model, used to investigate the overkill closure, uses a vertical closure to predict the effect on the stone diameters. Because the lower capacity seemed to form a problem, a horizontal overkill closure is not investigated. But using a dumping bridge could make it also possible to close horizontally.

17.2 Superbags/ Sand mattresses

The proposed geotextile structures (Superbag and sand mattress) seem to be very promising, however no experience with the proposed alternatives exists. Especially the scale of the structures is far beyond any experience. Besides that the following point should be clarified by model tests.

Stability
For the design of the Superbags it is crucial to be sure that they will not collapse after filling. This seems to be no problem for the Superbags in the dam body, but the stability at the corners of the dam body is more questionable. It might be necessary to place a special steel covering or frame around the corner bags.
For the sánd mattresses it is crucial to know if they will stay stable under heavy flow attack.

Filling of the structures
The filing process of the Superbags is crucial, but the suggested placing operation is realistic (the operation can be done with existing equipment). By placing the tubes vertically, densely packed sand can be realized. The proposed placing velocity might be too expensive, but is mainly determined by the assumed strength of the Superbags at the corners of the dam body.

It is very difficult to obtain densely packed sand in the mattresses. These 'tubes' are closed immediately after they are filled. The density of the sand mainly determines the resistance of the sand mattresses against internal transportation of sand. This internal transport is observed using single 'sand sausages'; the behavior of mattresses is unknown.
The proposed waterborne filling technique seems to results in lower densities than the proposed landborne technique.

17.3 Bottom protection

The bottom protection is not optimized. Since the bottom protection works form a major part of the total building costs great profits might be possible.
The mattresses are very expensive, these mattresses costs 2000-3000 crores rupees. The covering of these mattresses costs a 140-200 crores rupees. If it is possible to construct the bottom protections without mattresses, or with fewer mattresses, great benefits can be obtained. As design target reducing the costs with 30% can be set.

17.4 General Kalpasar recommendations

There are many uncertainties about bathymetric and the tidal wave in the Gulf of Khambhat. Before the final design is made, the following things should be clear:

- The development of the tide at the proposed closure location. There is a difference in amplitude between both ends of the dam, the tidal amplitude determines the head difference over the dam and therefore the stability of structures. Before any final design is made, the behavior of the tide is should be known. This can only be done by long-term (10 or more years) measurements;
- The only available bathymetric data are the depths on the Admiralty Charts. These Charts are not made for construction purposes but for use on ships. The depths on the Charts are therefore minimum depths. Besides that, the Charts are based on sometimes very old data. Before the final design can be made, the exact location of the bottom

should be known, but also the development of the gully system in time (are the gullies and the shallows stable or do they chance constantly?). The only way to obtain certainty about the bottom is yearly survey of the bottom of the whole Gulf. These data are also necessary to predict the change in tidal range near the dam after closure.

Without detailed information about tide and bathymetry constructors are not prepared to take severe risks, therefore they will increase their tender price. The measuring campaign should be started as soon as possible.

Part F ANNEXES

1 Literature

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2 Report of India Visit

This section describes the site visit of Erik Broos and Kees Wiersema to Gujarat, India.

From the 3rd to the 25th of April they visited de area around the Gulf of Khambhat, including the ports of Dahej and Bhavnagar, the tidal area near Hansot, the Sardar Sardovar dam and the Alang shipbreaking yard. They also did a number of interviews in Bhavnagar, Dahej, Ahmedabad, Gandinagar and Vadodara (Baroda).

List of visited places:
Dahej
Hansot
Sardar Sardovar dam
Bhavnagar (new port)
Alang
Ahmedabad

List of interviewed persons:
Dr. Anil S. Kane, Director of Essar Investments Limited
C.V. Prasad, Project manager of Essar Investments Limited
T. Vadodaria, Manager (Finance and accounts) Essar Investments Limited

K.G. Rathod, Superintending engineer, Narmada & Water Resources Department, Government of Gujarat
D.M. Pancholi, Superintending engineer (Geology), Narmada & Water Resources Department, Government of Gujarat

R. Page, Manager shorebase & logistics, Enron Oil & Gas India limited

Mrs. drs. G. Wagenaar, Trade Representative, The Netherlands Business Promotion Office (Gujarat)

