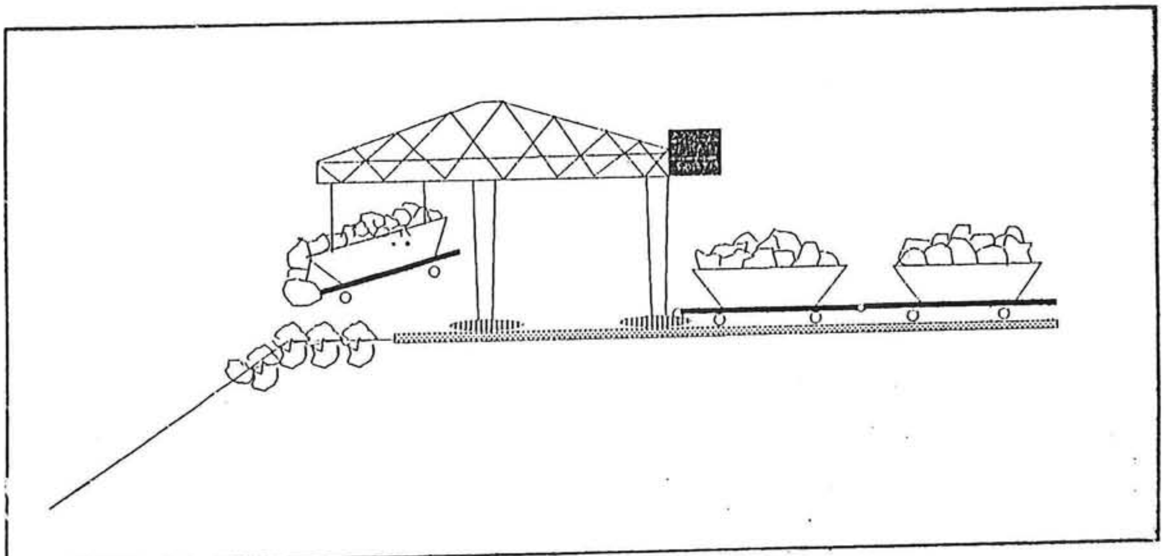
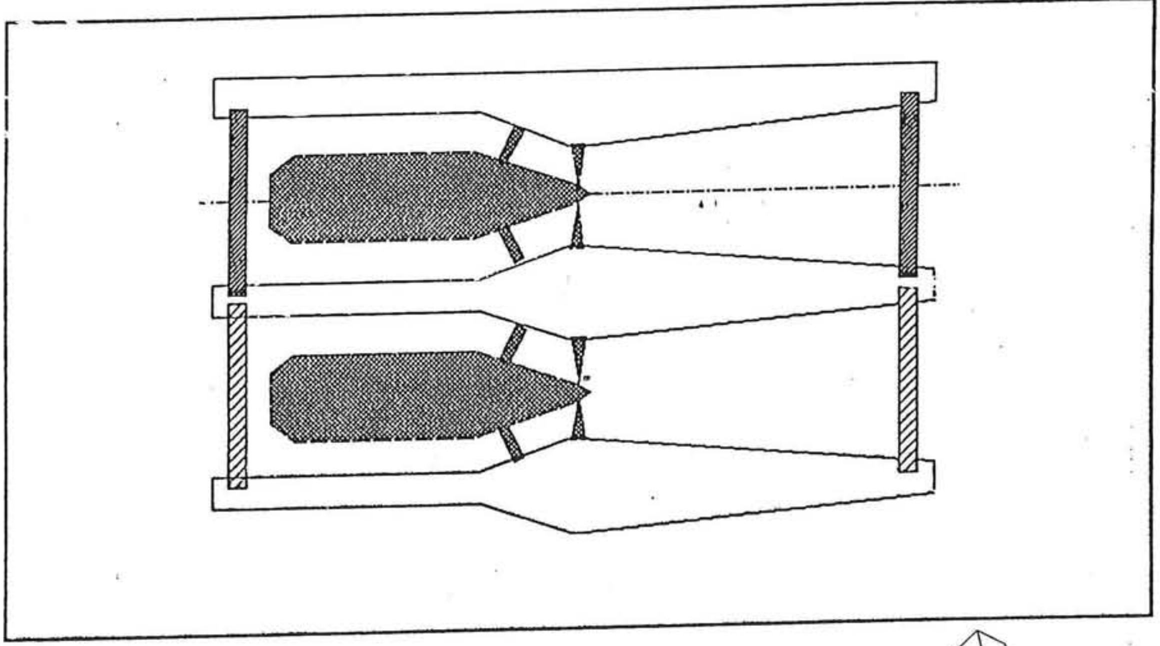
Gulf of Khambat

closure and tidal power station

final report





TU Delft

Delft University of Technology

Faculty of Civil Engineering
Hydraulic and Geotechnical Engineering Division
Hydraulic Engineering Group

Gulf of Khambat

closure and tidal power station

November 1992

Final report

P.L.M. Jansen

&

I.C. Vreeburg

Delft University of Technology
Faculty of Civil Engineering
Hydraulic and Geotechnical Engineering Group

Preface

In recent times people have developed a great interest in clean sources of energy. This is stimulated by the discovered polluting effects of the fossil fuels and the increased fuel consumption. The use of nuclear energy is not accepted by society due to the danger of a nuclear drop-out and the problems with nuclear waste. At first sight any possibility to produce clean energy should be taken.

One of the sources of clean energy is water. Hydroelectric dams are known for some time. A fairly new method is the use of the tide to produce electricity. The Gulf of Khambat in India is characterized by a large tidal range. Thus, it is one of the possible locations to construct a tidal power station.

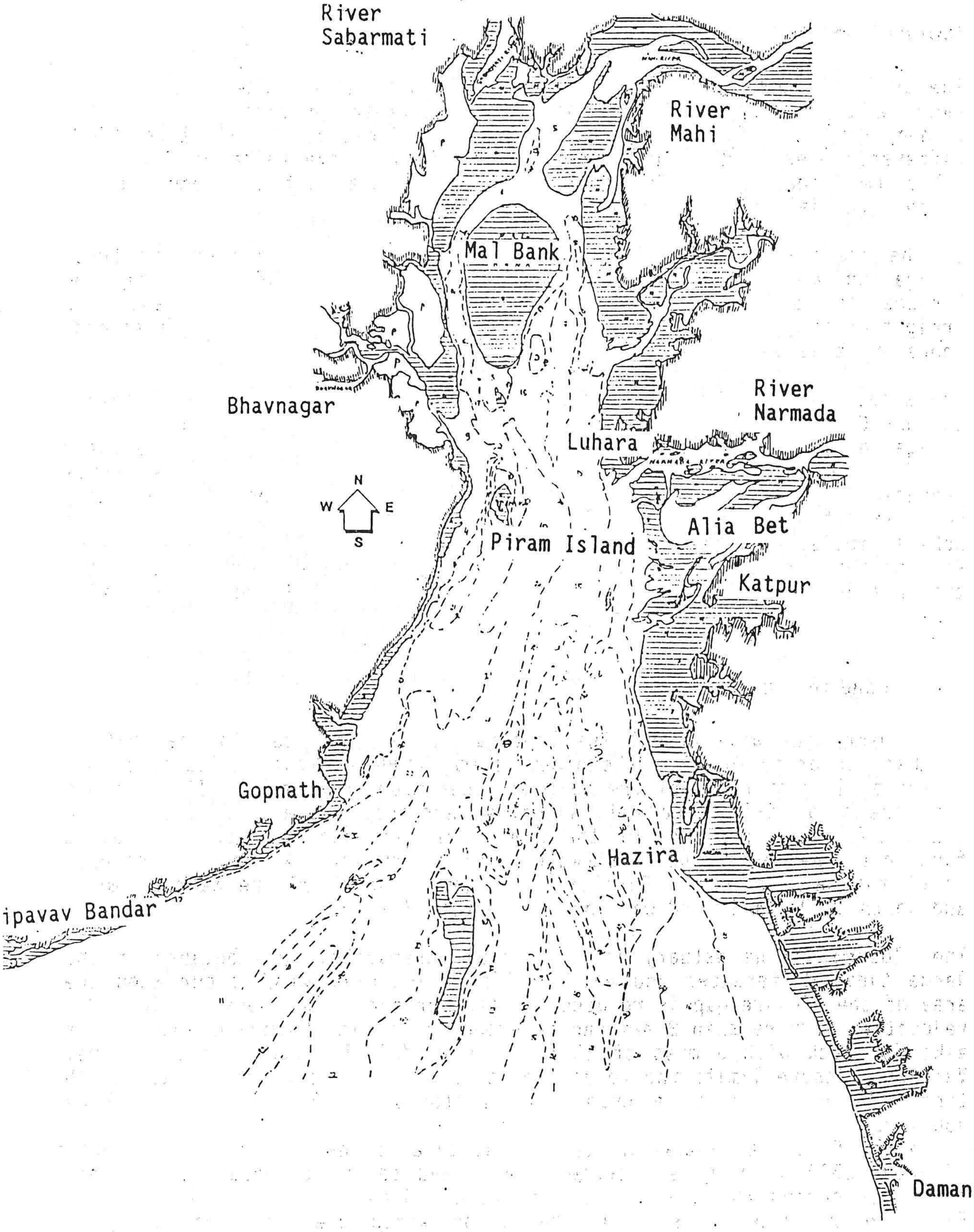
Around the Gulf of Khambat there is a demand for irrigation water. The fresh water discharging from the Narmada river can be used for that purpose. However, this water must be stored.

So the idea came up to close the Gulf of Khambat in order to accomplish both goals. In this final report the technical aspects of both the closure and the tidal power station are discussed.

The environmental impact of such a project will be enormous. The change of a salt water in a fresh water regime will cause a change in vegetation and fauna. This will take many years. Also the way of life of the local inhabitants will change considerably. Although these aspects are only slightly mentioned in this technical report we do realise that it is not self-evident that a project like this is finally realised.

With this report we expect to have contributed in the insight in the development possibilities of the Gulf of Khambat. We like to thank all persons mentioned in the list of references, and especially the members of the committee of supervision, for their help.

P.L.M. Jansen and I.C. Vreeburg


River
Sabarmati
River
Mahi
Mal Bank
Bhavnagar
River
Narmada
Luhara
N
W
E
S
Alia Bet
Piram Island
Katpur
Gopnath
Hazira
ipavav Bandar
Daman

Summary

Introduction

The Gulf of Khambat in India is an estuary with an extremely large tidal range which in average circumstances varies between approximately 4.5 m for a neap-tide and 8.5 m. for an average spring-tide. This large tidal difference goes hand in hand with a tidal flow in the range of 6 to 11 * 10^9 m^3 per tidal cycle. Due to these features the Gulf of Khambat has a large potential for tidal power energy.

In the areas surrounding the Gulf of Khambat there is a need for fresh irrigation water. Three rivers discharge in the Gulf, of which only the Narmada discharge is of significant importance when we speak about an irrigation project. In order to use this water it will have to be stored. Therefore a large basin is required.

The aspects above lead to the proposal to construct a tidal power station in the Gulf of Khambat, possibly combined with fresh-water storage for irrigation purposes. In this report only the combination of the two goals is discussed. An advantage of this development scheme is that the energy produced in times the demand for electricity is low, can be used to transport irrigation water to the areas where it is needed. The choice for this scheme implies the construction of two separate basins, one for fresh-water storage and a tidal basin used for the energy production. Somewhere in the entrance of the Gulf of Khambat a closure dam will have to be constructed.

Closure and final dam

The closure dam will affect the propagation of the tides in the Gulf of Khambat. In order to study the order of magnitude of this influence a one-dimensional tidal model of the gulf has been made [chapter 3]. It was found that the tidal difference will slightly increase by dam construction. The tidal difference in front of the dam is larger when the dam is situated further to the north. The fresh-water storage however asks for the inclusion of the Narmada mouth. Finally an alignment south of the Narmada mouth and north of Piram Island was chosen [section 4.1].

The closure of the estuary is a difficult operation. This because of the large tidal differences and extreme flow velocities which occur when the area of the closure gap is reduced. In case of horizontal constriction velocities of more than 8 m/s can be expected in normal circumstances. This asks for rock with a mass of 20 tons in the final stage of the closure. Vertical closure limits the velocities to 6 m/s. Now rock with a mass of 15 tons will do. At least a grading with stones of 15 - 20 tons will be applied.

In the final dam intake works for the tidal power station with a width of about 5500 m. must be embedded. This lead to the proposal to use this flow area during the closure to limit the velocities in the final stage. First, using rock, a sill is dumped on which prefabricated caissons, containing both the intake works and the turbine tubes, are sunk. Then a vertical closure is applied till a sill level of 5 m. below Benchmark. is reached. Above this level controlled dumping with barges will be difficult. On the sill a horizontal constriction is carried out in order to complete the closure. In this way, although the closure is partly horizontal, the velocities are limited to approximately 6.5 m/s. The maximum grading is 10 - 15 tons.

In order to prevent for severe scour close to the dam a bottom protection must be placed before the closure operation is started. The possibility discussed in this report is the use of stones on geo-textile. The maximum grading that has to be applied is 1 - 3 tons. The length is safely estimated as approximately ten times the downstream depth. Considering the scale of the project it may be favourable to develop a different method.

Except the provisions for the exploitation of the tidal power station two other structures have to be situated in the dam. In order to control the level of the fresh water basin a spillway is constructed east of the tidal basin. West of the intake works a ship lock is placed. This lock is the entrance to the tidal basin in general and Bhavnagar port specifically.

The closure dam is not suited to accommodate a double-tracked railway and a road. Besides the fresh-water basin must be protected for salt intrusion and fresh-water losses through the final dam. It is proposed to construct the final dam by means of hydraulic sand-fill, against the basin-side slope of the closure dam. This method can only be applied when the flow velocities are low (< 2 [m/s]). In order to accomplish this a filter layer and a ballasted geo-textile are placed first. After building the embankment core the slope protection on both sides is placed. It consists of rock on the sea-side and concrete blocks on the basin-side of the final dam. On the finished dam the railway and the road can be built.

Tidal power station

Tidal power stations can be designed for two different lay outs and for four different operating systems. If two tidal basins are made a continues energy output can be accompliced by keeping one of the basins low and one of the basins high. With one basin energy can be generated during ebb, during flood and during ebb and flood. Chosen is for energy production during ebb because the energy production is as high as it is for two way generation, and the investments are lower than for two way generation. Flood generation requires the same investments but gives less energy output.

Four alternative basins have been suggested, varying in seize from 250 km^2 to 510 km^2. All four alignments are situated north of the closure dam at the west side of the Gulf of Khambat. A cross section for the basin dam has been designed with a crest height of BM + 14 metre, and a slope of 1 : 15 under water and 1 : 3 above the water surface. With this cross section the amount of sand required for the dam can be calculated. The required quantities (average $108 * 10^6$ m^3) do not vary much because the longer basin dams have alignments running over shallower parts of the Gulf. The costs of the basin dams including the revetment have been determined at Rp 5400 crores (US$ 200 mln) for the two largest basins.

With the DUFLOW computer model a simulation for the tidal power station has been made for all four basins. The turbine has been simulated with an overflow varying in wide for different heads. In this way a realistic simulation can be made. The intake works have been simulated with an overflow with a sill at BM - 5 metres.

The turbine chosen for this project has a diameter of 8 metres and a design head varying with the number of turbines used for the basin. A higher number of turbines resulting in a lower design head. With 200 turbines the energy production of all four basins can be calculated. The result is a gross output of 17.7 GWh/day for basin 1 increasing to 36.3 GWh/day for basin 4. On basis of this calculation basins 1 and 2 have been abandoned as alternatives. Basins 3 and 4 have been put through a further economic evaluation. The seize of the intake structure was optimised at 6000 metres for basin 4 and 5500 metres for basin 3.

For basins 3 and 4 several DUFLOW calculations have been made simulating different numbers of turbines. The result of these calculations is rewritten in money by using a kWh price Rp 0.75. The cost of a turbine unit (Rp 59 crores) has been determined by using an cost equation. This equation has been calibrated with recent project of which costs are known. The costs of the intake structure has been estimated at Rp 1620 crores for basin 3 and Rp 1730 crores for basin 4.

The optimum number of turbines for basin 3 is 125 (annual production of 8440 GWh) resulting in a annual profit (capital loss as result of lack of interest included) of Rp 113 crores. The optimum number of basin 4 is 145 turbines (annual production of 9670 GWh) resulting in a profit of Rp 134 crores. Basin 4 has therefore been chosen as preferable basin. The total costs of the tidal power station (not including the closure dam and the bottom protection works) has been estimated at Rp 10,900 crores.

Table of contents

1 Introduction

1.1 Project Introduction

The Gulf of Khambat is located in the west of India (see Figures 1.1 and 1.2). This bay is distinguished by its very high tidal differences up to 11 metres at Highest Astronomical Tide (HAT). The extreme tidal movement offers the possibility for the exploitation of a tidal power station.

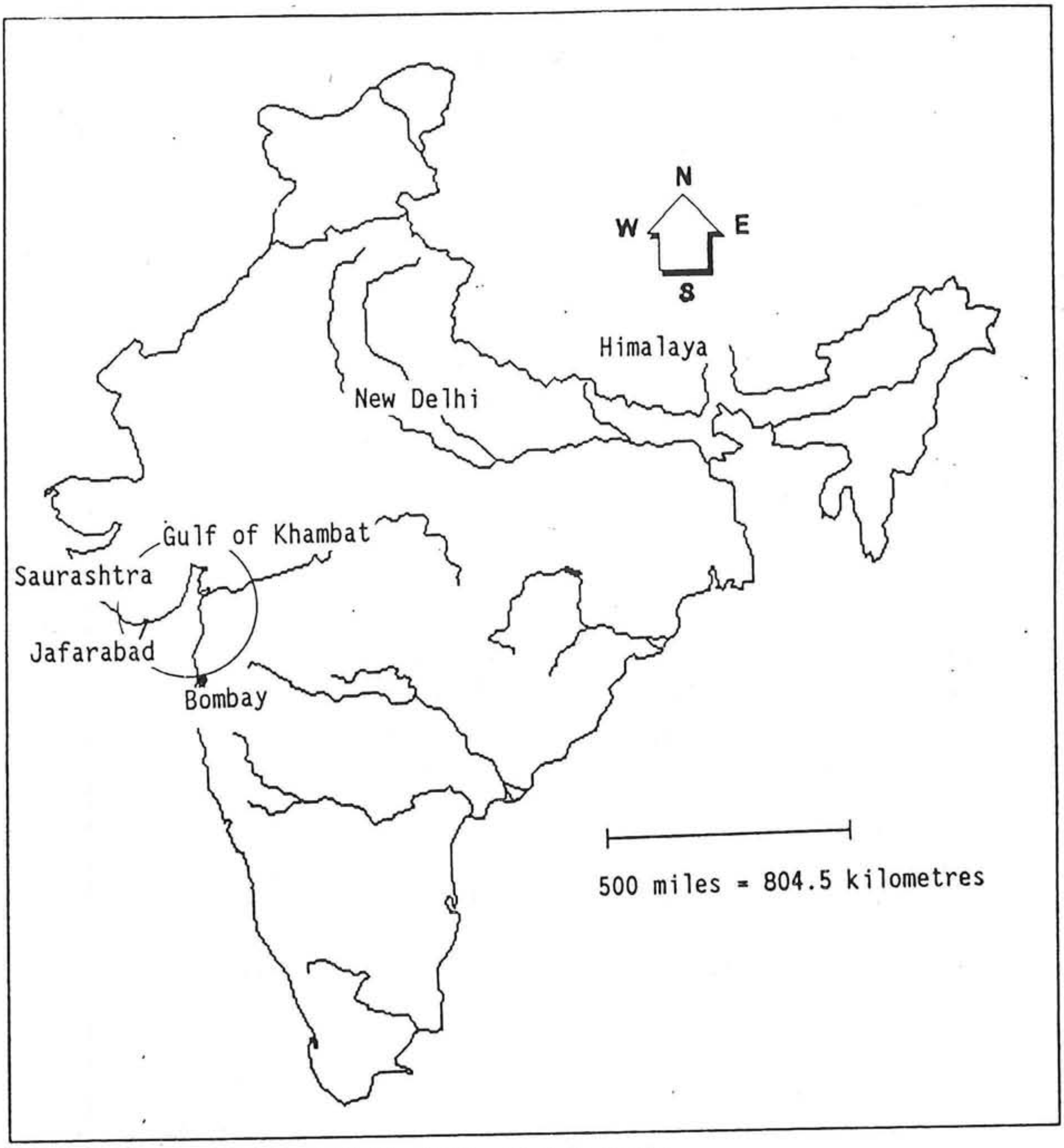


Figure 1.1 Location of Gulf of Khambat in India

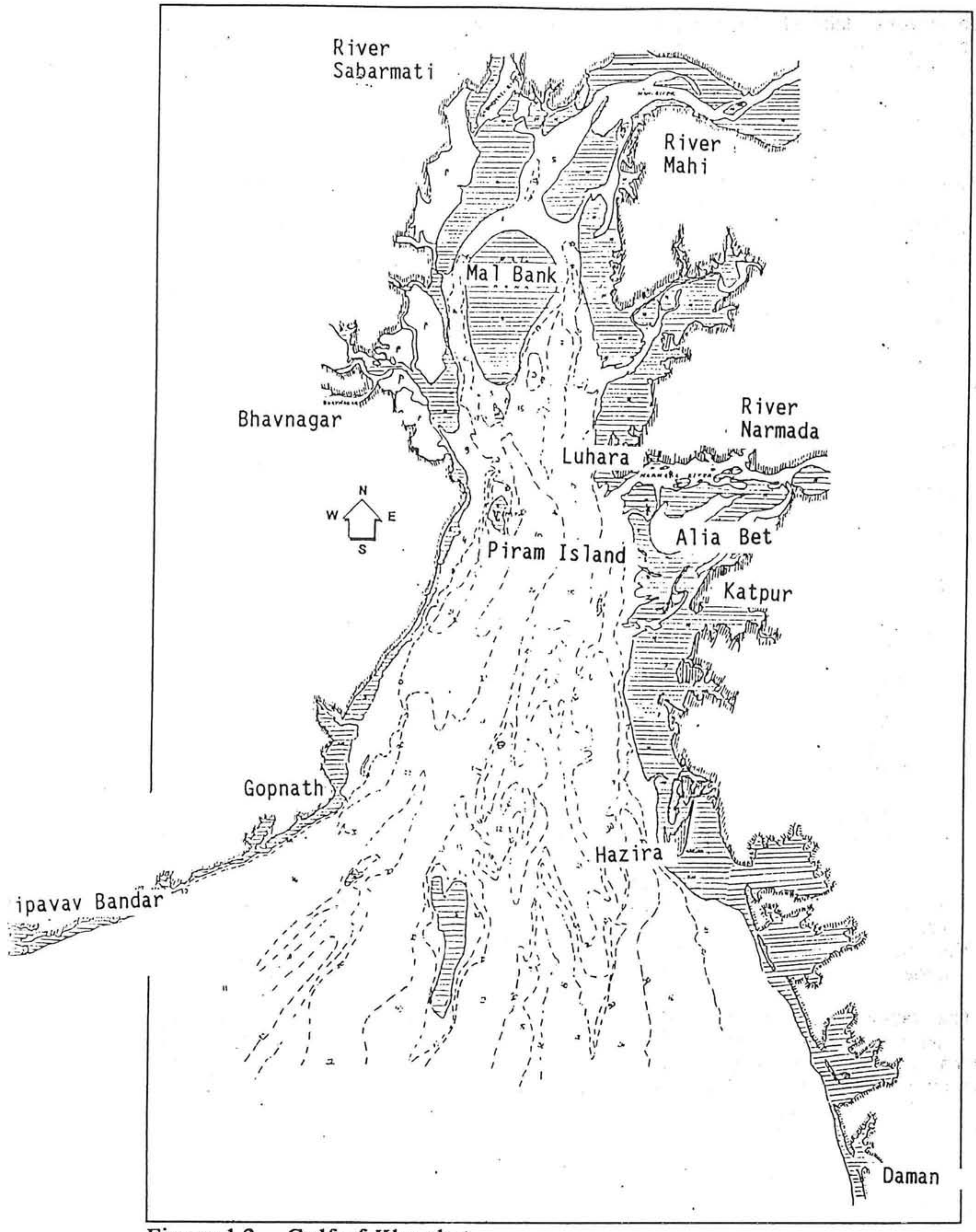


Figure 1.2 Gulf of Khambat

Some rivers debouch into this bay. The three most important and biggest rivers are in order of size: the Narmada river, the Mahi river and the Sabarmati river. Figures of the discharge can be found in Chapter 2, Table 2.1. With the water, sediment is transported into the Gulf.

Fresh water is a precious good in this part of India. Closure of the Gulf offers the possibility to store a large amount of fresh water in a reservoir, which otherwise would flow freely into the Arabian (salt) Sea. The fresh water can be used to irrigate land in a wide surrounding area.

On the basis of these characteristics a proposal has been made in 1989 to close a part of the Gulf. The proposal concerns a closure near the mouth of the Narmada river at which the total discharge of the river flows into the closed part of the Gulf. A summary of the reconnaissance report is given in the following section.

1.2 Summary of the reconnaissance report

On the 26th of September 1988 an agreement was signed between the Government of Guyarat, Water Resources Department and Haskoning Royal Dutch Consulting Engineers and architects for the preparation of a reconnaissance report on the construction of a dam across the Gulf of Khambat (this report is lit [5]). In May 1989 a third revised report of the study was presented. In this report the technical viability of the dam and additional benefits were presented.

Although various development schemes, related to the dam construction may be distinguished three benefits may clearly be considered as the most important viz.:
- fresh water storage for the irrigation and water supply of the Saurashtra Peninsula,
- tidal energy production,
- dam linking Saurashtra with the mainland of India.

These two schemes, therefore have been selected for further investigations and relevant dam alignments have been assessed accordingly.

The main criteria for the selection of the alignments have been:
1. Availability of sufficient water depth for the implementation of tidal power works.
2. The inclusion of the Narmada river flow in the fresh water storage scheme.

As the water depth north of Bhavnagar is insufficient dam construction has only been studied in three alternative alignments between the Bhavnagar Ghogha area on the western bank and the Narmada river mouth only. Dam construction costs in alignments north of Bhavnagar would be considerably lower. The same holds, however, for the benefits (volume of fresh water and quantity of energy production).

With regard to the costs, the soil conditions for the foundation of the dam are of utmost importance. The general information on the composition of the bottom of the Gulf of Khambat supports, however, the assumption that the foundation conditions may not form a major constraint. Further investigations and studies of the dam foundation are required to obtain sufficient information concerning: bearing capacity, deformations such as settlements, liquefaction (earth tremors), etc.

Various closure methods were examined and all considered technically feasible. Further optimization in terms of construction time and costs is required. Also material resources of construction are to be studied regarding suitability and economics. The least cost solution of the closure dam has been estimated at Rp 1950 crore (one crore Rupees is 10^7 Rupees) With a rate of 27 rupees to one US dollar the costs in US$ is 722 mln. This solution concerns the alignment Ghoga-Alia Bet - Hansol and includes a 65 km long rock-fill dam provided with a 700 m. wide spillway and a navigation lock. The impact on environment, and anticipated impact on tide and morphology is minor.

The development schemes that have been looked at:
1. Tidal power production in combination with navigation, port development and the crossing of the Gulf of Khambat.
2. Fresh water storage in combination with land reclamation irrigation navigation Gulf crossing and fishery.
3. Combined development in which part of the basin is used for fresh water storage and part for tidal power generation with the corresponding additional developments mentioned above.
4. Dam as an infra structural connection.

The costs of a tidal power station with an energy production of 20.000 GWh per year has preliminary been estimated at Rp 10.000 crores (including dam construction). Apart from the power production the road between Dahej and Ghoga will save Rp 50 crore per year of transport cost rising to Rp 100 crore within 15 years.

For the fresh water storage scheme an analysis was made of the availability of fresh water and the corresponding irrigation capacity. The average net yields discharging in the Gulf of Khambat are estimated at 14 x 10^9 m^3 per year. From this water some 5.4 10^9 m^3 could yearly be used for irrigation with 100% dependability and some 9.8 10^9 m^3 with 75% dependability. Water will be available to irrigate some 700.000 Ha. with 100% cropping intensity. This area could almost be doubled if 75% dependability is accepted.

Additional benefits of the closure are:
1. Some 65.000 Ha adjacent to the Gulf of Khambat can be reclaimed and irrigated.
2. Fish production may double from some 7.000 tonnes per year now to some 14.000 tonnes of fresh water fish.
3. Reduction of cost of drinking water.

The total costs of the fresh water scheme including dam construction, land reclamation, irrigation scheme and compensation for the salt industry has been estimated at Rp 3,750 crores.

In a combined option with a 500 km^2 tide basin and an energy production of 4.900 GWH per year the additional costs will be about Rp 2.460 crores. The introduction of the power scheme will reduce slightly the agricultural benefits since part of the storage capacity of the fresh water reservoir is lost. Also the area that can be reclaimed is reduced. The same holds for the benefits of the fisheries.

A very preliminary evaluation indicates that the first scheme looks attractive for a tidal scheme. The second scheme has also good potentials but the third scheme with combined tidal power and fresh water storage has the best potentials. Water is available for irrigation as is the energy to transport it. The alignment of the dam according to the 3th scheme is shown in Figure 1.3.

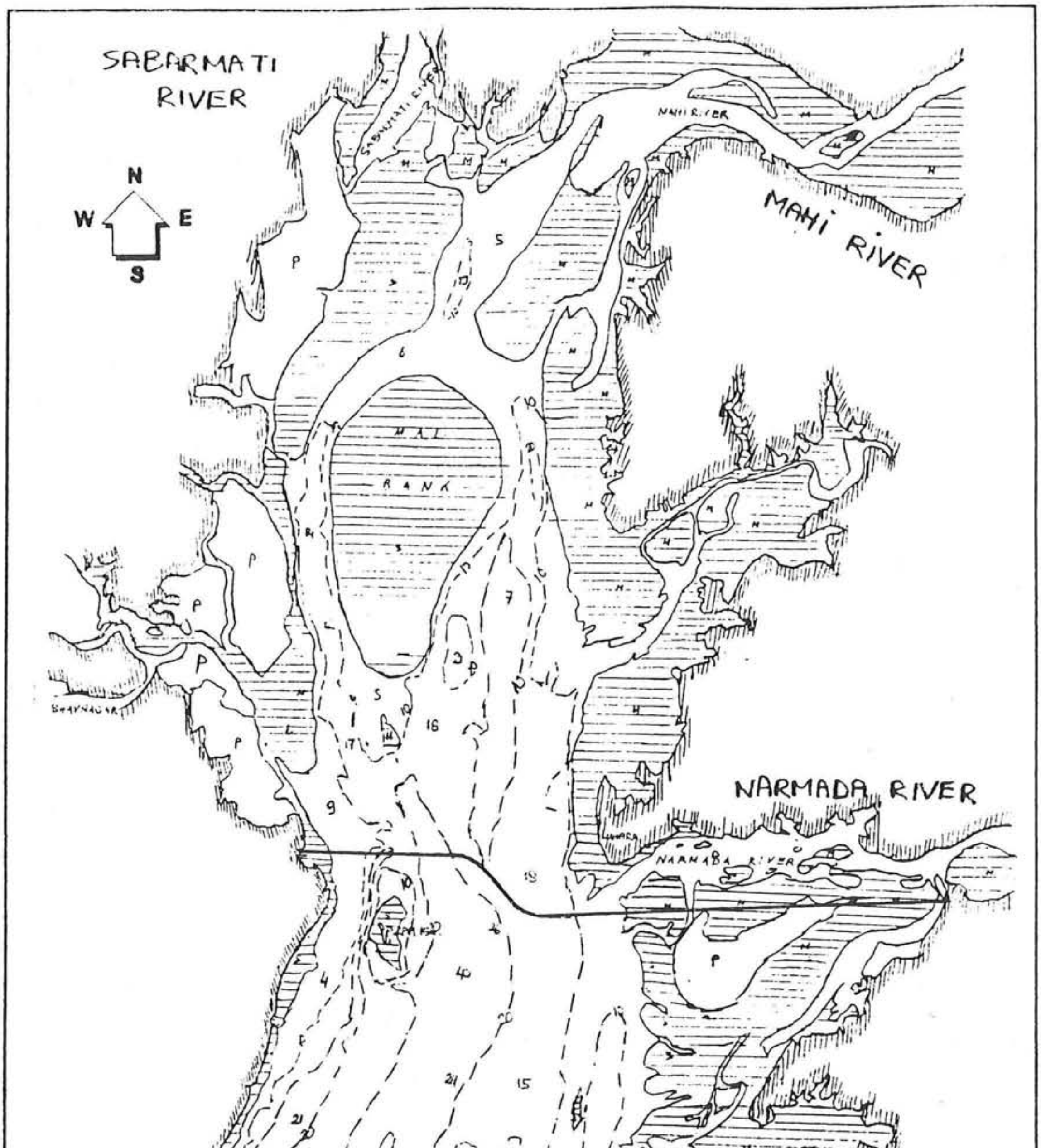


Figure 1.3 The dam-alignment according to scenario three of the reconnaissance study

1.3 Basic assumptions and preconditions

The scheme that has been evaluated as the most attractive in the reconnaissance study has the following features.

The part of the Gulf of Khambat that will be closed will be divided in:

- A tidal basin that will be used for the generation of energy by a tidal power station.
- A fresh water lake that will provide water for irrigation and drinking water supply.

This scheme will be the starting-point for this project. Possible land reclamations of parts of the created fresh water lake will not be elaborated. Such developments will affect the necessary capacity for the spillway in the dam. This influence will be taken into account in the design of the dam.

In the reconnaissance report a preference has been made for the alignment between Ghogha and Dahej. In this report however another evaluation will be made on basis of other possible alignments.

The Gulf of Khambat has a rapidly changing pattern of banks. At the same time maps of this area are scarce or out dated. In order to have a uniform starting point only one chart will be used. The admiralty chart 1486 India-West coast, approaches to Gulf of Khambat (edition 1991). This chart has been composed of several charts of the Indian Government. The dates of surveys for these Indian charts range between 1847 (northern part of the Gulf) and 1986.

1.4 Objectives of the Study

One of the objectives is the complete spatial design of the closure construction and the tidal power station. Accordingly the place, size and number of the structural elements of the dam and the station shall be determined.

Attention will focus on the following points:

Closure dam	- alignment
	- number and size of structural works
Tidal power station	- size of basin
	- intake structures
	- type, number and size of turbines

In addition to this spatial design two more detailed studies will be made. These studies comprise the following subjects:

1. The design of the cross-section of the closure dam.
   - closure method
   - material
   - development scheme
   - temporary bed protection works in closure gap

2. The design of the intake works as well as various generator and powerhouse aspects.
   - system of power station
   - kind of turbines
   - construction method
   - bed protection works
   - concrete structure
   - dimensions of stream duct

In order to get information about the water movements and levels in the Gulf after the closure, a tidal model will be made with the existing micro computer package DUFLOW. This package can be used for the simulation of one dimensional unsteady flow in channel systems. The tidal model will first be made for the present situation. Calibration of this model is possible by using known measurements and tide-expectations. After calibration of the model for the present situation a new model will be made, which will have the first model as a basis. The second model will describe the water movements in the Gulf after and during the closure.

In the reconnaissance report many advantages of the closure of the Gulf of Khambat have been discussed. The critical reader of that report will have missed the disadvantages of the closure. In this report an inventory will be drawn up of the disadvantages in chapter 6.

2 Description project area

2.1 India

India is a federal parliamentary democratic republic with New Delhi as its capital. The country can be found on the globe between 37,6° - 8,4° northern longitude, and 68,7° - 97,3° eastern longitude. The number of inhabitants is estimated at 840 million, and they live on an area of 3,3 million square kilometres. The national currency is the Rupee, at this moment the value of a Rupee is about 27 Rupees to one US Dollar. The official language is Hindi, the official name for India is Bharat.

About the economic standard in India it must be said that many people live in poverty. India is an agricultural state but at the same time, there is sometimes a serious shortage of food. An Indian proverb summarizes it in one key sentence; Live is gambling for rain. Lately irrigation projects have made a positive change in the amount of agricultural products that are produced.

The geological make up of India can partially be explained with the tectonic movements. India is situated on one plate, which in history has moved from the south of Africa to the south of Asia. Colliding with Asia the Himalaya mountains have been formed. This process is still taking place and earthquakes still occur in the Himalaya region. At the north-western edge of the Indian plate (in Pakistan) the plate is faulting past another plate, the Iranian plate. Sudden movements of the plates have great effects in this fault area. Focusing on the Khambat region it can be said that no earth movements of significance take place. The distance to the Himalaya is far too great to notice earthquakes. The distance to the fault area in Pakistan is also considerable, more then 750 km., and earthquakes taking place in this fault-line are reduced to earth tremors in the Khambat region.

Other mountain ranges in India are the western and eastern Ghats, located respectively along the western and eastern coast of India, while the Aravalli mountain range is located in the north western part of India, near the coast and three other mountain ranges run from the Gulf of Khambat to the Bay of Bengal namely the Vindhya mountain range, the Satpura mountain range and the Mahadeo hills.

The exact location and extent of the ranges can be seen in Figure 2.1.

The course of the most important rivers is shown in Figure 2.2. The largest river is the river Ganges, running from Tibet past New Delhi to Bangla Desh and the Bay of Bengal. Of more importance to the project under study are the river Narmada, the river Mahi and the river Sabarmati. These three rivers do not receive any water from the Himalaya mountain range. Large rainfall in the monsoon period provides the water for these rivers. The variability and the unpredictability of the rainfall leads to a variable discharge of the rivers. Figures of the discharges are given in Table 2.1. The source of these data is the Khambat Gulf Reconnaissance Report [lit (5)]. The figures show the irregularity of the discharges.

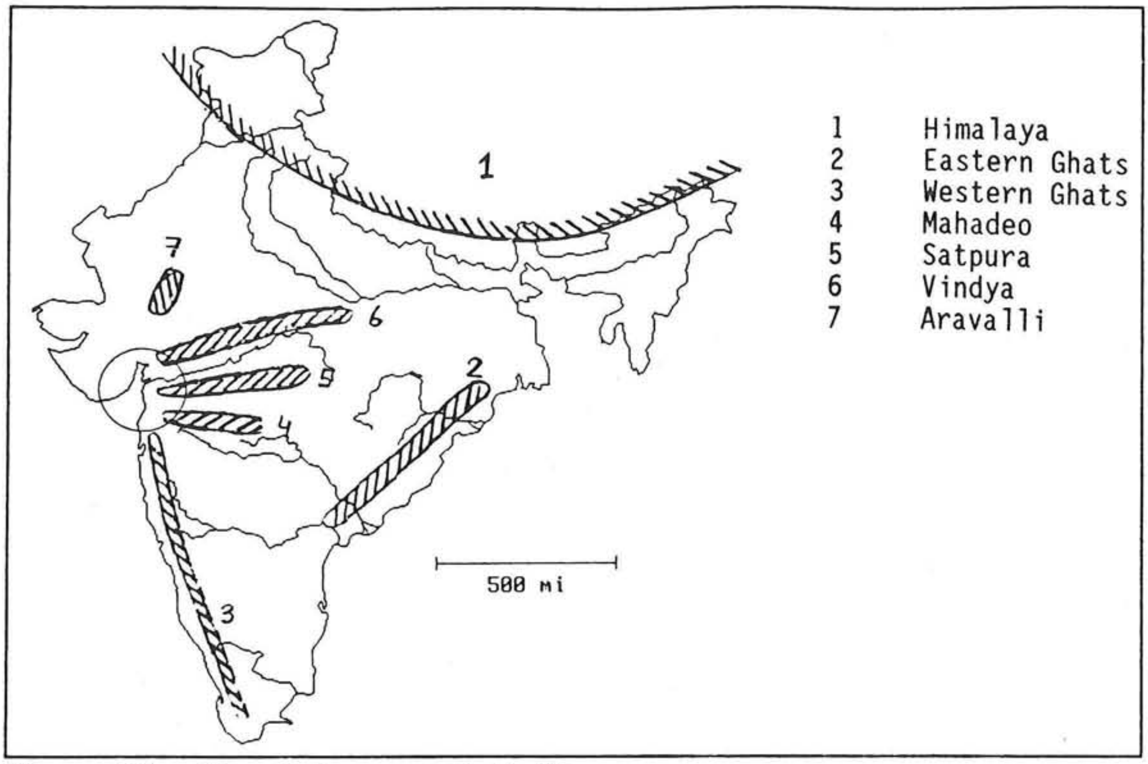


Figure 2.1 Mountain ranges in India

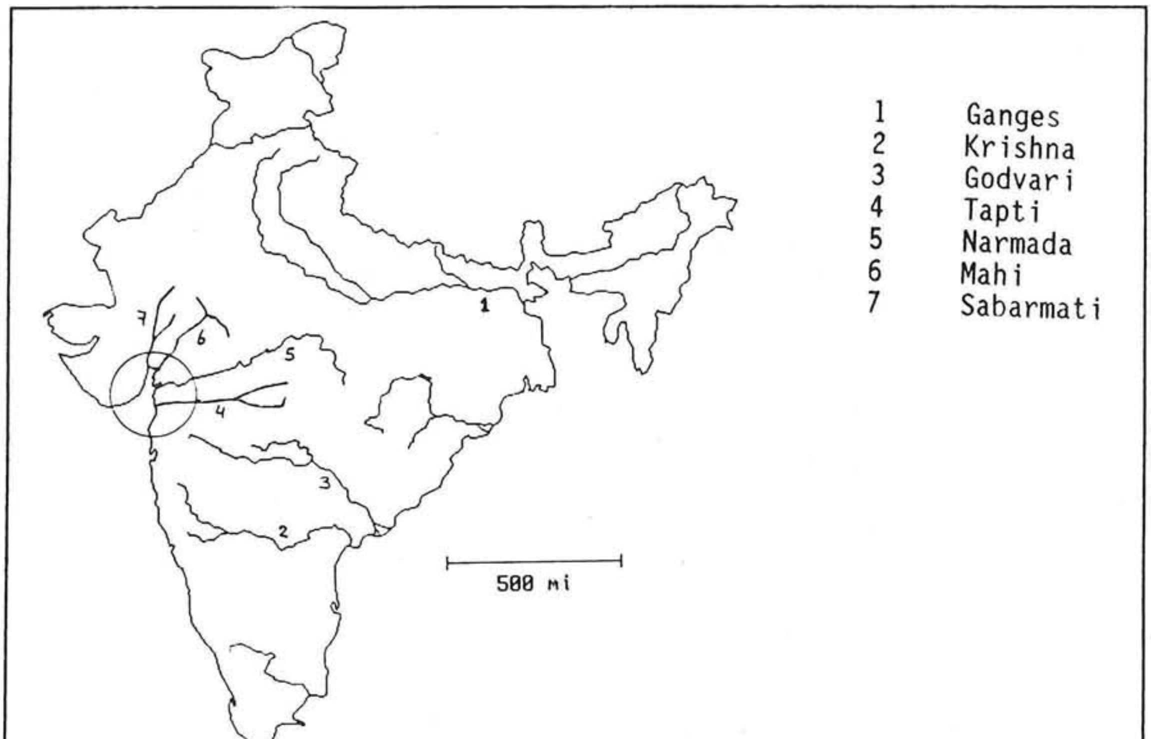


Figure 2.2 Rivers in India

Serial no.	Year	Annual Inflow from River Narmada	River Mahi	River Sabarmati	Annual Combined Inflows of these three Rivers into the Gulf
1	1915	10.22	0.32	0.15	10.69
2	1916	16.21	5.96	1.20	23.37
3	1917	21.62	8.70	1.30	31.62
4	1918	-	0.10	0.13	0.23
5	1919	23.48	3.14	0.55	27.17
6	1920	-	0.50	0.27	0.77
7	1921	-	2.12	0.46	2.58
8	1922	-	4.19	0.56	4.85
9	1923	13.64	0.68	0.10	14.42
10	1924	3.58	3.48	0.51	7.57
11	1925	-	0.35	0.21	0.56
12	1926	15.91	1.00	2.17	19.08
13	1927	-	4.34	0.94	5.28
14	1928	0.59	2.65	0.60	3.84
15	1929	-	0.76	0.49	1.25
16	1930	4.28	4.94	0.25	9.47
17	1931	17.59	8.70	0.82	27.11
18	1932	5.17	0.68	0.25	6.10
19	1933	19.44	13.20	1.38	34.02
20	1934	15.17	1.10	0.81	17.08
21	1935	-	0.74	0.33	1.07
22	1936	9.10	0.24	0.10	9.44
23	1937	12.49	5.73	0.80	19.02
24	1938	12.06	0.82	0.23	13.11
25	1939	2.26	0.42	0.16	2.84
26	1940	8.83	1.02	0.15	10.00
27	1941	-	6.93	0.76	7.69
28	1942	17.62	3.14	0.26	21.02
29	1943	11.91	1.07	0.58	13.56
30	1944	34.54	19.66	1.93	56.13
31	1945	7.95	6.03	0.66	14.64
32	1946	15.89	11.96	0.52	28.37
33	1947	11.96	0.93	0.24	13.18
34	1948	14.91	0.50	0.09	15.50
35	1949	4.79	0.80	0.20	5.79
36	1950	0.44	6.89	0.73	8.06
37	1951	-	0.28	0.13	0.41
38	1952	-	7.01	0.67	7.68
39	1953	-	0.84	0.68	1.52
40	1954	4.65	10.37	1.00	16.02
41	1955	14.22	3.53	1.28	19.03
42	1956	9.31	0.61	1.53	11.45
43	1957	-	0.50	0.08	0.58
44	1958	3.92	5.73	0.56	10.21
45	1959	16.38	1.90	1.32	19.60
46	1960	-	0.54	0.20	0.74
47	1961	30.88	9.12	0.94	40.94
48	1962	-	5.86	0.24	6.10

Table 2.1 Hydrological Series of Combined Inflow from River Narmada, River Mahi and River Sabarmati (in 10^9 m^3 / Annual Inflow).