2.1 *Site Visit Dahej, 8th of April 1998*

2.1.1 Birla Copper Jetty

Birla Copper is a copper refining company, which has just built a new refinery. Part of this huge complex is a 2.5 km jetty in the Gulf of Khambhat. Erik Broos and Kees Wiersema have visited this jetty, which is almost completed. Main objective of this visit was to investigate a marine structure in the Gulf of Khambhat. The jetty has a crest width of approximately 8 m, with a road on top. The jetty consists of a 1.3 km rock part and a 1.2 km concrete structure on steel-concrete piles. This concrete structure consists of one solid part (there is no space for dilatation). Each span is about 12 m.
The waterdepth is 8 m at LLW, and 15 m at HW. Marine rock with a diameter of approximately 0.6 to 0.8 m has been used for the top layer.

The jetty cost some 20 crores rupees (NLG 10 million) and will be completed within two years (expected to be finished in September).
During those two years 400 workers (about 30 engineers) worked continuously, in two shifts of 12 hours each.
In spite of a large number of billboards with slogans like 'safety first', the site looks dangerous. The work is mainly labor-intensive, and steel construction (including cleaning and conservation) is done without protection.

The contractor from which Erik Broos and Kees Wiersema received this information is Trafalgar House India limited (Mumbai), a company fully owned by Norwegian Kvaerner.

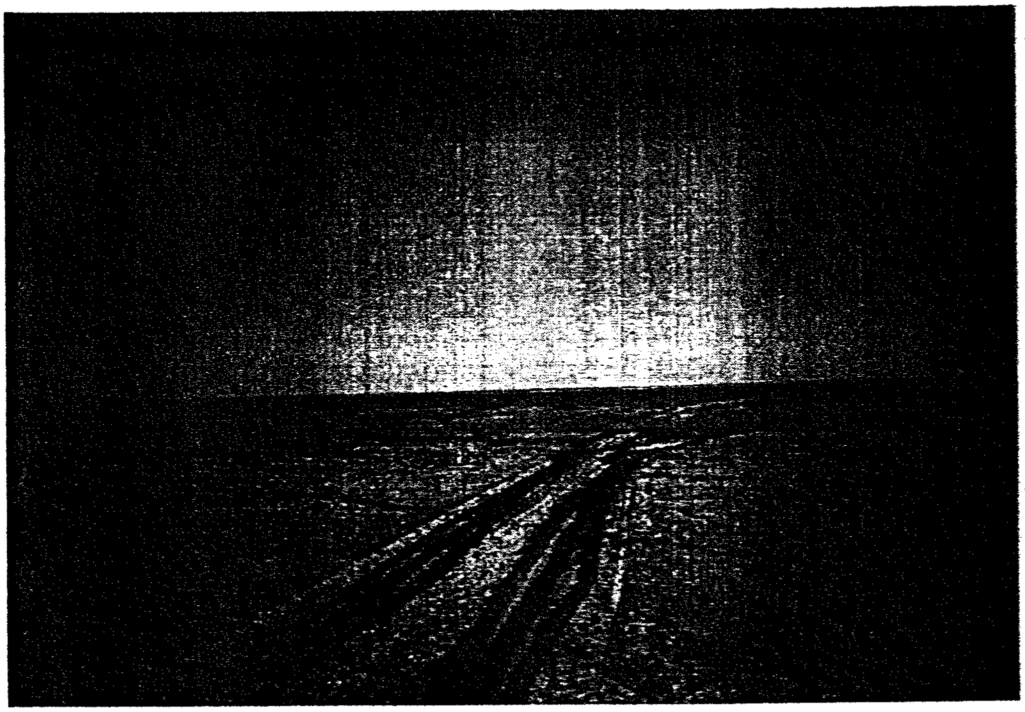
The same contractor is currently working on a second jetty, about 1 km to the south. Design is different, with only a small part consisting of rock. The biggest part is made on piles that are drilled offshore. With help of a gigantic gantry crane the jetty is constructed on top of the piles. Piling is only done when water velocities are low.