Climate

The climate of India and its adjacent waters is dominated by the monsoons or seasonal winds. In winter, under influence of an area of high pressure over Central Asia, winds are mainly blowing from the north or north-east with a high degree of constancy: north east monsoon. Temperatures in New Delhi rise to 25° C only during this cold season.

In summer an area of low pressure is lying over the Asia-mainland and winds are mainly blowing from the south west with a high degree of constancy: south west monsoon. Early in the summer temperatures rise to 40° C and more. After June however the south west monsoon brings cloudy-unsettled and very damp weather, with much heavy rain on the west coast of India south of about 20° north and occasional gales.

The rainfall in India is divided in a very unequal way. South-west of the western ghats and south of the Himalaya the total precipitation is more than 3.000 mm, locally the precipitation can even be more then 10.000 mm. In the rest of India the precipitation is less then 3.000 mm. In the north west region of India the precipitation is even less then 500 mm: the Harr-dessert. Precipitation values for India are given in Figure 2.3.

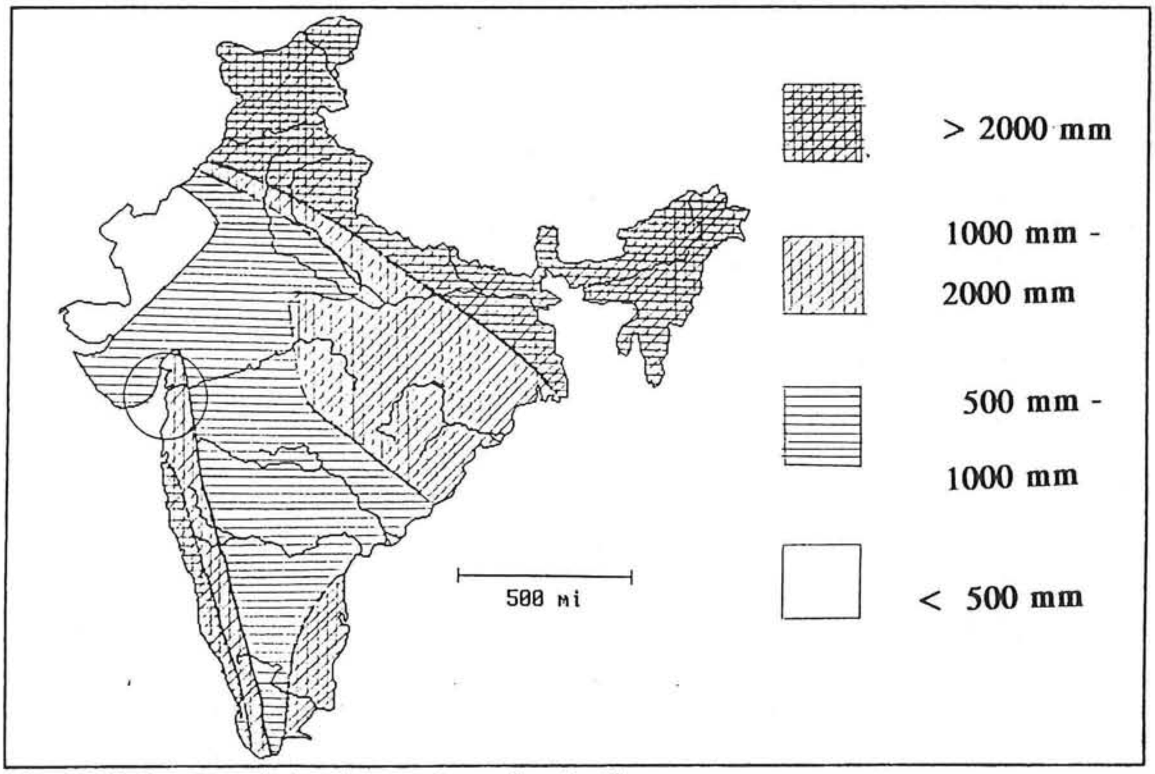


Figure 2.3 Precipitation values for India

These values are the average values. If, for example, the south west monsoon bears off to the north and reaches the province of Gujarat, the precipitation south west of the western ghats is lower then 200 mm, but the precipitation in the Gujarat exceeds the average by a factor three or four.

Land utilization

Being an agricultural country, most of the utilized soils are used for agriculture (80%). Agriculture in India means arable farming. Rice and wheat account for respectively 40 and 30%, other grains for 22% and vegetables for 8%. The precipitation determines what kind of crop can be cultivated. In places having a high precipitation rice is cultivated, in areas with a low precipitation millet, maize and peanuts are cultivated. In areas having a very low precipitation, as in west-India land is used for extensive stock breeding.

There are two major industrial areas in India. The biggest of the two is in east India, Calcutta. The other is situated north of the Gulf of Khambat.

2.2 *Gulf of Khambat*

The Gulf of Khambat is situated in the eastern part of the province Gujarat. Gujarat is one of the more thriving provinces of India. There is a large industrial zone in Gujarat and much business is taking place due to the good shipping facilities, and the long history of business in this area. The largest harbours near the Gulf of Khambat are.

*	Jafarabad	distance 120 km
**	Pipavav Bandar	distance 100 km
*	Bhavnagar	distance /-
*	Hazira	distance 60 km
***	Bombay	distance 250 km

The distance of the harbour to the Gulf of Khambat (Piram Island) is put behind the harbour name. The number of stars before the harbour name gives an indication of the size of the harbour.

Chart information

Information about the contours of the coast and the bottom lines is taken from Admiralty Chart 1486 India - West coast, approaches to Gulf of Khambat. This chart has been put together from various Indian Government Charts. These source data are not always very recent. The part of the Gulf north of Bhavnagar has been measured for the last time in 1850. The data used for the rest of the chart are more recent. Dates of surveys range from 1930 (only a very small area) to 1986 (the biggest part of the chart. Since this chart is the only obtainable source of information, it will be used as a basis for the design of the dam and tidal power station. It is recommended to arrange for a new set of data for the Khambat area if the project will proceed.

The depths in the chart are given in metres reduced to Chart Datum, which is approximately the level of lowest astronomical tide. This lowest astronomical tide (LAT) is not a horizontal plane. The LAT in the northern part of the Gulf is lower then in the southern regions due to the increased tidal range. In Table 2.2 Chart Datum is given for several harbours in the Gulf and near to the Gulf.

Location	Chart Datum below Mean Sea Level (MSL) in metres
Jafarabad	1.6
Port Albert Victor (Pipavav Bandar)	1.7
Gopnath	2.9
Bhavnagar	5.8
Luhara	4.9
Suvali (Hazira)	3.6
Daman	3.0
Bombay	2.4

Table 2.2 Chart Datum in relation to Mean Sea Level

The average depth during the tidal movements for every point in the Gulf can be determined by taking the given depth in the chart and add it to the locally difference between Chart Datum and Mean Sea Level. Interpolation between known differences gives an acceptable value.

Bottom Information

In the regions where rivers flow into the Gulf, the rivers bring very fine sediment in the Gulf. In these regions, the beach consists mostly of mud. At the west coast of the Gulf, there is only one small river discharging near Bhavnagar. This is the only location at the west coast where mud is found. The rest of the coastal bottom consists of sand or a mixture of mud and sand. At the east coast of the Gulf, several small rivers and a few large rivers flow out in the Gulf. The coastal bottom in this area consists for a very large part of mud.

For the project area information is used of two studies

The first study was spread out over the entire Gulf south of Bhavnagar. At locations between the beaches, some samples have been taken. The result of this investigation shows that the bottom is very layered. The samples were only taken to a depth of three metres, but sometimes more then ten layers could be discerned. A global result of the measurements is shown in Figure 2.4. Sand is indicated with a S, mud with a M and layers of sand and mud with M.S. For the deeper part of the Gulf, bottom borings show that the bottom consists mostly of fine sand. At some locations the sand is mixed with some thin layers of stiff clay.

The second study was more directed at the possible dam location. Near the proposed dam site (proposed in the reconnaissance study) two boring have been made to a depth of 30 metres below mean sea level. The borings can be found in Annex I. These two borings were used as a basis to describe the sub-soil for the dam. Three bore-logs were drawn for the different sections of the dam. One for the (deep) section in the middle of the Gulf of Khambat, one for the shallower parts of the dam location and one for the sections above Bench Mark (during ebb these sections have run dry). The boring for the shallower parts of the dam location is shown in Figure 2.5.

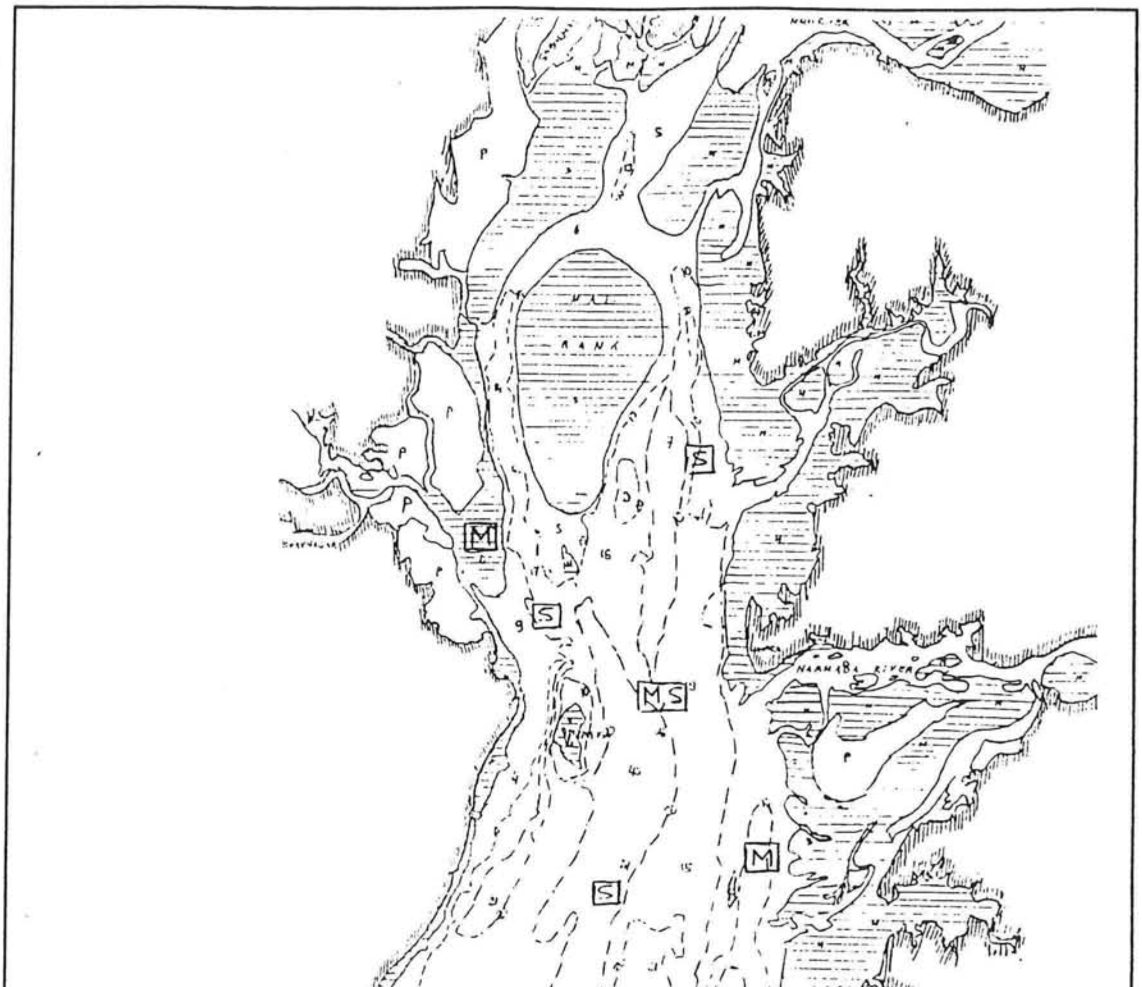


Figure 2.4 **Result and location of the low penetrating borings in the Gulf of Khambat**

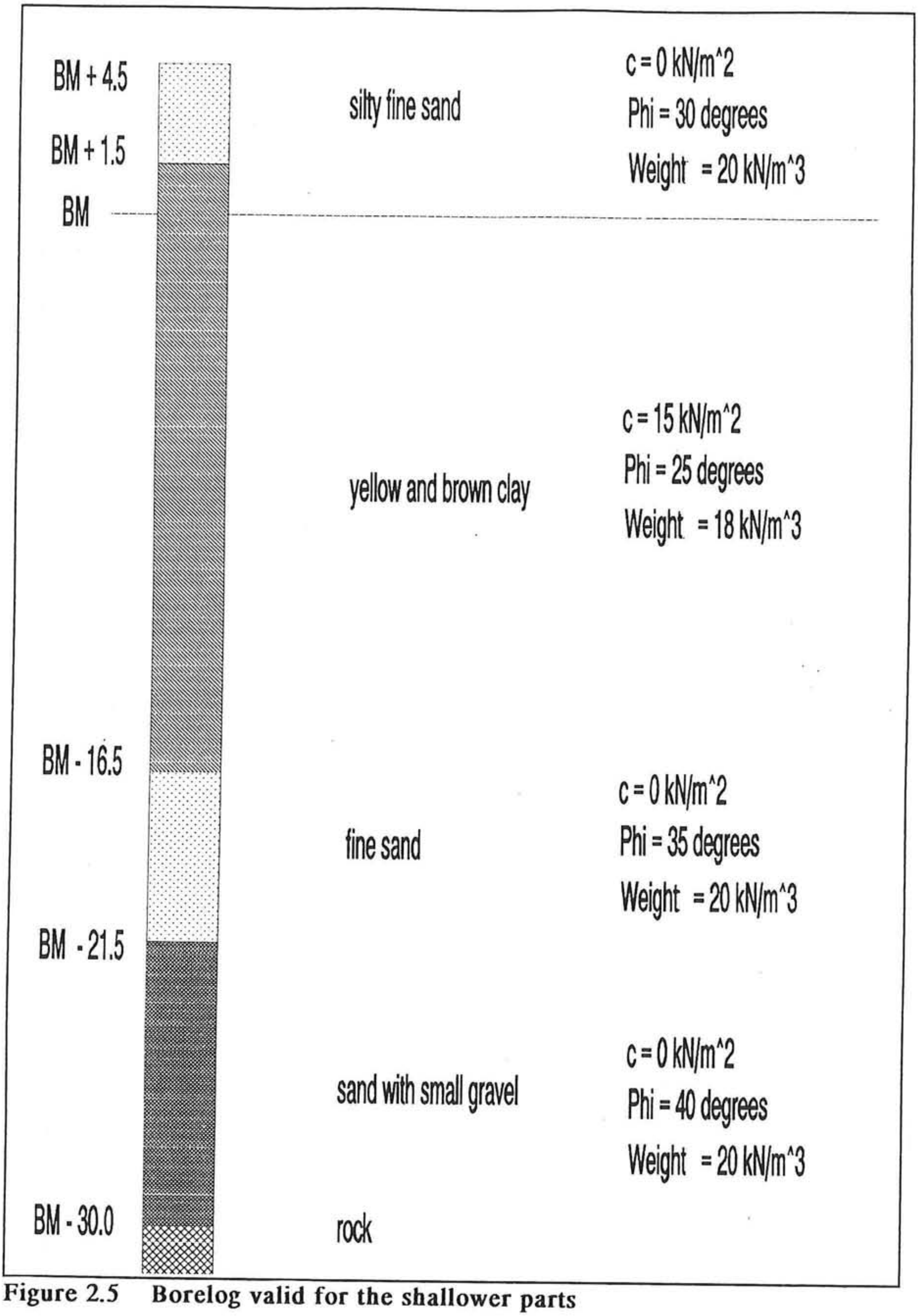


Figure 2.5 Borelog valid for the shallower parts

Even deeper then the alluvial depositions (several tens of metres below the bottom surface), the tectonic map shows a non-active north south fault in the centre of the Gulf and two non-active east west faults, one at the northern edge of the Gulf and one along the axis of the Narmada river. These faults are overlain with Tertiary and Quarternary rocks and have not shown much activity the last two centuries.

Climate

The province of Gujarat has a tropical continental climate. It is strongly influenced by monsoon winds. The rain season runs from June to October. Rainfall takes place in short downpours. Regular rainfall can only be expected in July and August (one rainy day every two or three days). This strongly concentrated rainfall causes temporarily very high discharges in the rivers with a high probability of flooding and erosion. The remaining part of the year there is usually not much rain. In the Khambat region the annual precipitation is about 1.000 to 1.500 mm. 80% of this total falls in the months July, August and September.

Wind and Waves

Wind and wave characteristics in the Khambat region are also dominated by the monsoon climate.

From December to February, when there is a high-pressure area over the mainland of India and a low pressure area over the north east of Africa, the winds blow from the north east. Wind is blowing from the irregular land surface and is therefore not very strong (4 m/s), thus waves in the Gulf of Khambat are not high because of the low wind speed.

From March to May the wind direction turns from west to north and the wind increases to 6 metres per second (average). In this situation there is still no strong wave field in the Gulf because of this low wind speed and the limited fetch. The maximum significant wave height with a wind speed of 6 metres per second and a fetch of 40 km is 0.8 m.

From June to September the average wind speed is 10 m/s and the wind direction is SW. In this situation the significant wave height reaches its maximum value. The high wind speed and unlimited fetch (unlimited for waves in the Gulf of Khambat) give an average significant wave height of 1.5 m.

From October to November the wind speed falls back to less then 5 m/s with no stable wind direction. Wave heights are limited to 1.0 m.

Studies for the Gulf of Khambat have been made by Delft Hydraulics (lit [26]). Results of these studies are presented in the next paragraph. The design conditions for wind and waves occur in the months June, July and August when wind speed and wave height can reach extreme values due to tropical cyclones. Data from weather measurement stations in the Gulf show that the chance of a tropical cyclone reaching the Gulf of Khambat with a wind speed of 26 m/s or more is 4.3 %. The chance of a cyclone with a wind speed of more then 30 m/s passing the Gulf is 2.6 %.

For more detailed information about wind speed, data are extrapolated of data from the Bombay weather station. Differences between Bombay and the Gulf of Khambat are governed by a multiplying factor. This factor is determined as 1.3.

Following from the study of the hourly wind data of Bombay applied to the Gulf of Khambat.

50 % chance of exceedance: 19 m/s.
10 % chance of exceedance: 26 m/s.
2 % chance of exceedance: 34 m/s.
1 % chance of exceedance: 39 m/s.

Near to the dam site a reduction is proposed of ten percent in view of the sheltering effect of Cathiawar. The new numbers become:

50 % chance of exceedance: 17 m/s.
10 % chance of exceedance: 24 m/s.
2 % chance of exceedance: 31 m/s.
1 % chance of exceedance: 35 m/s.

Wind speeds can increase considerably during a squall. A squall is defined as a sudden increase of wind speed by at least 3 stages on the Beaufort scale, whereby the wind speed reaches at least 11 metres per second (force 6 Beaufort) and remains so for at least one minute. The sudden increase of wind may take less then one minute. These squalls occur during the passage of huge convective clouds and are mostly connected with intensive rain showers and often with thunderstorms. Here they occur almost exclusively during the south west monsoon (May to September) with a maximum in July. Most of the squall last less then 15 minutes. For the wind speed during a squall, multiplication factors are used.

Period considered 5 seconds ("gust") -> factor 1.40
Period considered 1 minute -> factor 1.24
Period considered 10 minutes -> factor 1.10

Extreme wave heights to be exceeded on the average with a chance of 10 %, 2 %, or 1 % only are expected to be always associated with the extreme wind speeds of tropical cyclones. It must be stated beforehand that the estimation of probabilities of extreme wave heights associated with tropical cyclones is still a more speculative matter than the estimation of the probabilities of extreme wind speeds, because reliable observations of high waves in tropical cyclones are very difficult and therefore rare.

To translate the given extreme wind speeds in extreme wave heights, two calculations have been made. For the first equations given in the Shore Protection Manual (pp 3-77, 3-88) are used. These equations can be applied for wave prediction in deep water as a result of cyclones. The second calculation is made with the computer program HISWA (HIndcast Shallow water WAves).

Calculation with the equations in the Shore Protection Manual

As was mentioned before, the equations can be applied for wave prediction in deep water as result of cyclones. Since the waves in the Gulf of Khambat are not generated in nor travelling through deep water, the calculated waves are too high. Equations for wave prediction as result of tropical cyclones in shallow water have not been developed. To get a more realistic value for the wave height, and the wave period in the Gulf, the wave heights, and the wave periods calculated from the deep water equations are converted to the shallow water conditions (depth is 25 meters average).

The conversion takes place as follows:

1 With the calculated wave height for the deep water situation and the wind speed an equivalent fetch can be determined (the fetch combined with the wind speed would give the calculated waves).

2 With this equivalent fetch and with the given wind speed a wave prediction can be made for shallow water, the wave prediction consists of a prediction for the wave height and for the wave period

This process of wave prediction is presented and can be checked in annex II: Calculation wave prediction for the Gulf of Khambat

The result of the calculations are presented in Table 2.3:

chance of exceedance	wave height in metres	wave period in seconds
10 %	4.4	9.5
2 %	5.1	10.0
1 %	5.5	10.5

Table 2.3 Predicted significant wave heights and wave periods for the Gulf of Khambat with the equations of the Shore Protection Manual

These values for the maximum wave height are given with much reserve. Especially the wave heights near the dam site can vary because of limiting factors as:
- limited fetches,
- shallowness,
- turbulent currents.

So far all considerations and conclusions refer to the significant wave heights. It is well-known that some individual waves are much higher than the significant wave height and that the probable highest individual waves increases with the duration considered. In order to estimate the probable highest individual waves associated with the extreme significant wave heights as given earlier in this chapter by applying the rayleigh distribution the following should be done.

1 Determine the duration of the maximum wind speed. The duration of this wind speed is the size of eye of the cyclone divided by the speed of the travelling cyclone. In the former calculations the size of the eye of the cyclone is assumed as 80 kilometres, and the travelling speed of the cyclone as 10 metres per second. The duration of the storm then becomes 5400 seconds.
2 Divide this duration time by the most probable wave period to get the total number of waves.
3 The multiplication factor for the significant wave height to get the maximum wave height can be determined from Equation 2.1.

$$H_n = 0.707\ H_{sig} \sqrt{\log \frac{N}{n}} \tag{2.1}$$

In annex III the maximum wave heights are calculated. The results can be found in Table 2.4.

change of exceedance	maximum wave height in metres
10 %	7.9
2 %	9.1
1 %	9.7

Table 2.4 Predicted maximum wave heights for the Gulf of Khambat with the equation of the Shore Protection Manual

Calculation with the HISWA computer model

In the computer model calculations with wind speeds of 31 m/s, 24 m/s and 17 m/s have been conducted.The results for the calculation with 31 m/s are given in Figure 2.6. There are large differences between the two different calculations (the equations of the Shore Protection Manual and the HISWA model). The reason for these large differences lies partly in the (in)validity of the equations of the Shore Protection Manual. As was mentioned earlier in this section the equations are only valid in deep water. The conversion to shallow water does not seem to give the desired result. An other reason for the difference is that the computer model takes refraction and diffraction into account. The calculation with the SPM equations are made with average water depths. The figures as result of the HISWA model have to be considered as more reliable.

The maximum wave height for the different maximum wind speeds are given in Table 2.5.

chance of exceedance	maximum significant wave height (m)	wave period (s)
10 %	2.4	6.0
2 %	3.0	7.0
1 %	3.8	7.5

Table 2.5 Predicted significant wave heights and wave periods for the Gulf of Khambat with the HISWA model

The significant wave height presented in Table 2.5 is only valid for a small part of the proposed dam site. The wave height near Bhavnagar is considerable lower due to the sheltering effect of Piram Island. Maximum wave heights are reduced to 2.0 metres and lower. Near Alia Bet the wave height is also considerable lower than 3.5 metres due to the shallowness of this region. If the direction of the incoming waves and wind is varied the maximum significant wave height near Bhavnagar and Alia Bet does not rise noticeable.

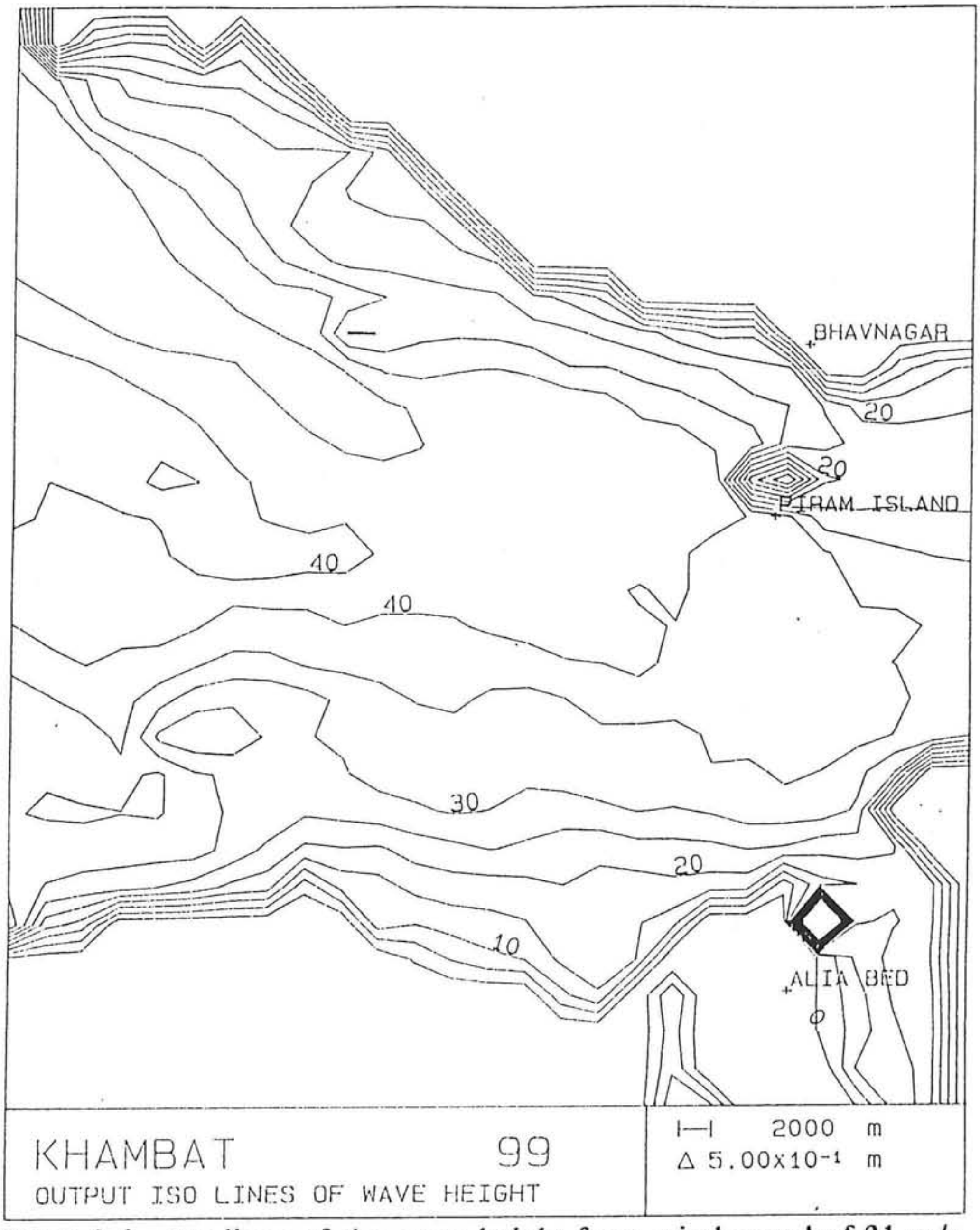


Figure 2.6 Iso lines of the wave height for a wind speed of 31 m/s

For the maximum individual wave height the same equations hold as are used in the previous section. The time in which this maximum individual wave can occur is in the time frame in which the water level is high enough to have the maximum waves calculated in HISWA. The water level for these calculations were set at B.M. + 8 metres. The length of this time frame is approximately 2 hours. The number of waves that reach the area of interest is 2 * 60 * 60 / T. The wave period varies with the wave height as does the multiplication factor for the maximum individual wave.

change of exceedance	maximum wave height in metres
10 %	3.0
2 %	3.7
1 %	4.6

Table 2.6 Predicted maximum wave heights for the Gulf of Khambat with the HISWA computer model

Again the values of the maximum individual wave height as result of the HISWA calculation have to be considered as more reliable.

The waves calculated with the Hiswa computer model may be more reliable, they are however not perfect. The HISWA model can not calculate correctly with a double frequency peak. So if waves generated in the Arabian Sea reach the Gulf of Khambat, these waves break over the shallow parts of the Gulf of Khambat, causing waves having decreased wave height but having the same relatively high wave period. Waves generated in the Gulf of Khambat region are new and have a relatively low wave period (and a high steepness). In the computer model these two wave fields can not be combined but are calculated more separately than they should be resulting in lower resulting waves. The wave height that will be used for construction of the dam and the caissons is a wave height somewhat higher than the computer results, the significant wave height with a change of 1 % of exceedance will be set at 5.0 metres, with a wave period of 10 seconds. The maximum wave height with this accepted significant wave in the time frame of two hours is 6.0 metres.

In extreme storm surges (the wind blowing from the south and exceeding 20 m/s) a rise of the sea water level as result of wind set up can be expected of 2.0 metres. [see lit. (5)]

3 Tidal movement

3.1 Tides in general

On the oceans a periodic rising and falling of the water level occurs. This phenomenon is known as tide. In this section the causes of tidal constituents will be discussed.

3.1.1 Astronomical Tides

The tidal movement is driven by the gravitational attraction of the moon and sun acting on the rotating earth. The influence of other celestial bodies has proved to be negligible.

The most important contribution in the tidal movement is the attraction of the moon. Although the mass of the sun is much greater than that of the moon, the influence of the latter more than doubles that of the sun because of the relatively short earth-moon distance. In the earth-moon system there are two types of forces: attraction and centrifugal forces (Figure 3.1). Newtons law of gravitation tells us that the earth and the moon exert attraction forces upon each other. These forces are balanced by the centrifugal forces due to the rotation of the two bodies around their common centre of mass. This equilibrium exists only for the earth as an entity. The distribution of the forces over the earths surface is characterized by resultant forces which cause high water on the line extending from the moon through the centre of the earth. When we assume that the earth doesn't rotate, the time of revolution of the moon is 24 hours and 50 minutes. This means that high tide occurs every 12 hours en 25 minutes. The described, cosine-shaped, tidal constituent is called the M_2-tide.

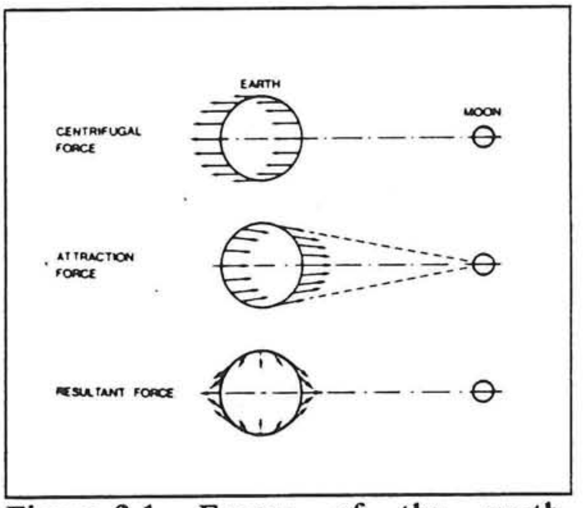


Figure 3.1 Forces of the earth-moon system [lit. (1)]

The influence of the sun can be explained in the same way. This is the S_2-tide which has an amplitude less than half the M_2 amplitude. The period of this constituent is 12 hours. The difference in period of the M_2 and S_2 constituent causes the tidal range to vary. When earth, moon and sun are aligned the M_2 and S_2 constituent have their maximum (or minimum) at the same time which results in a high tide that is higher than the average. In the same way low tides are lower than the average. This situation, which is called spring-tide, takes place every 14,3 days at respectively full and new moon. At the other extreme, when M_2 an S_2 are out of phase at the moons first and third quarter, the tidal range is small; neap-tide.

A characteristic which has been neglected so far is that the plane of the moons orbit is at an angle with the plane of the earths equator. This angle is called the moons declination.

The same counts for the sun. This diurnal phenomenon is described by the K_1, O_1 and P_1 constituents. K_1 is a combination of the influence of the sun and the moon. O_1 and P_1 are respectively solitary lunar and solar constituents. Because of the diurnal constituents two successive high tides are not

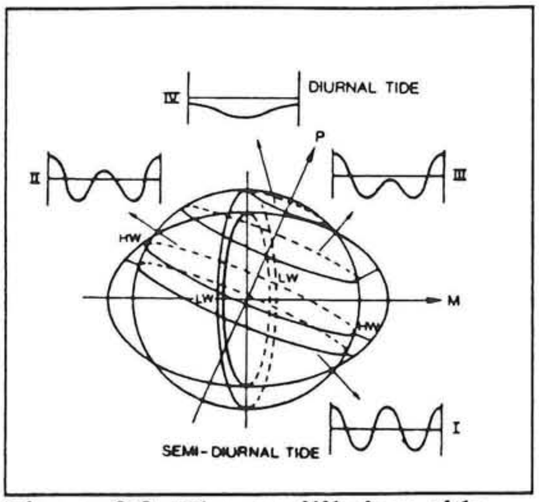


Figure 3.2 The equilibrium tide [lit. (1)]

similar in height. This is called daily inequality. At some places the diurnal tides even dominate the semi-diurnal tides (Figure 3.2).
The declination of the sun and moon is not constant in time. It changes during the orbital cycle. This is described by the semi-diurnal K_2 constituent. The distance from the moon to the earth also varies. This effects in varying tide-raising forces which are taken into account by a semi-diurnal N_2 constituent. The moon causes also a wave with a period of one cycle per fortnight,the M_f constituent, which is mainly significant in shallow water.

The initial tidal wave runs around the world on the southern hemisphere (the only world spanning sea-area) at about 65 ° latitude. There are two major waves synchronous with the moon, one wave top under the moon and one diametral across. The cd of the earth at 65 ° latitude is c. 16,000 kilometres, therefore the wave is about 8,000 kilometres.

This generated wave runs along the southern borders of the oceans and propagates northwards into these oceans.

3.1.2 Meteorological and shallow water tides

In the previous section the tide is described as a combination of astronomical constituents. In reality the ocean itself also plays an important role. The variations of the water-level propagate with a certain wave speed which depends on the water depth. When tidal waves reach shallow waters they are influenced by the bottom topography and coastline geometry. This influence is important when the non-linear terms play a part in the dynamic equations which describe the propagation of the tides. Two non-linear phenomena are described below:

- Because there is a difference in depth at low and high water the trough of the tidal wave is more retarded than its crest. This causes distortion of the tidal wave.

- The bottom friction is proportional to $u|u|$, where u is the flow-velocity.

Both phenomena are described by introducing over-tides of respectively four and six oscillations a day (M_4- and M_6-component).

Other non-linear effects originate from interaction between the astronomical constituents. This generates compound tides with frequencies equal to the sum or the difference of the frequencies of the original constituents. Over-tides do not have an astronomical background but they can be very important for coastal seas and estuaries.

The variations of the water-level due to tides are also influenced by meteorological phenomena like atmospheric pressure changes and wind stresses. The influence of these effects has not been taken into account for this project.

3.2 Tides in the Gulf of Khambat

The initial tidal wave, with a wave-length of 8,000 kilometres, runs along the southern borders of the oceans and propagates northwards in these oceans. The propagation of the tidal wave in the Indian ocean can be seen in Figure 3.3. The wave speed of the tidal wave can be calculated with Equation 3.1. (Shallow water equation, because L = 8,000 kilometres; D = 4,000 metres)

$$c = \sqrt{gd} \tag{3.1}$$

In the present situation the following values were used:
$g = 9.8\ m/s^2$
$d = 4{,}000\ m$
this results in: $c = \sqrt{9.8*4000} = ca.\ 200\ m/s$

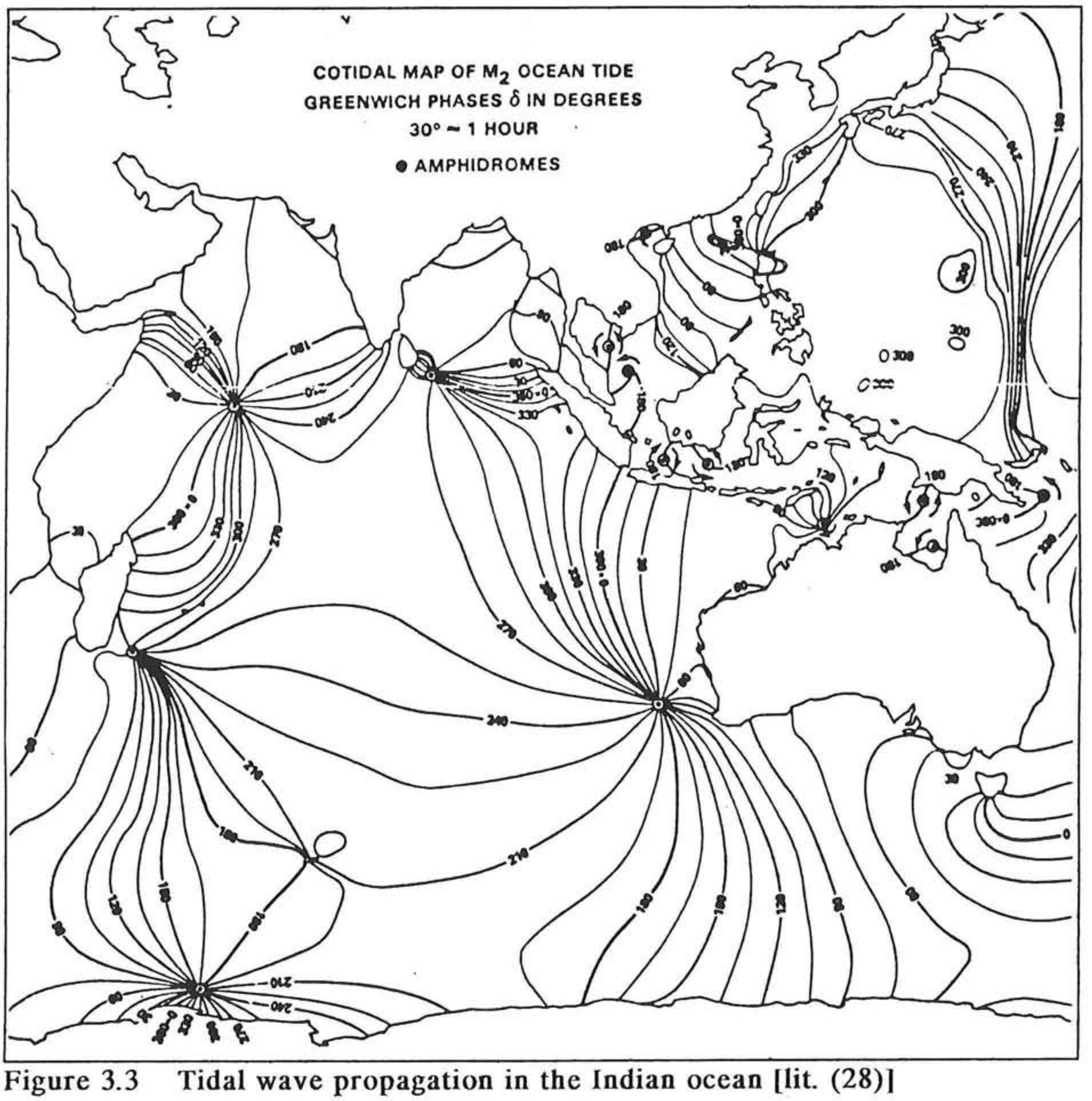


Figure 3.3 Tidal wave propagation in the Indian ocean [lit. (28)]

The tidal wave reaches the Gulf of Khambat some 16 hours after departure from the place of origin.

The Gulf of Khambat is characterized by a large tidal difference. This varies, for an average spring-tide, from 5.1 m at Bulsar to 8.8 m at Bhavnagar[1]. The location of these places, and others mentioned in this chapter can be found in Figure 1.2, in chapter 1. For a neap-tide these values are respectively 2.4 and 4.8 m. Further to the north the tidal difference is even bigger. When we take the tide at Bulsar as a reference it is possible to determine an amplification-factor f which is the ratio between the tidal differences at Bhavnagar and Bulsar. The value of this factor f is 1.7 for a spring-tide and 2 for a neap-tide.

Besides a difference in amplitude there is also a difference in time when high water is reached. High tide in Bhavnagar occurs (according to the Admiralty Tide Tables [lit. (2)]) approximately two hours later than in Bulsar. Dependent on the tidal amplitude at a certain moment this value can be 15 minutes more or less. The distance between Bulsar and Bhavnagar is 120 km.

In Section 3.1 the astronomical constituents of the tides were treated in general. In Table 3.1 the amplitudes of the most important tidal constituents of three places "in" the Gulf of Khambat are presented.

heights in m	Bulsar	Pipavav Bandar	Bhavnagar
amplitude M_2	1.86	0.88	3.14
amplitude S_2	0.72	0.35	0.96
amplitude K_1	0.74	0.50	0.76
amplitude O_1	0.25	0.23	0.34
amplitude ¼-diurnal	0.02	0.04	0.03

Table 3.1 Tidal amplitudes [lit. (2)]

At the southern entrance of the Gulf of Khambat a big difference in tidal ranges exists when we compare the eastern and the western edge of the estuary. When we consider a normal tide, the average of a spring and a neap tide, the tidal ranges at Bulsar and Pipavav Bandar are respectively 3.6 an 1.8 m. The difference of 1.8 m. can be explained by the Coriolis effect. Due to the rotation of the earth a force (Coriolis-force) is acting perpendicular to the streamlines. At the northern hemisphere this force is acting to the right. The Coriolis-force is balanced by a slope of the water-level that can be described as a pressure-gradient.

$$\frac{1}{\rho_w}\frac{dp}{dn} = F_{Cor} \tag{3.2}$$

$$F_{Cor} = 2\ U\ \omega_e\ \sin\varphi \tag{3.3}$$

1 *This values represent the differences between the averages of successive high and low waters.*

with:

U = flow velocity, estimated value from Duflow calculations: 1 m/s

ω = angular velocity of the earth = 0.73 * 10 $^{-4}$ rad/s

φ = geographical latitude, 21°

$$dp = dh\,\rho_w g \tag{3.4}$$

With Eq. 3.3, Eq 3.4 and some algebra Eq. 3.2 becomes:

$$\frac{dh}{dn} = \frac{2U\omega_e \sin\varphi}{g} \tag{3.5}$$

Together with the estimated values of the variables, dh/dn can be computed:

$$\frac{dh}{dn} = \frac{2*7,3*10^{-5}\sin 21°}{9.8} = 5.3*10^{-6}$$

The distance between Bulsar and Pipavav Bandar is approximately 150 km. This results, with u is 1 m/s, in a difference in water-level of:

$$\Delta h = 150*10^{3} * 5.3*10^{-6} = 0.8\ m$$

When we consider a rising tide the highest point of the slope is at Bulsar. When the direction of flow is reversed, at falling tide, the direction of the slope is reversed too. Now the lowest point is at Bulsar. Thus, the difference in tidal range of both locations should be equal to two times the computed value of 0.8 m., which gives 1.6 m. This value is close to the existing difference of 1.8 m., mentioned above.

Points of interest for this consideration is the used value for U, and the phase difference in high water caused by tide and high water caused by Coriolis. The value for U differs over the stretch Bulsar - Pipavav Bandar, so a more differentiated calculation should be made. Furthermore a little difference in the mean velocity changes the difference in water level linear.

The maximum Coriolis force, and maximum water-slope, takes place when the velocity of the water is at its maximum. This maximum velocity does not take place at high (or low water) but some hours before. This phase difference can be overcome if the inertia of the water-mass is considered. Some reservation concerning the calculated water-difference should have been made if the calculated values were not in agreement with the given values in the Admiralty Tide Tables [lit. (2)].

3.3 Tide modelling with DUFLOW

3.3.1 The existing situation

network

To be able to carry out a one-dimensional Duflow calculation, the Gulf of Khambat was schematized as a network consisting of 39 nodes which are connected by 51 channels (see Figure 3.4).

boundary-conditions

At the boundaries of the model one has to introduce boundary conditions. At the seaward boundaries of the model, being nodes 37, 38 and 39, the tidal wave is introduced. The conditions at Bulsar (Valsad) are considered to be representative for the seaward boundary. An average spring-tide is introduced by means of a Fourier-series. The daily inequality (difference between two successive high tides caused by the moon's declination) is included in this boundary-condition.

At the other boundaries of the model a discharge of 0 m^3/s is assumed which means that the river discharges are neglected.

results

The results of the calculations described in appendix A, section A2 are presented in Figure 3.5 and Figure 3.6. Figure 3.5 shows a gradually increasing tidal difference, when we go northward from the seaward boundary into the Gulf of Khambat, which rather accurately matches with the situation as predicted in the admiralty tide tables. The ratio of the tidal differences at Bhavnagar and Bulsar (Figure 3.4) can be computed using the values at nodes 38 and 11. The amplification-factor has a value of 1.7 which matches with the roughly estimated value [see section 3.2].

The discharges[1] show an also estimated gradual decrease (Figure 3.6). For further information about the DUFLOW-calculations, e.g. results, interpretation and sensitivity analysis, the reader is referred to appendix A.

[1] Note: A positive discharge is a flow out of the Gulf of Khambat

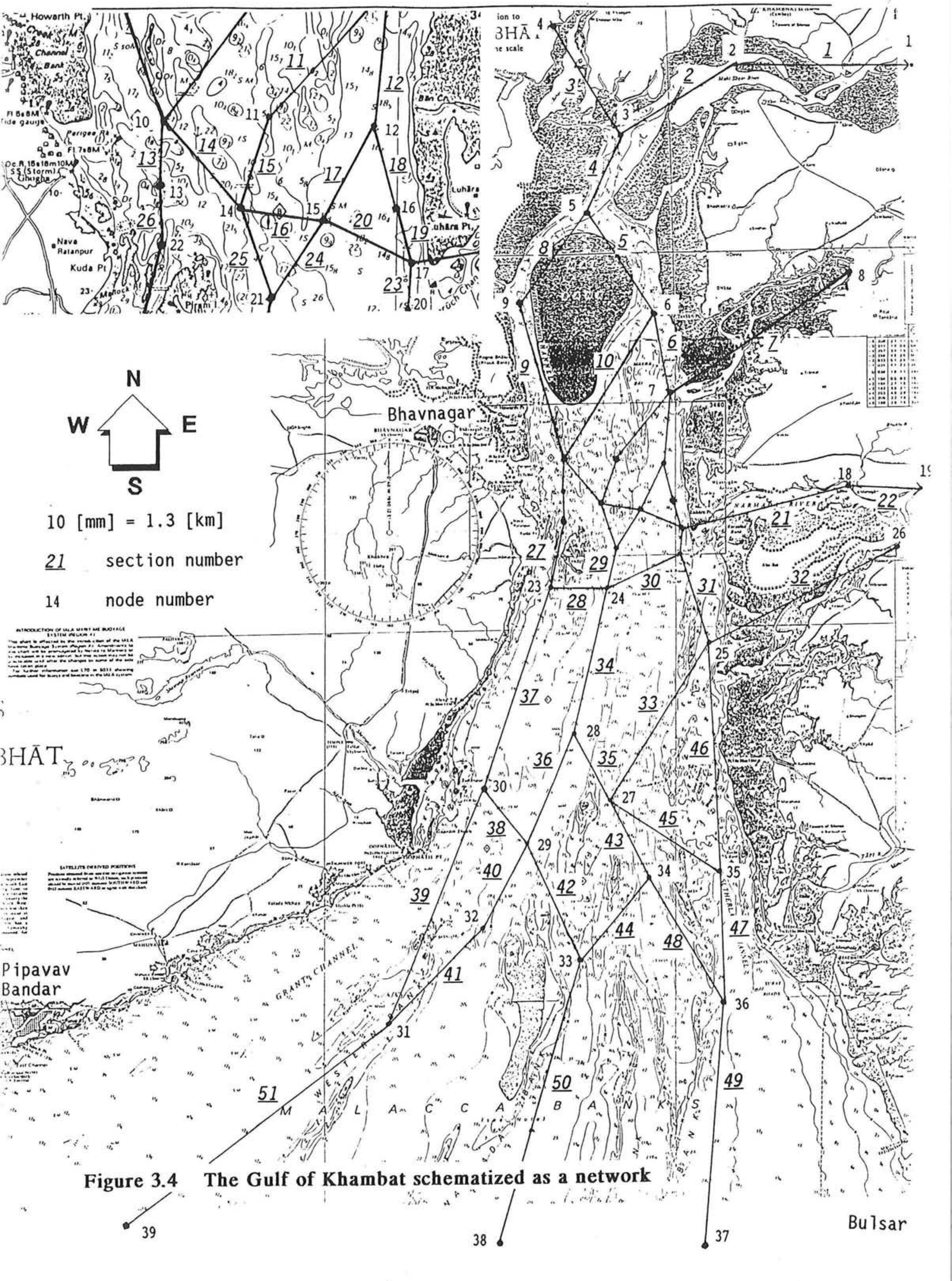


Figure 3.4 The Gulf of Khambat schematized as a network

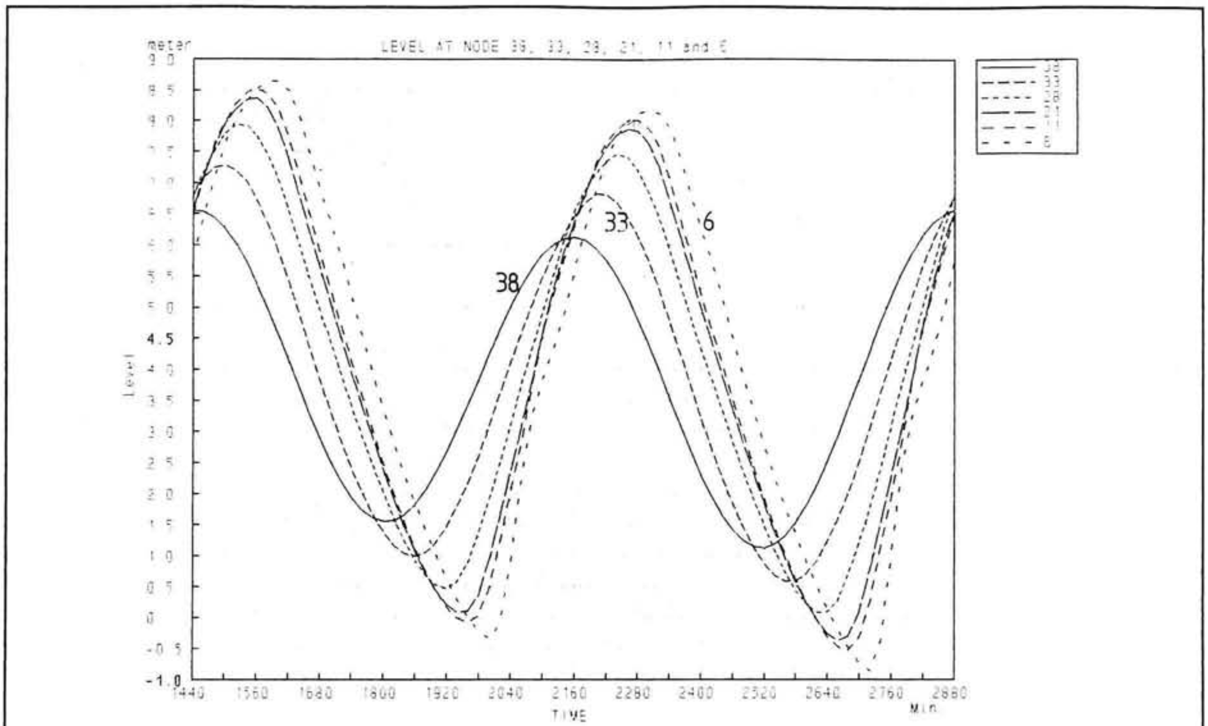


Figure 3.5 Tidal amplitudes for several nodes in the Gulf of Khambat

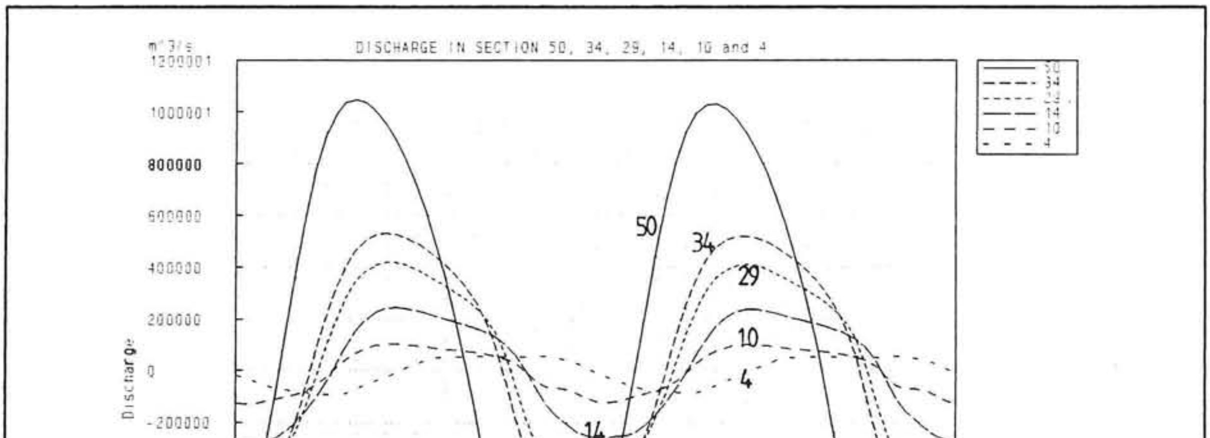


Figure 3.6 Discharges along the axis of the Gulf of Khambat

3.3.2 Gulf partly closed

By closing a suitable set of sections (which cross a possible dam-alignme-nt) the influence of the closure-dam has been investigated. Of the two routes which have been used one represent an extreme location of the dam in the north, the other one is a realistic route in the south. The southern alternative does not differ much from the proposed routes in the reconnais-sance report.

The construction of a dam in the Gulf of Khambat will increase the tidal differences.

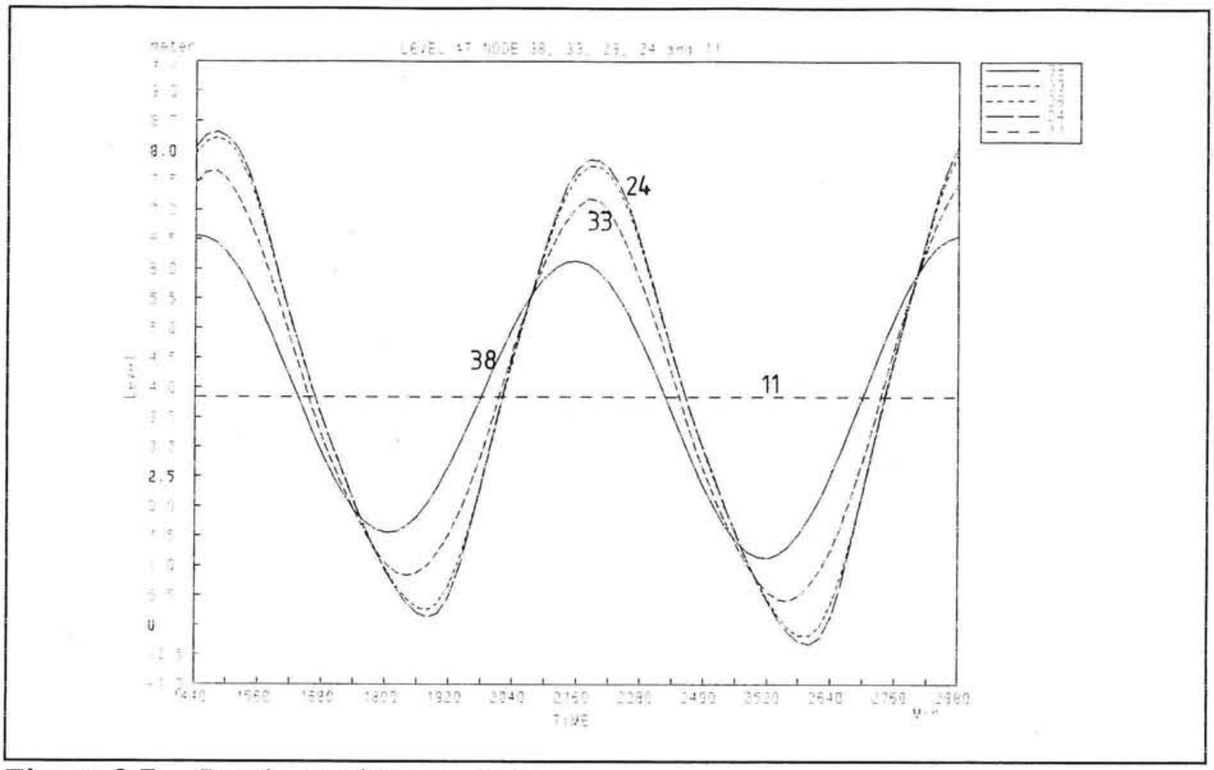


Figure 3.7 Southern closure route

Comparing Figure 3.7 with Figure 3.5, it shows that the tidal range for node 28, close to the dam, has increased. In Figure 3.5 it did amount to 7.4 m. (7.9 to 0.5 m.) and here it is 7.9 m. (8.2 to 0.3 m.).

From the calculations it became clear that a location of the dam further to the north (not too far, because than the damping becomes stronger) results in a bigger tidal difference in front of the dam. This is important for the tidal power station. A choice for a location of the dam only based on the expected tidal range would result in a route rather far to the north (just avoiding damping), provided that the depths are sufficient to transport the water.

3.4 Note about the reference-level

In the DUFLOW-model of the tidal movement in the Gulf of Khambat a fixed reference-level has been used. For this level **Chart Datum** at Bulsar was chosen. Chart Datum is defined as approximately the level of lowest astronomical tide. This implies that Chart Datum is not a horizontal plane because the lowest astronomical tide is a local feature.

In the reconnaissance report all levels were related to Chart Datum without further specification. Undoubtedly a place near the dam-alignment was chosen, for instance Piram Island, Bhavnagar or Luhara. This level is much determined by local conditions. The difference between Chart Datum at e.g. Piram Island and at Bulsar is determined by two effects:

a - The difference in still-water level;

b - The difference in tidal amplitudes.

sub a:

The still-water level in the Gulf of Khambat is not horizontal. It shows a slope. In the table of annex A-II of appendix A the still-water levels of Bulsar (node 38) and Piram Island (node 21) can be looked up (for an average spring-tide). They amount respectively to 3.88 and 4.07 above C.D. at Bulsar. This is a difference of 0.19 m. When an extreme spring-tide is considered it amounts to 0.30 m..

sub b:

The second effect is the increase in tidal amplitude. This varies, for an extreme spring-tide, from 3.84 m. at Bulsar to 5.50 m. at Piram Island.

Chart Datum near Piram Island can now be approached using C.D. at Bulsar and the mentioned values. It can be estimated that C.D. at Piram Island lies 3.88 + 0.30 - 5.50 = 1.32 m. below C.D. at Bulsar.

Because of the construction of the closure-dam the tidal amplitudes and the still-water level in the neighbourhood of the dam will change. Then Chart Datum will have to be determined again. Because of these effects and to prevent mistakes in the calculations it is most reliable to maintain C.D. at Bulsar as the reference for the closure-operation, the dam-heights etc. Relating heights to a reference-level which is a feature of a place at considerable distance may seem a problem. This can be solved however by placing a benchmark near the construction-site. Further in this report except for the DUFLOW-calculations, levels and heights will be related to **Benchmark [B.M.]**, which is the same level as Chart Datum at Bulsar. If only the term Chart Datum is used this is always meant to be the Chart Datum at Bulsar.

4 Design of the closure dam

4.1 Determination of the dam-alignment

The optimum alignment of the closure dam depends upon a lot of different factors which have technical, environmental and economic backgrounds. Here, a set of possible alignments is determined, based on some basic assumptions. These alternatives will be compared using a rough calculation of the quantities of building-material necessary to construct the dam. It is supposed that the difficulties with the final closure of the dam will not differ much between the alternatives because the same tidal channels are crossed. So the quantities of building-material can be considered as a measure for the costs of the dam. Costs do form an important selection-criterion in the choice between different alternatives.

4.1.1 Contributing factors

The first and most determining factor for the alignment is how well the dam will comply with the main purpose of the dam. Based on the proposed development scheme, this main purpose is twofold in the Gulf of Khambat namely:

- the construction of a tidal power station;

- the creation of a fresh water basin.

Some complementary factors that also can contribute to the determination of the alignment are the crossing of the Gulf of Khambat, and the development possibilities of Bhavnagar Port.

In the preceding chapter it was concluded that the tidal difference in front of the closure dam is larger when the dam is projected further to the north. On the other hand the depth of the gulf decreases in this direction. When the area of the flow profile of the intake works is kept constant they will have to be wider in order to compensate for the smaller available depth. So one has to find an optimum between the tidal difference and the water depth needed to construct and exploit a tidal power station. A study by E.M. Wilson (1975) showed that the implementation of a tidal power scheme north of Bhavnagar is not feasible because of insufficient water depth.

The second goal of the project is to create a fresh water basin. A big area (i.e. a more southern alignment) results in a large storage potential. A smaller quantity of fresh water originating from the Narmada river will have to be spilled. Another reason to project the dam far to the south is the inclusion of the Narmada-mouth. This river has the biggest discharge of the three rivers which flow out into the Gulf of Khambat. The Narmada will deliver a considerable quantity of fresh irrigation-water and will also speed up the desalinization process. Without inclusion of the Narmada-mouth a large storage potential will be useless.

The construction of a dam in the Gulf of Khambat provides a possibility to decrease transport costs in the area considered. A road or railway can be constructed on top of the closure dam. This will considerably reduce the transport distances between e.g. Bombay and the Saurashtra peninsula. The connection with the existing infrastructure is not expected to form a real problem. Also during construction the dam must be accessible for construction traffic.

The environmental impact of the closure dam will be considerable and should be investigated thoroughly. The outcome will probably not differ much for the possible alignments when the proposed development scheme is maintained. This scheme implies a fresh water basin, production of tidal power energy and gulf crossing. The environmental impact may lead to a decision to carry out the project on a much smaller scale, for instance the construction of two small separated basins excluding gulf crossing. One basin can be used for irrigation and another one for tidal power energy. This alternative however is not investigated here.

The features discussed before are more indicative than that they determine the alignment of the dam. Factors which determine the location of the dam in detail are discussed below:

- <u>length of the dam</u>

  The length of the closure dam can be minimized in order to limit costs of material and construction time. However, the quantities of material needed for the construction of the dam not only depend on the length of the dam but also on its cross-section. When a decrease in the length of the dam implies an increase in cross-section this may not be advantageous.

- <u>water-depth and gullies</u>

  As the quantities of construction material increase quadratic with the depth, it will be clear that the chosen alignment as much as possible must avoid deeper parts of the Gulf of Khambat. Tidal channels that can not be avoided must be crossed perpendicularly, if possible. This saves construction material and creates an optimal flow pattern during the closure. Also confluences and divisions of channels close to the closure-gaps should be avoided.

- <u>the sea-bed</u>

  The composition of the sea-bed is of major importance for the foundation of the dam. The expected loss of material due to scour is determined by the bottom material. The bottom of the Gulf of Khambat consists mainly of fine sand, which forms a good foundation but has to be protected against scour. At the western side, near Piram Island a rocky bottom can be found. At the eastern fringe the sand is covered by a layer of mud, originating from the rivers discharging in the gulf. The inclusion of the Narmada-mouth implies an alignment south of Dahej. An alignment south of Piram Island does not increase the possible gains. In the remaining area no significant differences in composition of the sea-bed are evident.

- <u>new part of the coast</u>

  The final dam will form a part of the new coast line. The closure influences the pattern and the magnitude of the currents. Also the sediment transported by the Narmada no longer can deposit in the tidal basin. (This deposition already has changed due to the construction of the Narmada dams.) The morphological processes resulting from the changes have to be studied. The alignments considered here, using the described indicative factors, do not differ much in location. Important differences in morphological impact between the alternatives are not expected.

- natural points of support

  In general, in determining an alignment for a dam, it is attractive to use islands as fixed points. Islands can provide a possibility to store building-material and equipment near the construction site. Dependent on the shape of the island, the length of the dam is reduced too. In the Gulf of Khambat the use of Piram Island is a possibility. An alignment crossing the gulf perpendicularly to Luhara Point can also be considered although it is not an island. From there a second closure is required in the delta of the Narmada.

4.1.2 Alternative alignments and selection

The alignment of the closure dam in the Gulf of Khambat can be roughly divided in two sections which connect on the Kerselea Bank in the Narmada delta, south-east of Luhara Point.

A: The first section runs from the western side of the gulf to Kerselea Bank. Three alternatives for this part will be compared later in this chapter.

B: The second section starts at Kerselea Bank, crosses the mud-flats and connects with the higher parts on the coast.

sub B:

Here two main alternatives are possible. These are (Figure 4.2):

1 an alignment north of Alia Bet, more or less parallel to the main channel of the Narmada river, with a length of approximately 25.8 km.;

2 crossing Alia Bet in south-eastern direction, ending near Katpur, 16.1 km. long.

The consequence of the second alternative is that a large area, consisting of mud-flats and sand-banks is isolated from the tidal, salt-water regime. In this case the mangroves, present on and south-east of Alia Bet, will disappear. On the other hand the storage area will increase, compared with the first alternative. A second advantage is that the quantities of construction-material needed, will be less.

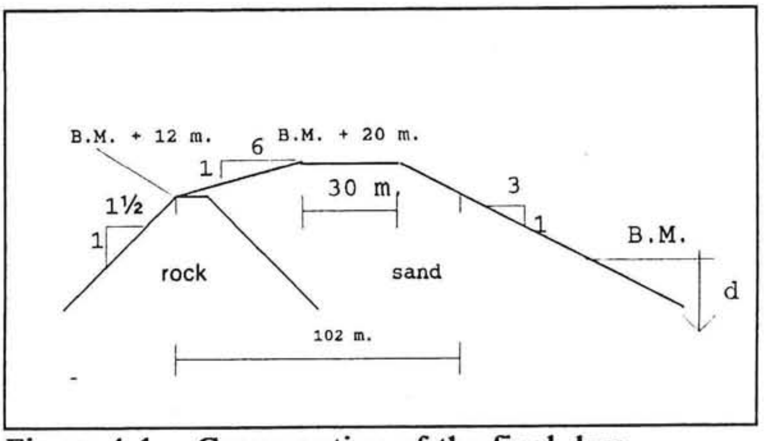


Figure 4.1 Cross-section of the final dam

The differences in these quantities are calculated applying a cross-section of the closure dam as shown in Figure 4.1. The final dam consists of a rubble closure-dam with slopes of 1 : 1½ and is assumed to be completed with sand that is covered by a revetment. At the sea-side the wave-attack will be more severe, so the slope here is less steep than at the basin-side. The levels mentioned in Figure 4.1 will be subject of detailed study further on together with the final shape of the dam.
Every 600 m. the bottom-level is taken from the admiralty chart. It is assumed that Alia Bet has a constant level of 10 m. + B.M. (d = -10 m.).

The area of a cross-section of the closure dam is calculated using Equation 4.1. This equation is used to determine the area of the part of the cross-section above the level of B.M. + 12 m. which has a maximum of 528 m^2 when d > -12 m. At this level a change in dam slopes is supposed. The area below this level is calculated in the same way, using other slopes. The sum of this two values is the local area of the cross-section.

d = bottom-level in m. below C.D. (d has a negative value when the level is above C.D.)

$$A = 30\cdot(20+d) + \frac{1}{2}\cdot 6\cdot(20+d)^2 + \frac{1}{2}\cdot 3\,(20+d)^2 \quad [m^2] \qquad \textbf{(4.1)}$$

The amount of material needed for a section of 600 m. is determined by linear interpolation of the areas at the edges of the section, whereafter this value is multiplied with the length of the section.

The results for this particular cross-section are:

alignment 1:	alignment 2:
$30.5 * 10^6$ m^3	$16.9 * 10^6$ m^3

This means that alignment 1 requires 1.8 times as much construction material as alignment 2.

It can be considered to build this dam section by means of hydraulic sand-fill. Then the cross-section of the dam will be totally different. When we assume slopes of 1 : 30 on both sides of the dam the quantities of construction-material are respectively 167 * 10^6 m^3 for alignment 1 and 90 * 10^6 m^3 for alignment 2. Here the difference in use of material is a factor 1.85. So it can be concluded that, almost independent of the shape of the cross-section, alignment 2 is much cheaper than alignment 1.
Here, only on economical grounds, alternative 2 is chosen. However, this does not imply that the final alignment exactly runs over Kerselea Bank.

<u>sub A:</u>

The other and most important part of the closure dam is the part crossing the large stream-gullies in the centre of the gulf. Based on the characteristics described in section 4.1.1 several alignments were proposed. Here three rather different variants are discussed, see alignment I, II and III in Figure 4.2. The longitudinal sections are shown in Figure 4.3.

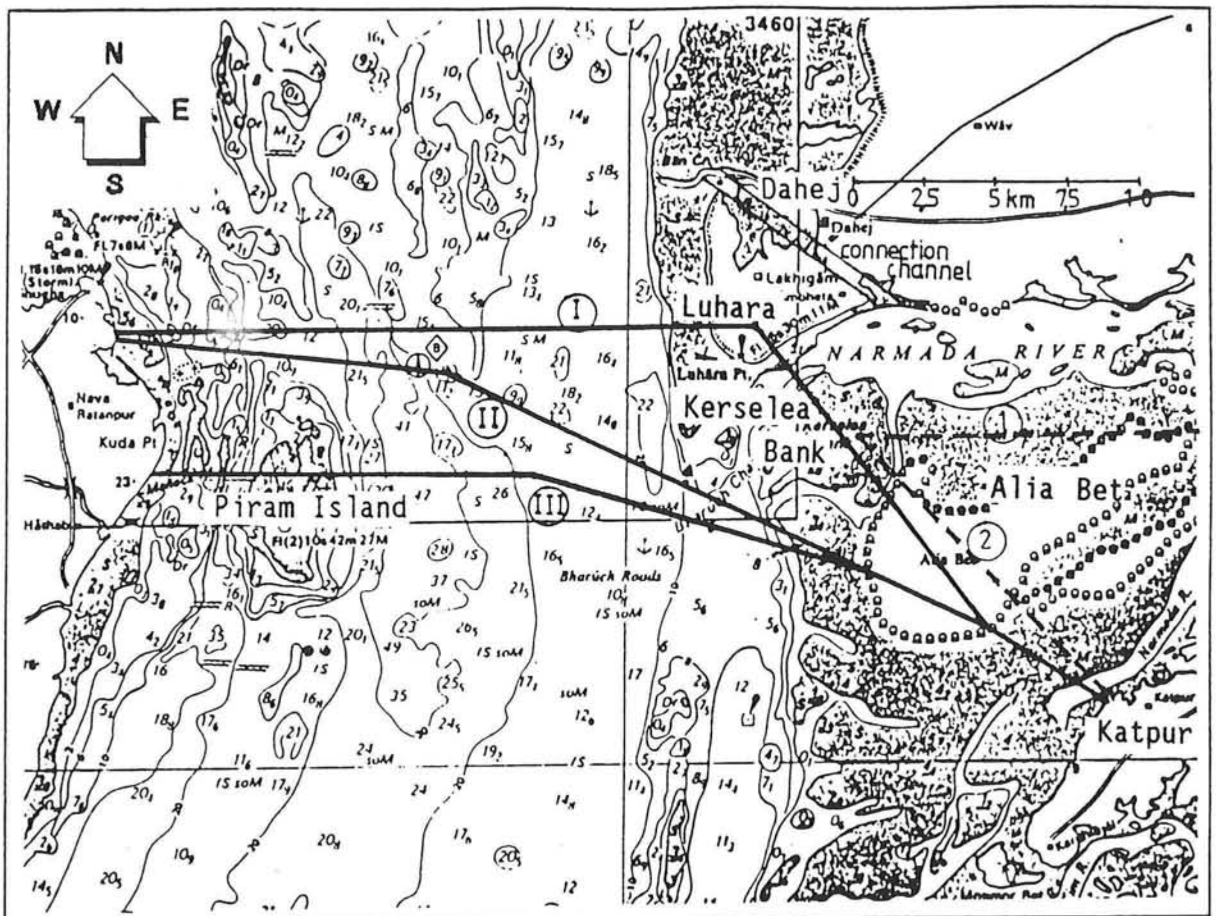


Figure 4.2 Three possible alignments for the closure-dam

The features of these alternatives are:

I A straight alignment at the narrowest part of the Gulf of Khambat;

II This alternative is oriented at sandbanks and shallow parts in the gulf;

III Alignment III crosses the Gulf straight in the direction of Kerselea Bank and Alia Bet, using Piram Island as a natural point of support.

For these (and other) alignments the quantities of construction material were calculated again using the same method as described above (Table 4.1). Here only the cross-section of Figure 4.1 was used.

alignment	construction material	length of new dam
I	$141 * 10^6$ m^3	47.3 km (Including the section over land, along Luhara it becomes 52.8 km.)
II	$157 * 10^6$ m^3	48.6 km
III	$169 * 10^6$ m^3	46.1 km

Table 4.1 Comparison of the alignments

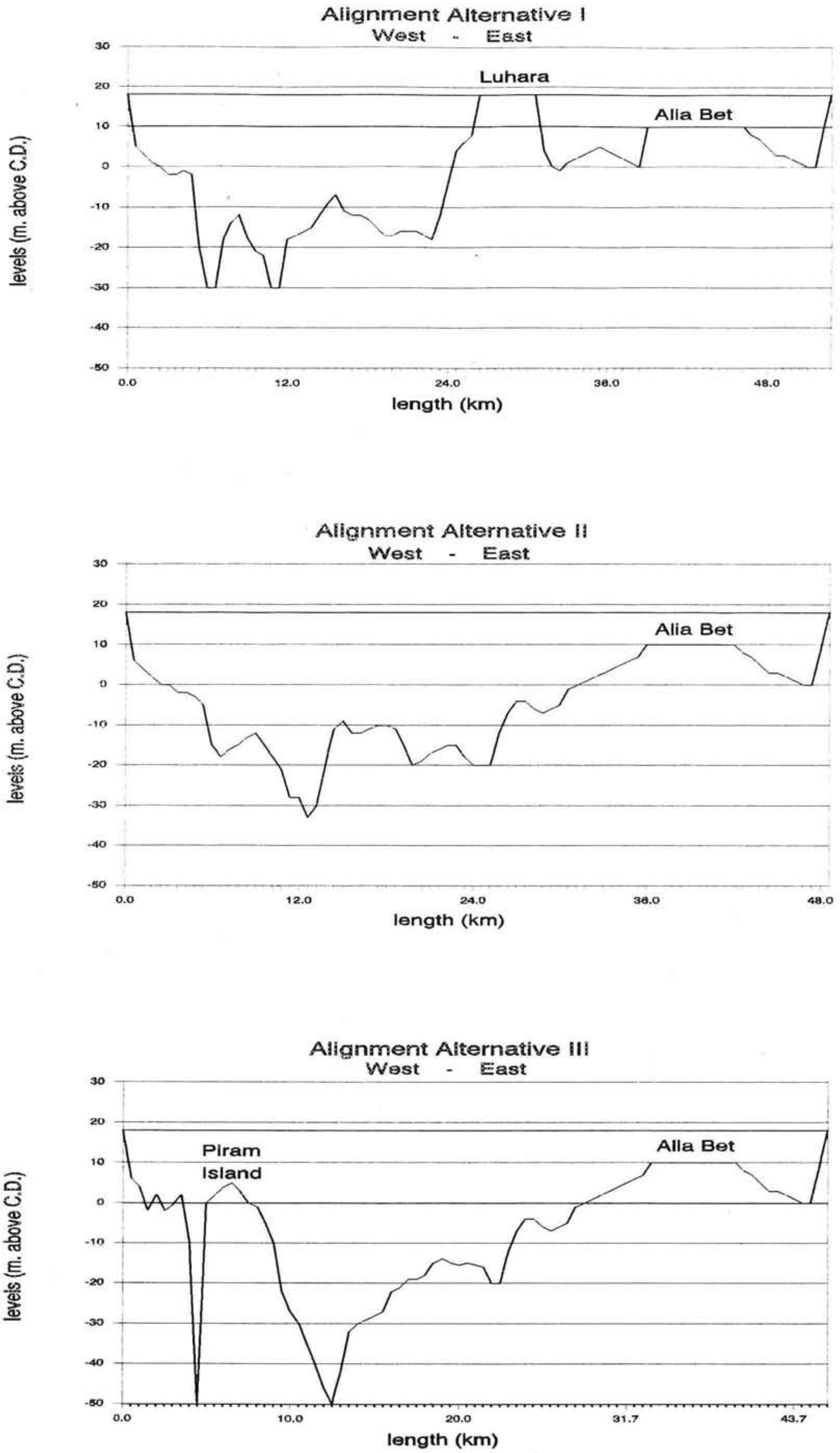


Figure 4.3 Longitudinal section of the alternatives

The calculations were based on the depths read from the Admiralty Chart. These charts are made for navigation purposes. In general the depths shown are less than in reality. So the absolute values of the calculated quantities will probably appear to be too small. However, the differences between the alternatives will increase with increasing depths because of the quadratic relation between the depths and the quantities of material needed for construction. Certainly the difference between alignment III and the other two will increase because the average depth is larger here. The average depths of I and II differ less but alignment II is slightly deeper.

alignment I

Alignment I crosses the Gulf of Khambat perpendicularly and seems to be a very good alternative. The length of the dam is only 2.7 km longer than alternative III and the material needed here is much less compared with the other alternatives. The main problem here is the inclusion of the Narmada mouth. The dam itself blocks the Narmada discharge. To solve this problem a channel must be dredged south-west of Dahej to accomplish a connection between the Narmada delta and the Gulf of Khambat. The dimensions of this channel will become very large. Also the quantity of material to be dredged is enormous.

rough estimation of channel dimensions

In the reconnaissance report the flood with a frequency of exceedance of 10^{-2} per year is mentioned and estimated to have a peak discharge of 67 000 m^3/s. The flood is supposed to last 60 hours and is described as a parabolic function with its maximum at t = 30 hours. The frequency has been chosen as the design frequency. Flooding will cause considerable damage but in view of the return period of 100 year this can be accepted.

In the small basin where the Narmada discharges water-accumulation will occur. So the peak-discharges of the Narmada will be reduced in the connection channel. The maximum discharge in the connecting channel is estimated at approximately 2/3 of the maximum Narmada discharge: 44 600 m^3/s. Suppose that the velocity in the channel section must stay below 1.5 m/s in order to prevent erosion and that the effective depth d = 20 m. Than the channel-width (W) will still have to be:

$$W = \frac{Q_{max}}{u \cdot d} = \frac{44600}{1.5 * 20} = 1487\ m \tag{4.2}$$

This is only a very rough estimate, but the expected necessary channel width will be in the order of 1 to 2 km. Between Dahej and Ambheta there is hardly space for a channel this wide.

Two sections are distinguished in longitudinal direction of the channel. These have respective lengths of about l_1 = 8 and l_2 = 3 kilometres and an average lowering of the bottom-level, by dredging, of respectively 10 and 20 m. is needed there. The volume (V) of material that has to be dredged becomes approximately:

$$V = (l_1 \cdot 10 + l_2 \cdot 20) \cdot W = (8000 \cdot 10 + 3000 \cdot 20) \cdot 1487 = 208 \cdot 10^6\ m^3 \tag{4.3}$$

This almost doubles the quantity necessary for dam construction. Most of the material probably can not be used for dam construction. The channel is situated in an alluvial area partly consisting of mud-flats.

Another consequence of the channel would be the isolation of the villages Luhara, Ambheta and, somewhat larger, Lakhigam. A ferry or the construction of a bridge, which is rather expensive, can solve this problem. A bridge has the advantage that the railway and the road near Dahej can be reached from the closure dam too.

alignment II

Assuming the line that the alignment will end near Katpur, another alternative was determined. Alignment II lies somewhat more to the south than alignment I but avoids the gullies near Piram Island. After crossing the deepest part this alignment bends directly in the direction of Alia Bet. In this way Kerselea Bank is not crossed. This diminishes the length of the dam with 3 km (compared to an alignment that crosses Kerselea Bank, which is not discussed here). The necessary quantities of material are also less, however compared with alternative I they are larger.

The Narmada discharge must pass now the gully between the closure-dam and Luhara-point. When the flow-area is too small here the mud-flats near Luhara-point have to be dredged.

The quantities of construction-material needed for the dam are less than when alignment III is applied. Because the dredging necessary for the Narmada-flow will be considerably less this alternative is favourable to alignment I too.

alignment III

This alternative seems favourable because of the shortness of the alignment and the use of Piram Island as a natural point of support. The deep gullies around Piram Island however do increase the quantities of material needed for construction considerably. The gully between the island and the coast is rather deep. Here a maximum depth of 50 m. is assumed. This is somewhat more than could be read from the Admiralty Chart. Local authorities however estimate the depth of this gully up to 100 m. This increases the quantities, for this part of the alignment between the coast and Piram Island, with $16 * 10^6$ m^3 to $30 * 10^6$ m^3, depending on the assumed width of the deepest part of the gully.

selection of the alignment

Making a well-considered choice between several possible alignments asks for a comparison of many different aspects. The total benefits should be weighed against the costs and all negative effects caused by the closure of the Gulf of Khambat. For instance a thorough study has to be made with regard to the effects of the disappearance of the tide and the desalinization upon the flora and fauna.

Here, only based on economical grounds concerning the construction of the dam, alignment II is chosen as the starting-point for the design.

hydraulic aspects

The selected alignment is rather far to the south. This implies that an enormous amount of water passes through this cross-section every tidal cycle. During the inflow (flood) it amounts from approximately $6 * 10^9$ m^3 for a neap-tide to $11 * 10^9$ m^3 for a spring-tide.

For the definite alignment again some DUFLOW-calculations were made to study the range of waterlevels that can be expected in front of the dam. This is important for the tidal power station as well as for the closure-operation and the design of the final dam. The results are presented in section A5.5 of Appendix A. Here only the situation for an average spring-tide is shown in Figure 4.5. Compared with the situation before closure (Figure 4.4) the waterlevel in the basin (node 14) is constant now and the maximum waterlevel in front of the dam (node 21) increases by approximately 0.30 m.

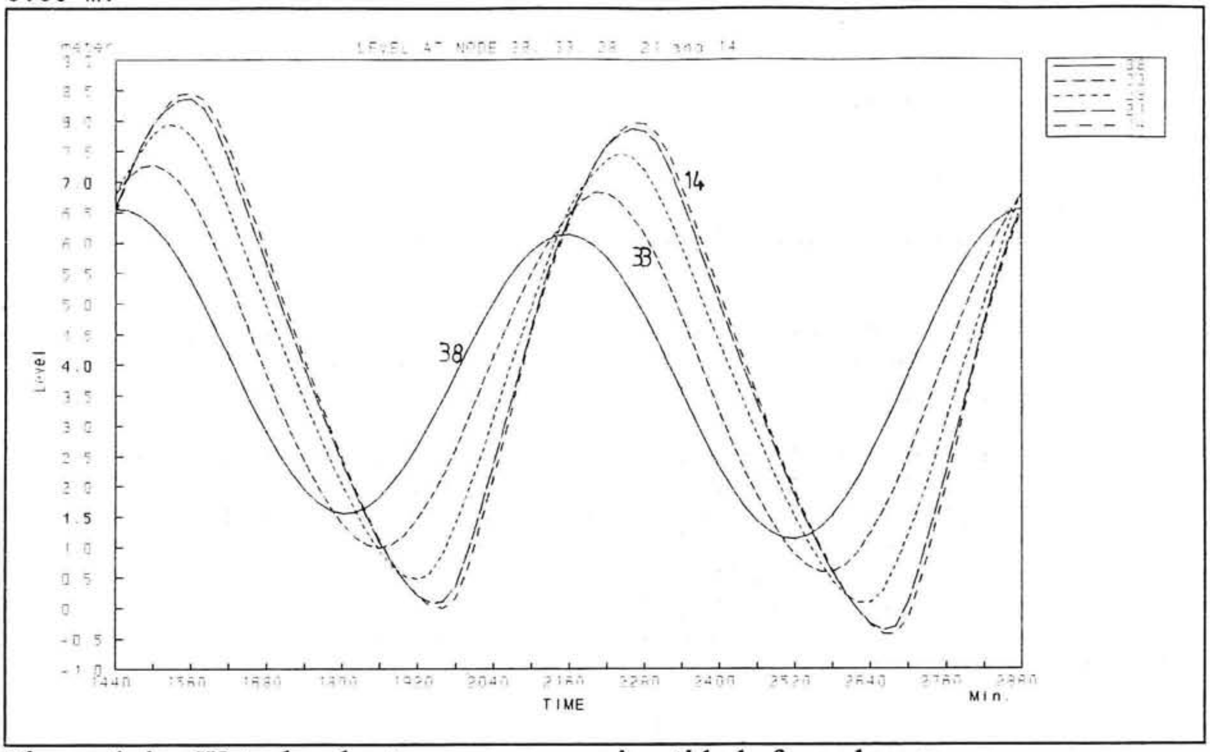


Figure 4.4 Waterlevels at an average spring-tide before closure

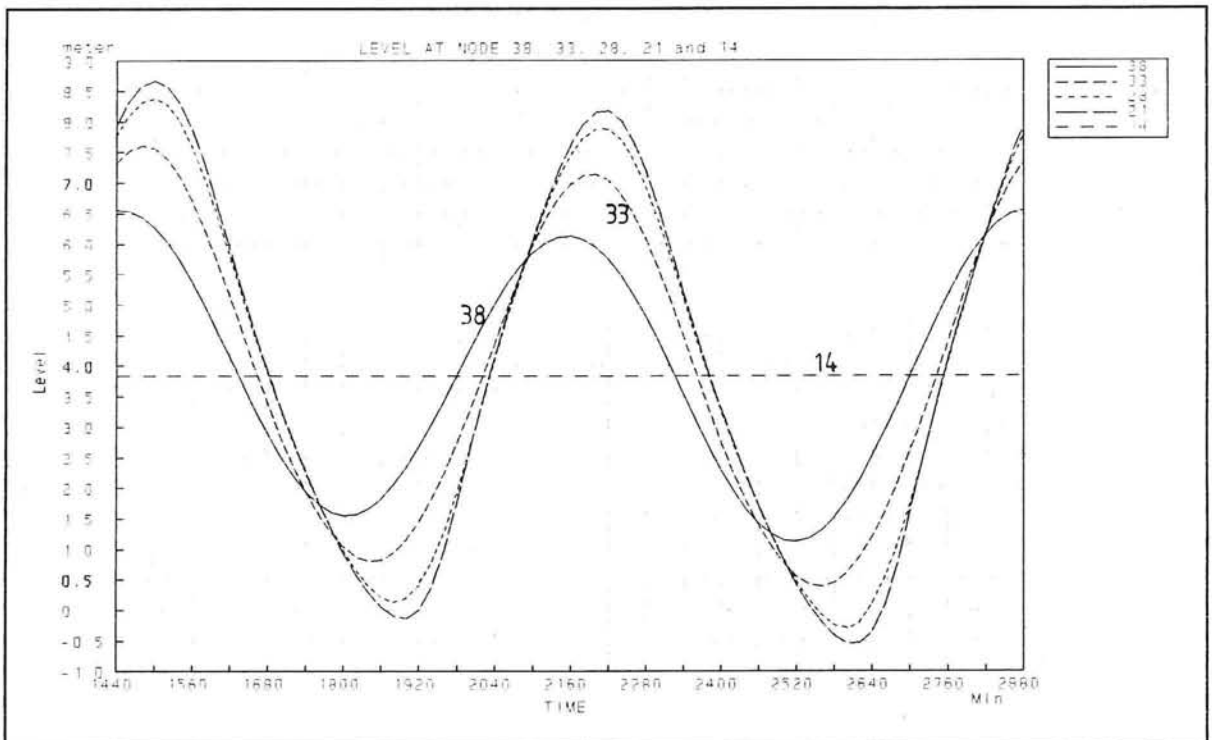


Figure 4.5 Waterlevels at an average spring-tide with a completed dam at the chosen alignment

4.2 Closure methods

When building a dam to close off a tidal basin, different methods can be used. These methods can differ in the material used, and the way in which the closure is carried out. The material can be sand, rock or large elements like caissons. As far as the method of closure is concerned, four alternatives can be distinguished:

- horizontal closure;
- vertical closure;
- a combination of horizontal and vertical closure;
- sudden closure.

4.2.1 Horizontal closure

The horizontal closing method is characterized by a construction-sequence where the closure gap is gradually narrowed. Working from both sides (this depends on the position of the final closure gap) the waterway is horizontally constricted (Figure 4.6 and 4.7). This can, at small depths, easily be carried out by trucks tipping their load. Narrowing the cross-section available for the tidal current, the velocities increase considerably. This makes it necessary to use very heavy stones for the final closure. Also the bottom will be attacked severely because of the high velocities occurring close to the bottom. It will be clear that a heavy bottom-protection will be necessary.

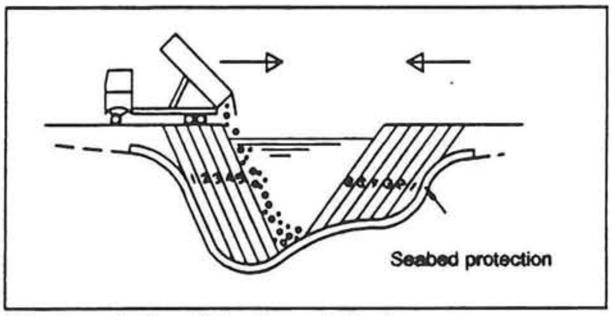


Figure 4.6 Horizontal closure

When closing a large tidal basin with a considerable tidal difference the current-velocities reach such high values (order of magnitude 8 till 10 m/s for a spring-tide) that a pure horizontal closure is very difficult to achieve.

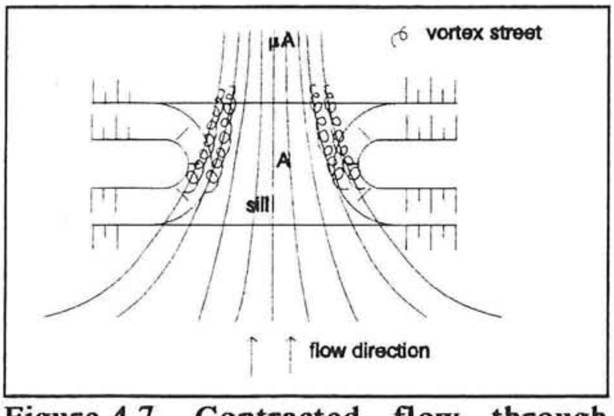


Figure 4.7 Contracted flow through the closure gap

The discharge through a narrowing is described by Equation 4.4:

$$Q = \mu \cdot A \cdot \sqrt{2 \cdot g \cdot z} \qquad \textbf{(4.4)}$$

μ = contraction-coefficient, varying from 1, for a smooth shape of the dam-heads, to 0.8 for a narrow and steep dam-head. The dam-heads are supposed to be rather narrow ($\mu \geq 0.8$) and not smooth ($\mu < 1.0$). A sill diminishes the contraction of the flow. The sill-level is deep so the influence will be small. A value of μ = 0.85 has been chosen for the last 100 m. When the closure-gap is wider the contraction will be less. So different values of μ have been used.