2.1.2 Dahej Old Port

The Old Port is nothing more than a small harbor, only accessible during high tide. It is used for transportation of sulfate. Only few ships visit this port.

2.2 Hansot, 9th of April

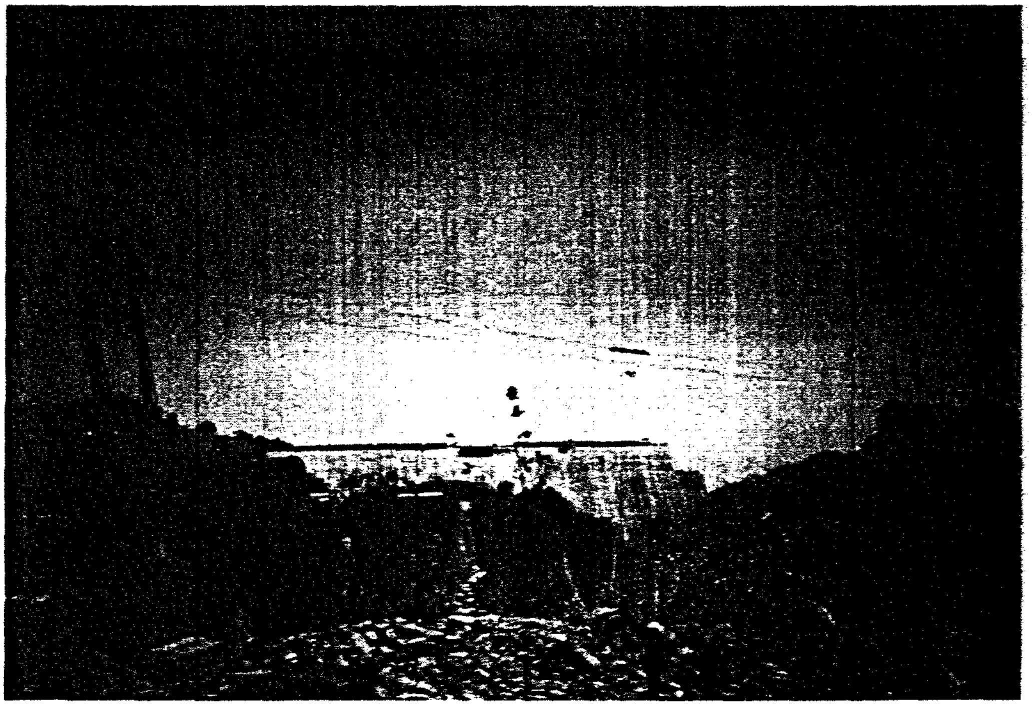
Hansot is a small town just south of the mouth of the Narmada River. It is expected to be the one end of the dam.
At Hansot Erik Broos and Kees Wiersema visited the enormous tidal wetlands. These areas don't submerge with each high tide, but still the top layer (clay) is quite soft.
The only use that is currently made of this area is production of salt on a very small scale.



2.3 *The Sardar Sarovar Dam (Narmada Dam Project), 9th of April*

This gigantic dam is part of a large-scale irrigation and hydropower project in Gujarat. From the proposed lake water is diverted via great channels with a total length of over 200 km to a network for irrigation.
The original dimensions were: a crest length of 1200 m and a total height of 162 m. Due to financial problems it is decided that the dam will be completed, only with a height of 139 m. Consequently, the amount of power that will be generated will be lower and as a result of this, the cost per kWh will increase.

At the moment, very little progress is made.
The cableway is clearly visible on the pictures. Remarkable is the single-cable fixation on both ends. The underground powerhouse was not accessible at the moment.



2.4 *Visit Alang Shipbreaking, 13th of April 1998*

It was not allowed to enter the yard itself, but from the stocking area and the long-distance view we had a good image of the size of the complex. About 15 large ships were currently demolished.
Main objective of this visit was to investigate whether it would be possible to construct barges or caissons from partly demolished ships. It is clear (from recent articles and an interview with Ron Page, who buys equipment from time to time) that it is a sort of anarchistic situation, where workers start tearing ships apart without any plan or supervision. So far, there is no recycling of steel in Alang; steel is transported to countries like Taiwan. It is not likely that construction is possible at the moment.