22 - 4 km.	μ = 1.0
4 - 2 km.	μ = 0.95
2000 - 250 m.	μ = 0.90
250 - 0 m.	μ = 0.85

z = the head-difference over the closure-gap

When the closure-gap is gradually narrowed the waterlevels in the basin can not follow the waterlevels outside because of the flow profile A becoming smaller. This diminishes the discharge Q.

The influence of a horizontal closure on the velocities occurring in the closure-gap have been studied using the DUFLOW-model of the Gulf of Khambat. For a detailed description of the way in which this has been done, the reader is referred to Appendix A, section A6. The results of the calculation in the situation of Highest Astronomical Tide (HAT) and an average spring- and neap-tide are presented here.
This section deals with a horizontal closure. However a combination of different methods is considered here. The horizontal closure is supposed to take place on a sill at 20 m. below B.M.. This has been done to be able to compare the results with those in the reconnaissance report, where a calculation based on the storage-equation has been made applying a sill at the same height.
In Figure 4.9 and 4.10 the situation for HAT is presented when the closure-gap has a width of 7000 m. Figure 4.9 shows the relation between the momentary waterlevels just in front of and behind the dam. In the undisturbed situation this would result in a straight line with a slope of 1. This slope decreases when the discharges through the closure-gap decrease and the basin-level approaches a constant value.
The tidal range inside the basin is approximately 1.2 m. less then at the sea-side. In the undisturbed situation it was slightly larger at node 42 than at node 43 (Figure 4.8).

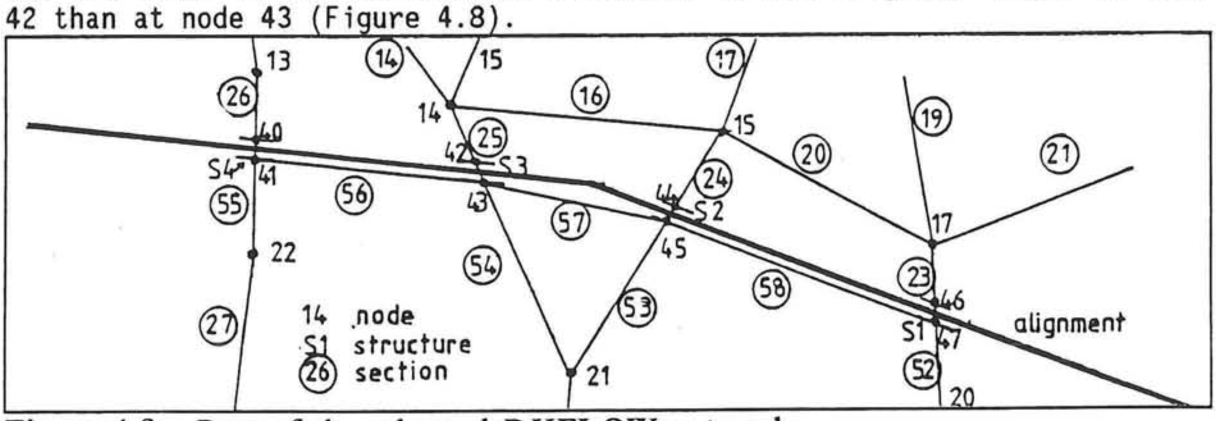


Figure 4.8 Part of the adapted DUFLOW-network

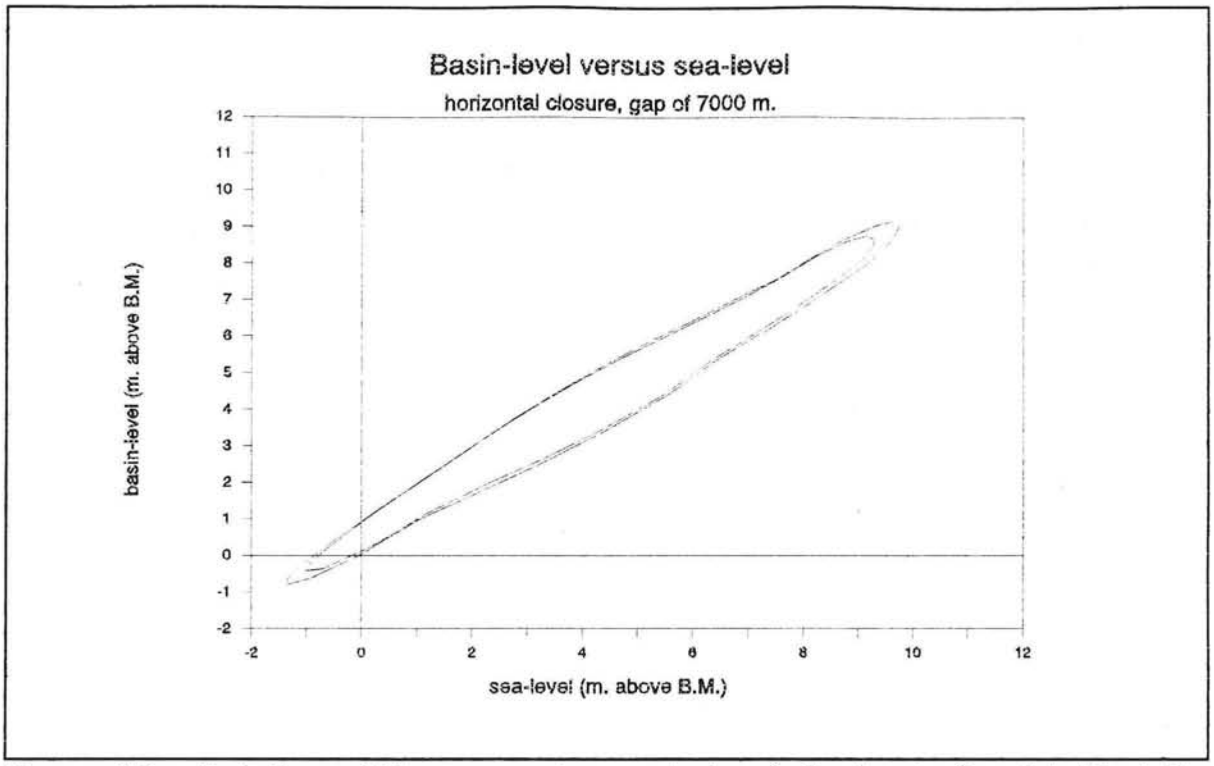


Figure 4.9 Relation of the momentary waterlevels in front of and behind the closure dam in the situation of HAT and a gap of 7000 m.

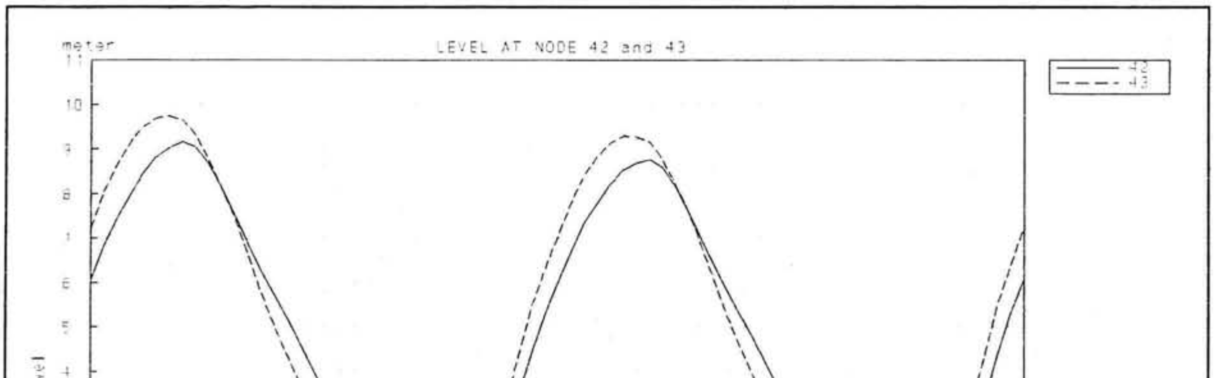


Figure 4.10 Waterlevels in front of (43) and behind (42) the closure dam at HAT with a closure gap of 7000 m.

When the closure-gap is narrowed to 500 m. the situation is completely different (Figure 4.11 and 4.12).

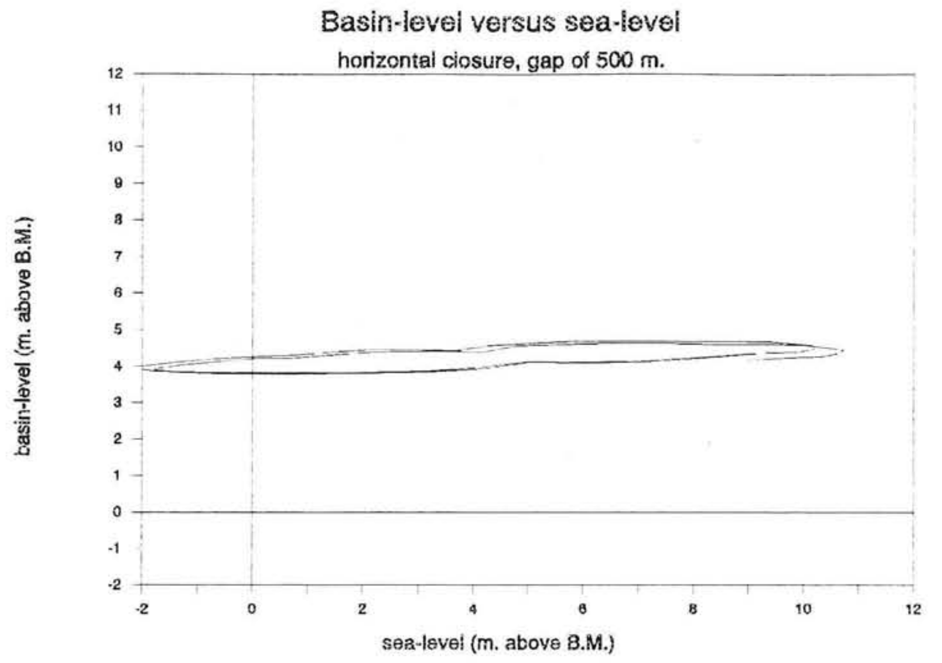


Figure 4.11 Relation of the momentary waterlevels in front of and behind the closure-dam in the situation of HAT and a gap of 500 m.

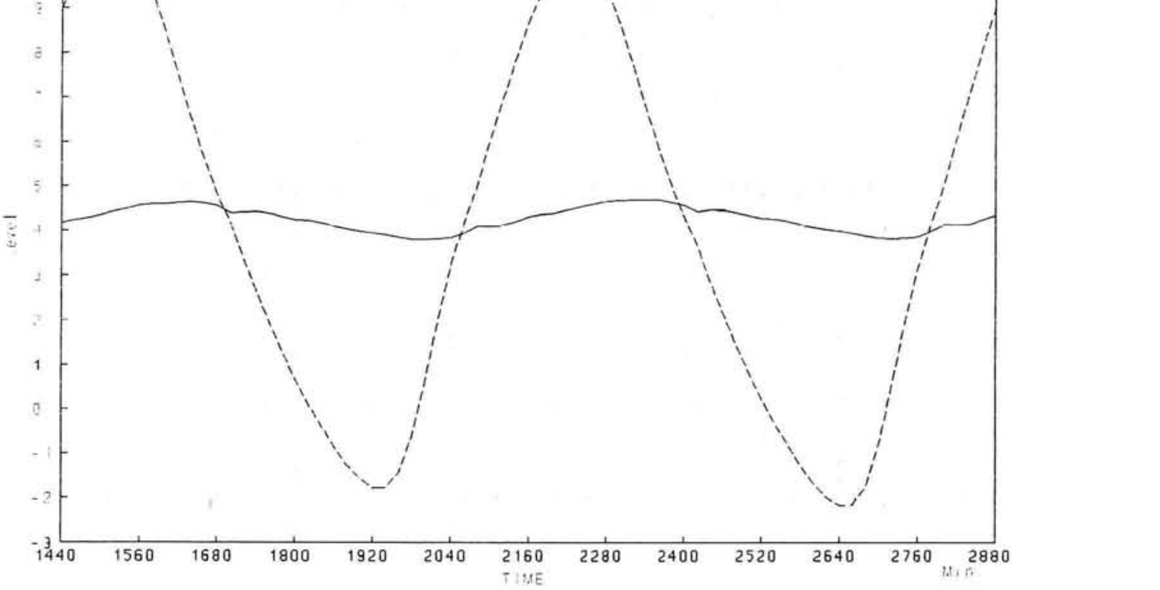


Figure 4.12 Waterlevels in front of (43) and behind (42) the closure dam at HAT with a closure gap of 500 m.

The flow profile has become rather small. The waterlevel in the basin is almost constant now at a level of approximately the still-waterlevel at sea-side. Note that in the calculation the equilibrium-situation is not reached yet. The discontinuity in the curve of Figure 4.11 shows that the average waterlevel in the basin is still rising. The equilibrium-situation where the inflow is equal to the outflow is not reached yet. This can be solved by a very long time of computation.

However, in nature the equilibrium is also never reached. Before this is the case the astronomical tides will have changed.

Closing the last kilometre gap the velocities in the closure-gap increase to more than 8.5 m/s, when a flood of an average spring-tide is considered. (Figure 4.13). In case of HAT this value is 10.6 m/s. Velocities during the ebb-flow are slightly smaller. Using Eq. 4.4 the velocities in the closure-gap can be computed too.

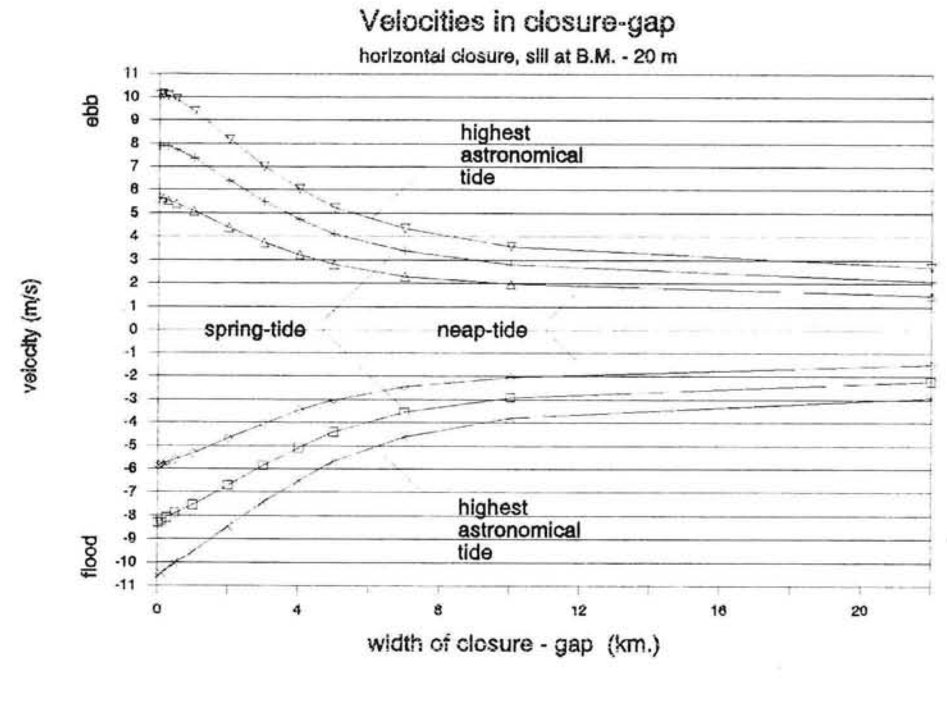


Figure 4.13 Range of velocities for different tidal situations when a horizontal closure is applied

The maximum value of the head-difference is equal to half the tidal range at sea-side: 0.5 * 12.90 = 6.45 m. for HAT. The maximum velocity becomes (Eq 4.5):

$$v_{gap} = \frac{Q}{A} = \mu\sqrt{2 \cdot g \cdot z} = 0.9\sqrt{2 \cdot 9.8 \cdot 6.45} = 10.1\ m/s \qquad \textbf{(4.5)}$$

This value corresponds quite well with the DUFLOW-results where the maximum values are 10.2 m/s for ebb-flow and 10.6 m/s for flood-flow.

Stability of rock-fill

Rock can be used to construct the closure-dam because it is available rather close to the construction-site. Here it is supposed that basalt with a specific density of 2900 kg/m^3 is available for the final stage of the closure. Large quantities of stone are needed with a rather small mass. Then rock with a specific density of 2700 kg/m^3 is supposed to be used. For more information about rock see annex III.

The stability of the stones depends on the stone-parameter ΔD_n. Out of the DUFLOW-results (HAT and average spring-tide) the stone-parameter ΔD_n is calculated, using the head-difference over the sill (annex IV). This stone-parameter consists of:

$$\Delta = \frac{(\rho_{stone} - \rho_{water})}{\rho_{water}} = \frac{2900 - 1020}{1020} = 1.84$$

= relative density of the stones (in saline water)

$D = D_n =$ nominal diameter of the stones representing the linear dimension of a cube of the material considered, based on a mass which is exceeded by 50 % of the total number of stones (M_{50})

The calculation is based on a schematization of graph IV.1 in annex IV. For each time-step the momentary result is determined using equations IV.3 and IV.4 and afterward the maximum value is taken. The results are presented in Figure 4.14.

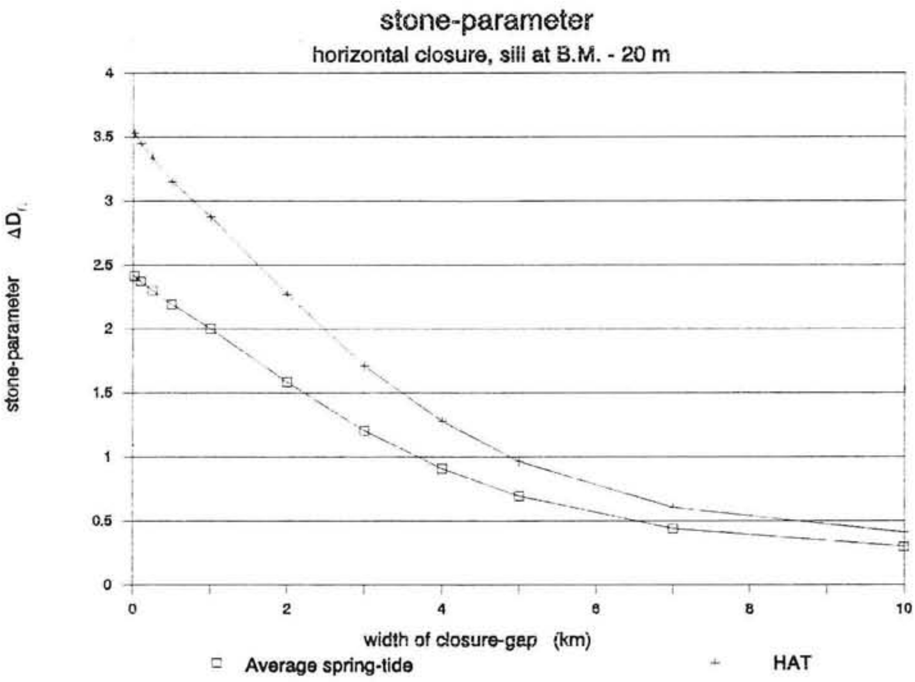


Figure 4.14 Values of ΔD_n for decreasing width of the closure gap for two tidal amplitudes applying a horizontal closure

During construction of the dam a spring-tide with a maximum waterlevel higher than at average spring-tide will certainly occur. Also the influence of waves and wind set-up has to be taken into account. For this reasons HAT has been chosen representing a combination of these effects. One should realise that a combination of a very high spring-tide (approaching HAT) and a large wind set-up together with high waves is very rare. For a more definite design the chance of occurrence of extreme waterlevels, wind set-up and wave-heights and their possible combinations will have to be studied in detail.

The final closure can be planned between two monsoon-periods when the weather is quiet. The occurring tidal difference can also be kept relatively small by picking a period with spring-tides smaller than average. In that case a less severe design condition than HAT can be taken. So the results for average spring-tide are presented too.

So far both the head-effect and the porosity of the dam are neglected. The vortex streets (Figure 4.7) severely attack the top-layer of the sill. A method to take this head-effect into account is discussed in annex IV. It is found that a factor 1.2 on the velocity will be appropriate. This implies a factor 1.2^2 on the value of ΔD_n. The porosity of the dam in its turn diminishes the head difference over the dam. This decreases the flow-velocities and thus the value of ΔD_n. The influence is discussed in Annex IV but neglected here.

A more detailed construction strategy has been worked out for the chosen method of closure (section 4.3)

The value of ΔD_n is used to determine the necessary mass of the stones used for the closure. As an example the maximum-value of ΔD_n = 3.5 is taken. The values for HAT accidentally more or less correspond with an average spring-tide combined with the margin for the head-effect (3.5 (HAT) $\approx 1.2^2 * 2.4$ (average spring-tide including head-effect)). The necessary stone-mass can be determined as in Eq. 4.6.

$$M_{50\%} = D_n^3 \rho_s = \left(\frac{\Delta D}{\Delta}\right)^3 \rho_s = \left(\frac{3.5}{1.84}\right)^3 \cdot 2900 = 20.0 \cdot 10^3 \ [kg] \qquad \textbf{(4.6)}$$

Stones of this mass are not easy available. It has to be studied if a quarry, that can be opened in the closest mountains, will be able to deliver stones in the range of 20 to 25 tons (When the final closure is planned during neap-tide in a season that the weather is quiet, stones in the grading of 15 to 20 ton will be heavy enough). Otherwise the production of concrete blocks with heavy aggregate can be considered.

Except the mass and diameter of the stones the total quantities of stone and the way of transport of the material play an important role in the closure operation. This asks for further detailed study of the logistics of the operation.

4.2.2 Vertical closure

When a vertical closing-method is chosen the width of the river or basin is maintained. Gradually the depth at the dam-site decreases by building the dam in horizontal layers (Figure 4.15). The material is brought in by dump-barges (which can not complete the total closure because the available depth decreases) or by cranes, cableways, bridges or even helicopters.

Here a bottom-protection is necessary over the whole width of the basin.

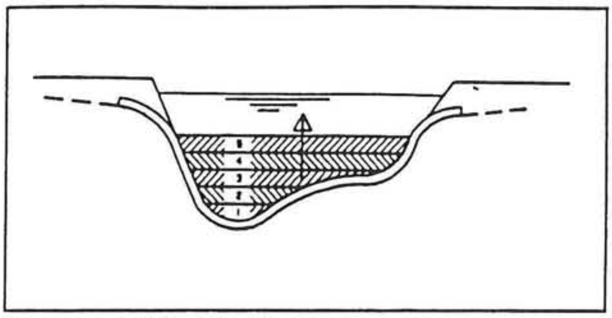


Figure 4.15 Vertical closure

The discharge per metre width, q, over a drowned weir is described by Equation 4.7.

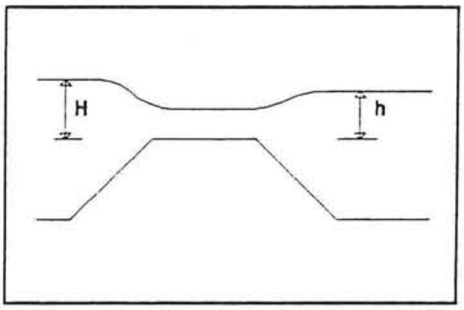


Figure 4.16 Flow over a weir

$$q = m \cdot h \cdot \sqrt{2 \cdot g \cdot (H-h)} \qquad \textbf{(4.7)}$$

When h becomes smaller than 2/3 * H (Figure 4.16) critical flow sets in. In this situation of a free fall weir Eq 4.7 changes into Eq. 4.8.

$$q = m \cdot \frac{2}{3} H \cdot \sqrt{2 \cdot g \cdot \frac{1}{3} H} \qquad \textbf{(4.8)}$$

m = discharge coefficient depending on the loss of energy of the flow. The value of m varies between 1.3 for a broad sill with smooth slopes, and 0.9 for a narrow sharp-crested sill. Here m = 1.0 has been used. The sill is rather broad but certainly rough when rubble is used.

Also the vertical closure has been simulated with the DUFLOW-model. The flow-area was decreased by raising the sill-level over a width of 22 km. It is assumed that the part of the alignment in the mouth of the Narmada is already closed. Again three tidal amplitudes have been studied to determine the velocities over the sill.

When the sill has a level of 7 m. - B.M. the tidal differences of node 42 and node 43 differ about 1 m. The effect of the decreased values (compared with a very deep sill e.g. B.M. - 20 m.) of h and H is partly balanced by an increase in the head-difference (H-h). So the discharge q is still rather large. The water-levels inside the basin (node 42) show no large difference compared with the levels at sea-side (node 43). See Figure 4.17 and 4.18. The shape of the curves is the same as when a horizontal closure is applied, compare respectively Figure 4.9 and 4.10 with 4.17 and 4.18.

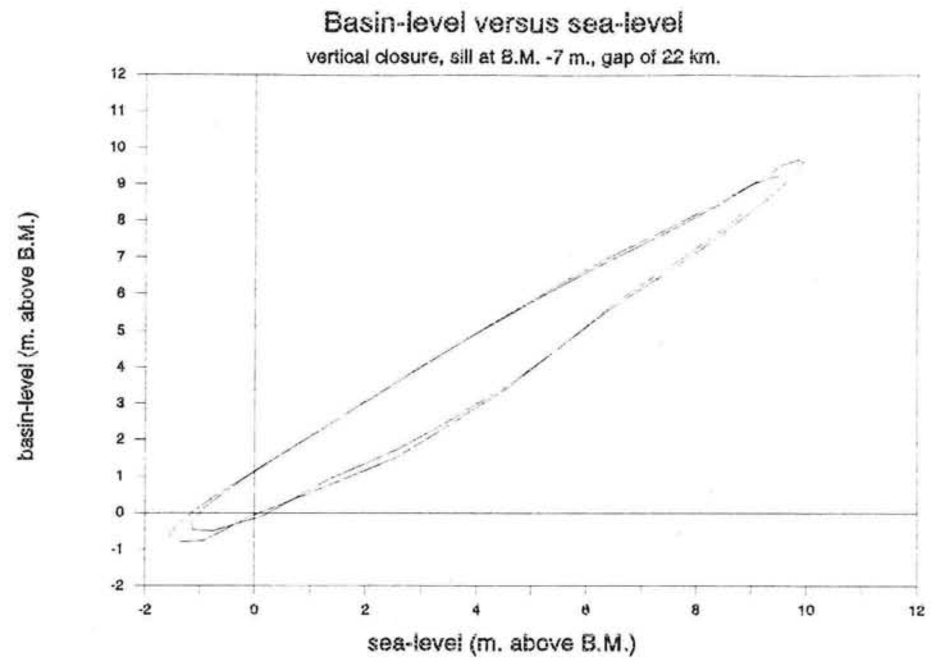


Figure 4.17 Relation of the momentary water-levels in front of and behind the closure-dam in the situation of HAT and a sill at 7 m. below B.M.

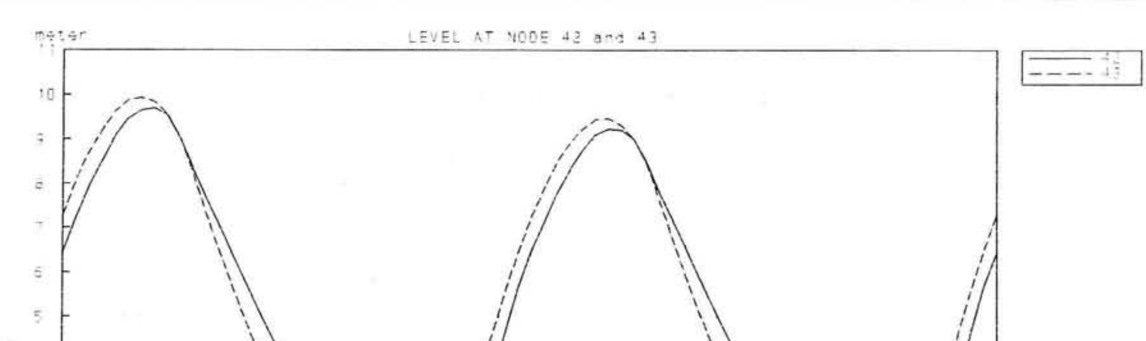


Figure 4.18 Waterlevels in front of (43) and behind (42) the closure dam at HAT with a sill at 7 m. - B.M.

When the sill is heightened till a level of 4 m. above B.M. the difference in water-levels is far more evident (Figure 4.19 and 4.20).

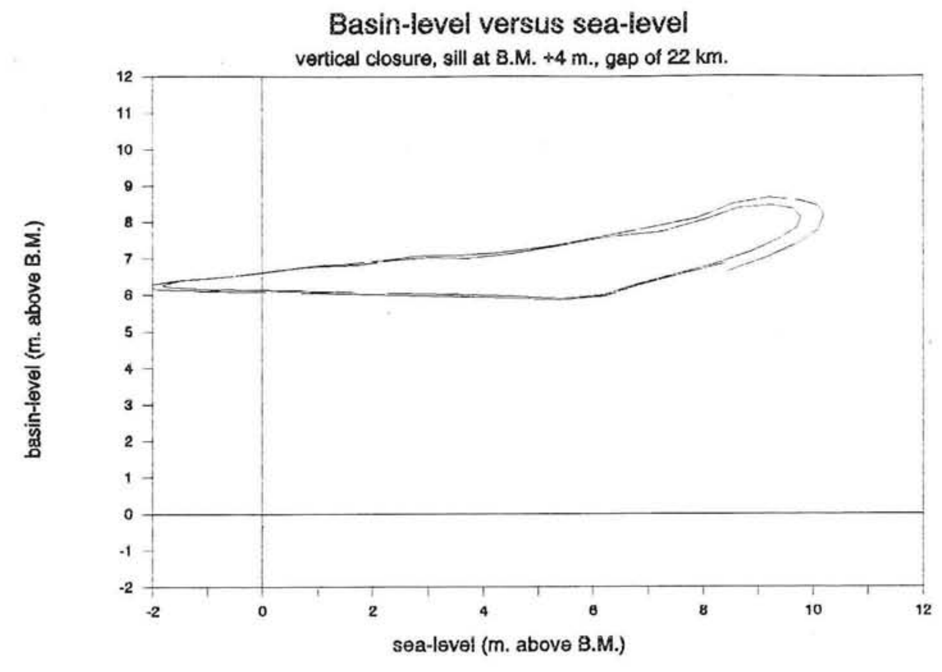


Figure 4.19 Relation of the momentary water-levels in font of and behind the closure dam in the situation of HAT and a sill at 4 m. + B.M.

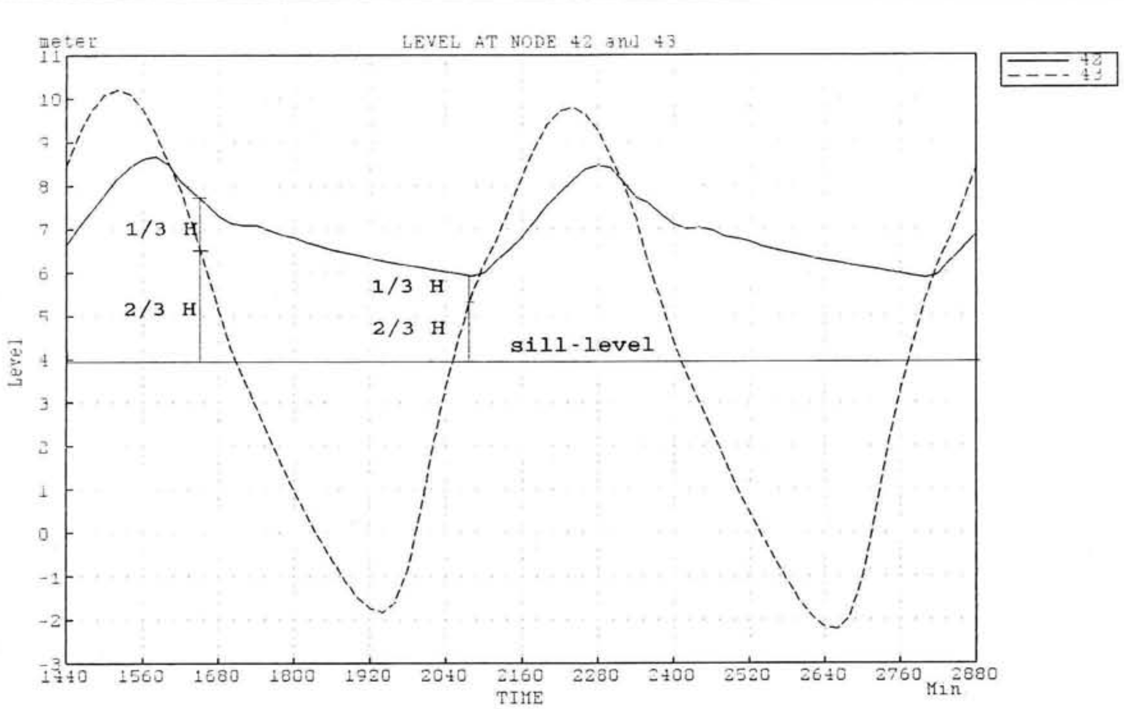


Figure 4.20 Waterlevels in front of (43) and behind (42) the closure dam at HAT with a sill at 4 m. + B.M.

The level inside the basin can not follow the level at sea-side any longer. Although the head difference over the sill is much larger now, the inflow is limited by the small value of h (maximum h = 4.7 m.). During falling tide the water-level at sea-side drops below that at basin-side (which becomes H now). When h becomes smaller than 2/3 H, e.g. at time t = 1650 min., critical flow sets in (Figure 4.20 and 4.21). After the maximum flow velocity have been reached, here 4.95 m/s, it decreases because H becomes smaller (Eq 4.8). When the tide comes up, again at the time that h = 2/3 H, the free fall weir changes in a submerged weir (t = 2075 min.). The direction of flow turns shortly after that.

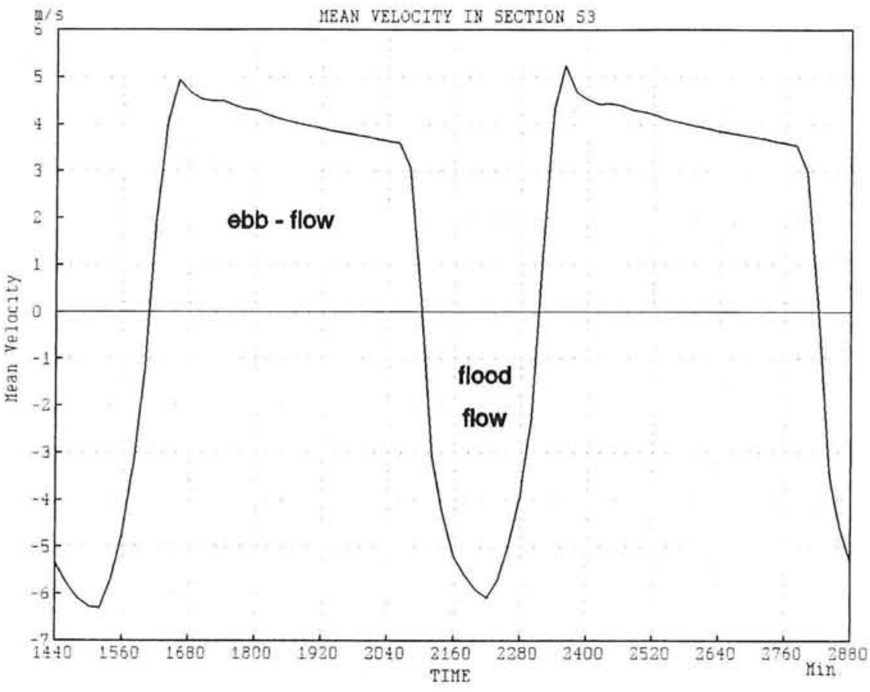


Figure 4.21 Mean velocity above the sill which has a level of 4 m. + B.M.

During low tide the basin-level drops from approximately 8.6 m. + B.M. to 6.0 m. + B.M. (still 2 m. above sill-level). The time between two high tides is not long enough to empty the basin. The outflow is more hampered than the inflow because the relative depths above the sill are larger during flood. The maximum flow-velocities in the two directions differ about 1 m/s here.

The flow-velocities which are reached during a vertical closure have been calculated for three different tidal amplitudes (Figure 4.22). The over-all maximum velocities of the flood- and ebb-flow are almost the same but they do not occur at the same sill-level. During low-tide the situation of critical flow is reached at a sill-level of 0 m. + B.M. for a neap-tide. Further heightening of the sill only reduces H which causes the (ebb-)flow velocity to decrease. The maximum velocities occur when the critical-flow situation is just reached. During high tide no critical flow situation occurs.

The maximum calculated velocities, taking Highest Astronomical Tide, are approximately 6.5 m/s. This is considerable less than in the case of a horizontal closure (10.2 m/s).

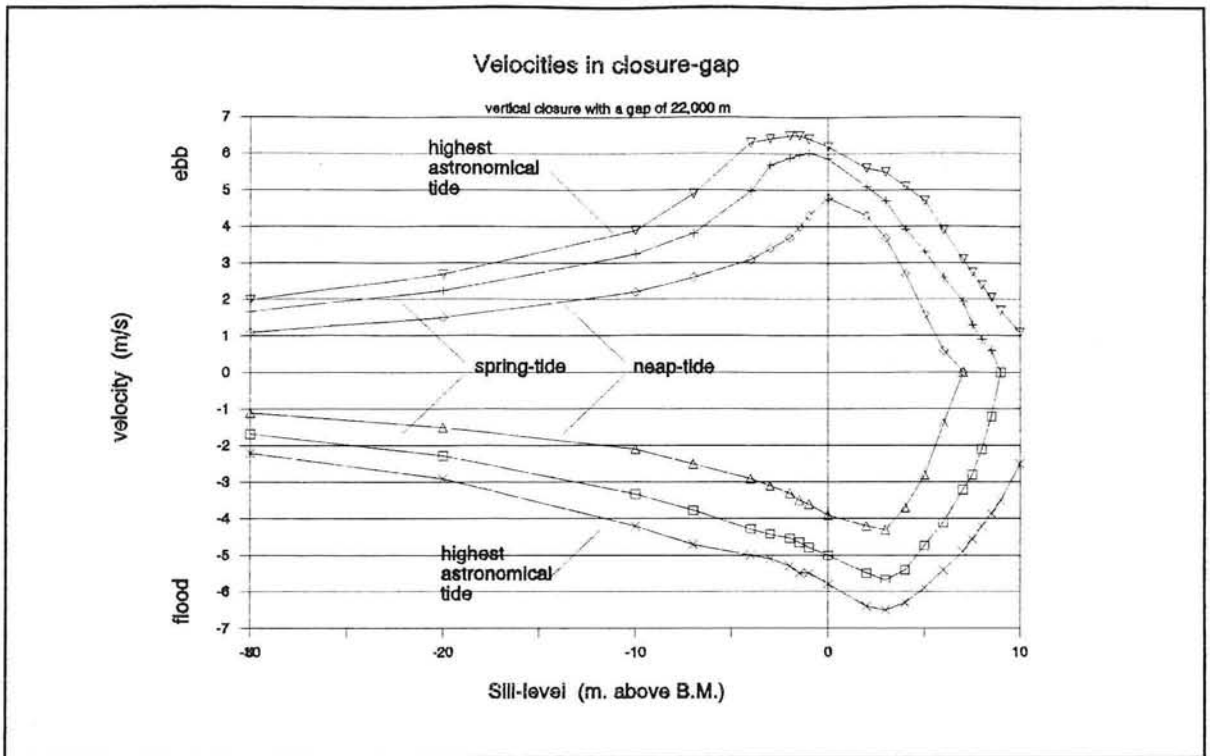


Figure 4.22 Maximum velocities for increasing sill-heights

Stability of rock-fill

The stone-parameter ΔD_n is determined again. The results for two tidal amplitudes are presented in Figure 4.23.

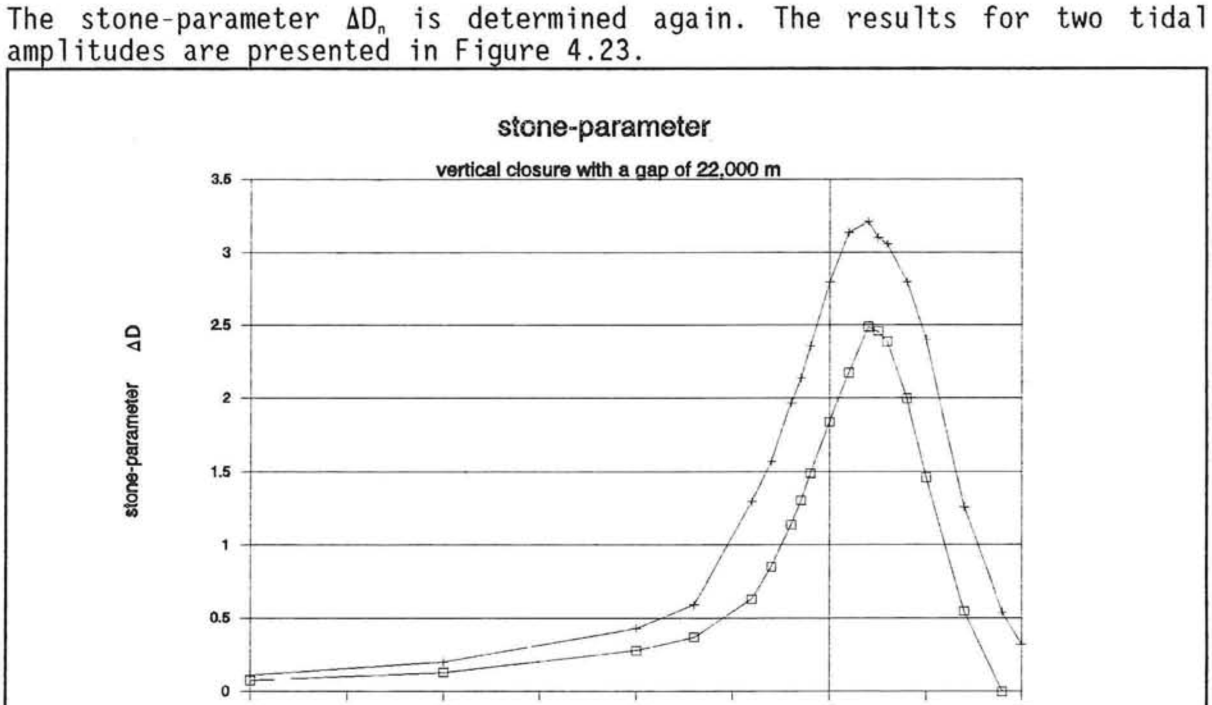


Figure 4.23 Values of ΔD_n for increasing sill-heights for two tidal amplitudes applying a vertical closure

Here, again, the porosity of the dam is neglected. The value of ΔD_n is maximum when the sill level is 2 m. + B.M. This means that the top-layer of the closure dam can be constructed using smaller stones.

The maximum value of the ΔD-parameter (HAT)is 3.2 here. This leads to a stone-mass of:

$$M_{50\%} = \left(\frac{3.2}{1.84}\right)^3 \cdot 2900 = 15.2 \cdot 10^3 \ [kg]$$

The stones have to be less heavy than in the situation of a horizontal closure for the same boundary condition. Here in both cases HAT has been chosen as the governing situation for the determination of the stone-mass. The choice for the governing situation depends on the design strategy and the construction scheme.

4.2.3 Combined closure

Another possible method to close an estuary is a combination of the two earlier described methods. First a vertical closure is applied till a certain level is reached. This level depends for instance on the draught of the stone-dumping vessels and the flow-velocities. Secondly the closure-gap is narrowed from the sides. In fact a horizontal closure is carried out on a high sill (Figure 4.24).

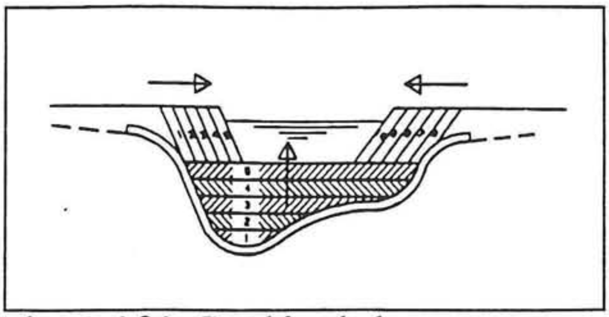


Figure 4.24 Combined closure

The maximum velocities at which use can be made of a stone-dumping vessel are approximately:

* 0.70 m/s under transverse current attack, round the turn of the tide, and

* 2.5 to 3.0 m/s under head current attack.

When we look at the graph of Figure 4.22 we see that at a sill-level of 5 m. below benchmark the velocities of a neap-tide stay within the allowed range, considering a head current. Then stone-dumping will be possible during the whole tidal cycle. At spring-tide dumping will not be possible during a small part of the tidal cycle. As an example this sill-level is chosen for detailed study. Also for a sill-level of 10 m. below benchmark the flow-velocities have been determined by varying the gap-width at a constant sill-level (Figure 4.25). In the graph the values of the earlier discussed horizontal closure are presented too.