2.5 *Bhavnagar New Port, 13th of April 1998*

The small new port of Bhavnagar was built in 1952, when the old port close to the city became inaccessible due to sedimentation. The distance between Bhavnagar City and the new port is about 7 km.

A single lock closes the port, which is only opened at higher tides. Its sluice gate is a British design, and German made. A crane lifts the door out the water, and turns it 90 degrees, parallel to the canal.

Facilities include some quays (one in concrete), a few storage areas, small jetty's and two dry-docks.

Currently the port is mainly used for shipment of cotton. There are also two oil companies that use the port for supply of offshore locations in the Gulf (Hardy's oil India limited (drilling in the Gulf) and Enron oil & gas India limited (drilling offshore)). We had an interview with Ron Page, manager shorebase and logistics.

Greatest problem is sedimentation. Suspended material enters the port due to the turbulence of the inflowing water when the sluice is opened. This material has plenty of time to settle, and each time the depth decreases.
Bhavnagar's new port is the only port in the region that can be used throughout the whole year.

2.6 Ahmedabad and Gandhinagar, 15th-17th of April

In Ahmedabad Erik Broos and Wiersema contacted mrs. Gauri Wagenaar of The Netherlands Business promotion Office for Gujarat. This office was installed by the Dutch Embassy two years ago to promote business and trade between India and the Netherlands.

With help from Mrs. Wagenaar we contacted the Government of Gujarat and Dr. Kane.

On April 16 we had an appointment in Gandhinagar with Mr. P.K. Lehri, Principal secretary of the Chief Minister. For specific details he referred to K.G. Rathod, Superintending engineer, Narmada & Water Resources Department and D.M. Pancholi, Superintending engineer (Geology).

Summary of this interview:
Time: Kalpasar will definitely be a long-term project. They expect that governmental and political discussions, definitive studies and updates will cost at least 20 years.
They do not expect that construction will start within 40 to 50 years. Constructing time is estimated at 10 years.

Haskoning: Dutch Haskoning has recently finished a pre-feasibility study. First, the gaps in knowledge have to be filled. Then a final study can be made, which is estimated to cost about NLG 100 million.

Power: According to the Government of Gujarat there are still three options. Fresh water (no power), a tidal area of about 500 km^2 (5000 MW) and a total tidal option (6200 MW). We had doubts hearing those figures.

Rock: At both sides, at a distance of app. 30 km a large amount of basalt is available. Also limestone is available.

Water: The Saurashtra peninsula receives no water from outside. There are no inflowing rivers. Pumping of groundwater and inflow of rainwater are not in balance. The groundwater-level decreases yearly. Result of this is a increasing siltation. Salt water from the Gulf enters the land with a speed of about 1 km per year.

The Sardar Sarovar project does not influence Kalpasar in any way. Sardar Sarovar will be realized earlier and in a different area. Erik Broos and Kees Wiersema think that a disastrous hydropower project in the same state will not be advantageous for Kalpasar.

Dam: There will be a shiplock in both the fresh and the salt-water area. The shiplock near Dahej should have a capacity of up to 80.000 DWT.
No alternative dam designs have been studied in Gujarat. An exception is Dr. Anil Kane.
On the dam will be a 4-lane road. A railway is not considered, as this is a national decision.

Building costs: no power, 19.500 crores rupees
5000 MW, 35.000 crores rupees
6200 MW, 35.000 crores rupees

Transition from salt to fresh water in the basin is expected to take 4 years. Technique is to empty the basin at LLW, and to fill it with rainwater and river discharges during the Monsoon.
The morphology is stable on a short-time basis. However, it is necessary to investigate this thoroughly in the definitive study.

2.7 *Interview with Dr. Anil S. Kane, 18th of April*

In Baroda (also known as Vadodara) we had a long interview with Dr. Anil S. Kane, Director of Essar Investments Limited. Also attending were C.V. Prasad, Project manager and Tushar Vadodaria, Manager (Finance and accounts).
Dr. Kane is a driving force behind Kalpasar. In articles he is named 'well known technocrat', but according to himself he is the 'Lely' of India. He has good connections with both the Government en the major industries

Dr. Kane possesses a copy of the study by Haskoning. He thinks that the Dutch approach is far too conservative. Very safe designs are a luxury for the rich Dutch, driven by the disaster of 1953.
The safety factors that have been used are too high. Too much bottom protection and too much concrete are used.

He sets much value on the study on the Severn Tidal Power Plant (Great Britain). In this report a definitive design is made for a 16 km, 8600 MW dam in an area with a tidal difference of 13 m. This design fully consists of caissons, no sandfill dam will be constructed afterwards. The closing caissons in the Haskoning study will be used only once, which he believes is a waste of money.

Tidal energy is the pivot of the whole project. With power money is earned. Second are the fresh water and the road connection. Later harbors and land reclamation can be developed. Dr. Kane is convinced that the whole project is commercially feasible and can be developed with help of private investors. Dr. Kane has written an article about Kalpasar in March 1998, this with the intention to attract investors to create a basis of acceptance among the public.

Haskoning calculated the total cost at some 35.000 crores rupees (about 9 billion US$). He believes that 20.000 crores should be an acceptable price for investors.

Construction:
Caissons should be used permanently, not only once during closure
Factor of safety should be considerably lower
Several sizes of caissons should be used, so there is no need to construct large sills

Power:
There should be an optimum between the areas of salt and fresh water, not just an arbitrary value. Tidal power should be 700 instead of 500 km^2, maybe even more
Turbines (9 m diameter) can produce 40 MW instead of 25 MW
Turbines can be cheaper
India's demand for electricity exceeds 60.000 MW in the next 5 years, increasing to 70.000 MW in the following 5 years
30% of India's power is used for irrigation. Farmers receive subsidized electricity (subsidizing will stop within a few years, according to G. Wagenaar). Dr. Kane wants to sell (cheap) electricity to farmers. A timetable of delivery is available for years.

Finance:
Total cost should be reduced to some 20.000 crores rupees.
No money from the Government is necessary. Industry and other private persons are interested in financing
Concessions for toll, water, energy should be auctioned for a period of 25 years
Reclaimed land will be sold
For energy produced by tidal power no oil or coal have to be imported, therefore there will be a reduction on the amount of US$ that flow out India
5000 MW costs 16.000 crores, production will be 12.000 GWh, which results in 3.600 crores a year (current price 3 rupees/ kWh). This is a yearly benefit of 22.5 %. Corrected on

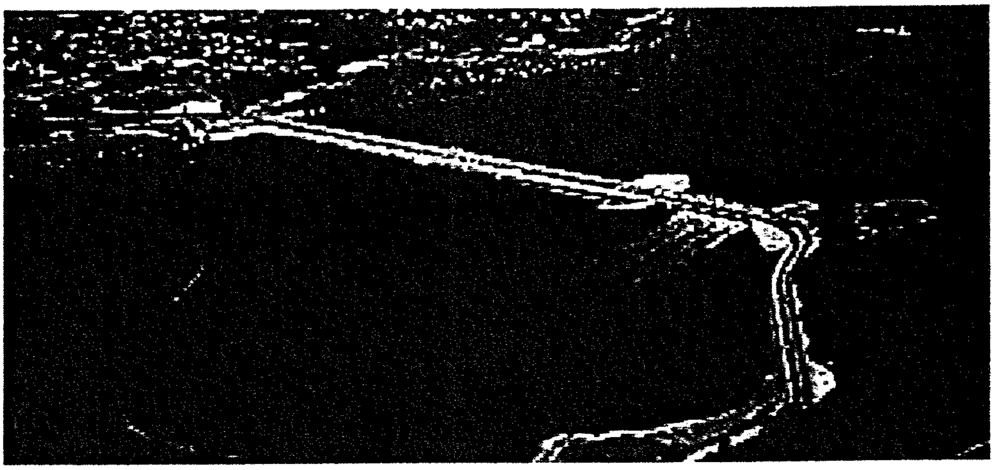
inflation and interest, it should be possible to earn 12.5% per year. Thus in 8 years investments will be paid back
Dr. Kane wants 6200 MW, resulting in 16.200 GWh. 1 Ton of oil produces max. 5000 kWh, so Kalpasar saves 3 million tons of oil each year. With 120 US$ per ton, a saving of 360 million US$ is made.
According to Dr. Kane, these calculations are justified, as India is dependent on import of oil (production 32 million tons, consumption 65 million tons).
Haskoning calculated a price of 3.2 crore per MW, Dr. Kane wants this to be reduced
Bonus: In Kyoto it was decided that countries that reduce the emission of CO_2 should be rewarded for this financially. It is not clear whether the reduction by Kalpasar compensates the growth of the rest of India.