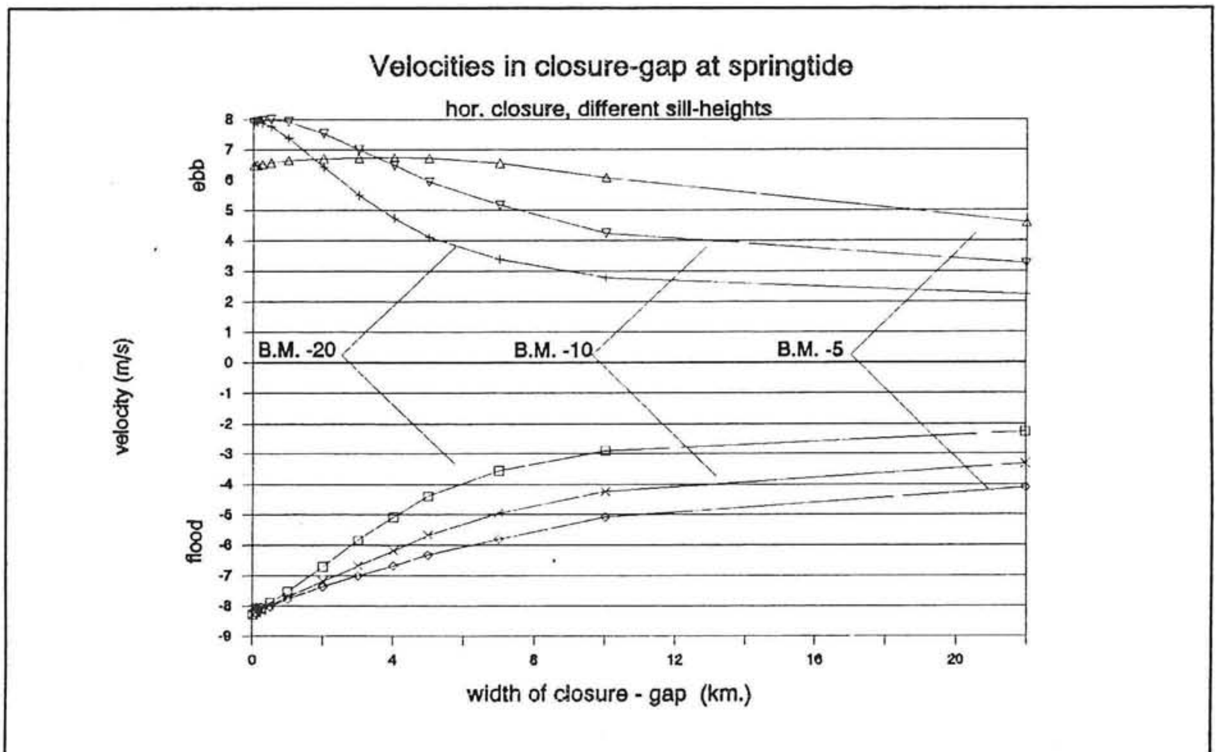


Figure 4.25 Maximum flow-velocities with different sill-levels

For the largest part of the closure counts that the velocities are more extreme when the sill is higher. However when the closure-gap becomes smaller (smaller than 8 km.), the differences in velocities start to decrease. When the gap is almost closed the velocities during flood approach to the same maximum value of about 8.2 m/s. During the ebb-flow however the over-all maximum velocities decrease with a higher sill. This is caused by the critical-flow situation which limits the flow-velocities more than horizontal constriction. Critical-flow occurs during the ebb-flow because the sea-level falls more than the basin-level. Further closure doesn't influence the ebb-velocities much because the critical-flow situations remains present. In the last four kilometres the flow-area becomes so small that the upstream waterlevel H decreases. This causes the maximum velocity during ebb-flow to decrease.

The curve of the stone-parameter ΔD has been determined for the situation of an average spring-tide and a sill at 5 m. - B.M., see Figure 4.26.

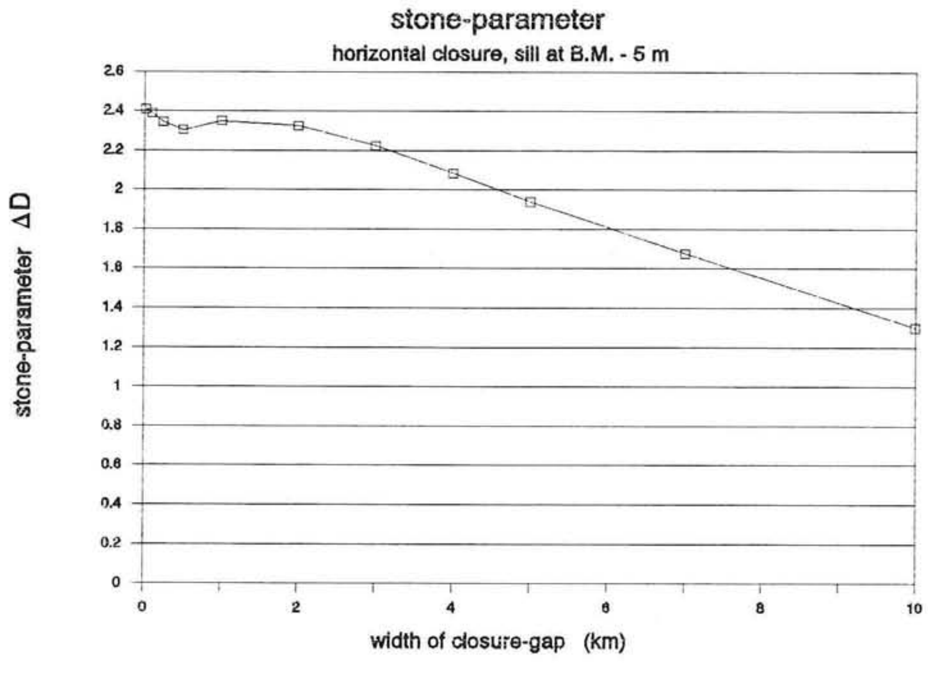


Figure 4.26 Values of ΔD_n for decreasing width of the closure gap with a sill at 5 m. - B.M.

The maximum value of ΔD is the same as at the application of a horizontal closure with the same boundary condition. Now, first a sill is built in the closure-gap on which a horizontal closure takes place. This effects in heavier stones during the whole horizontal constriction compared with a sill at 20 m. below B.M. The necessary quantity of heavy stones (e.g. ΔD > 2.0) is about 20 % larger applying a combined closure, although the cross-section of the dam-part to construct horizontally is smaller.

It is observed here that it is possible to reach a higher sill-level with dump-barges. Then less time is available for dumping. At approximately B.M. - 3 m. (depending on the tidal amplitude) critical flow sets in (when the gap is still 22 km. wide). One can build the sill even higher than this level. However, then dumping has to be carried out very accurate to prevent for extremely high local velocities caused by unevenness of the sill. Now the time during which an acceptable head-current is present is critical.

4.2.4 Sudden closure

Sudden closure is, in fact, a special way of combined closure. Here also a vertical closure is applied till a certain level. On the sill, large elements are placed. These elements either close the gap one by one or can be closed simultaneously.

For a small closure-gap (two or three caissons) where the velocities are relatively small the elements are caissons which are placed into the gap during a slack water period and thus reduce the gap one by one. In some cases even old ships have been used to accomplish the closure.

Another possibility is the use of sluice-caissons. Large concrete caissons with a gate are floated into position and sunk on the sill during slack-water periods (Figure 4.27). Usually the gates are positioned in the centre of the cross-section of the caissons. At the long sides of the caissons boards are placed to make them float. The gates stay open until all caissons are placed. The caissons do not diminish the flow-area much, so the increase in velocities is limited. When all caissons are sunk in their position, at slack water, all gates are closed simultaneously. The closure is a fact now. Around the caissons the final dam can be built. In this way the caissons only have a temporary function. The use of the caissons as definite elements of the barrage is also possible. These elements will have to be stronger because the design criteria are more severe. Also problems with the stability of the foundation (piping) do occur.

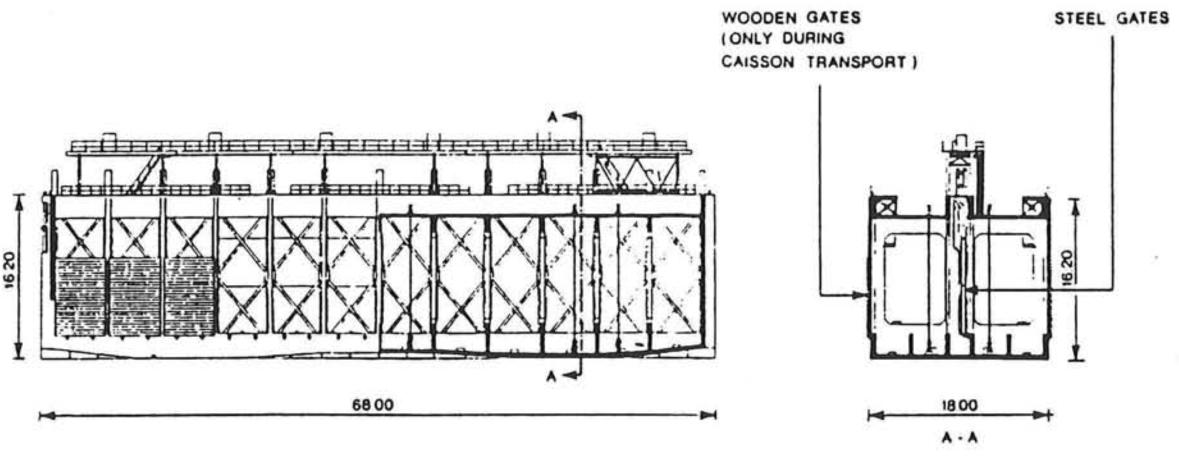


Figure 4.27 An example of a sluice-caisson used in the dutch Delta works

In the Gulf of Khambat the tidal range is large and the flow-velocities are high. When closed caissons are used the increase in velocities will be very large. The period available for the placing of the last few caissons will be too short. The placement operation can start when the velocities fall below 2.0 m/s and must be finished before the current reaches a velocity of 0.5 m/s in the opposite direction. Meanwhile the caisson must be sunk. A period of at least one hour is needed. This is certainly not possible with closed caissons. So only sluice-caissons are considered here for the final closure. Closed caissons can be used to constrict the gap till dimensions that still allow sinking of the sluice-caissons.

determination of the sill-level and caisson-height

The period available for the placement-operation is influenced by the sill-level. A higher sill means higher maximum velocities above the sill because of the smaller flow area. Thus the period in which they stay within the allowed range will be smaller. The available time for placing the caissons is bigger during a neap- than during a spring-tide (velocities are smaller). The time required for the sinking-operation of a caisson is less at low water slack than at high water slack. Then the increase in draught of the caissons will have to be less because the keel-clearance is smaller when the draught of the caisson (approximately half the height) and the sill-level are the same.

Out of this features it can be concluded that placement at least must be possible at low water slack during a neap-tide because this is a favourable time for a sinking-operation. The expected waterlevel at that time is about 1.5 m. above benchmark. Over the complete transport route the available depth must be sufficient. This probably asks for a dredged channel from the building site to the deeper channels.

The level of the upper side of the caisson is determined by the design-level that is chosen. On top of the caissons a road is necessary to transport equipment and materials. Overtopping of waves is acceptable under extreme circumstances. No labourers will have to set foot on the dam then. A water flow over the top of the caisson is not allowed. This strongly attacks the ballast placed on top of it. The stability of the ballast in relation to the overtopping waves must be studied in a further stage.

Here, as a first approach, a situation has been chosen with waves caused by a wind-speed which is exceeded during approximately 1 % of the time during the monsoon period.

	wind speeds [m/s]									
Direction [°]	0.00	0.51	1.80	3.34	5.40	8.50	11.10	14.10	17.20	20.80
150	0.81	0.81	0.81	0.81	0.54	0.27	0.00	0.00	0.00	0.00
180	2.02	2.02	1.08	0.67	0.13	0.13	0.00	0.00	0.00	0.00
210	7.02	7.02	6.07	4.32	2.29	0.67	0.40	0.00	0.00	0.00
240	37.65	37.65	37.11	35.36	28.07	15.25	7.96	2.83	0.81	0.00
270	28.34	28.34	27.94	24.16	17.00	8.10	3.10	0.94	0.00	0.00

Table 4.2 Exceedance probabilities [%] of wind speeds; period May-September; [source RNMI, lit. (10)]

Only waves out of the directions 180° and 210° can reach the dam with a considerable wave-height. In the directions 240° and 270° the fetch is limited or the waves are generated parallel to the dam. In direction 150° the wind-speed is limited. Here, the governing situation is a wind-speed of 11 m/s out of direction 210° (Table 4.2). The estimated fetch amounts to 50 km. (Figure 4.28). According to Brettschneider [lit. (15)] the wind-speed must be converted to a wind-stress factor U_A by the following equation (Eq 4.9):

$$U_A = 0.71 \cdot U^{1.23} = 0.71 \cdot 11^{1.23} = 13.6 \ [m/s] \qquad \mathbf{(4.9)}$$

In the nomogram of annex II a significant wave-height (H_s) of approximately 1.60 m can be read. The wave-period amounts to 5.5 s.

The wind set-up can be neglected in these circumstances (order of magnitude 1 to 4 cm). In combination with the high-water level of an average spring-tide (8.7 m + B.M.) a level of 10.3 m. is reached. It is noted that the combination with a spring-tide reduces the chance of occurrence.

When we assume that the resultant force R crosses the bottom at 1/3 b, b can be solved out of the ratio of Equation 4.12:

$$\frac{V}{H} = \frac{f}{\frac{1}{6}b} \qquad (4.12)$$

$$\frac{5.0 \cdot b \cdot 29}{1821} = \frac{12.8}{\frac{1}{6}b} \quad <=> \quad b = 31.1\,[m]$$

The forces V2 and V3 together increase the turn-over stability. The caisson can also be ballasted to increase the stability on the foundation. For the width of the caisson b = 30 m. is chosen here.

The length of the caisson is determined by nautical aspects during transport and by the foundation-conditions. As far as the foundation is concerned the main problem is the unevenness. When the length of the caisson is rather large and the bottom is smooth, the loads will not be evenly distributed but bore by the sill at unknown places. This problem can be avoided by a special design of the caisson where the bearing-points are known.

From a nautical point of view it is desirable that the ratio between the width and the length of the caisson is about 1 : 3 or 1 : 4. Also the number of placement-operations can be reduced by the application of larger caissons. For now the caisson-length is taken rather arbitrarily as 110 m. Including the space between the caissons the effective width of a caisson will be about 112 m. This means the construction and placement of 90 caissons.

summary

When sluice-caissons are used as discussed here, the closing of the Gulf of Khambat consists of a combination of three methods:

* vertical closure till a sill-height of 18 m. below B.M. is reached,
* horizontal constriction until the width of the closure-gap is reduced to 10 km.
* the placement of 90 prefabricated sluice-caissons with a length of 110 m., 29.5 m. high and 30 m. wide.

stability during transport and sinking

Here only the mass of the caisson itself is taken into consideration. The (positive) influence of the added mass is neglected. Added mass is the mass of the water that is influenced by and moves together with the caisson.

static stability

The caissons have to be stable during transport and sinking. They must maintain their vertical position. This can be achieved by building them symmetrical around their longitudinal axis. The stability can be checked by a calculation. A floating unit is stable if the meta centre M is above the centre of gravity G (Figure 4.32). A commonly used and safe value is a distance of at least 0.5 m. between M and G. For calculation, distance MB is needed (Eq. 4.13).

$$MB = \frac{I}{V} = \frac{\frac{1}{12} l b^3}{V} \tag{4.13}$$

M	=	transverse meta centre
B	=	centre of buoyancy (at 0.5 * draught above the bottom)
G	=	centre of gravity (at distance g above the bottom)
I	=	transverse moment of inertia of the water plane [m^4]
V	=	immersed volume [m^3]
l	=	length of the caisson [m]
b	=	width of the caisson [m]
d	=	draught [m]
MG	=	h_M = height of the transverse meta centre above the centre of gravity

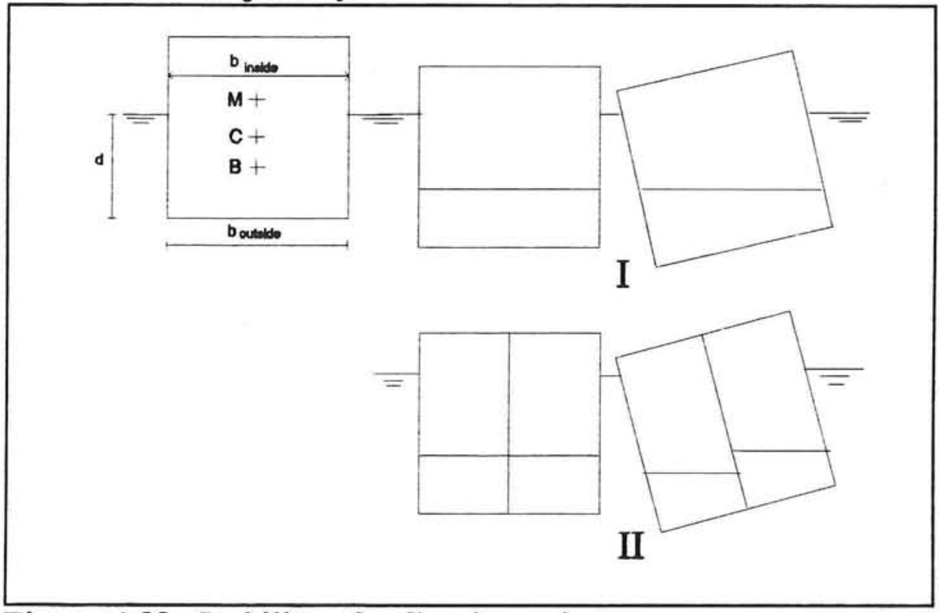


Figure 4.32 Stability of a floating caisson

During sinking the situation changes. The increase in draught let B move upward with respect to the bottom of the unit. The position of G (in general) moves in the direction of the bottom. Distance GB becomes smaller. Water inside the caisson (sit. I) decreases the moment of inertia (Eq 4.14).

$$MB = \frac{I_{outside} - I_{inside}}{V} = \frac{\frac{1}{12} l b_{out}^3 - \frac{1}{12} l b_{ins}^3}{V} \tag{4.14}$$

In case of a caisson the width outside and inside do not differ much. The value of MB approaches 0. The solution is the use of stabilizing boards. A single board in the centre (sit II) decreases I_{inside} with a factor four. (Eq 4.15). The steel sluice gate can be used for this purpose.

$$MB = \frac{I_{outside} - I_{inside}}{V} = \frac{\frac{1}{12} l b_{out}^3 - 2 \frac{1}{12} l (\frac{1}{2} b_{ins})^3}{V} \tag{4.15}$$

The stability of the caisson with dimensions L*B*H of 110*30*29.5 m^3 have been checked.

$$d = \frac{\gamma_{caisson}}{\gamma_{water}} \cdot H = \frac{5.0}{10.2} \cdot 29.5 = 14.46 \quad [m]$$

$$MB = \frac{I}{V} = \frac{\frac{1}{12} \cdot 110 \cdot 30^3}{110 \cdot 30 \cdot 14.46} = 5.19 \quad [m]$$

The centre of gravity is assumed to be located at a distance g of 0.4 * H above the bottom, thus g = 11.8 m.

$$h_M = MB + 0.5 \cdot d - g = 5.19 + 0.5 \cdot 14.46 - 11.80 = 0.62 \quad [m]$$

During transport the stability of the caisson is found to be sufficient. The distance h_M is larger than 0.5 m.

The situation during sinking is more critical. A small layer of water (suppose an increase in draught till 14.55 m.) decreases I not much if it is evenly distributed in the compartments between the ribs on the caisson floor. Here it is assumed that five longitudinal ribs divide the lower part of the caisson in six compartments. The width between the ribs becomes 3.83 m. (Figure 4.33).

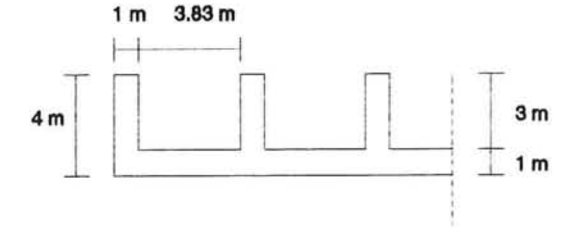


Figure 4.33 Ribs on the caisson bottom

$$MB = \frac{\frac{1}{12} \cdot 110 \cdot 30^3 - 6\frac{1}{12} \cdot 110 \cdot 3.83^3}{110 \cdot 30 \cdot 14.55} = 5.09 \qquad [m]$$

The small layer of water hardly influences the position of the centre of gravity. This small positive influence is neglected here, so:

$$h_M = 5.09 + 0.5 \cdot 14.55 - 11.80 = 0.57 \quad [m]$$

The stability of the caisson is still sufficient.

The next critical stage is when the water-level in the caisson rises above the ribs, which means a layer of 3 m. water. Meanwhile the draught has increased considerably and the position of G has moved in the direction of the bottom. It is assumed that 60 % of the bottom area is covered with water. (When the keel clearance between the sill and the bottom of the caisson is less than 0.6 * 3 = 1.8 m. this situation is never reached.)

The draught becomes:

$$d = d_{old}+0.6\,h_{rib} = 14.46 + 1.8 = 16.26 \quad [m]$$

$$MB = \frac{\frac{1}{12}\cdot 110\cdot 30^3 - 2\frac{1}{12}\cdot 110\cdot 15^3}{110\cdot 30\cdot 16.26} = 3.46 \quad [m]$$

$$W_{caisson} = 30\cdot 110\cdot 29.5\cdot 5.0 = 486.75\cdot 10^3 \quad [kN]$$

$$W_{water} = 0.6\cdot 3\cdot 30\cdot 110\cdot 10.2 = 60.59\cdot 10^3 \quad [kN]$$

$$g_{new} = \frac{g_{cais}\cdot W_{cais} + g_{wat}\cdot W_{wat}}{W_{cais} + W_{wat}} = \frac{11.80\cdot 486.75 + 2.5\cdot 60.59}{486.75 + 60.59} = 10.77 \quad [m]$$

$$h_M = 3.46 + 0.5\cdot 16.24 - 10.77 = 0.81 \quad [m]$$

The stability again is not expected to cause any problems. If the even distribution of the water during sinking turns to be rather expensive, the compartments between the ribs can be filled with ballast. Then similar computations can be made.

<u>dynamic stability</u>

The dynamic stability is determined by the ratio between the wave period and the specific period of the caisson. If these two values are almost similar resonance occurs. The specific period can be determined using Eq. 4.16 to 4.18.

$$T_s = \frac{2\pi i}{\sqrt{g h_M}} \tag{4.16}$$

with:

$$i = \sqrt{\frac{J}{\rho V}} = \sqrt{\frac{J}{m}} \tag{4.17}$$

and:

$$J = \int r^2\, dm \approx \sum m_i\cdot r_i^2 \tag{4.18}$$

in which: r_i = distance of element i to the rotation centre

In order to estimate the specific period of the caisson the following assumptions were made:

- the mass of the floating caisson m = 46.875 * 10^6 kg;
- the bottom floor is schematized as a uniform slab of concrete of 1 + 0.4 * 3 = 2.2 m. thick (60 % free space between the ribs);
- the deck is assumed to be 0.80 m. thick, including ribs;
- concrete: 2.5 * 10^3 kg/m^3

The masses m1, m3 and m2 that are supposed to be concentrated in respectively G1, G3 and G2 (Figure 4.34) become:

$m1 = 30 \cdot 110 \cdot 0.8 \cdot 2.5 \cdot 10^3 = 6.600 \cdot 10^6$ [kg]

$m3 = 30 \cdot 110 \cdot 2.2 \cdot 2.5 \cdot 10^3 = 18.150 \cdot 10^6$ [kg]

$m2 = m - m1 - m3 = 23.925 \cdot 10^6$ [kg]

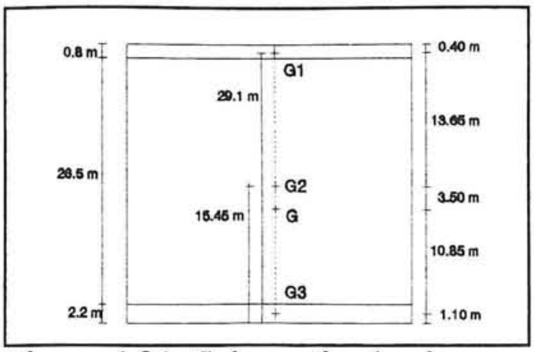


Figure 4.34 Schematized caisson

The position of G can be determined out of the values of M_i and g_i, when g_i is the distance between M_i and the bottom.

$$g = \frac{\sum m_i \cdot g_i}{\sum m_i} = \frac{6.600 \cdot 29.1 + 23.925 \cdot 15.45 + 18.150 \cdot 1.1}{6.600 + 23.925 + 18.150} = 11.95 \quad [m]$$

The distance g = 11.85 / 29.5 = 0.405 h, which is approximately equal to the supposed value of 0.4 h.
With Equation 4.18 J becomes:

$$J = 6.600 \cdot 10^6 \cdot 17.15^2 + 23.925 \cdot 10^6 \cdot 3.50^2 + 18.150 \cdot 10^6 \cdot 10.85^2$$
$$= 4.37 \cdot 10^9 \quad [kgm^2]$$

$$i = \sqrt{\frac{4.37 \cdot 10^9}{48.675 \cdot 10^6}} = 9.48 \quad [m]$$

with the height of the meta centre h_M = 0.62 m. (see section static stability) and Eq. 4.17 the specific period T_o becomes:

$$T_o = \frac{2\pi \cdot 9.48}{\sqrt{9.8 \cdot 0.62}} = 24.16 \quad [s]$$

The response of the caisson can be predicted with Eq. 4.19. when a value of the period of the actual waves is known. For the determination of the caisson height a wave height of 1.60 m. had been chosen with period of 5.5 s. No caisson will be placed under this conditions. However it is possible that long-periodic swell of the Indian Ocean penetrates the Gulf of Khambat. Therefore still a value of 10 s has been chosen here (a smaller value decreases the response). Even then the response of the caisson is limited to 1/5 of the wave amplitude:

$$\frac{T^2}{T^2 - T_o^2} = \frac{1}{1 - \frac{T_o^2}{T^2}} = \frac{1}{1 - \frac{24.16^2}{10^2}} = -0.21 \qquad \textbf{(4.19)}$$

The negative value implies that the caisson rotates in the opposite direction of the moving torque.

4.3 Chosen method of closure

In order to determine the way of closure one should realise that no uniform dam-profile is planned to be constructed along the whole chosen alignment. In the final situation of the development scheme, in the Gulf of Khambat several main construction parts can be distinguished:

* a closure dam along a large part of the chosen alignment;

* a combined structure in which the intake works (with a width of approximately 5500 m., determined in chapter 5) and the turbines of the tidal power station are embedded;

* a dam dividing the Gulf of Khambat in a fresh water basin and a tidal basin belonging to the tidal power station;

* additional structures like a spill-way and ship-locks.

In India the network of railways is rather dense. It is assumed that a permanent railway will be placed on the final dam. On both sides of the alignment new railway sections will have to be built to connect the existing network with the starting-points of the dam. These sections will be constructed preceding the closure operation. Thus they already can be used to transport the necessary rock from the quarries (that are also close to the railway, see the maps of annex III) to the dam site or loading site of the dump barges.

4.3.1 Alternative construction sequences

Mainly the construction of the intake works and power houses greatly influences the possible closure methods. Roughly, three construction strategies can be distinguished:

I First the closure of the Gulf of Khambat is completed using one of the methods described in section 4.2. Then, in the basin that is (almost) free of the influence of the tides, behind the closure dam the tidal power station is built. This can be done by construction at the site in a enormous building pit or placement of prefabricated caissons. After completion the dam is removed where necessary.

II A second option is the immediate enclosure of the structures of the tidal power station in the closure dam. This also can be done by construction at the site or prefabrication preceding the total closure.

The turbines of the tidal power station have to be located sufficiently deep under the water surface to prevent for cavitation. Thus, they have to be located where the depth is sufficient and dredging will be limited. The necessary bottom level is estimated to be 20 m. - B.M.. This is only available in the deepest gully that crosses the alignment (section 25 in Figure 4.29). Because the turbines are situated at considerable depth it is possible to locate the intake works on top of them. Thus, two parts can be embedded in one structure.

Construction at the site asks for a building pit of enormous dimensions which has to be impervious to water. Little seepage can be accepted and removed by draining. In case of a building pit when the Gulf of Khambat is still open, it blocks a large part of the discharges during the tidal cycle because it is situated in the main gully. A totally different flow pattern will develop, which, because of the long time of construction, will probably result in large morphological changes. The dams surrounding the trench will be attacked severely by currents parallel to the dam.

The disadvantages described are supposed to be so large that building at the site of the tidal power station in case of an open Gulf of Khambat is out of the question. In case of a preceding closure, the dams surrounding the pit all have to be constructed and removed again after completion, which is very expensive. Besides, also the supply of construction-materials will be more difficult than at a building-site that can be reached over land. Also in this case construction at the site is rejected.

A total closure preceding the placement of the caissons for the tidal power station asks for large stones or the use of caissons (section 4.2). The advantage is that the elements of the tidal power station can be placed in quiet water. After that however, about 6 km. of the closure dam will have to be removed again.

The use of specially designed sluice-caissons can be avoided. When the intake works and turbines are embedded in turbine caissons the intake works can be used for this purpose. However, this turbine caissons will have to be positioned very accurately. This, because they are part of the final structure and not embedded in a dam which is usually done with sluice-caissons. Accurate positioning of caissons is only possible if the flow velocities are low (see section 4.2.4). Limitation of the velocities is reached by keeping the flow area as large as possible during the sinking procedure. This implies that first a sill is built on which the turbine caissons are placed. After that the closure is completed using rock. Here a combined closure is chosen because dump barges can not complete a vertical closure. For a purely horizontal closure the transport capacity on top of the dam is too limited.

The use of the intake works as sluices during the final closure limits the stone-masses. On the other hand it asks for a stronger bottom protection than necessary during exploitation of the tidal power station.

The proposed closure operation becomes:

* First a sill is constructed at the position of the tidal power station by means of dump barges. The sill-level is 20 m. - B.M.. To get a good foundation for the turbine caissons the sill must be even. This implies controlled dumping. The barges will have to be anchored and transversely shifted. Measurements will show whether further levelling is necessary or not. The rock is transported by rail from the quarry to the loading berth.

* The turbine caissons containing the intake works and turbine-tubes of the tidal power station can be placed now accurately on the sill. After placement the sluices are opened. The dimensions of the turbine caissons are somewhat different from these of the sluice caisson described in section 4.2.4. They are part of the permanent construction and have to withstand the loads in governing conditions. When the upper level is estimated at 20 m. above B.M. the height of a turbine caisson becomes 40 m. The width of the turbine caissons is determined by the length of the turbine tubes. The estimation of the width of a turbine caisson becomes 60 m (see chapter 5).

* Stones are dumped now parallel to the turbine caissons to prevent for sliding. Both the abutment caissons are protected at the heads against instability, caused by the high velocities which will occur during final closure. This will be done by small dam-parts constructed by means of dump-barges and crane-ships. When heavy stones are used the length can be restricted to some tens of meters.

* Meanwhile the sill in the two remaining closure gaps can be heightened. The chosen sill-level is 5 m. - B.M. A higher sill is possible but then the dumping process will be more critical. At the chosen level the influence of the unevenness of the sill will be limited. The local differences in flow velocities will not be very large. This will limit possible erosion of the sill during the monsoon period. The sill-level can be further optimized comparing costs of horizontal and vertical closure and expected damage.

* Finally the horizontal constriction can be carried out from both banks. This is planned to take place with the help of special equipment. The rock is transported by rail to, and over, the closure dam in wagons that can be unloaded by a special designed gantry crane [see section 4.3.2 and Figure 4.41].
  The foundation of the temporary railway must be protected against large eroding influences. Considerable overtopping of water and waves attacks the upper layer of the closure dam. This layer consist of small material in order to level the crest. As the design level of the crest 11.0 m. + B.M. has been chosen. This level is somewhat higher than HAT (10.9 m. + B.M.). In section 4.2.4 a comparison has been made with the combination of waves which are exceeded during 1 % of the time in the monsoon period and a high water of an average spring-tide. This resulted in a level of 10.30 m. + B.M.. Wave run-up is not taken into account there. The level of 11.0 m. + B.M. gives some margin for waves. Some damage caused by overtopping in case of higher waves will be accepted.
  When the constriction is finished the sluices of the intake works can be closed to complete the closure.

4.3.2 Design strategy and working-out of the chosen alternative

The construction of the closure dam will take several years. So the design conditions can be differentiated for different parts of the closure dam. The following items must be taken into account:

* The boundary condition of average spring-tide is not sufficient for the total construction time. During this period most likely a higher waterlevel will occur. For the final closure it can be used if this closure is planned in a period that this waterlevel is not exceeded.

* The influence of wind set-up and wave-loads is important for parts of the construction that have to survive a monsoon period. However when a sill is constructed far enough below the mean sea level (B.M. - 5 m. will be sufficient) the wind set-up penetrates in the basin too. So the extra head-difference will be limited. The use of Highest Astronomical Tide as a boundary condition is regarded to be safe enough, representing the combination of an average spring-tide and waves. In more extreme circumstances some loss of stones will be accepted.

- * The dam is pervious to water. This reduces the momentary head-differences over the closure dam. Here, when the final closure takes place a considerable gap is present by means of the sluices of the intake works. Then the favourable influence is relatively small (see Figure IV.5 in annex IV), certainly when we may only take into account half the reduction. Therefor the permeability of the dam is neglected in the calculations here.

- * The upper layer of the sill is attacked by the vortex streets caused by the horizontal constriction. Here a factor on the stone parameter ΔD must be applied to prevent severe damage. In Annex IV.2 it is found that a value of 1.2 on the velocity is appropriate which results in factor $1.2^2 = 1.44$ on ΔD_n. In the situation of a combined closure the sill aligns the flow. Still, because of the uncertainty about the magnitude of this favourable influence a factor 1.44 has been applied on ΔD_n.

- * Some loss of stones can be accepted during severe circumstances when this does not lead to excessive damage. For a more definite design a risk - cost analysis should be carried out.

In general it can be said dat further research concerning the waterlevels that can be expected during the construction period is necessary. This will show if the boundary conditions assumed here are allowed.

In order to be able to determine the necessary grading on a certain location the values of Table 4.3 have been used in combination with graphs like Figure 4.35. For more information about this table see annex III.

grading	M_{50}	average D_n [m]	ΔD_n limits [m] 2700 kg/m³	2900 kg/m³
10 - 60 kg	35 kg	0.23	0.0 - 0.4	0.0 - 0.4
60 - 300 kg	180 kg	0.41	0.4 - 0.6	0.4 - 0.7
300 - 1 000 kg	650 kg	0.62	0.6 - 1.0	0.7 - 1.1
1 000 - 3 000 kg	2 000 kg	0.90	1.0 - 1.4	1.1 - 1.6
3 000 - 6 000 kg	4 500 kg	1.18	1.4 - 1.9	1.6 - 2.1
6 000 - 10 000 kg	8 000 kg	1.42	1.9 - 2.3	2.1 - 2.5
10 000 - 15 000 kg	12 500 kg	1.67	2.3 - 2.7	2.5 - 2.9

Table 4.3 ΔD_n limits and dimensions of gradings

foundation sill for caissons

The upper level of the sill becomes 20 M. - B.M.. Here the value of ΔD, applying a uniform sill over the whole width, stays below 0.25 m. even when HAT is considered. This means that during normal conditions the use of tout-venant or the stone-class 10 - 60 kg. will be sufficient. However the use of the intake works as sluices during closure and the exploitation of the tidal power station asks for a stronger armour layer. This layer can be brought in place after placement of the caissons. The necessary stone-mass of the armour layer has been determined using Figure 4.35. When the last gap is closed the width of the sluices will still be 5500 m. This final

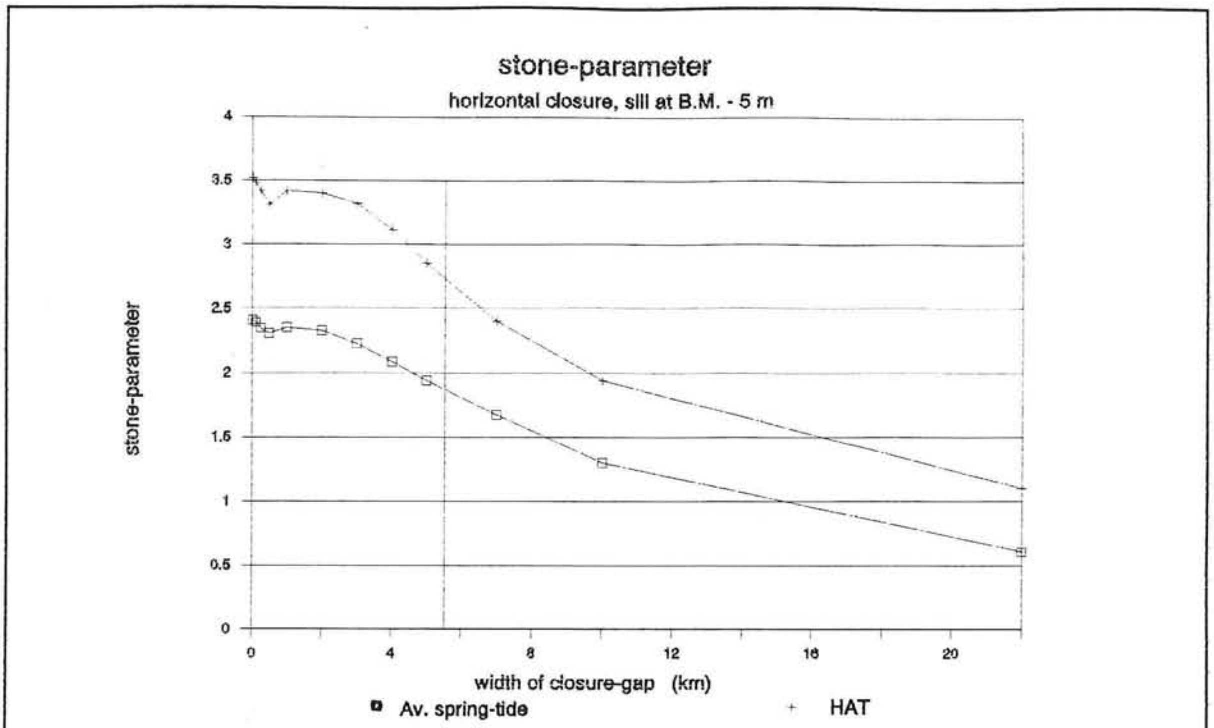


Figure 4.35 ΔD_n as function of the width of the closure gap

closure will take place during quiet weather in absence of extreme high waters. So the graph of average spring-tide will be safe enough, certainly when it is kept in mind that the flow not directly attacks the sill, assumed that the turbine tubes are closed. (During exploitation of the tidal power station the head-differences and discharges will be less.) A value of approximately $\Delta D = 1.8$ can be read. This leads to the grading of 3000 - 6000 kg [see Table 4.3]. It is wise to dump the three intermediate layers too, in order to get a good filter. The thickness of the layers of these gradings is estimated to be three to five times D_n (Table 4.3). The armour layer is thicker because of its permanent function.

grading	thickness
60 - 300 [kg]	2.0 [m]
300 - 1 000 [kg]	2.0 [m]
1 000 - 3 000 [kg]	3.0 [m]
3 000 - 6 000 [kg]	5.0 [m]

The sill-crest must accommodate space for the placement of turbine caissons with an estimated width of 60 m. (The sluice-caissons were 30 m. wide.) The width of the sill-crest is set on 70 m. Considered the enormous caissons that have to be placed on top of it and the space needed to dump rock after sinking the total width becomes 80 m (Figure 4.36). The cross-section is drawn for a location with a depth equal to the schematized depth of the DUFLOW-section. It is expected that the local depth will be less.

the sinking operation of the caissons

When the sill is finished over a width of about 7000 m. (5500 is the net width of the intake works, including parts of construction the gross width becomes approximately 6500 m.) the caissons of the tidal power station can be sunk on top of it. The accuracy of the operation is very important. So the use of winches and fixed anchor points is demanded. It could be advantageous to start in the middle of the sill with the first caisson. Then in turn on both sides of the first one, a caisson can be placed. Thus, the time for preparation before the next caisson arrives is larger, when the time between two subsequently sinking operations is kept constant. After sinking the temporary boards of the caisson are removed and the sluices opened. Now also the stabilizing rock and the armour layer can be placed or carefully dumped (class 3 - 6 tons in Figure 4.36). The heads of the caisson row are stabilized by rock in the grading of 10 - 15 tons (Figure 4.37).

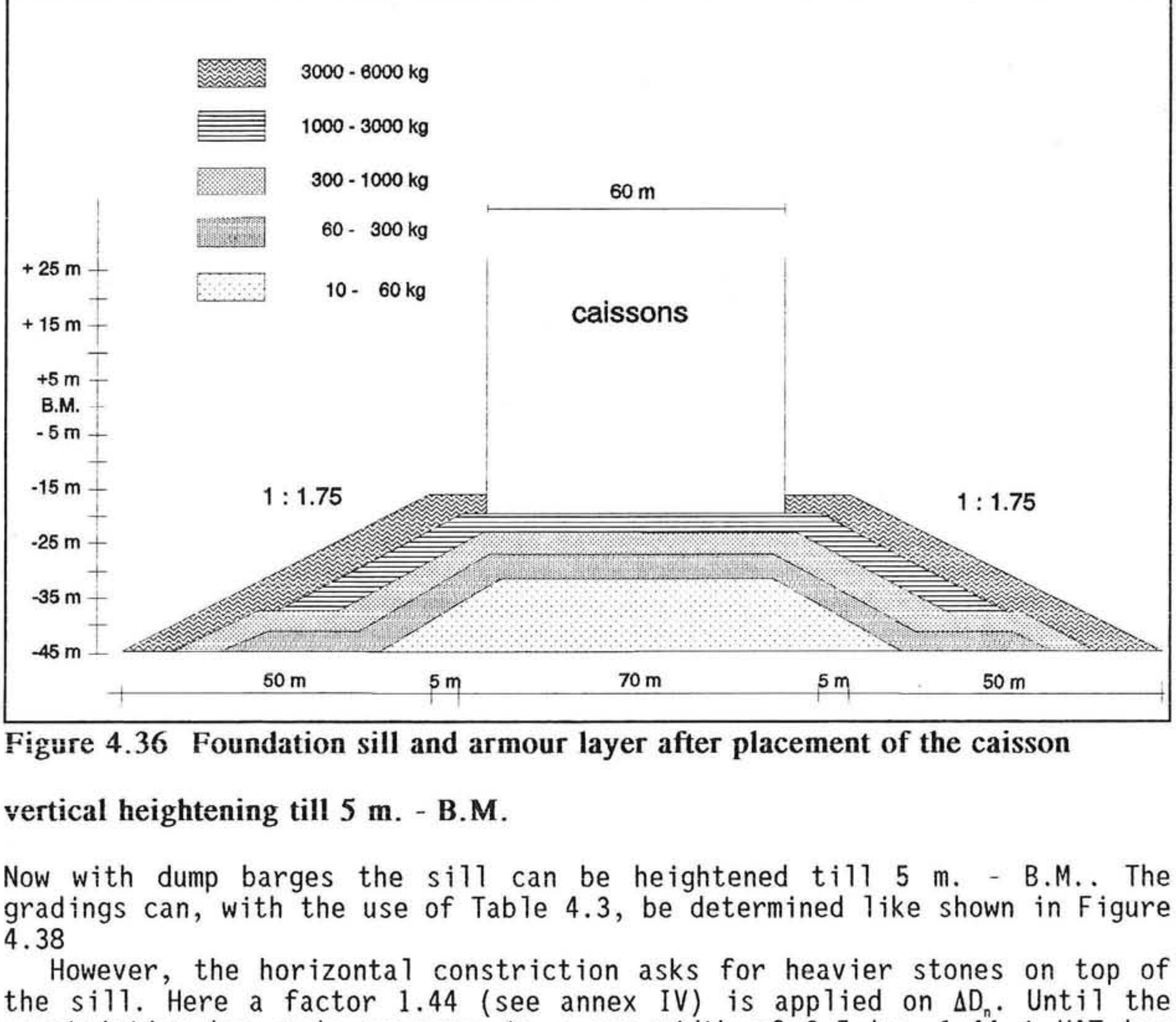


Figure 4.36 Foundation sill and armour layer after placement of the caisson

vertical heightening till 5 m. - B.M.

Now with dump barges the sill can be heightened till 5 m. - B.M.. The gradings can, with the use of Table 4.3, be determined like shown in Figure 4.38

However, the horizontal constriction asks for heavier stones on top of the sill. Here a factor 1.44 (see annex IV) is applied on ΔD_n. Until the constriction has made progress to a gap-width of 9.5 km, 1.44 * HAT has been taken as a boundary condition (see the upper step-line of Figure 4.39)[1]. Further constriction will take place under less severe conditions. Grading 10 -15 tons is used for the rest of the upper sill layer. Notice that the factor 1.44 just slowly diminishes and the porosity of the dam helps to prevent damage. The heavy top layers imply the application of intermediate layers (Figure 4.40). Not all intermediate gradings are used in a vertical cross-section. The largest step is a layer of 10 -15 tons on top of one of 1 - 3 tons. Table 4.3 shows that the ratio of the D_n-values

1 *In Figure 4.39 for a single rubble class the stones with a specific* density *of 2700 kg/m³ and 2900 kg/m³ have been distinguished. In Figure 4.40 this has not been done.*

then amounts to 1.67/0.90 = 1.86. For a filter construction this is allowed to be 3 - 5. Thus no problems concerning geometrical closeness are expected.