Construction:
Turbines should be built in India, with help of the French. About 250 turbines will be needed.
In the basin the water level will be lifted 6 m. CD will rise to current MSL. This creates a deeper lake, which is beneficial for ships. Only in the Monsoon this level will be slightly lower.
Land reclamation will be limited to areas 5 to 6 m above MSL. This means no polders like in The Netherlands. Land reclamation will therefore be very cheap.
Great shiplocks should be built
There should be a 4-lane road and at least 1 railway track
Concessions for toll should be auctioned in advance

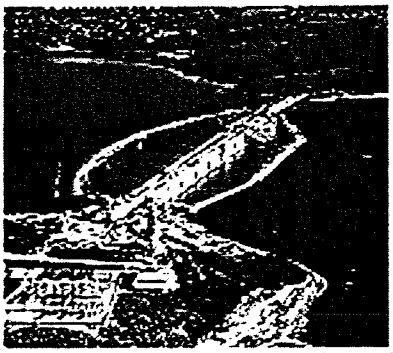
Future:
There will be almost no sedimentation in the basin, as all rivers are dammed.
The basin can be developed to a big port, safe from rough seas. Disadvantage according to Erik Broos and Kees Wiersema is the shiplock, this is not a problem for Dr. Kane
When politics remain stable Dr. Kane is able to start the project in 2 years.

Quotes:
'This dam will be completed before 2010.'
'Within 20 years after construction this will be the biggest harbor of the world.

3 Report of La Rance visit

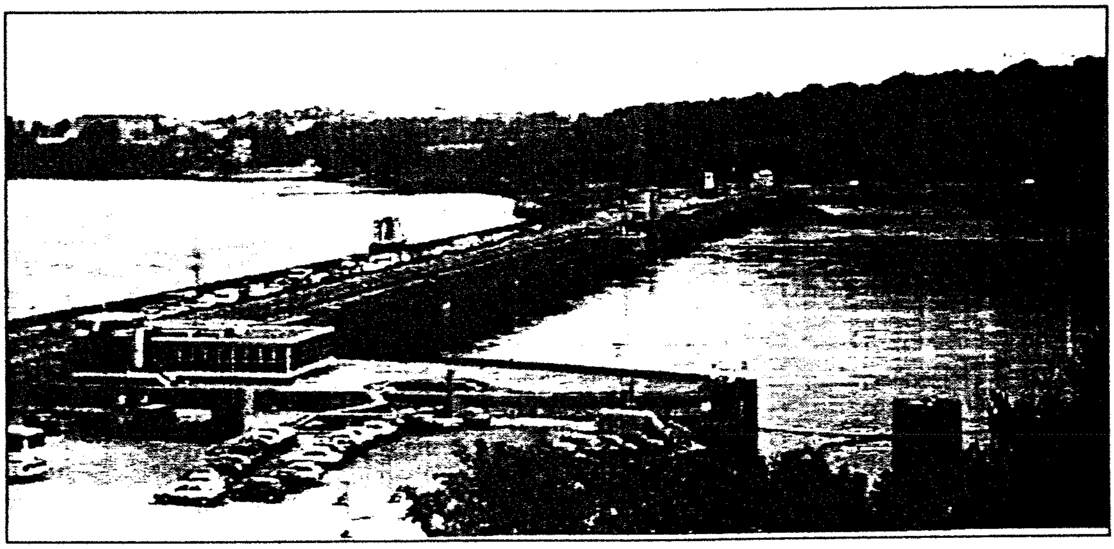


To get more insight in the design and use of a tidal power facility, Erik Broos and Kees Wiersema paid a visit to Europe's only tidal power station, located in the Rance, near St. Malo in France. This visit was done from June 3rd to June 6th.