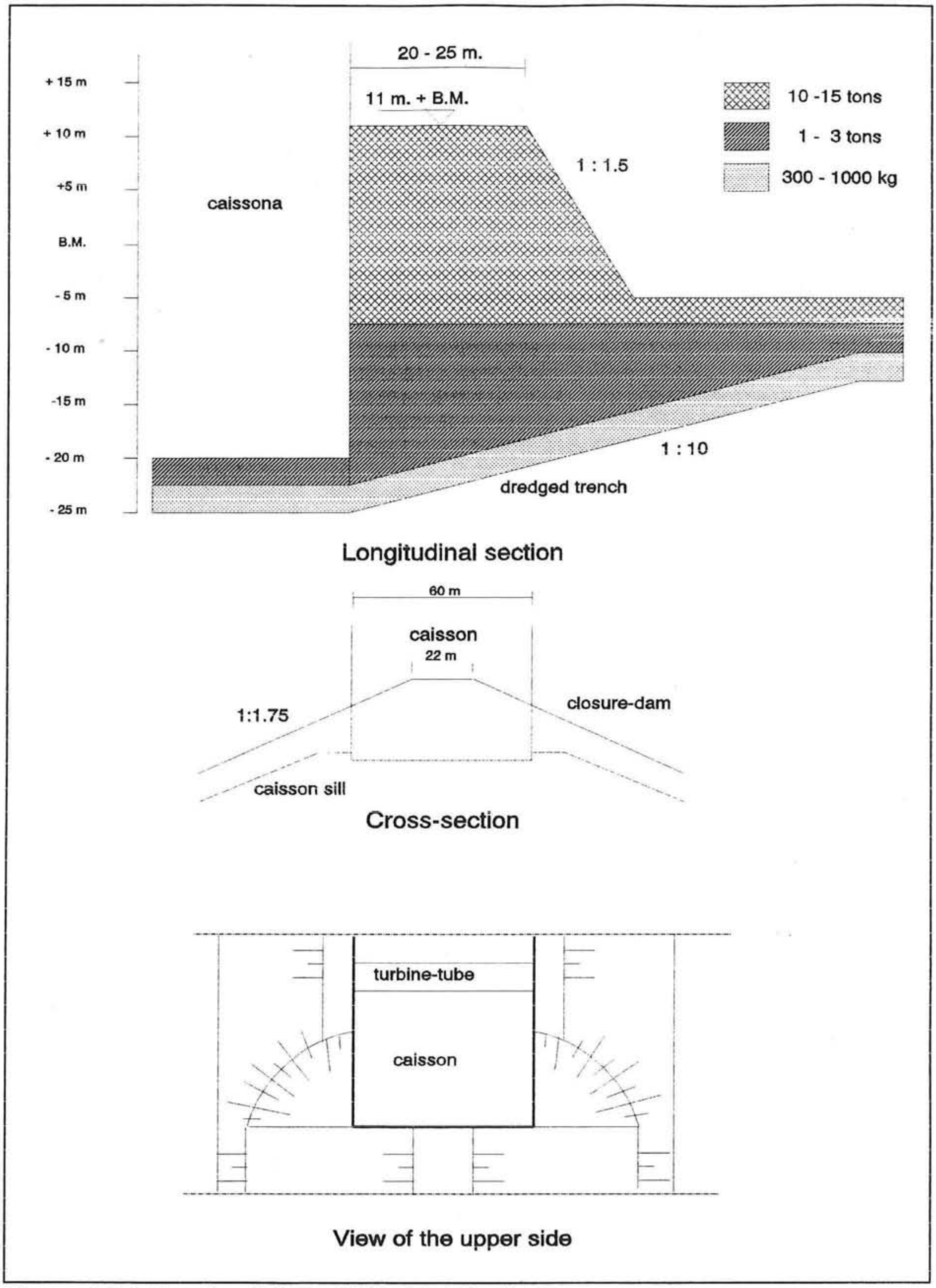


Figure 4.37 Details of the stabilized head caisson

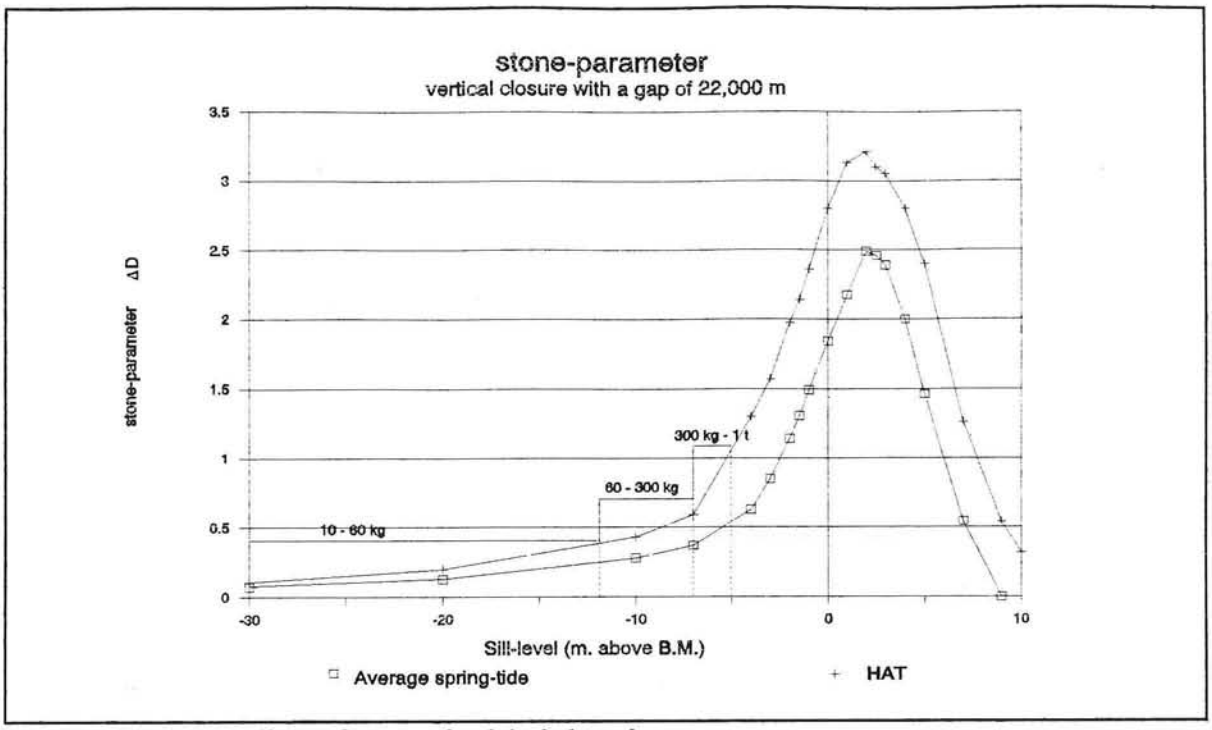


Figure 4.38 Gradings for vertical heightening

horizontal constriction

On the sill the closure gap can now be constricted horizontally working from both banks. The gradings have been determined as shown in the lower step-line of Figure 4.39. Here HAT is taken as a boundary condition except for the last 2.5 km. Here a grading of 6 - 10 tons will be sufficient when the final closure is planned in the non-monsoon period in the absence of extremely high waters.

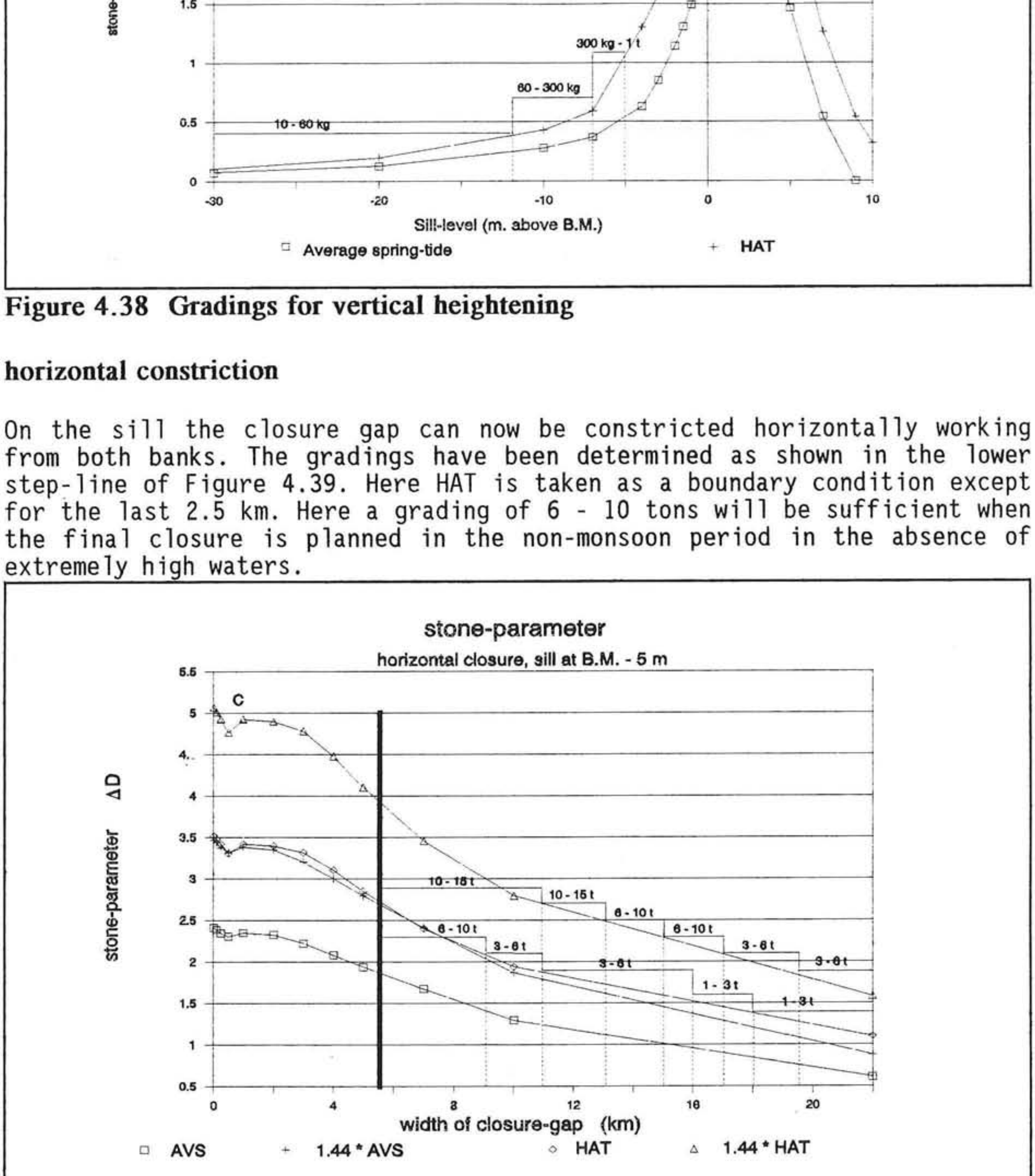


Figure 4.39 Gradings used for horizontal constriction, sill at B.M. -5 m.

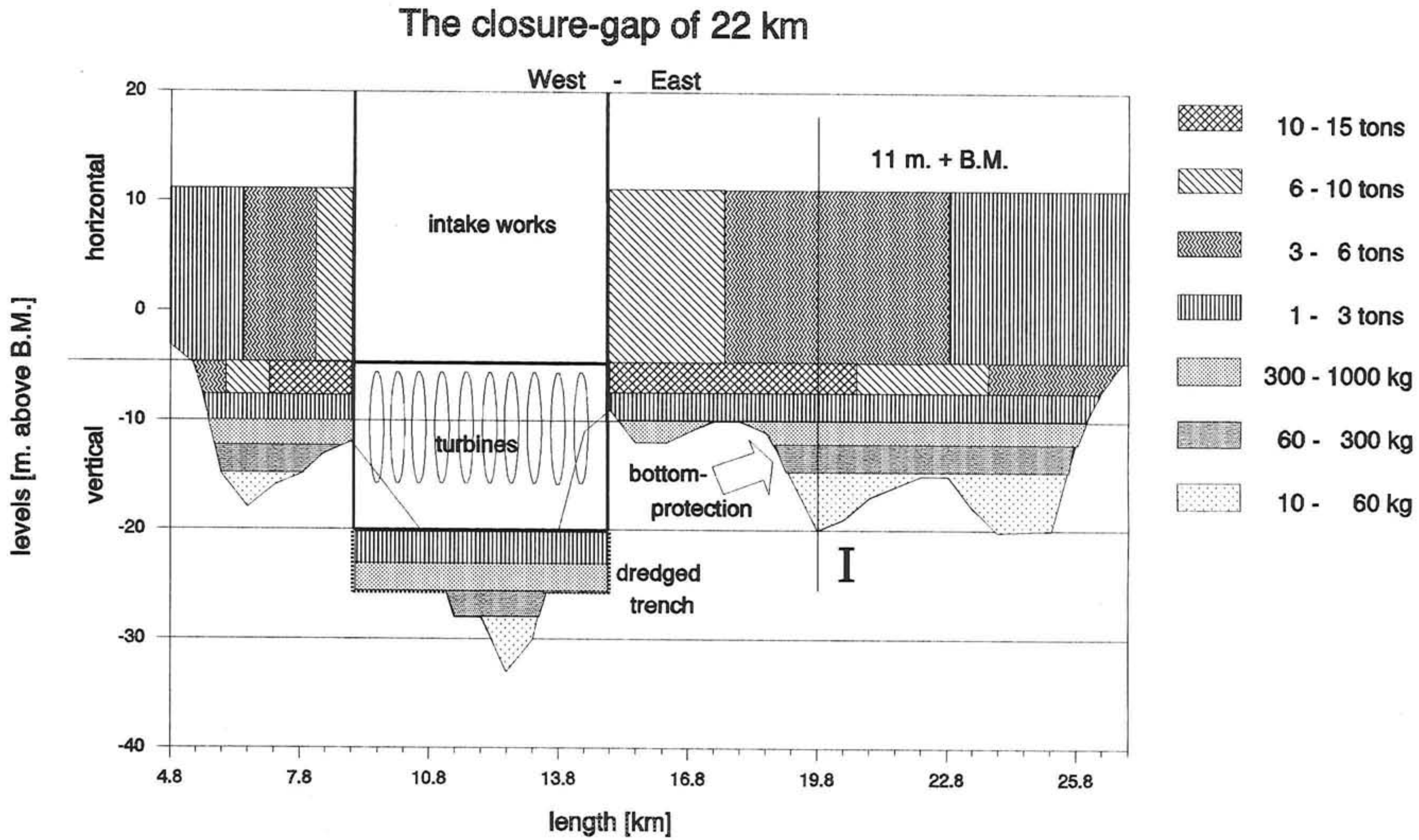


Figure 4.40 Overview of the gradings used during the closure

The ratio of the length of the gaps west and east of the tidal power station is approximately 1 : 3. In order to contract the flow in the direction of the sluices the speed of propagation of the constriction will approximately have to be 1:3 too. The length of the sections with different gradings is distributed proportionally (Figure 4.40). The location of the tidal power station is disadvantageous concerning the quantities of rock that have to be dumped on either side of the caissons (ratio 1 : 3). When the total alignment is taken, including the dam section crossing Alia Bet, the situation concerning transport distances is even worse than 1 : 3.

work-method

The situation described asks for the application of special equipment, certainly at the eastern part of the closure gap. It is proposed to place a temporary railway on top of the closure dam. The rock is transported by rail from the quarry directly to the dumping site. The containers are picked up by a specially designed gantry crane on caterpillar tracks that dumps their load (Figure 4.41). At least over the last few km. a second railway for the empty wagons is embedded in the road c.q. space for other equipment. After dumping the crane will put the wagons on this railway. By means of points the empty wagons return on the other railway behind the loaded wagons. So no complete double-track will be necessary.

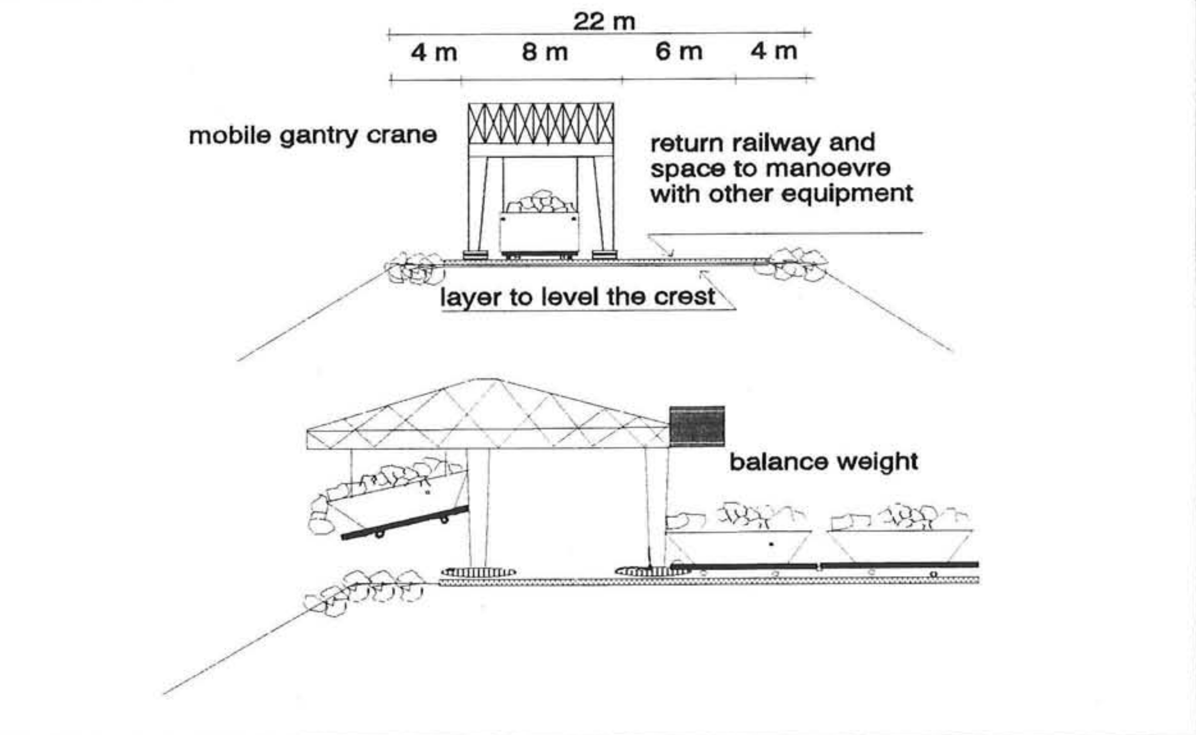


Figure 4.41 Dumping stone with a gantry crane

The necessary width of the closure dam is determined by the space that is required to apply the described working method. It is estimated at 22 m. Then cross-section I can be drawn (Figure 4.42). A slope of 1 : 1.75 has been chosen. A slope of 1 : 1.5 can hardly be accomplished when stone is dumped at considerable depth.

At the western side of the closure gap the speed of propagation is allowed to be less. Here dump trucks can possibly be used. This implies extra transhipment assuming the rock is transported by rail to the starting-point of the dam.

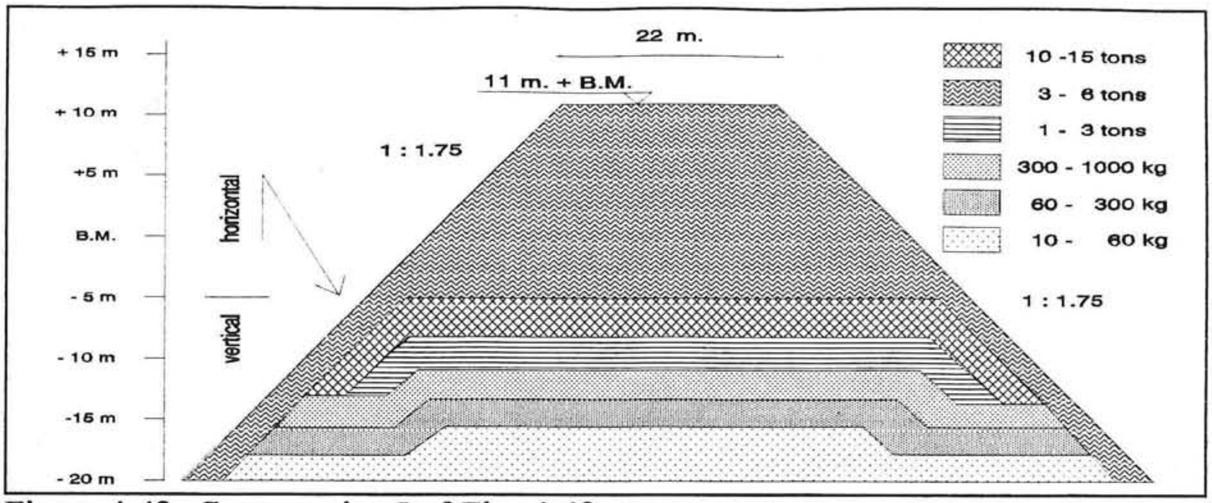


Figure 4.42 Cross section I of Fig. 4.40

Seepage

The turbine caissons are placed on a pervious sill. This implies underseepage and seepage around the abutments when dam construction is completed. Underseepage causes no dangerous situation. The way of calculation of the necessary gradings is such that a flow through the sill is already taken into account. The governing circumstances taken for closure will not be exceeded because the head-differences over the dam will be limited. The area of the tidal basin is decreased after construction of the basin dams. In extreme circumstances the head-difference can be limited by manipulation with the intake works. Though the porosity of the dam has a negative influence on the energy production. The head difference present in order to produce electricity causes a flow through the turbines but through the sill as well. This loss of energy production must be kept as small as possible.

The only way that seems possible to limit the flow after placement of the turbine caissons is a decrease of the sill porosity by means of grouting with cement grout. The success of this method is doubtful because the period during which the flow velocities are acceptably low to prevent for washing of the cement grout is very small. This can be improved by filling the large holes with gravel, if possible. Both the grouting and the gravel transport ask for provisions in the caisson bottoms and the applicability is questionable.

At the abutments of the caissons the final dam meets. This final dam consists of rock and sand which must be protected against erosion. Here filter layers can be used to prevent for erosion of the dam profile. Vertical ribs on the heads of the abutment caissons can be used to increase the leakage length. Rock is dumped between them.

No satisfying solution has been found yet to limit the seepage resulting from the chosen construction method. Further study combined with experiments will be necessary.

4.4 Bottom protection works

The construction of the closure dam in all phases causes a disturbance of the existing vertical velocity distribution of the flow. Heightening of the sill results in turbulence, eddies and higher local velocities. If the seabed is composed of loose sediments severe scouring can occur. If the scour hole becomes to deep and/or is situated to close to the dam, the stability of this dam is endangered. In order to ensure its stability, a certain part of the seabed under and, in case of a flow in two directions, on both sides of the dam will have to be protected.

In the Gulf of Khambat the seabed consists of sand, so scouring can be expected. Partly the bottom protection has a permanent function here. On both sides of the tidal power station a bottom protection is necessary during operation. Due to the choice of an ebb-generating system (see chapter 5) the seabed in the basin is attacked by the flow through the intake structures and the water-flow entering the turbines. On the sea-side the exit flow of the turbines attacks the bottom. The flow-velocities during exploitation are much smaller then during closure. The velocities of the intake-flow will be smaller because of the smaller basin-area. During energy production both the entrance and the exit speed of the turbines reach a maximum value in the range of 1.5 - 2.0 m/s. Under these conditions small gravel is already stable. Thus, the closure operation is taken as the governing situation.

functions of bottom protection works

<u>sand-tight</u> The purpose of the bottom protection is primarily to keep the bottom material in place. It can be constructed as a filter that is pervious to water. This to prevent for the occurrence of water overpressures that will lift the protection. A heavy impervious protection is only used when a large head-difference must be phased out over a considerable distance in order to limit the velocities and the discharge of the groundwater-flow.

<u>length</u> The length of the bottom protection must be such that the scour hole is situated sufficiently far from the dam or construction to prevent for a slide or other instabilities.

<u>stable</u> The upper layer must hold the lower layers and resist the flow velocities, wave-loads and water-pressure. In the case of a pervious bottom protection the governing load is the flow velocity.

sand-tight

The bottom protection placed before the closure of the Gulf of Khambat has to fix the bottom material. A pervious filter construction is chosen to fulfil this function.

Again the use of locally available materials is desirable in order to limit the costs. This leads to the use of rock. The stability of the upper layer under governing conditions asks for rather heavy stones. To prevent erosion of the bottom material several intermediate layers are necessary. The thickness of these layers is at least 0.5 m. for small gravel to 3 times D_n. The construction height of the filter in this way becomes several meters. A common way to limit the construction height is the use of a fascine mattress in combination with quarry stone. The construction of

fascine mattresses is very labour-intensive. Nowadays a combination of geo-textiles and fascine is used. In India the use of bamboo can be considered in stead of fascine. Bamboo however is less flexible than fascine.
Considering the enormous bottom-area that has to be protected it may be advantageous to design special equipment to produce and place bottom protection mattresses. These mattresses can also consist of geo-textile and blocks that are connected to it. Here the stability of an upper layer of loose stone is determined.

length

The length of the bottom-protection (L in Figure 4.43) depends on:
- the maximum depth of the scour hole, h_{max};
- the slope of the upstream end of the scour hole, ß;
- the foundation of the construction;
- the materials present in the construction and of the bottom.

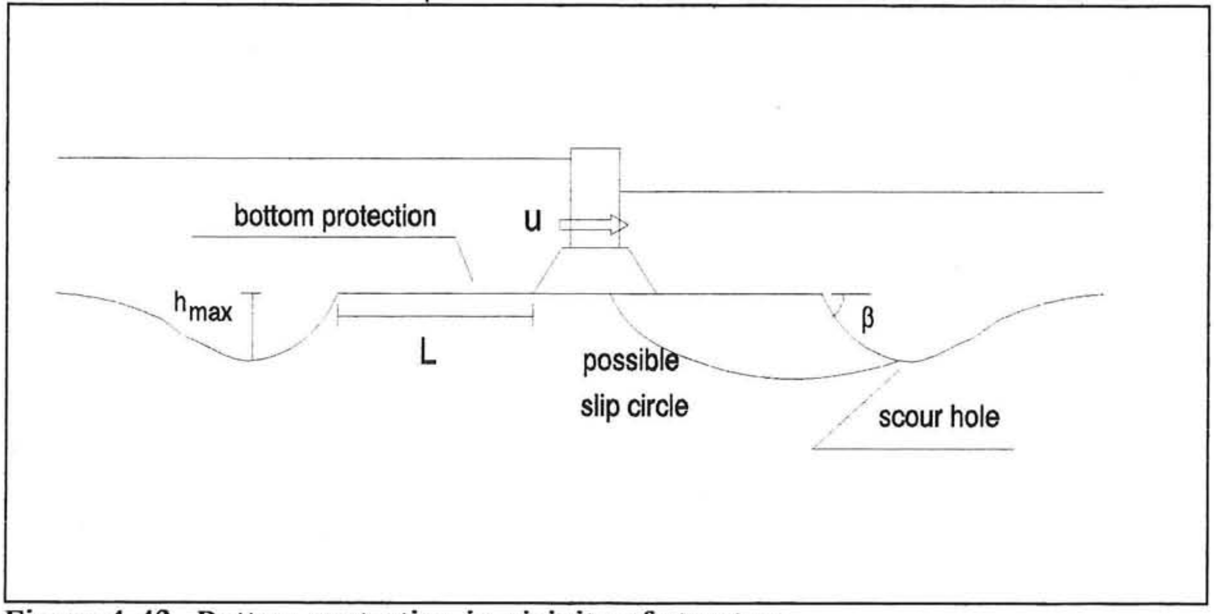


Figure 4.43 Bottom-protection in vicinity of structure

When the equilibrium situation of the scouring is reached the stability of the construction must be assured. This asks for calculations in order to predict the development of the scour hole and to check the stability of the structure. Here a, in most cases rather safe, value of approximately ten times the downstream depth has been taken for L.

stability of the upper layer

If a bottom protection is used with an upper layer that consists of rock, the stability of it depends on the flow conditions and the mass of the rock. In [lit. (16)] a relation has been deduced from data available out of experiments (Equation 4.20).

$$\Delta D_n = A \cdot U_o^2 \qquad with \qquad A = \frac{1}{\Psi \cdot C^2} \tag{4.20}$$

Δ	=	relative density of rock	[kg/m³]
D_n	=	nominal diameter of the stones	[m]

$$C = 18 \cdot \log\left(\frac{6 \cdot (h_3 + d)}{D_n}\right) \qquad \textbf{(4.21)}$$

U_0	=	reference velocity above the sill, equal to U/μ where U = flow-velocity in the closure gap computed using a hydraulic model, μ = discharge coefficient of the closure gap	[m/s] [m/s] [-]
Ψ	=	damage parameter Ψ = 0.03 start of movement Ψ = 0.04 some transport	[-]
C	=	Chezy roughness parameter	[$m^{0.5}$/s]

The C-value (Eq. 4.21), necessary to determine the ΔD_n parameter in its turn depends on the value of D_n. This asks for an iterative calculation using a first estimate of D_n.

When a bottom protection with a permanent function is considered it is advisable to take a safe Ψ-value of 0.03. Here the governing conditions occur during closure. Taking the conditions of Highest Astronomical Tide, which are not exceeded during the final closure, a Ψ-value of 0.04 will be safe enough. The bottom-protection is supposed to be constructed using rock with a specific gravity of 2700 kg/m^3. This implies a Δ-value of 1.65.

For several widths of the closure gap necessary stone masses have been determined for the range of bottom-levels that is present in these gaps. As an example the calculation for a width of 7 km. is discussed below.

Taking HAT, the maximum velocities can be read from the DUFLOW-calculation. For the same time-step the momentary discharges are taken. Dividing this discharges by the velocities and the gap-width the actual waterlevel above the sill can be determined (Table 4.4). For a gap-width of 7 km. structure 2 has a width of 2 and structure three of 5 km. The bottom level varies between approximately 10 and 20 m. below B.M. (The bottom protection for the gully with the maximum depth of 33 m. below B.M. (Figure 4.40) is determined, using the conditions for a gap-width of 5500 m.)

	V_{max}	$Q_{momentary}$	h_{sill}	level	h_{sill}+d	ΔD_n	Dn	M_{50}
	[m/s]	[m^3/s]	[m]	[m.+B.M.]	[m] (d=10 and d=20 m.)	[m]	[m]	[kg]
structure 2								
flood-flow	-7.55	-172230	11.41	6.41	16.41	0.84	0.51	708
					26.41	0.65	0.39	160
ebb-flow	7.97	101520	6.37	1.37	11.37	1.31	0.79	1331
					21.37	0.86	0.52	380
structure 3								
flood-flow	-7.49	-426930	11.40	6.40	16.40	0.82	0.50	338
					26.40	0.63	0.38	148
ebb-flow	7.99	251930	6.31	1.31	11.31	1.33	0.80	1382
					21.31	0.86	0.52	380

Table 4.4 Calculation results for several conditions

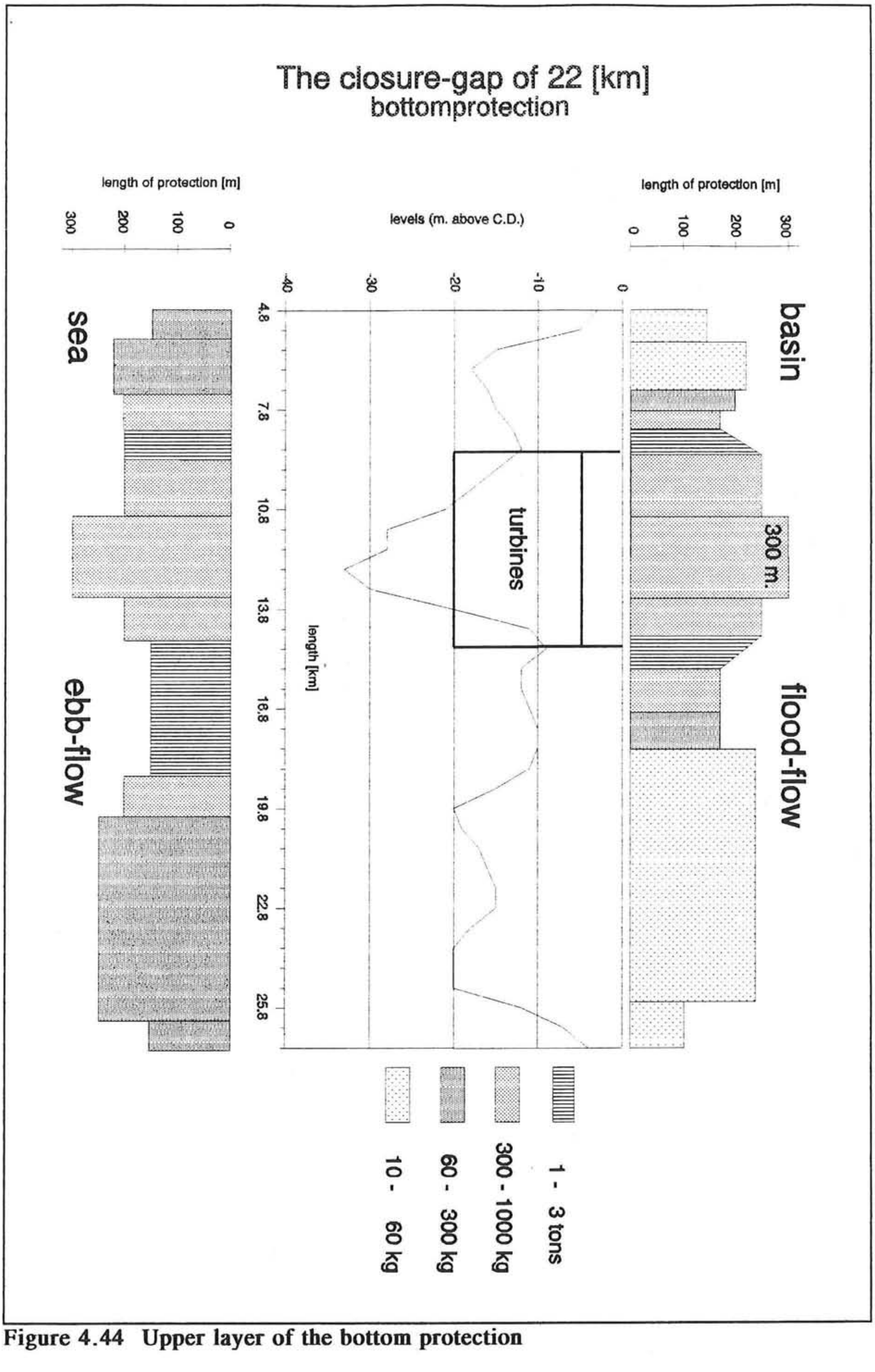


Figure 4.44 Upper layer of the bottom protection

The iterative calculation of the upper situation is shown in Table 4.5.

U	7,55						
h3+d	16,41						
Dn	0,700000	0,577658	0,535275	0,519745	0,513925	0,511725	0,510891
h3/Dn	23,44286	28,40780	30,65716	31,57320	31,93073	32,06801	32,12038
C	38,66691	40,16860	40,76430	40,99446	41,08248	41,11602	41,12878
A	0,016721	0,015494	0,015045	0,014876	0,014812	0,014788	0,014779
ΔD_n	0,953136	0,883203	0,857579	0,847976	0,844346	0,842969	0,842447

Table 4.5 Iterative determination of the ΔD_n value for a depth of 10 m. during flood-flow in structure 2.

The maximum stone-mass in Table 4.5 is 1382 kg. which asks for rock in the grading of 1000 - 3000 kg. This kind of calculations have been carried out for several widths of the closure gap with the sill at a constant level of 5 m. below B.M. The areas with upper layers consisting of different gradings and with different lengthsare presented in Figure 4.44.

In a further stage more detailed calculations can be made when the actual bottom-levels are known. It is expected that the stability calculations then will result in a decrease in the length of the bottom-protection.

4.5 Structures in the dam

Up till now only the constructions necessary for the tidal power station have been discussed. Provisions however will have to be made for ships that will reach the ports in the Gulf of Khambat and the discharge of superfluous fresh water origining from the Narmada.

ship-transfer

In the Gulf of Khambat two ports are in use under the present circumstances:

- the all weather direct berthing port of Bhavnagar;
- the fair weather lighterage port in Dahej.

Bhavnagar port is accessible for 4 - 5,000 DWT seagoing vessels. Entering Bhavnagar port is only possible during high water time. During low waters the necessary water-level in the docks is maintained with the help of a single lock-gate. Vessels larger than mentioned anchor outside and lighterage takes place. This means outside trans-shipment of cargo into smaller vessels.

Bhavnagar port is severely affected by siltation. The annual dredging requirements are estimated at 4 million m^3 to maintain the depth of both the docks and the approach channel.

Dahej is a lighterage port that is used mainly for the import of fertilizers. The cargo is transported inland by train.

Future developments with respect to cargoes and required handling facilities have to be analyzed as a part of an overall transport study for this region. Gujarat State intends to develop Pipavav Bandar as the main direct berthing port for ships up to 25,000 DWT. It is expected however that Bhavnagar will remain an important regional port [lit. (5)]. Two alternatives to maintain the accessibility of Bhavnagar port are presented here:

- Bhavnagar port is situated in the future tidal basin of the tidal power station. This implies that the high water periods which allow ships to enter the docks will stay present. Entering the tidal basin asks for a ship transfer system in the dam. A lock will be most appropriate. The lock is supposed to be constructed in a temporary reclaimed polder prior to the closure operation.

- The dam surrounding the basin bends in the direction of Channel Bank. The access channel of Bhavnagar New Port is passed at the eastern side (Figure 4.45). In this way the construction of a lock can be omitted and Bhavnagar Port lies outside the tidal power basin. However, when the planned routes of the road and railway are maintained a bridge over the access channel must be built. Another disadvantage is the probably increasing siltation of Bhavnagar Port.

In a further stage a decision based on economic grounds can be made. Then more information about the bottom and the use of Bhavnagar port is required. The costs of a bridge and a longer dam must be weighed against the costs of a lock.

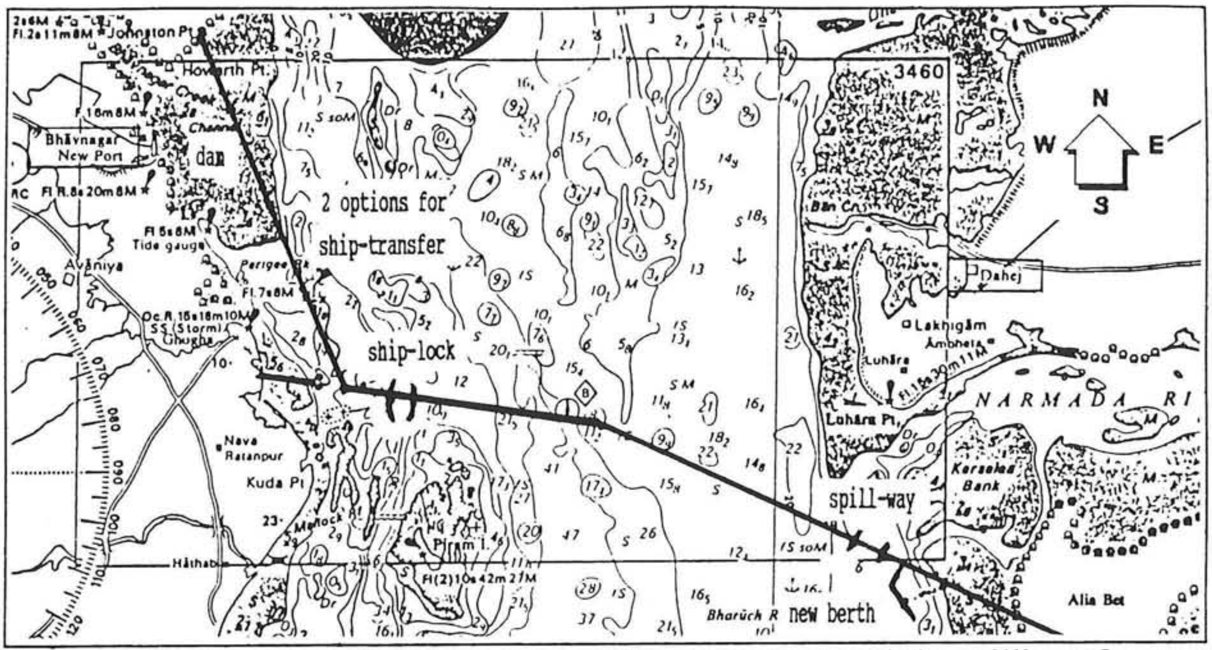


Figure 4.45 Provisions for the access of Bhavnagar Port and the spilling of superfluous Narmada water

The presence of the dam in the Gulf of Khambat decreases the transport distance from Bhavnagar to the areas east of the Gulf of Khambat. Two alternatives for the handling of the fertilizers are mentioned here:

- Dahej port is closed, after completion of the dam, and the fertilizers are handled in Bhavnagar too. The cargo can be transported by rail or road to the areas east of the Gulf of Khambat.

- A new deep water unloading berth is constructed along the dam. This asks for a breakwater protecting the berth.
  Because of the presence of a railway on top of the dam the method of inland transport can be maintained. This berth can be reached by larger ships than Dahej port, which decreases the necessity of lighterage.

In both cases no transfer through the dam will have to take place. Thus, no ship transfer system have to be situated between the salt sea and the fresh-water basin. This partly prevents the salt intrusion.

Detailed study will be necessary in order to make a final decision concerning the future role of the two ports and the required provisions. In order to complete the layout of the dam, here the construction of a single lock is supposed to guarantee the accessability of Bhavnagar Port. Dahej Port is replaced by a new unloading berth at the sea-side of the final dam.

spillway

The fresh Narmada discharge will be stored in the created basin. However in average circumstances the yearly Narmada discharge will be larger than the irrigation demand. The fresh water surplus will have to be spilled in order to keep the basin level under a safe limit. Here the maximum basin level is taken equal to the waterlevel that is exceeded during Highest Astronomical Tide in the present situation, being 10 m. above B.M..

In the reconnaissance report [lit. (5)], a simulation is described of the safe release of the flood wave with a chance of exceedance of 1/100 per year. The peak discharge amounts to 67,000 m^3/s. The flood wave is schematized as a parabolic function described by Equation 4.22.

$$Q(t) = \frac{t(T-t)}{\left(\frac{T}{2}\right)^2} Q_{\max} \tag{4.22}$$

where	T	=	total duration of the flood wave for which a value of 60 hours is taken	
	t	=	time variable	[t]
	Q(T)	=	momentary discharge	[m^3/s]
	Q_{max}	=	peak discharge of the flood wave: 67,000	[m^3/s]

Integrated over the period t = 0 to t = 60 hours the total content of this schematized flood wave amounts to $9.6 * 10^9$ m^3. Without spilling this causes the basin level to raise by approximately 5 m.. When the level before the start of the flood is higher than 5 m. above B.M. this rise can not be accepted. Spilling will be necessary. The simulation shows that a spillway with a sill-level at 2 m. below B.M. and a width of 700 m. will be sufficient to guarantee a safe release of the fresh water surplus.

These calculations were based on discharge data of the Narmada river during the period from 1915 - 1962 [see Table 2.1 in chapter 1]. More detailed calculations are not sensible now. This because since 1962 several dams have been constructed in the Narmada. These dams certainly have influenced the flood hydrographs. Water is withdrawn upstream of the dams and the flood waves may be smaller. First new data must be collected and insight in future developments must be obtained before a final design of the spillway can be made.

The spillway is planned to be situated close to the Narmada mouth, in Bharuch channel. Two rather different ways of construction can be distinguished:

- The spillway is composed of gated caissons sunk on top of a rock-fill sill.

- A temporarily reclaimed polder is used to construct the spillway on the site.

The spillway is situated between the fresh water basin and the sea. Here the problem of salt intrusion is important. As already mentioned in section 4.3 the prevention for underseepage is very difficult in case of caissons. Without further provisions the discharge through the sill, and consequently the salt intrusion, will be considerable. Unless a reliable solution for this problem is found, construction in a polder seems more appropriate and is assumed here.

The spillway consists of 28 radial gates with a width of 25 m. each. They must be designed for a head-difference of 12 m. (maximum basin-level minus sill-level). The choice for radial gates is based on the relatively small power that is required to open them.

4.6. *Design of the final dam*

4.6.1 Profile of the dam

The closure dam constructed in the Gulf of Khambat is not suited to accommodate a railway and a road in a permanent situation. Although the crest level of this dam lies at a level of 11 [m] above B.M., the foundation of both can be endangered in extreme circumstances. Not to mention the fact that the railway and road can not be safely used during several days a year.

Besides, the closure dam is pervious to water. The final dam, to a certain extent, will have to be impervious in order to prevent for salt intrusion in the fresh water basin.

available construction materials

Again the use of local available materials is desirable to limit costs. Rock was already used for the construction of the closure dam. Considering the enormous amounts used already, capacity problems can be expected when rock is used for the enlargement of the dam-profile too. However it can be considered. One has to keep in mind that the large porosity of rock combined with the large grain size enables water to be transported through the dam very fast. This implies that the construction of an impervious layer, embedded in the dam, will be necessary.

The bottom of the Gulf of Khambat mainly consists of sand [see section 2.2]. It is assumed that a sufficient quantity of sand is available. The final dam than can be built by means of hydraulic sand-fill.

In the vicinity of the dam several kinds of clay can be found. Clay is a suitable material for the construction of an impervious layer in the dam or an impermeable facing. It can be used as under layer too when a facing of concrete blocks is chosen. However, the demands on clay used in the slope protection are very high. The properties of the available clay are not known.

Construction of the embankment core

Around or next to the closure dam the core of the final dam will have to be constructed. This core is planned to be built by means of hydraulic sand-fill. This building method can only be used when the flow velocities stay below a value of 2 [m/s]. Even then the sand-loss will be considerable.

The core will be located on the northern side of the closure dam. In this way the temporary railway on top of the closure dam can be used during dam completion. On the sea-side the present rock slope is sufficient to resist the wave-loads during normal circumstances. The crest of the closure dam will at least partly become a berm in the profile of the final dam. Through the closure dam still a significant tidal flow takes place. This means that the sand can not be placed directly on the northern rock-slope of the closure dam. Here a filter is planned to be constructed which will be covered by a sand-tight geo-textile. This geo-textile will be made heavier by means of concrete blocks connected to it. In this way it will be able to resist the head-differences during the tidal cycle.

Finally the core of the dam can be made by means of hydraulic sand-fill. Below the water-line the steepness of the slope will be maximum 1 : 15. No equipment is able to steepen the slope there. By means of manipulation with the intake works the waterlevel can be lowered to 2.0 [m] + B.M.. Above this level a slope of 1 : 3 will be applied. In this way the core will get a width of approximately 110 [m] on the Bench Mark level.

The discharge through the dam will be very small because of the considerable seepage path. Thus, the salt intrusion through the dam can be neglected. Like in Dutch practice of sea-defence design, embedding an impermeable layer in the dam will not be necessary.

The crest of the closure dam lies at a level of 11.0 [m] + B.M. and is equipped with a railway that is situated at the sea-side. During construction of the final dam this railway can still be used. The outer 10 [m] of the crest are not embedded in the dam. In this way a berm with a width B of 10 [m] will stay present in the final design.

Slope-protection sea-side and wave run-up

The dam-slope on the sea-side must be able to resist the wave-attack. In order to limit the crest-height it is advantageous to reduce the wave run-up. When we look at the run-up Equation (4.23) it is clear that the wave run-up can be reduced by the application of a rough slope-protection, resulting in a relatively small f-value, and a less steep slope. In (4.23) the influence of oblique waves is neglected.

$$z_{2\%} = 0.74 \cdot f \cdot T\sqrt{g \cdot H_{1/3}}\tan\alpha \cdot (1 - B/L_o) \qquad \textbf{(4.23)}$$

In which:	$z_{2\%}$	= 2 % wave run-up	[m]
	f	= roughness parameter	[-]
	T	= mean wave-period = $T_{1/3}/1.15$	[s]
	$H_{1/3}$	= significant wave-height	[m]
	α	= slope-inclination	[°]
	g	= acceleration of gravity	[m/s^2]
	B	= berm-width	[m]
	L_o	= wave-length on deep water	[m]

Based on some run-up calculations for the slope at the sea side a steepness of tan α = 1 : 6 has been chosen. The armour layer of the slope protection consists of rock which has a f-value of approximately 0.6.

Also on the basin-side of the final dam a slope protection will be constructed. Because of this the 2% wave run-up becomes a very safe value. It is applied in Dutch practice when the inner slope of a dam often consists of clay and grass.

Lately a method has been developed to deduce a z-height from the overtopping that can be accepted [lit. (19)]. The necessary equations are presented below (Eq. 4.24 - 4.26).

$$z = \frac{X\sqrt{(H_{1/3}/1.6)\, g\, (T_{1/3}/1.15)^2/2\pi}}{cotg\,\alpha}\gamma_f \cdot \gamma_B \qquad \textbf{(4.24)}$$

The value of X can be found using:

$$\log Y = -0.214X^2 - 0.787X + 0.103 \qquad (4.25)$$

Y should be determined using:

$$Y = \frac{\overline{q} \cdot T_{1/3}\sqrt{cotg\,\alpha} \cdot 1.6 \cdot 10^{-3} \cdot 2\pi}{0.1 \cdot 1.15\, H_{1/3} \cdot g\left(\frac{T_{1/3}}{1.15}\right)^2} \qquad (4.26)$$

in which:
- z = wave run-up height [m]
- $\overline{q}$ = mean overtopping [l/m/s]
- γ_f = reduction factor slope-roughness
- γ_B = reduction factor berm = $1 - B/L_o$

The results of the different methods are presented in Table 4.6. During calculation the significant wave with a frequency of exceedance of 1/100 year^{-1} has been applied:

$H_{1/3}$ = 5.0 [m]
$T_{1/3}$ = 10.0 [s]
H/L_o = 0.03 [-]*

* Deep water: $L_o = 1.56\ T^2$ = $1.56 \cdot 100$ = 156 [m]
Shallow water: $L_o = T\sqrt{(g\ d)}$ = $10\sqrt{(9.8 \cdot 30)}$ = 171 [m]

With an intermediate L_o of 165 [m], H/L_o becomes 0.03 [-]

Also the values of $z_{5\%}$ and $z_{10\%}$ are presented, being respectively 0.87 and 0.76 times $Z_{2\%}$.

z-heights	no berm	B = 10 m
$z_{2\%}$	4.51 [m]	4.21 [m]
$z_{5\%}$	3.92 [m]	3.65 [m]
$z_{10\%}$	3.43 [m]	3.20 [m]
$\overline{q}$ = 1 [l/s/m]	4.84 [m]	4.50 [m]
$\overline{q}$ = 5 [l/s/m]	4.08 [m]	3.80 [m]
$\overline{q}$ = 10 [l/s/m]	3.73 [m]	3.47 [m]

Table 4.6 Values of the wave run-up z for different ways of computation

Due to the slope of 1:6 and the rough slope-protection the values do not differ much. An acceptable overtopping of 5 l/s/m is chosen here which results in a z-height of 3.80 m.

determination of the crest level

Determining the crest level of the dam, many aspects must be taken into account. They are discussed below.

design-level — First the expected maximum water-level must be determined. When enough data are available the design-level can be determined with the help of a statistical calculation. Then directly a combination of tide and wind set-up is considered. Here, these data are not available so a combination of the separate items high tide and wind set-up will have to deliver the maximum level. In a report of Delft Hydraulics [lit. (6)] a wind set-up of 2 [m] was mentioned for conditions with a recurrence-interval of 100 years. In the reconnaissance report a value of 1.5 [m] was taken. The combination of Highest Astronomical Tide and an extreme wind set-up is extremely rare. Here two combinations are considered:

- HAT (B.M. + 11 [m]) in combination with a small wind set-up of 0.5 [m] gives a level of B.M. + 11.5 [m]
- When the wind set-up is extreme, here the value of 2.0 [m] is taken, a level of 10.7 [m] + B.M. is reached, when it is combined with average spring-tide (B.M. + 8.7 [m]).

As a first estimate the level of 11.5 [m] + B.M. is taken as the design-level.

wave run-up

The wave run-up based on an overtopping discharge of 5 [l/s/m] amounts to 3.80 [m] as calculated above.

crest-height reducing factors

The crest-level reduces in time because of some time-dependent factors. The aspects which are important here are settlement of both the sea-bottom and the dam itself and the relative rise of the sea-level. For these effects a freeboard of 1 [m] is taken into account

The crest-level of the final dam becomes 16.30 [m] + B.M. when the three components are joined.

crest width and inner slope

As already mentioned the crest must be wide enough to accommodate rail and road. When the length of the dam, being 48.6 [km] is considered it seems necessary to construct a double tracked railway. A crest-width of 25 [m] is assumed to be sufficient.

The wave-attack on the slope at the basin-side will be considerably less. When we bear in mind that it is not certain that prior to a cyclone the basin-level is equal to the maximum level it seems sufficient to take a design-condition with a smaller recurrence interval (of 50 years) here (only for the wave run-up and not for the design of the revetment). A wave with $H_{1/3}$ = 2.3 [m] and a period $T_{1/3}$ = 5.0 [s] is chosen. With the f-value taken equal to 1 and a slope of 1:3 first the 2% wave run-up is computed. This amounts to 5.09 [m] (Eq. 4.23). When the sum of the maximum basin-level (B.M. + 10 [m]) and the wave run-up is compared with the crest-level of the dam (B.M. + 16.3 [m]) the freeboard amounts to 1.21 [m] which is sufficient. So there is no need to accept more wave run-up. Now a cross-section of the dam can be drawn (Figure 4.46).

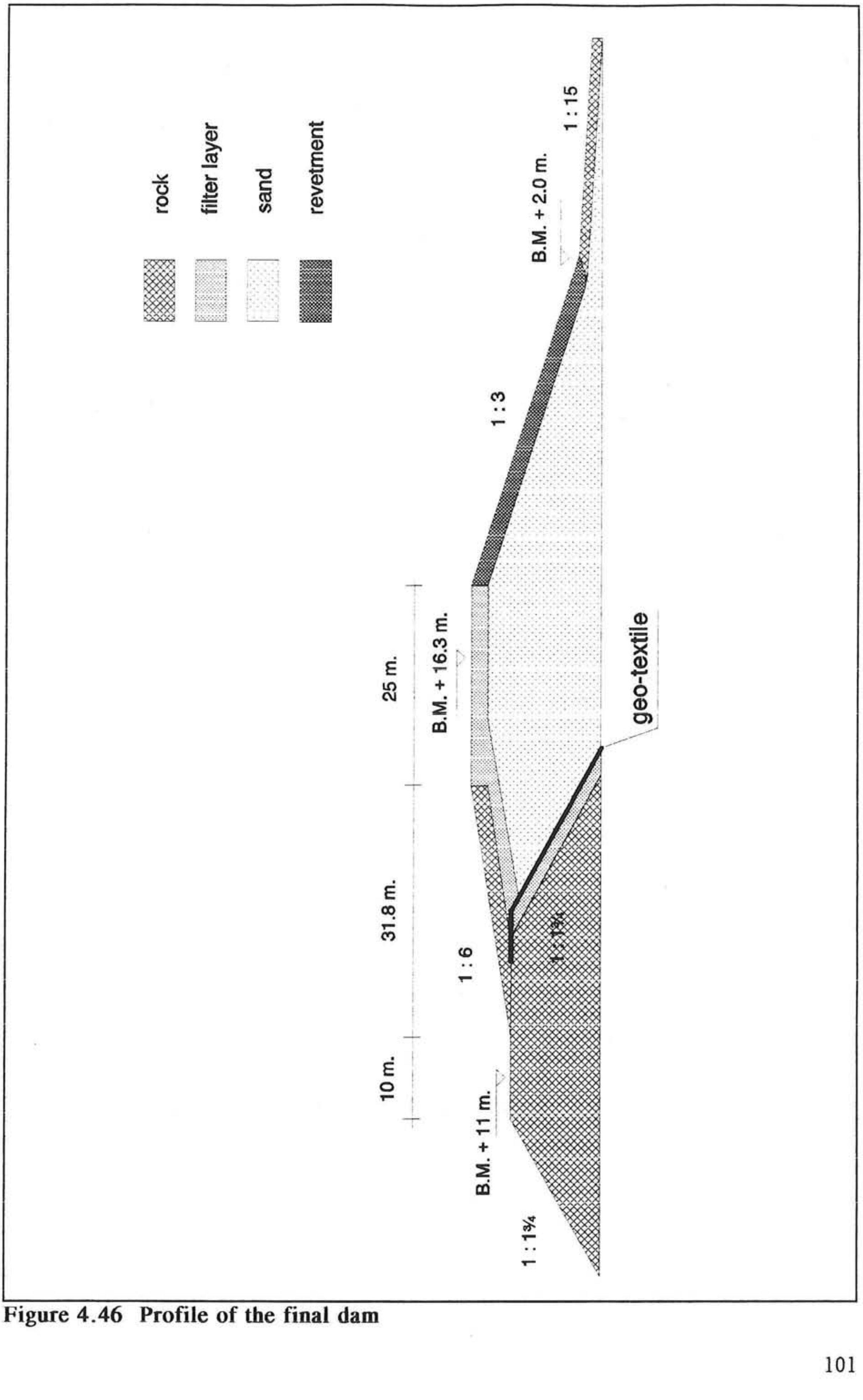


Figure 4.46 Profile of the final dam

4.6.2 Protection of the sea-side slope

As already mentioned, the sea-side slope will be protected with rock. The necessary stone-mass will be determined in this section for both the slope of the closure dam and the slope above the berm. Two formulae were available for this purpose. Firstly the well-known Hudson relation (Eq. 4.27) and secondly a more exact formulae recently derived by v.d. Meer (Eq. 4.28).

Hudson:

$$M_{50} = \frac{\rho_s H_s^3}{k_D \Delta^3 \cot\alpha} \qquad (4.27)$$

where:

ρ_s	= specific gravity of the stone	[kg/m³]
H_s	= significant wave height (= $H_{1/3}$)	[m]
k_D	= damage coefficient (= 0.4 for 0-5% damage for angular quarry-stone)	
Δ	= relative density of the stones	[-]
α	= slope angle of the front slope	[°]

V.d. Meer (for plunging waves), [lit. (21)]:

$$\frac{H_s}{\Delta D_{n50}} \cdot \sqrt{\xi_m} = 6.2\, P^{0.18} (S/\sqrt{N})^{0.2} \qquad (4.28)$$

where:

$$\xi_m = \frac{\tan\alpha}{\sqrt{H_s/L_o}} \qquad (4.29)$$

Equation 4.28 for plunging waves can be used when:

$$\xi_m \le (6.2\, P^{0.31}\sqrt{\tan\alpha})^{\frac{1}{P+0.5}} \qquad (4.30)$$

When ξ is larger than this transitional value an equation for surging waves must be applied (which is not described here).

parameters:

H_s	= significant wave height (= $H_{1/3}$)	[m]
N	= storm duration or number of waves	[-]
D_{n50}	= nominal diameter of the stones	[m]
Δ	= relative density of the stones	[-]
α	= slope angle of the front slope	[°]
P	= permeability of the structure	[-]
ξ_m	= surf similarity or breaker parameter	[-]

Slope above the berm

First the protective layer on the slope of 1:6 is considered. Using v.d. Meer (Eq. 4.28), first ξ_m must be computed using Eq. 4.29.

$$\xi_m = \frac{1/6}{\sqrt{0.03}} = 0.96 \quad [-]$$

It is very clear that these are plunging waves (Figure 4.47) so Equation 4.28 can be applied.

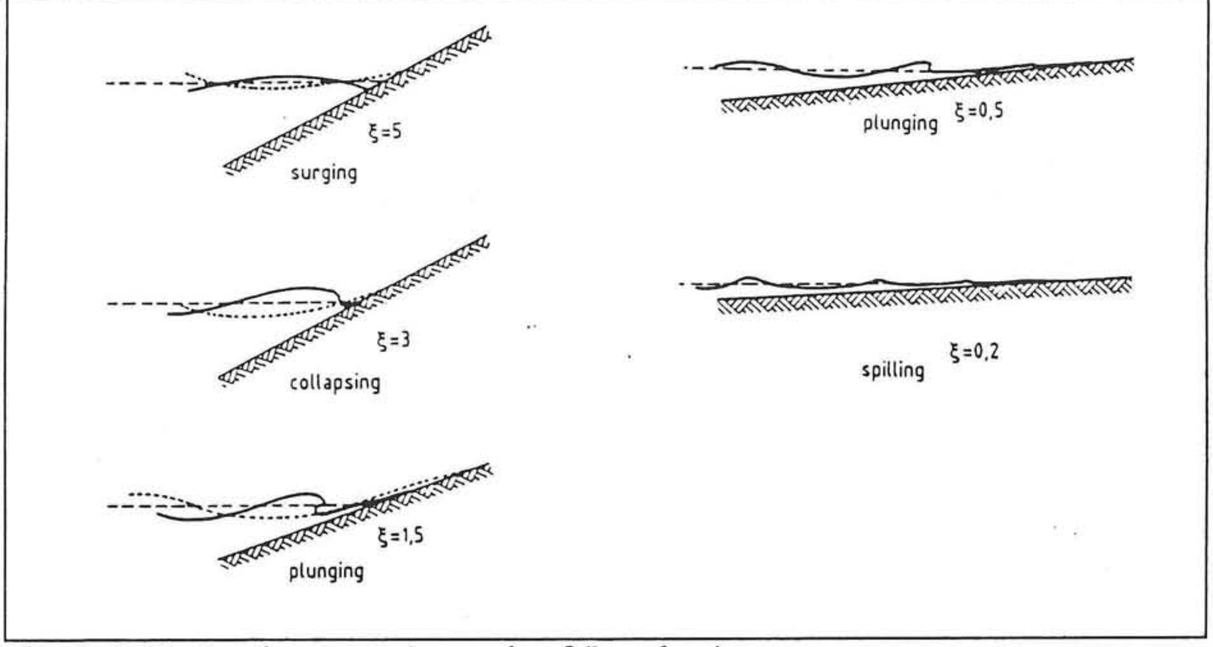


Figure 4.47 Breaker types (example of ξ_m-values)

In order to be able to use this Equation the values of the parameters P, S and N must be estimated.

P P represents the permeability of the structure. P varies from 0.1 for a impermeable core to 0.6 for an homogeneous permeable core. Here the slope protection is permeable and the necessary filter layers underneath too. The core consists of sand which behaves impermeable in case of wave attack. An intermediate P-value of 0.2 has been chosen.

S The value of the damage-coefficient S depends on the accepted damage. In general S lies between 1 and 3 when the no-damage criterion is applied. Here S = 2 has been applied.

N For the number of waves, N, an often applied value is 3000. Because the wave attack on the slope considered is extreme in combination with an extreme storm-surge level the chance of occurrence is small. There seems to be no reason to raise N. So N = 3000.

Now the value of ΔD_{n50} can be computed:

$$\frac{5}{\Delta D_{n50}} \cdot \sqrt{0.96} = 6.2 \cdot 0.2^{0.18} (2/\sqrt{3000})^{0.2} = 2.39$$

$$\Delta D_{n50} = 2.05$$

It is assumed that rock with a specific gravity of 2900 kg/m^3 is used for the slope protection. In Figure III.7 of annex III the required grading can be read. It shows that the grading 3000 - 6000 [kg] must be applied.
When we compute the exact mass of a stone with ΔD_{n50} = 2.05, 4011 [kg] is found (Δ = 1.84). This mass lies between the grading limits.
The application of the Hudson-Equation (4.27) leads to the following result:

$$M_{50} = \frac{2900 \cdot 5^3}{4.0 \cdot 1.84^3 \cdot 6} = 2425 \quad [kg]$$

This mass leads to the same grading.

The thickness of the armour layer can be determined now. In general the number of layers of armour units, m, is taken as 2. In order two compute the thickness t, the layer-coefficient k_Δ, must be known. For angular quarry-stone the value of this coefficient is approximately 1 to 1.15.

$$t = mk_\Delta D_{n50} = 2 \cdot 1.05 \cdot \frac{1.81}{1.84} = 2.07 \quad [m] \tag{4.31}$$

Slope of the closure dam

The same procedure can be followed for the sea-side slope of the closure dam which becomes part of the final dam. The main difference is the slope angle which causes a considerable change in the value of ξ_m.

$$\xi_m = \frac{1/1\frac{3}{4}}{\sqrt{0.03}} = 3.28 \quad [-]$$

This means that we have to deal with collapsing waves. With the help of Equation 4.30 the applicability of Equation 4.28 can be checked. Now the value of P is raised to 0.6 because the closure dam is permeable and homogeneous.

$$\xi_m \leq (6.2 \cdot 0.60^{0.31} \sqrt{1/1\frac{3}{4}})^{\frac{1}{P+0.5}} = 3.53$$

Because 3.28 is smaller than 3.53 Equation 4.28 can still be used.
The slope of the closure dam is also attacked when the storm-surge level is rather low, for instance at low tide. In order to take this effect into account the number of waves is raised till N = 10,000. On the other hand the rock-fill core can be considered as a many-layer slope protection. This implies that some damage can be accepted. The S-value is raised to 5.

$$\frac{5}{\Delta D_{n50}} \cdot \sqrt{3.28} = 6.2 \cdot 0.6^{0.18} (5/\sqrt{10000})^{0.2} = 3.11$$

$$\Delta D_{n50} = 2.91$$

Here a grading of 10 - 15 tons is required. This is rather heavy. However, during the closure-operation this grading has already been used over a certain length. For the sections where this is not the case, it can be studied in detail if the grading determined here is really necessary on that particular spot. The HISWA-calculations (section 2.2) showed that the expected wave-height depends on the location along the dam alignment. Up till now the wave at the most severely attacked section is considered. So, detailed study or wave-measurements will reduce the amount of slope protection that has to be placed during dam completion.

The same distribution of the wave-heights counts for the grading of 3 - 6 tons on the 1 : 6 slope. Here also reduction can be considered.

The rock for the slope protection can not directly be placed on the sand of the dam-core. Because of the wave-attack the sand underneath the armour layer will erode. Two options for the construction of the transition between sand and rock are mentioned here.

- The sand is levelled smoothly in a slope of 1 : 6. On the sand a sand-tight geo-textile is laid. Placing the stones of several tons directly on this geo-textile, the risk for damage is considerable. The dimensions of the armour layer are on the average 1.18 [m]. To prevent the under-layer for erosion the ratio between the D_n of the two layers may vary between 3 and 5 (filter rules [lit. (b)]. When a D_n of 0.30 [m] is chosen for the under-layer the geometrical closeness is guaranteed. These stones are still rather large and heavy, 10 - 60 [kg]. Another similar step leads to a layer of gravel with a mass smaller than 1 [kg] placed directly on the geo-textile. For reasons of construction the thickness of this layer should at least be 0.30 [m].

- Instead of using a geo-textile the filter can be completed until the grain-size of the sand is reached. This asks for many layers.

In the design the alternative with a geo-textile is chosen (Figure 4.48). The costs of this solution should be weighed against the costs of a complete granular filter in order to make a final decision.

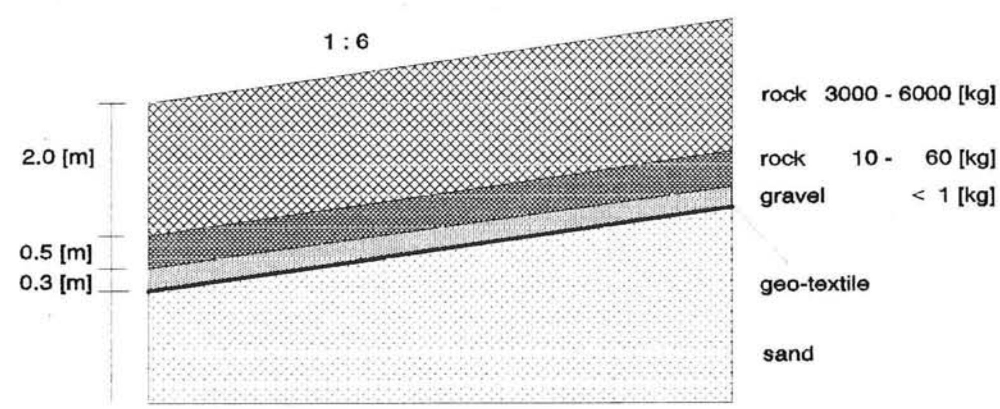


Figure 4.48 Slope-protection of rock on geo-textile

4.6.3 Protection of the slope on the basin-side

The slope protection must be stable under design-conditions and prevent for the loss of base-material. A revetment consists generally of a top-layer, that must resist the hydraulic loads, and an underlayer that prevents the base-material to erode. A third failure-mechanism is the soil mechanical loss of stability. This must be considered too in the design.

top-layer

When a slope of 1 : 3 is applied on the basin-side the wave run-up will be sufficiently limited by a revetment with a f-value ≤ 1 (See section 4.6.1 and Eq. 4.23). This f-value allows to apply every slope protection except for a closed and very smooth one ($f = 1.1$), like asphalt.

Open asphalt will do but problems with the high temperature and the burning sun can be expected. Asphalt behaves rather viscous when its temperature is high. Wave-loads attacking such a protection will cause deformations of the slope. Also the ability of asphalt to follow deformations of the foundation is small.

When the enormous surface is considered that has to be protected it is favourable to select a mechanical construction method. Then a revetment of pitched stones can be applied. When uniform concrete stones are applied for instance a block claw and a hydraulic crane can be used. In this section a revetment of pitched concrete stones is designed.

under layer

Pitched stones can be placed on a granular filter but also on a layer of good clay. The properties of the available clay are not known. Besides, a revetment of pitched stone on clay may not be exposed to daily occurring hydraulic (wave-)loads. In the fresh-water basin and also in the tidal power basin the water-level varies considerably in time. A large zone is regularly attacked by hydraulic loads. So the use of clay as under-layer for the stones is rejected.

The proposed construction is a revetment of pitched concrete stones on a granular filter. In order to limit the thickness of the filter the core of sand is first covered by a sand-tight geo-textile. As we will see later a thin filter diminishes the requested dimensions of the stones of the upper layer.

design

First a simplified design method [lit. (20)] will be applied. When pitched stones on a granular filter are used a subdivision can be made between: good, moderate and poor designs. Here we aim at a good design which implies the following demands (see also the flow-chart of Figure 4.49):

- The filter layer is thin: $b/D < 0.5$, where b and D are the thicknesses of respectively the filter layer and the stones.

- The filter material is fine: $D_{f15} < 10$ [mm], where D_{f15} is the particle size of the filter that is not exceeded by 15 % (by weight) of the particles.

- The outer layer is open, the value of Ω (relative opening surface) must be bigger than 3%, and the openings are not filled. For details about the holes see the flow chart.

The design can start under the assumption that a good construction will be possible. The simplified design method establishes a relation between the breaker parameter ξ_m and the parameter $H_s/\Delta D$.

First the design-wave must be known in order to compute ξ_m. For the check on the wave run-up $H_{1/3}$ = 2.3 [m] has been used, having a recurrence interval of 50 years. For the loads on the revetment however, the wave with a chance of occurrence of 1/100 per year is chosen:

$H_{1/3}$ = 3.0 [m]
T = 5.5 [s]

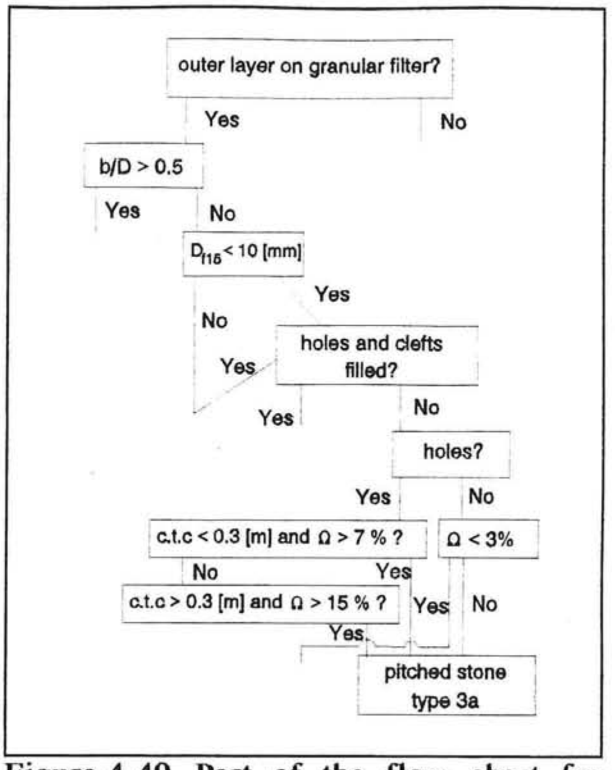


Figure 4.49 Part of the flow chart for the construction type

The mechanism of the uplifting of a block is ruled by the wave attack and specifically by the way the wave breaks. This process is described by the breaker parameter ξ_m (Eq.4.29). First the wave-length L_o must be computed. For this relatively short wave the deep-water equation can be used:

$$L_o = 1.56\ T^2 = 47\ [m]$$

ξ_m becomes:

$$\xi_m = \frac{1/3}{\sqrt{3.0/47}} = 1.32$$

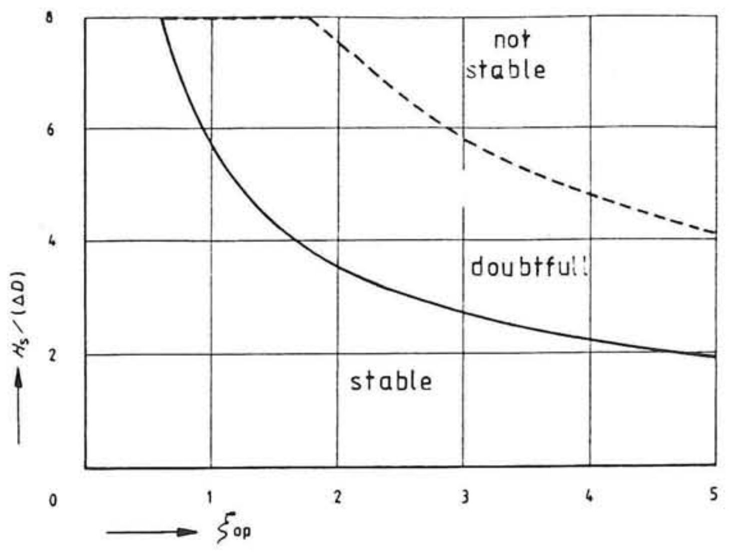
Now the required value of $H_s/\Delta D$ can be read in the graph of Figure 4.50. In this case:

$$H_s/\Delta D = 4.6$$

The value of Δ is determined using a specific gravity of concrete of 2500 [kg/m^3]. Then Δ becomes (2500 - 1020)/1020 = 1.45. A first estimate of the necessary thickness of the blocks is:

$$D = \frac{H_s}{4.6\,\Delta} = \frac{3}{4.6 \cdot 1.45} = 0.45 \quad [m]$$

In a further stage this value must be checked with the analytical method [lit. (20)].



Stones on a granular filter, favourable construction (type 3a) (check the result with the analytical method).

Figure 4.50 Design graph for a good construction of stones on a granular filter

The layer thickness found for the stones implies that the filter thickness is maximum 0.22 [m]. In the design a layer of 0.20 [m] thick is proposed. Stones without holes are chosen here, which implies that relative open surface Ω must be larger than 3 % . Clefts of 10 [mm] between the blocks satisfy this requirement when blocks are applied with dimensions of 0.60 * 0.60 * 0.45 [m^3]. A view of the design is presented in Figure 4.51.

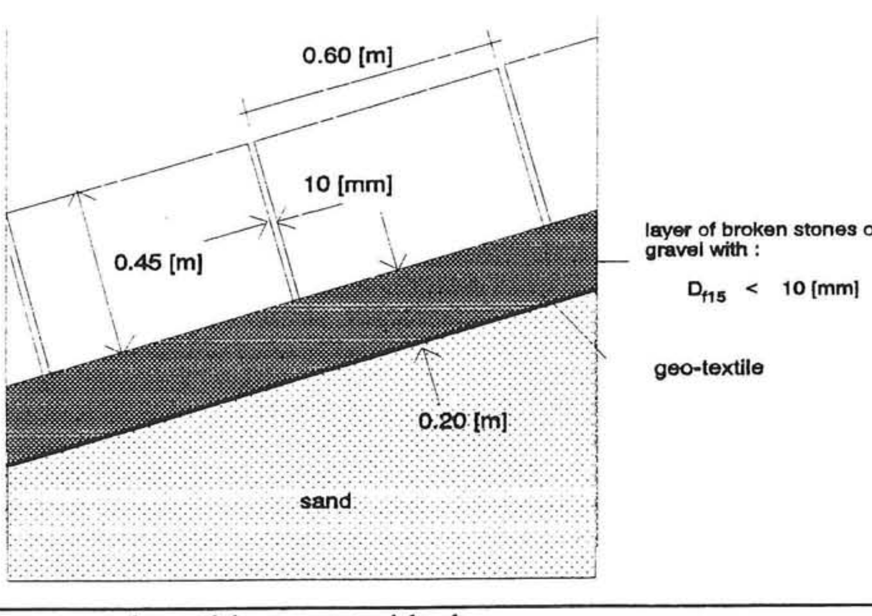


Figure 4.51 Slope protection with concrete blocks

transition-structure at the toe

As already mentioned the basin-level can be lowered till a level of 2.0 [m] + B.M. for construction purposes. Below this level the proposed revetment can not be applied. This asks for a transition structure at the toe. An example of a possible transition structure is shown in Figure 4.52. In order to limit the wave-attack on the toe one can keep the minimum water-level of the basin some 1.5 [m] - 2.0 [m] above the transition. However, when the waterlevel drops below this level the waves will partly break on the 1 : 15 slope before they reach the 1 : 3 slope. This also limits the wave-attack. Thus, the water-level constriction can be discarded.

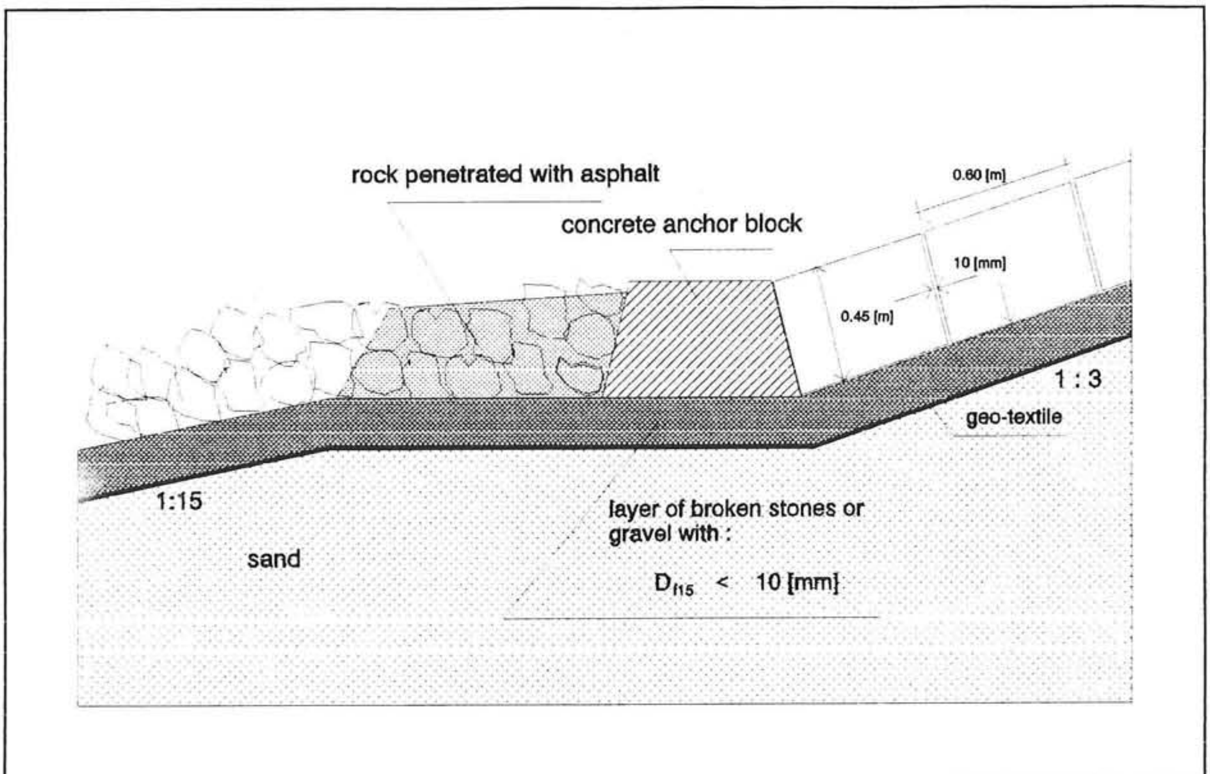


Figure 4.52 Example of a transition structure at the toe

4.7 Estimation of costs final dam

The shape of the final dam and its slope protection is known know. This makes it possible to estimate the quantities of building material required for construction. Based on this quantities a very rough estimation of the construction costs can be made. For the cost estimation unit prices have been used. These unit prices (in Rupees) were partly collected during the reconnaissance mission and presented in the report of may 1989 [lit. (5)]. The values have been corrected here for inflation using an inflation rate of 7.5 % per year. Except unit prices also the estimated prices of the spillway and the navigation lock are corrected prices obtained from the reconnaissance report. Table 4.7 shows the corrected values.

material	**unit price**
rock	261 R/m^3
sand	34 R/m^3
bottom-protection	373 R/m^2
construction part	**price**
spillway caisson	112 * 10^6 R/piece
navigation lock	1243 * 10^6 R/piece

Table 4.7 Prices obtained from the reconnaissance report [lit. (5)]

Other unit prices used here are:

Geo-textile	80	$[Rp/m^3]$
Geo-textile with concrete blocks connected on it	150	$[Rp/m^3]$
Slope-protection of concrete blocks (0.6 * 0.6 * 0.45 $[m^3]$)	305	$[Rp/m^2]$

In Table 4.8 the unit prices are multiplied with the estimated quantities. The prices are not only shown in Rupees [Rp] but also in US Dollars [US$].

1 US Dollar [US$] = 27 Rupees [Rp]

In the reconnaissance report the spillway did consist of 11 sluiced caissons. The total cost-estimate is maintained here although the construction of the spillway in a temporary reclaimed polder is preferred.

The total construction costs are estimated to be 21.71 * 10^9 [Rp] or 805 *10^6 [US$]. For contingency a margin of 15 % is taken and the overhead amounts to 10 %. The total estimated costs become:

$$27.14 * 10^9 \text{ [Rp]} = 1.007 * 10^9 \text{ [US\$]}$$

	quantities (* 10^6)	unit price [R]	total cost [* 10^9 R]	[* 10^6 $]
CLOSURE DAM				
rock	35.8 [m^3]	261	9.35	346
bottom-protection	9.7 [m^2]	373	3.62	134
FINAL DAM				
geo + blocks	1.39 [m^2]	150	0.21	8
sand	110.0 [m^3]	34	3.74	139
SLOPE PROTECTION				
rock	5.47 [m^2]	261	1.43	53
geo-textile	4.09 [m^2]	80	0.33	12
concrete blocks	1.85 [m^2]	305	0.56	21
CONSTRUCTIONS				
navigation lock	1 [piece]	$1.242*10^6$	1.24	46
spillway	11 [pieces]	$0.112*10^6$	1.23	46
TOTAL CONSTRUCTION COSTS:			21.71	805
contingency (15 %)			3.26	121
overhead (10 %)			2.17	81
TOTAL COSTS:			**27.14**	**1007**

Table 4.8 Estimation of the total construction costs of the final dam

5 Tidal Power Station

In chapter 4 the alignment for the estuary-dam has been selected. In the next sections the size of the tidal basin and the number of turbines will be optimised.

5.1 Ebb-generation as mode of operation

In a tidal power station three modes of operation are possible to generate energy. These three are:
- Ebb-generation mode
- Flood-generation mode
- Two-way generation mode

In the ebb-generation mode, see Figure 5.1, the sluices are opened on the flood tide allowing the basin behind the barrage to fill. On the ebbing tide this retained water is used to power the turbines until the operating head becomes too low, sometime after low water. The turbines are then closed down until the rising flood tide reaches the level of the drawn- down basin and the cycle restarts. This mode of operation has been shown repeatedly to be the most cost-efficient method of tidal energy production as it combines high energy productivity with comparatively low-cost turbines and sluices.

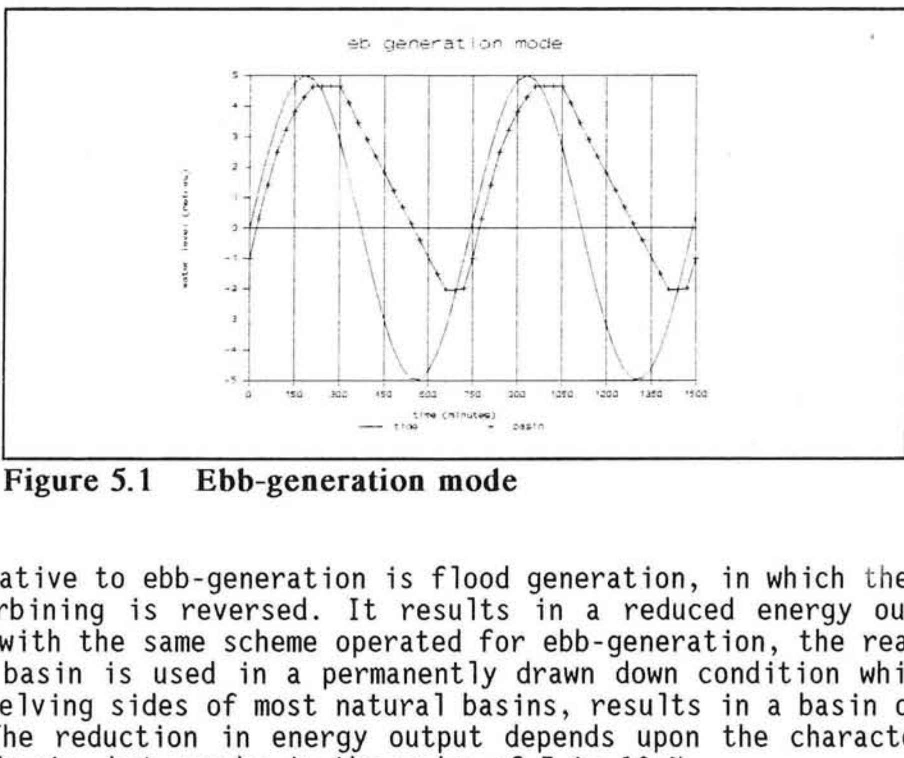


Figure 5.1 Ebb-generation mode

An alternative to ebb-generation is flood generation, in which the direction of turbining is reversed. It results in a reduced energy output when compared with the same scheme operated for ebb-generation, the reason being that the basin is used in a permanently drawn down condition which, owing to the shelving sides of most natural basins, results in a basin of smaller volume. The reduction in energy output depends upon the character of the enclosed basin, but may be in the order of 5 to 10 %.

Generation from and to the same basin (two-way generation) on both ebb and flood tides, see Figure 5.2, has two inherent advantages:

1. power is being produced for a larger proportion of the tidal cycle,and is therefore, more likely to correspond to the peaks of the system demands;

2. Since the generating head is generally reduced the maximum power level is less which tends to alleviate some of the problems of integrating large power blocks, characteristic for single-effect operation, into the electricity supply system.

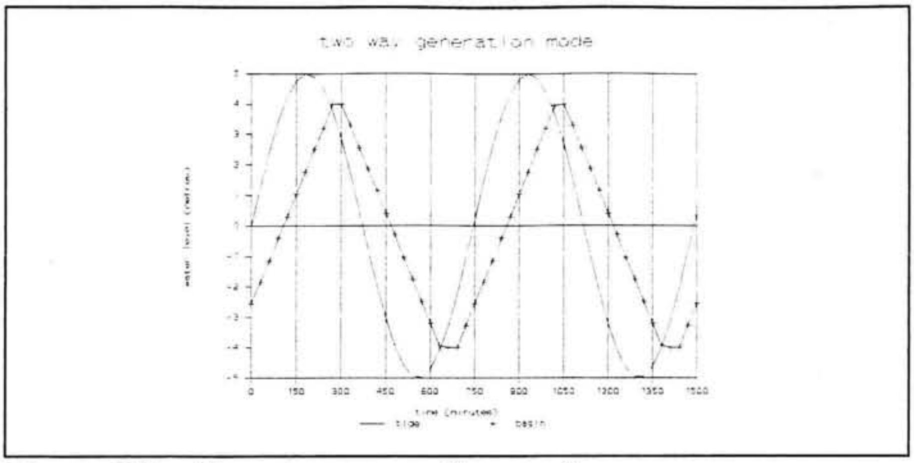


Figure 5.2 Two-way generation mode

However, as a result of the compromises on runner blade design, water passage geometry and distributor positioning which have to be made to allow the turbines to operate in both directions, turbine-efficiency is substantially less than for a specially designed one way generation machine. The reduced efficiencies lead to an energy-loss which in certain circumstances results in an overall performance in a two way generation mode which is inferior to ebb-generation. Energy gains of two way generation over ebb-generation are significant if the tidal range exceeds eight metres. Energy gain for two way generation is not, however, of primary importance because what is required ultimately is to maximise the revenue generated (or costs saved) by a tidal power scheme. The main attraction of two way generation is that power can be produced, to a limited degree, to correspond to high system demand and hence acquire high value for its output.

There is a possibility to generate energy over the whole tidal cycle if there are two tidal basins available. This double-basin scheme has one high tidal basin and one low tidal basin. The exploitation curve for this system is given in Figure 5.3.

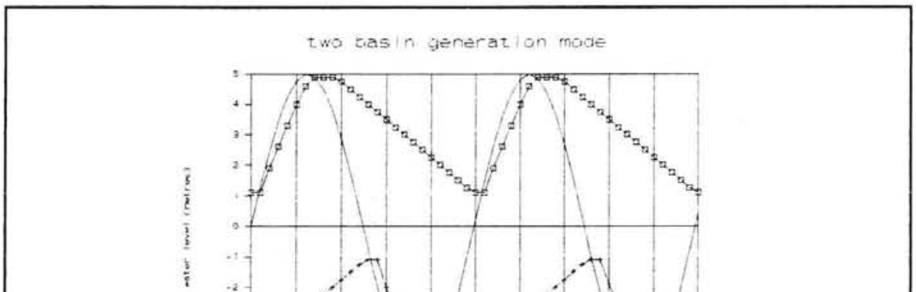


Figure 5.3 Two basins generation

The high basin A is filled through the intake structures A during the rising tide until some time after the high tide. When during ebb the water-level at sea has the same level as in basin one the intake structures are closed. During the whole tidal cycle the water flows from basin A through the turbines to the low basin B. Therefore the water-level in basin B rises. However when the water-level at sea drops below the water-level in basin B the scour sluices connecting basin B with the sea are opened. These scour sluices are closed again when the water-level at sea has risen to the water-level of basin B. In the mean time water kept on flowing through the turbines from basin A to basin B. When the water-level at sea rises to the water level of the high basin the intake structures A are opened again completing one tidal cycle.

There are two major complaints for this double-basin scheme:

1 the costs required to build the tidal power station (two basins, two sluices and the power station itself);
2 the total amount of generated energy (this is only 50 % of the energy that could be generated with an ebb generation mode).

Consideration of the generating cycles shows that ebb- or flood-generation (in either one way or two way generation mode) could be boosted if the basin were 'primed' by pumping prior to the generation period. In the case of ebb-generation, pumping would be from the sea to the tidal basin shortly after high tide against a low head, thus raising the reservoir level, see Figure 5.4.

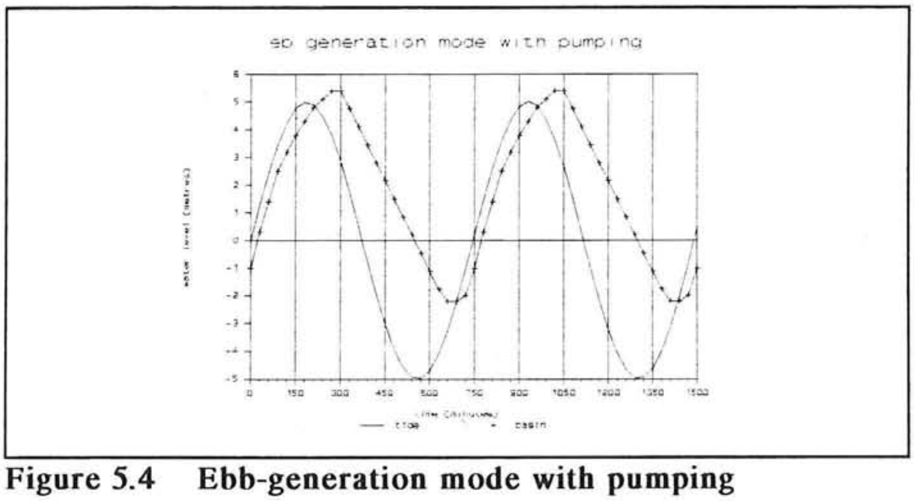


Figure 5.4 Ebb-generation mode with pumping

When optimised, the energy used in pumping can be more than recovered because of the increased head available for the subsequent generating period. Increase in production can be achieved for any tidal range, with the largest energy gain occurring with moderate tidal ranges. The magnitude of the energy gain over ebb-generation operation which can be achieved in this way is variable, depending on scheme parameters. Evidence from the analyses of operating regimes at La Rance indicates an energy gain of about 10 %, whereas the energy gain for the proposed power station in the Severn is no more than 2 %.