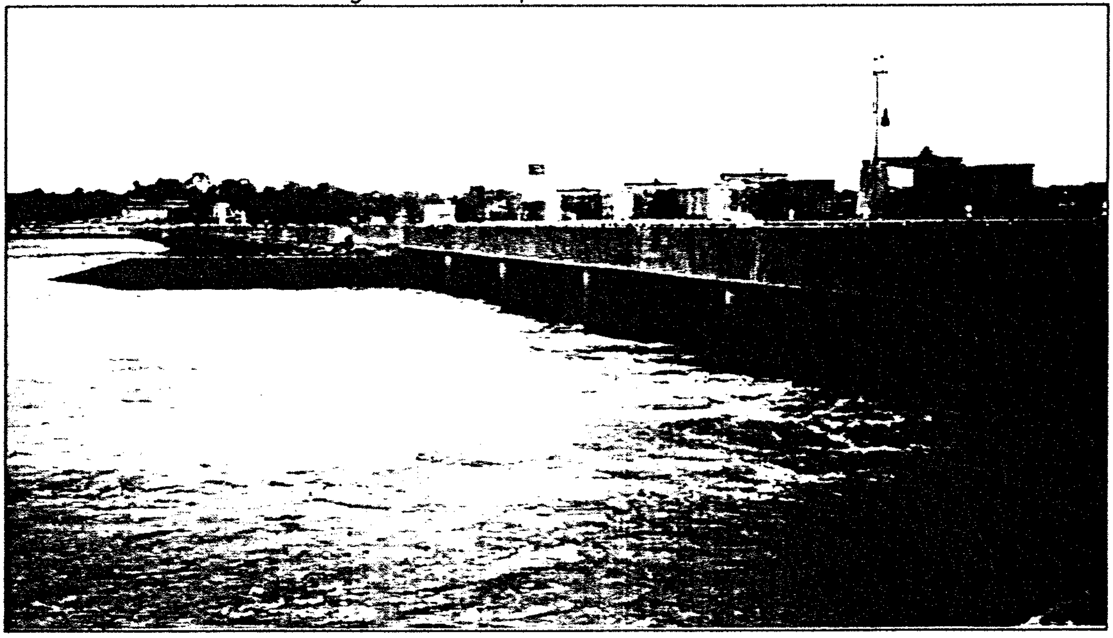
The Rance is in only one way comparable to the Gulf of Khambhat: its tidal range has a maximum of 13,5 m at Spring tide. The width of the closure is 750 m, the bottom consists of firm rock and the storage area is quite small (22 km^2). It produces 240 MW. The basin is for tidal use only.

Construction started in January 1961 and was finished in December 1967. During this period, the Rance was temporarily closed off by a huge cofferdam. The inlet sluices were built first, and they served as a temporary orifice that reduced the flow through the other closure gap. In that gap the powerhouse with the turbines was constructed. Also a shiplock with single (!) doors forms part of the dam.



An important benefit of this dam is the short connection between St. Malo and Dinard. Over the dam a four-lame road reduces the distance between those two cities with 40 km.

In this picture the shiplock is shown on the foreground. The turbine section is situated between the building and the island in the back. On the background the sluices are shown. These sluices are on the foreground of the picture below.



As La Rance's tidal power station was the first in the world, great research has been done to construction in open sea, hydraulics and corrosion. A 1 to 150 model of the Rance was built and the turbines were tested in a 1 to 1 scale situation. Moreover, it was decided to design the facility in such a way that not only ebb-generation, but also flood and two-way generations was possible. The bulb-shaped turbines also provided the possiblity of pumping to increase the head difference over the dam.

In La Rance mr. Alain Barreau, manager of the 'Marémotrice de La Rance' was interviewed. He showed that tidal power can be really competitive when compared to other sources of electrical power. In France, the tidal power station is connected to the national electrictiy network. When the head difference over de dam in La Rance is such that power generation is cheaper than other sources, the station produces electricity. This can be done up to 17 hours per day. When production is too expensive, the turbines are not used.