The preoccupation of the operators of any scheme with pumping will be to derive the maximum revenue from this facility. This will entail use of pumping only at those times of day when energy value is at or below some economical limit. This means that pumping will be used only on times when its use is financially advantageous, so that the percentage gains mentioned above will not necessarily be achieved in practice.

The project in the Gulf of Khambat consists of a tidal power station in combination with a fresh water storage. This fresh water will be needed in a wide surrounding area as irrigation and drinking water. The amount of energy required for this transport is estimated at 1,400 GWh per annum (see lit. [5]). If reservoirs are set up near the water consuming areas, these reservoirs can be filled with water pumped up with energy which is not required in the energy supply system. In this way maximum profit can be made from the energy produced in the night hours, and energy produced during a spring tide. A spring tide results in an energy peak in the output of energy. Since these large outputs can not totally be absorbed by the energy supply system for the full price of a kWh, the rest can be used for pumping. The possibility of using the energy produced by the tidal power station for pumping fresh water has the positive effect of a buffer and makes it less important to have a continuous energy production.

A second point of interest to decide what kind of production method is required is the cost of the tidal power station. The issue of costs is always important, but in this case the total costs of the project are enormous no matter what scheme is chosen. Every reduction in costs would assist in making the scheme feasible.

The double-basin scheme requires more investments and has a lower energy production as the ebb-generation scheme. Therefore this scheme has been abandoned as not feasible. The two-way generation scheme also requires much investments, with no extra energy production compared to the ebb-generation scheme. Finally the flood-generation scheme can be ruled out since it has the same disadvantages as the ebb-generation scheme plus a lower energy output. An extra advantage of the ebb-generation mode is the relatively high average water-level in the tidal basin. The harbour of Bhavnagar has the same water level as the water level in the basin; all four possible alignments for the basin dams cut off the harbour from the rest of the Gulf of Khambat. A high water-level in the basin means that the harbour is longer or even continuously accessible to ships with certain draught.

The reasoning for taking an ebb-generating scheme is very rough, and with very little argumentation of figures. A further study to the possibilities of the other schemes is desirable, as is a study to the feasibility of pumping to increase the head over the turbines. In this report pumping is not applied to keep the starting investments as low as possible. It may very well be that after a further economic evaluation, pumping will be considered as desirable.

5.2 Basin for the tidal power station

The tidal basin will be made in the closed part of the Gulf of Khambat, this makes it more simple to build the tidal basin dams. The basin for the tidal power station has to be located adjacent to the dam to have a connection with the non-closed part of the Gulf of Khambat. Another restriction for the location of the basin is the Narmada inflow. Since this water must flow into the fresh water storage, the basin can not be placed at the eastern part of the Gulf. Only one location remains for the basin: north of the dam, at the western part of the Gulf.

This location contains the earlier mentioned advantage for the Port of Bhavnagar. Since the port has no connection to the fresh water lake, it will not have the same water-level as the lake. This water-level can drop to low values in years of low precipitation and the port would become unaccessible for ships with certain draught. An extra advantage of the location is the absence of salt intrucion in the fresh water basin as result of sluicing the ships to Bhavnagar.

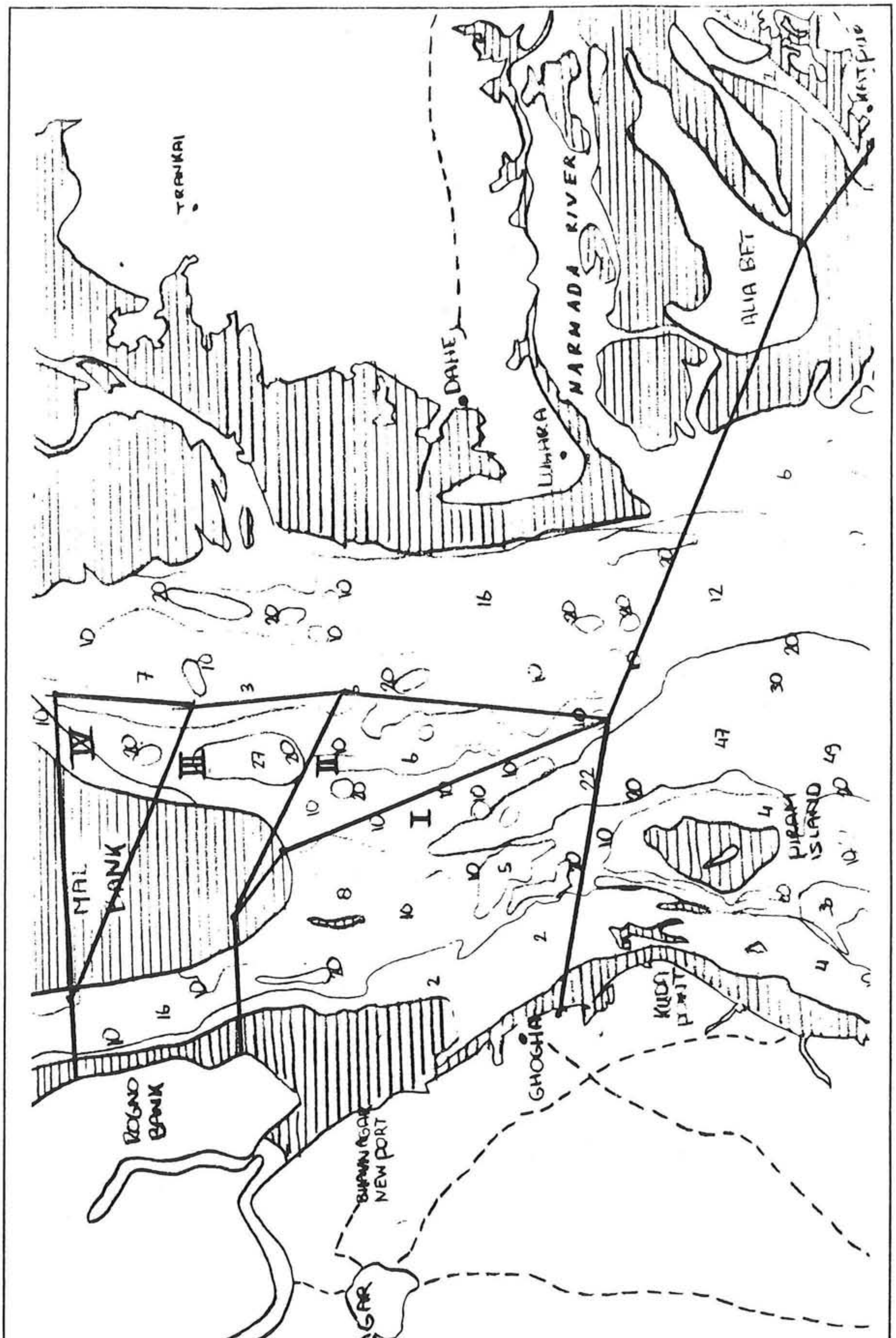


Figure 5.5 Alignments for the basin dam

With the positioning of the basin, nothing has been said about the size of the basin and the alignments of the basin dams. In this section some suggestions will be made about the size of the basin and the possible alignments.
In this case the best alignments for the basin dams will be determined by choosing the alignments over the shoals and sand banks in the Gulf, this making the total amount of sand required for the tidal basin dams as small as possible. The size of the basin will follow from the chosen alignments.

The tidal basin dam alignments considered in this section, see Figure 5.5, all have the same starting point at the estuary-dam; right at the middle where there is a slight twist in the dam. At this point the bottom is not as deep as westward of this point; the depth is only 12 metres. More to the east the basin dam would come too close to the eastern shore of the Gulf hindering the inflow of the Narmada at times of large discharges. From this point, four possible alignments have been determined.

The first basin dam alignment goes from the fixed starting point straight to the southern point of the Mal Bank. From the Mal Bank the shortest route to the main land is chosen, to the Rogno Bank. The length of the dam is 30 kilometres. The basin surface at BM + 6 metres is 256 square kilometres.
The reason for determining the surface of the basin at this height is because energy generation in an ebb generation mode takes place with the slice of water between BM + 4 metres and BM + 8 metres.

The second basin also begins in the fixed point but has a more easterly course over the southern part of the Makra Bank, resulting in a larger basin surface. The second alignment has the same alignment between the Mal Bank and the Rogno Bank as the first. The length of the alignment is 34,5 kilometres and the enclosed surface at BM + 6 metres is 363 square kilometres.

The third alignment has an extension to the north. The eastern part of the alignment is located on the Makra Bank with depths of only three to five metres below BM. The length of the extension to the north is eight to ten kilometres. The length of the dams is 43,5 kilometres and the basin has a surface of 473 square kilometres.

The fourth and last alignment is quite similar to the third but has a small extension at the north eastern point. The Makra Bank is followed until it deepens and than the alignment takes the shortest route to the western shore of the Gulf. The length of the alignment is 49,5 kilometres and the surface of the basin is 510 square kilometres.

To be able to calculate the quantity of sand needed for the closure, the cross section for the tidal basin dams has to be determined. The crest height of the tidal basin dams is less than the crest height of the estuary dam since the wave attack on the tidal basin dam is much less severe. The shape and height of the tidal basin dam will be determined in Annex V. The result is given in Figure 5.6. For the precise dimensions of the revetment is referred to Annex V.

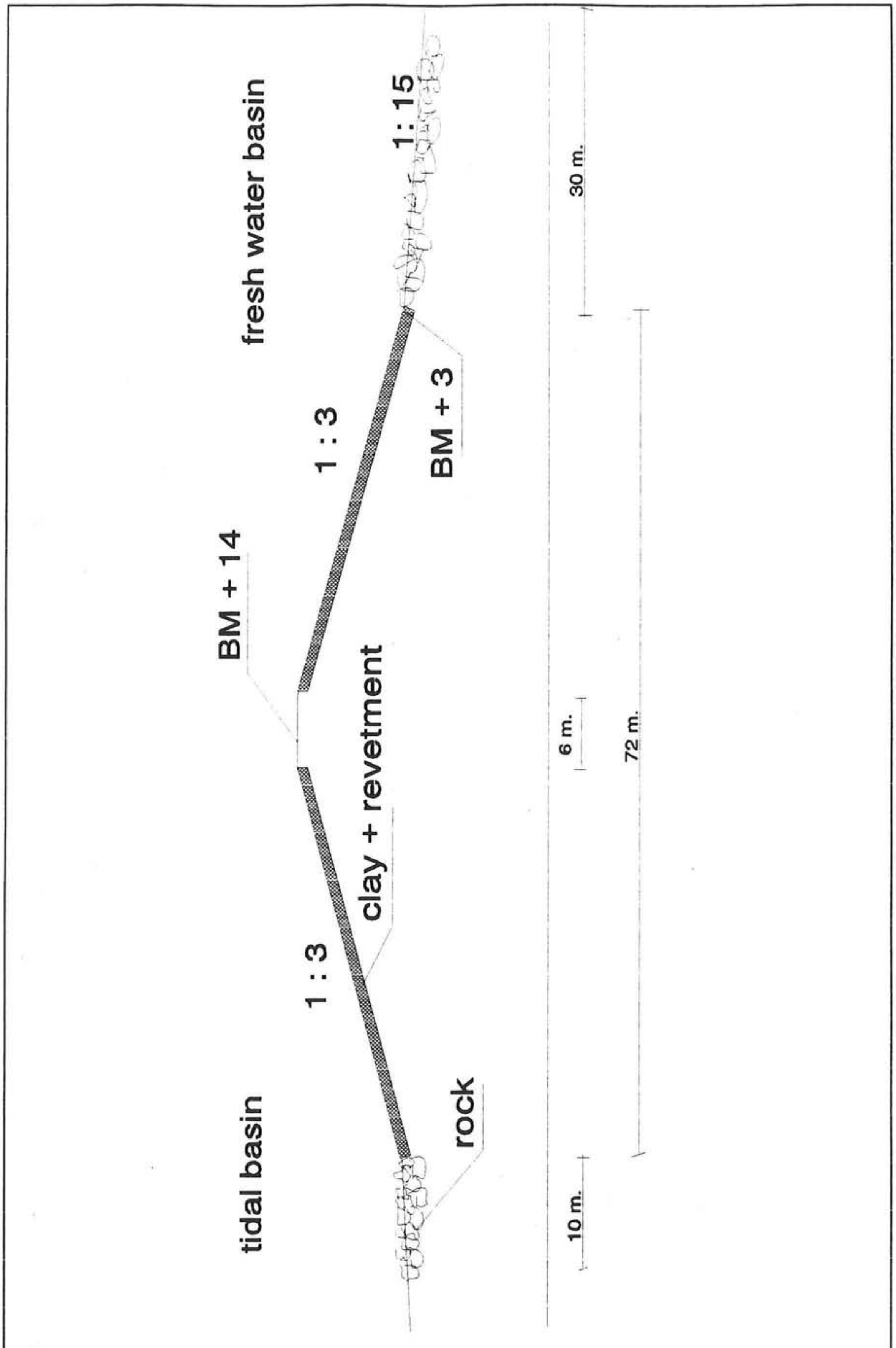


Figure 5.6 The cross section of the basin dam

With this cross section the quantity of sand can be calculated. The figures are given in Table 5.1.

Alignment	Length (km)	Surface (km^2)	Quantity of sand (10^6 m^3)
1	30.0	256	111
2	34.5	363	111
3	43.5	473	106
4	49.5	510	108

Table 5.1 Characteristics for the four tidal basin dams

From the table it follows surprisingly that the quantity of sand required for the alignments does not vary much. For alignment four less sand is required than for alignment one, but the basin surface area is more than double. Between alignments 3 and 4 no significant difference is found. The cost of the dam depends also on the length of the dam. The longer the dam, the more road, clay and revetment are needed. In this case the cost difference is obvious.

However, since more than half of the costs are made for the enormous quantity of sand required, the conclusion is that the larger basins are relatively cheap.

5.3 The use of the Duflow model for tidal power station.

The three main elements for a tidal power station are:
- the basin and basin dams,
- the intake structures,
- the generator houses and turbines.

To optimise these elements the water-levels inside and outside the dam have to be known with reasonable accuracy. It is possible with the earlier used model in Duflow to calculate the water-levels outside the dam in case of a fully closed dam. Water-level predictions can be found in Appendix A for neap tide, mean tide and spring tide.

In case of an intake structure or outlet structure or turbines in operation the water-levels at both sides of the dam are influenced. If water is let into the basin, the water-level outside the dam will drop and the water-level at the closed side of dam will rise in comparison with the rest of the basin. The basin does not react uniformly to an inflow. To determine this loss in head new calculations have been made with Duflow adapting the existing model to the new situation.

Firstly the basin had to be re-schematized. The schematization in the original model was too coarse to handle small discharges. Another reason for the new schematization is the limits of the basin. In the existing model these limits would lie in the middle of chosen sections. In the new model the limits of the basin correspond with the end of a section or with side limits of sections. The new sections and nodes are given in Figure 5.7 along with some nodes and sections outside the basin area.

To be able to simulate a tidal power station the inflow through the intake structure, and the outflow through the turbines has to be described.
In Duflow a limited number of structures types can be used namely:

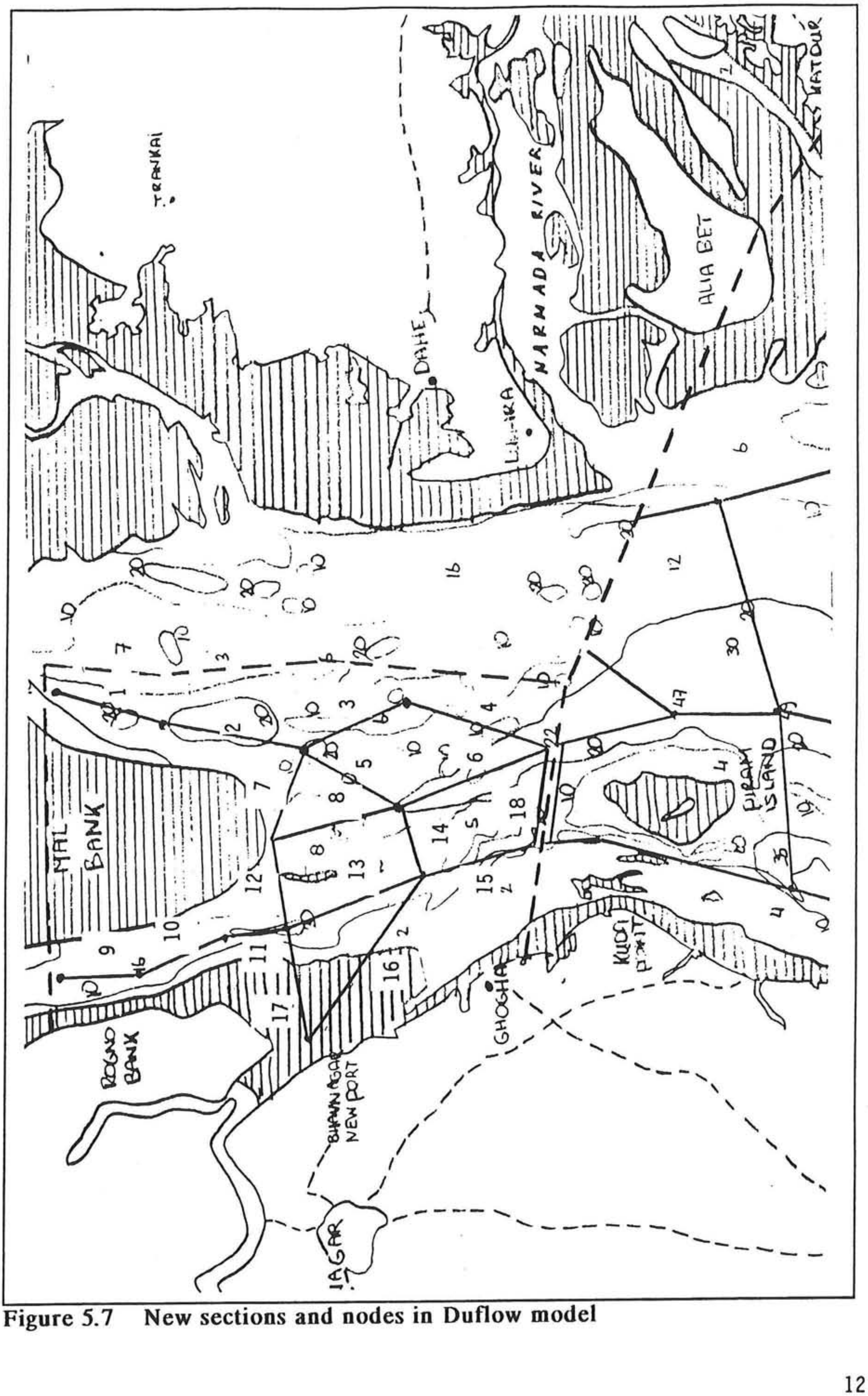


Figure 5.7 New sections and nodes in Duflow model

- overflow;
- underflow;
- pump;
- syphon;
- culvert.

Some of these structures can be activated by differences in head. The head over a structure is the difference in water-level at both sides of the structure. The pump however can not be activated in case of a head difference but is only activated if a water-level at a certain point is exceeded.

To model the intake structure an overflow is used with a sill at 5 metres below BM. This is a realistic model; if the turbines and the intake structure are combined in one structure the sill of the overflow would not be lower then 5 metres below BM. If on the other hand the turbines and the intake structures are not combined it is possible to use an intake structure with a sill lower than 5 metres below BM. This would make the intake structure much shorter.

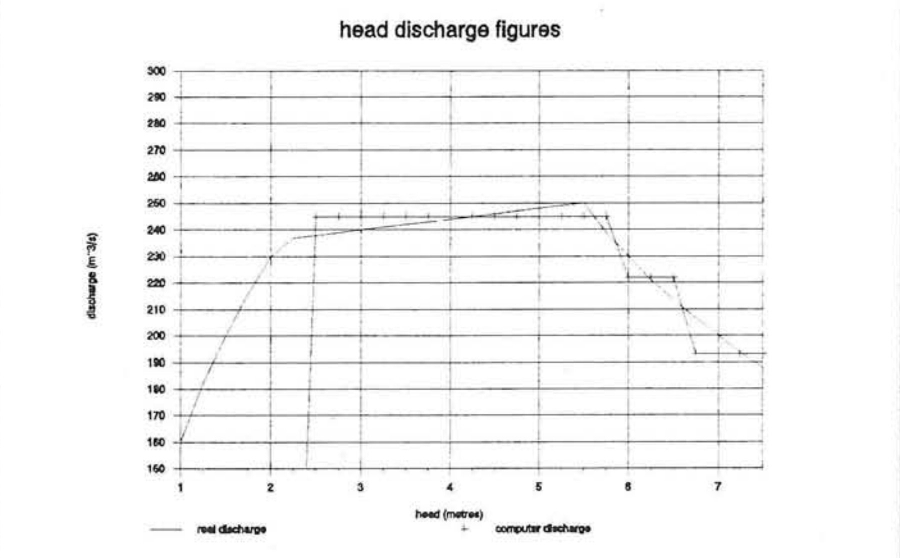


Figure 5.8 Discharge-head figures for ideal situation and simulation

For the turbines some improvisation was needed to get a realistic model. A pump does not satisfy since it can not be steered according to the existing head. Chosen is for the overflow with a sill at 25 metres below BM. The advantage of the overflow is that a discharge can be steered for a given head by adjusting the length of the overflow. The low sill-level is chosen because this insures the discharge depending only on the head and not of the water-level, as it should be in case of simulating a turbine. The result is a more or less constant product of discharge and head, quite similar to the discharge head figures of a turbine. The ideal maximum discharge-head figures (of a turbine) and the realised maximum discharge-head figures (with the DUFLOW model) are shown in Figure 5.8.

At the first stage of simulation the size of the intake structure was with a length of 10.000 metres much too large. This was done in order to obtain a clear optimum for the discharges in the turbines section. If the number of turbines and the size of the intake structure are optimized at the same time the process would take much longer and would be more indistinct. The cost of the turbines and the powerhouses are furthermore much higher than the costs of the intake structure, so an optimum for the turbines is more important and decisive in the process. In a later stadium the dimension of the intake structures is optimised.

5.4 Choice of turbine and control strategy

Control strategy for the turbines

As preconditions for determining the best control strategies a maximum energy output, and a maximum number of turbines is taken. The intake structure is chosen with the earlier mentioned width of 10,000 metres. A choice of basin is not necessary since for a larger basin, a higher number of turbines is taken resulting in the same water level changes.

The different control strategies for the turbines that have been looked at are the following three:

I Start the energy production at a head of 2 metres and adjust the discharge if the head becomes larger than 3 metres and 4 metres, the energy production is stopped if the head becomes less than 2 metres.

II Start the energy production at a head of 3 metres and adjust the discharge if the head becomes larger than 4 metres, the energy production is stopped if the head becomes less than 3 metres.

III Start the energy production at a head of 4 metres, the energy production is stopped if the head is less than 4 metres.

The strategies have been checked for neap tide, mean tide and spring tide.

Strategy number III gives the largest energy output in case of a spring tide but gives no energy production at all if there is a neap tide. Since some continuity in energy production is desirable this strategy has been abandoned.

If strategies I and II are compared on output of energy and maximum discharge a simple decision can be made. Strategy II gives an higher energy output for neap, mean and spring tide. Furthermore the maximum discharge as a result of strategy II is lower than the maximum discharge as a result of strategy I. Figures are given in table 5.2 for basin one.

	I		II	
	energy (MWh/day)	max. discharge (m^3/s)	energy (Mwh/day)	max. discharge (m^3/s)
neap	6,000	54,000	8,000	68,000
mean	16,000	64,000	17,700	62,000
spring	25,700	90,000	28,000	83,000

Table 5.2 Energy and discharge figures for different control strategies

A bigger discharge means more turbines or a higher velocity. In both cases more investments have to be made to start the energy production.

In Figure 5.9 the energy output is shown as a function of the time for strategy I and II. In this figure the (only) advantage of strategy I is shown: energy output takes place during a longer interval of the tidal cycle. Since the energy output for strategy II is on average more then 10 % higher than for strategy I, and since it can be generated with no extra costs strategy II is chosen as generating strategy.
To determine how much turbines are required to generate the net output calculated with DUFLOW the total discharge must be divided by the discharge per turbine. In the next paragraph a choice of turbine shall be made and the characteristics of this turbine shall be given.

In this project the head over the turbine can vary from 1.5 metres (the minimum head required to generate energy with turbines) to somewhat less then 10 metres (in case of a high spring tide). In normal conditions however the head over the turbine will vary from 2,5 metres to 6 metres. With normal conditions is meant a mean tide and an economic generating strategy as is chosen in this chapter. For these head different types of turbines can be selected. The Pelton and Francis turbines can not be used since these only work with a high head (higher then 20 metres).

Turbines that can satisfy our demands for the head are the Propeller turbines. Propeller turbines have guide-vanes mounted peripherally around the runner hub and upstream of the runner blades. These vanes impart an initial whirl to the incoming discharge so that it meets the runner blades at an appropriate angle. The vanes may be fixed or movable, the latter form being more expensive. The whole assembly of guide-vanes is called the distributor, and so there may be fixed distributor (FD) or variable distributor (VD) turbines. A turbine which is VD and has movable runner blades (VB) is double-regulated, Whereas a FDVB or a VDFB one is single-regulated. [see lit. (22)].

Fixed distributor, VB, propeller turbines operate efficiently over a wide range of heads (although not so wide as the double-regulated machine). This type requires a downstream gate in the draft tube for operational and emergency purposes since closure of the runner blades does not stop the flow of the water completely.

Variable distributor, fixed blade (FB) turbines have a narrower range of efficient operation and do not maintain efficiency well when operated away from the design head. Nevertheless, because VDFB types can discharge larger volumes at low head conditions although at lower efficiency, on balance there is little to choose (± 0.5 %) between the energy-generating potential of VDFB and FDVB types.

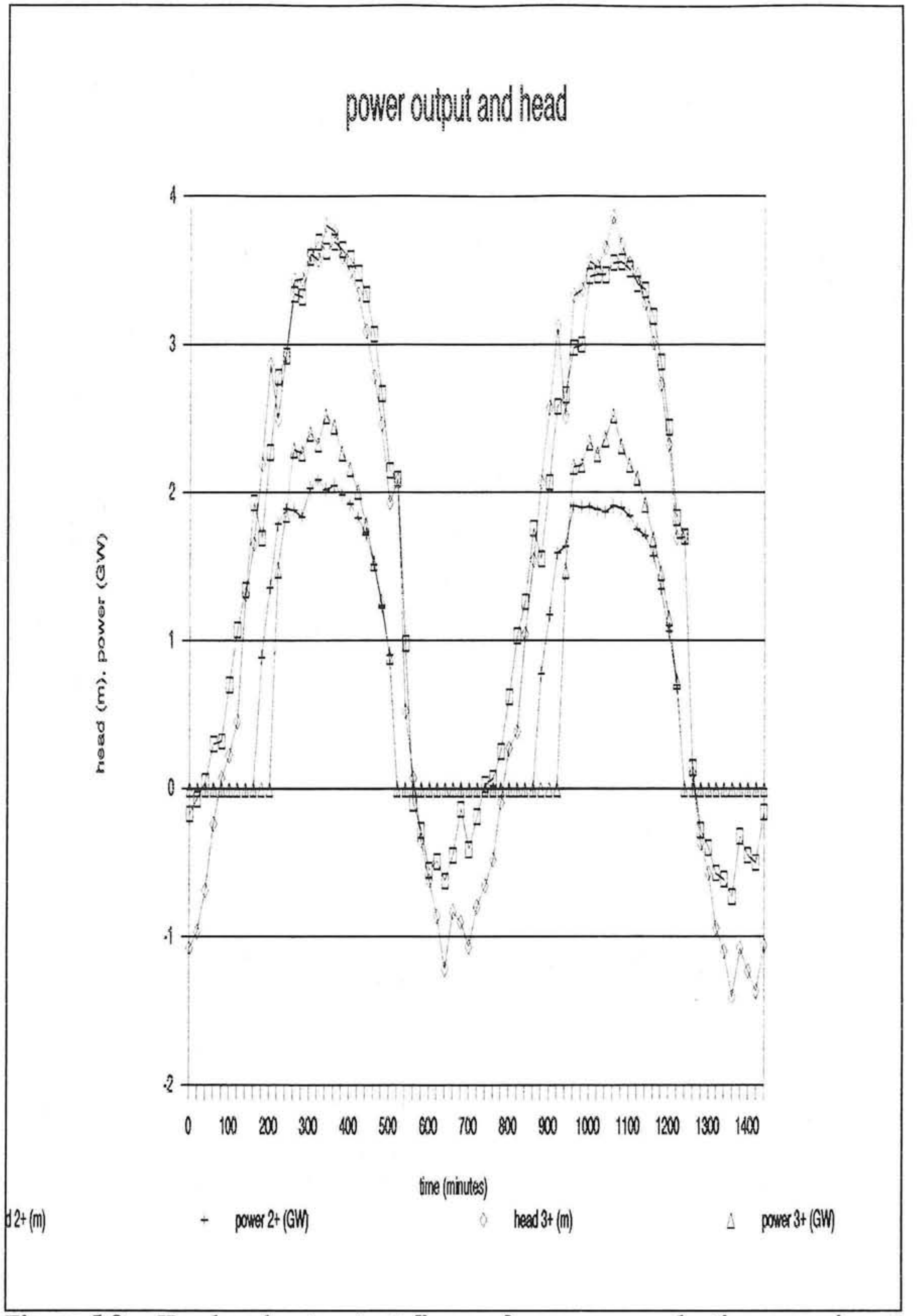


Figure 5.9 Head and net output figures for energy production engaging at 2 metres and at 3 metres

Double regulated turbines have the widest range of efficient operation and can mantain efficianty when away of the design head better than the earlier mentioned two single regulated turbines. Since the head over the turbine in this project is rather variable, the head can vary between 3 and 6 metres in case of a spring tide, the double regulated turbine is chosen. The extra costs as result of the more expensive turbine is expected to be regained by the higher efficiency of the turbine.

Turbine characteristics have been taken from a preliminary study for a tidal power scheme in the Wyre, England (lit [23]). The turbine used in that project has a design head of 5.5 metres. The tidal power station in the Gulf of Khambat requires turbines with a design head ranging from 3.0 metres in case of the maximum number of turbines, to 6.0 metres in case of not more than 50 turbines. The turbine of the Wyre Estuary is therefore of the right kind. However some small modifications for the design head will have to be made.

The characteristics of this turbine can be found in Annex VI. The efficiency curves of this turbine are not given but are assumed not to be very different from other efficiency curves. The possibility of using these curves in the Duflow model is limited, therefore a constant efficiency factor is used of 90 %. The turbine that will be used is of the type VDVB, the same as the turbine used in the Wyre Estuary.

As can be seen in Annex VI, the turbine used in the Wyre Estuary (the basis turbine), is a turbine with a diameter of 6 metres and a design head of 5.5 metres. With these dimensions a maximum discharge can be processed of 250 m^3/s, resulting in a maximum net power output of 12,5 MW (efficiency = 91 %).

The tidal power station in the Gulf of Khambat is much larger than the power station designed for the Wyre. Therefore turbines with larger diameters can be used in the Gulf of Khambat. In the present technology the maximum diameter for turbine with a horizontal axis is 8 metres. This diameter is chosen for this project to reduce the number of turbines. Disadvantages of such large diametered turbines is the sensitivity of the structure for vibration. This sensitivity determines the maximum attainable diameter. The second disadvantage is the depth of the total structure. When a larger diameter is used for the turbine, the foundation of the power-house has to go down 1,0 times the difference in diameter. One half times the difference because the axis of the turbine has to be located on the same hight, and a second half times the difference because the powerhouse structure below the turbine equals half the size of the turbine diameter.

With this new diameter for the turbine a new set of characteristics for the turbine can be calculated with the help of the two expressions given below in equations 5.1 and 5.2. These expressions give the relationship between similar turbines for different diameters (D), different heads (H), different rotation speeds (n) and different maximum discharges (Q).

$$\frac{Q}{n\,D^3} = constant \qquad \mathbf{(5.1)}$$

$$\frac{n^2\ D^2}{H} = constant \tag{5.2}$$

The use and results of these expressions can be found in Annex 7. A short summary of the results is given in Figure 5.10. The maximum discharge through one turbine (and the rotational speed of the turbine) is for given diameter still depending from the head over the turbine. This relation can be described in an equation and is done so below.

$$Q = 190\ \sqrt{H}\ \ m^3/s \tag{5.3}$$

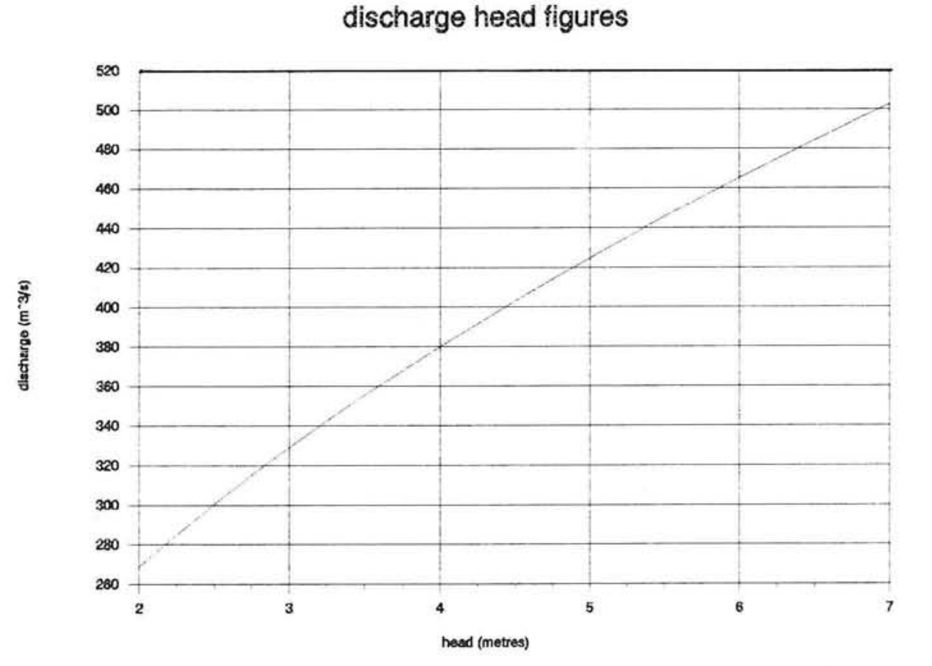


Figure 5.10 The relation between discharge and head for a turbine with a diameter of 8 metres

In Annex VII an outline design for the turbine caisson is given.

5.5 Optimisation of the elements of the tidal power station

For all four basins Duflow calculations have been conducted. The calculations that have been made had to give a solution for three questions.

- what basin is most profitable ?
- What number of turbines is most profitable ?
- What seize of the intake structure is most profitable ?

These three questions are interrelated. To get an optimum for the number of turbines the basin has to be known, and to know what basin has to be chosen the energy output of all four basins has to be known inclusive the number of turbines etc. etc. In this section the three questions will be answered separately. In a later stadium the effect of changing preconditions for the first answered questions will be taken into account.

Choice of basin

The first choice that has to be made is about the basins. As was shown in section 5.2. the amount of sand needed for the four basin dams does not vary much. However the enclosed tidal basin behind the basin dams, and the potential energy that goes with it, does. Firstly a calculation was made with Duflow to determine how much energy can be generated with the chosen strategy for all the four basins in case of a mean tide.

The results of these calculations are shown in Table 5.3

	time of energy-production per day in minutes	maximum gross output capacity in MW	gross output per day in GWh
basin 1	600	2450	17.7
basin 2	560	3600	29.5
basin 3	600	4500	38.5
basin 4	620	5100	41.7

Table 5.3 Maximum energy output for the four basins

As is shown in Table 5.3 the net output of basins 3 and 4 is much larger then the net output of basins 1 and 2. But on the other hand, so is the number of turbines needed to generate this energy.
In the case of the basin calculations given in Table 5.3 the average head over the turbine was 3,5 metres. With this head the discharge through the turbines is 350 m^3/s, and the maximum gross output of the turbine is 12,5 MW. The number of turbines needed for the four basins are:
for basin 1 : 180 turbines;
for basin 2 : 290 turbines;
for basin 3 : 350 turbines;
for basin 4 : 360 turbines;

The total number of turbines needed for the basins seems very high, and it would if this number of turbines was used, but the figures given above only describe the maximum number of turbines possible (and working) in the basin. To make a choice as to which basin is the best some more calculations were made for basins 3 and 4 with less (180) turbines. For basin 3 the gross energy output drops from 38.5 Gwh a day to 32.9 Gwh, and for basin 4 the output drops from 41.7 GWh to 35.3 GWh. These are considerable losses in energy (14 % and 15 %) but even more considerable reductions in number of turbines and costs (49 % and 50 %). After a first evaluation follows that the energy output for basins 3 and 4 is for the same number of turbines much higher then the output for basins 1 and 2.

In Table 5.4 the energy output all four basins as result of 180 turbines is given. The costs necessary to build the power stations do not vary much for these four scenarios. The power stations for basins 3 and 4 are somewhat more expensive since the intake structures are larger and the basins are larger (less storage for drinking and irrigation water and more sand required for the dams.

	gross output of energy in GWh a day
basin 1	17.7
basin 2	26.0 (estimated)
basin 3	32.9
basin 4	35.3

Table 5.4 Energy-output for the four basins as result of 180 turbines

On basis of these results basins 1 and 2 will be dropped for further study, both give much less energy for more or less the same costs. The difference between basins 3 and 4 are not so extreme, these two basins therefore will be further evaluated. This evaluation concerns the size of the intake structure and the most economical number of turbines.

Intake structure

The size of the intake structure determines the water level in the basin. After generation water is let in to refill the basin. If the intake structure is small the water level inside will not be able to follow the water level outside the dam. Resulting in less water available to generate energy at a lower head. With sufficient size of the intake structure water levels inside the dam will reach the same height or even get slightly higher than outside the dam resulting in a higher possible energy output. Further enlargement of the intake structure will give some better energy output, but not enough to compensate the extra costs made for the intake structure.

For basin 4 several calculations were made with different sizes for the intake structure. The width of the intake sluice was varied between 3000 metres and 8000 metres. The exact figures of the calculation can be found in Appendix A7. In Figure 5.11 the influence of the seize of the intake structure on the possible gross energy output can be seen. If the intake structure has a width of 3000 metres energy production on a mean tide are as high as 33.6 GWh a day. When the structure has a width of 8000 metres the production rises to 43.0 GWh a day.

The possible energy output does not rise very much if the intake structure gets any wider then 6000 metres. Therefore the size of the relatively cheap intake structure has been set at 6000 metres. For basin 3 a smaller intake structure will do since the basin is smaller then basin 4. The width of this intake structure has been set in relation to the basin surface at 5500 metres. This width of the intake structure has been calculated without using the possibility of letting some of the water for the basin flow through the turbines. This will lead to a reduction of the necessary width of the structure. The seize of the reduction is depends on the number of turbines used.

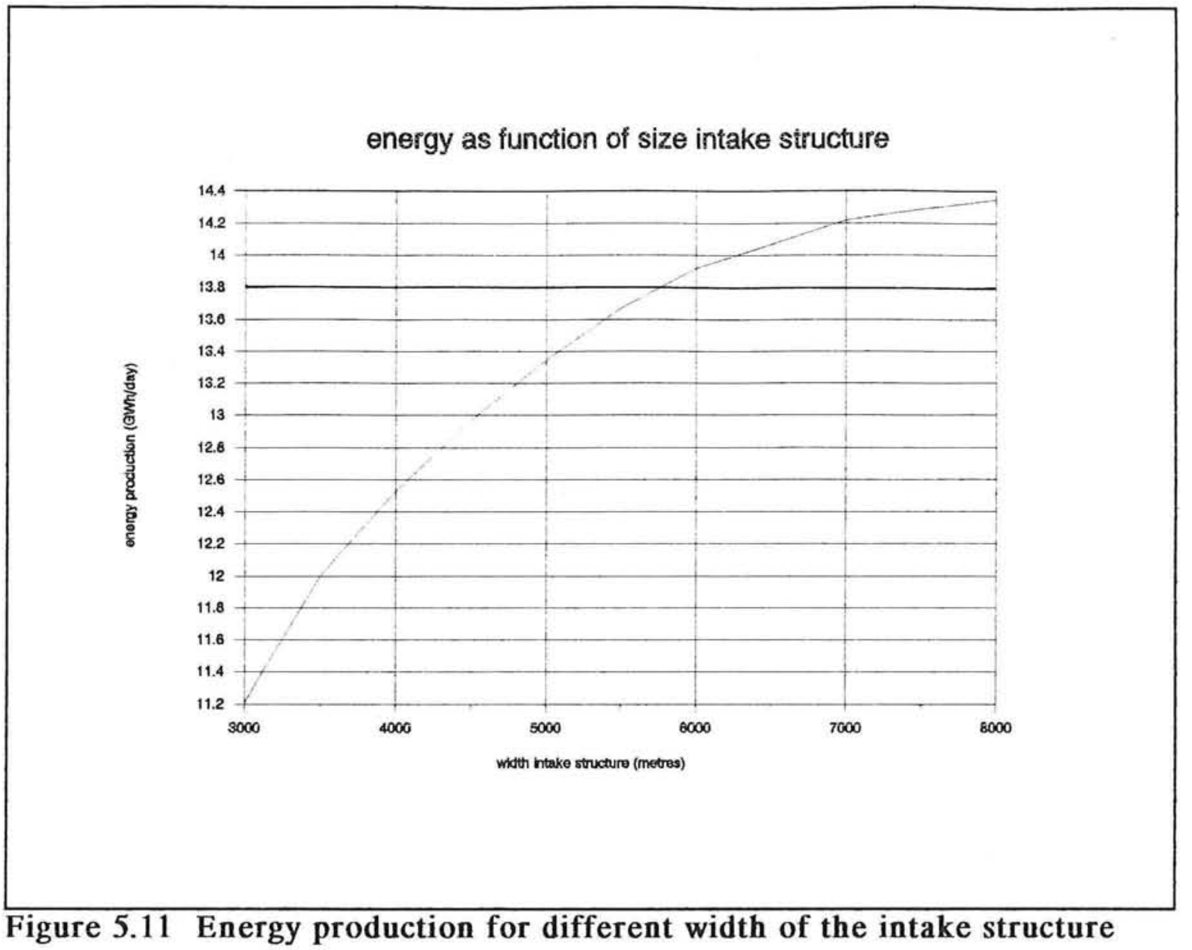


Figure 5.11 Energy production for different width of the intake structure

Number of turbines

For basins 3 and 4 calculations have been made with the Duflow model to determine how much energy can be generated in each basin with 50, 75,100, 125, 150, 200, 250 ,300, 350 and 400 turbines. The results are given in Table 5.4 along with the power of the turbines. This power is not proportional to the number of turbines. If there are less turbines, the head over the turbines will be bigger, resulting also in a bigger discharge through one turbine. The power of a turbine is proportional to the discharge and the head, thus the power per turbine rises if less turbines are used.

The figures for the nett energy output are also given in figure 5.12 to get a better insight. To decide how much turbines is desirable the costs of an extra turbine versus the profits of this turbine has to be determined. This will result in a line in Figure 5.12, there where this line is the tangent line of the given curves the optimum number of turbines can be read off.
In the next section the costs and the benefits of an extra turbine will be determined. After the optimum number of turbines for both basins are determined a choice will be made between the two basins on basis of costs and benefits of the total tidal power station.

number of turbines	basin 3		basin 4	
	nett energy output GWh	nett power (MW)	nett energy output GWh	nett power (MW)
50	11.500	1220	11.900	1260
75	16.500	1670	16.800	1710
100	20.400	2160	21.100	2210
125	23.900	2480	24.600	2570
150	26.700	2840	28.000	3060
200	30.500	3240	32.700	3470
250	32.200	3650	35.600	3870
300	33.700	3870	37.300	4320
350	34.700	4050	37.500	4590
400	33.400	4190	37.300	4640

Table 5.4 Energy output and power for various numbers of turbines for basins 3 and 4 per day

5.6 The cost and benefits of an extra turbine

To determine how much turbines will be used for the two basins all the costs and all the benefits of an extra turbine has to be taken into account.

The costs of an extra turbine are obvious; an extra power house with turbine and generator has to be built and placed on an extended sill.
The benefits of an extra turbine however are more divers. Most important is the extra energy output which can be determined from Figure 5.11. Other advantages are a larger reduction on the intake structure and less metres of closure dam that have to be built.

In the determination of the number of turbines used in the project the avails of the turbines regarding reduction of intake structure and reduction of the costs of the closure dam have been neglected since the costs of the turbines is much larger than the costs of the other two (the cost of the turbines with generator and powerhouse are more than twenty times as high than the costs of the closure dam).

The benefit of a turbine can be taken from Figure 5.11, but this benefit is given in kWh. To get an optimum in the number of turbines a kWh has to be given a price. The value of the production per kWh is directly related to the alternative costs of production. These alternative costs of production seem to be about Rp 1.25 per kWh (see lit [5] Reconnaissance study with a rate of inflation of 7.5 % annual). The value of a kWh produced by the tidal power station is lower than an alternatively produced kWh since the production by the tidal power station is subjected to lack of continuity of production related to the tides and the type of generation mode chosen in section 5.1.

If the energy is used for irrigation purposes the full price can be counted. After all there is no restriction as to when the water has to be pumped to the places where it is needed. A condition for this full price is

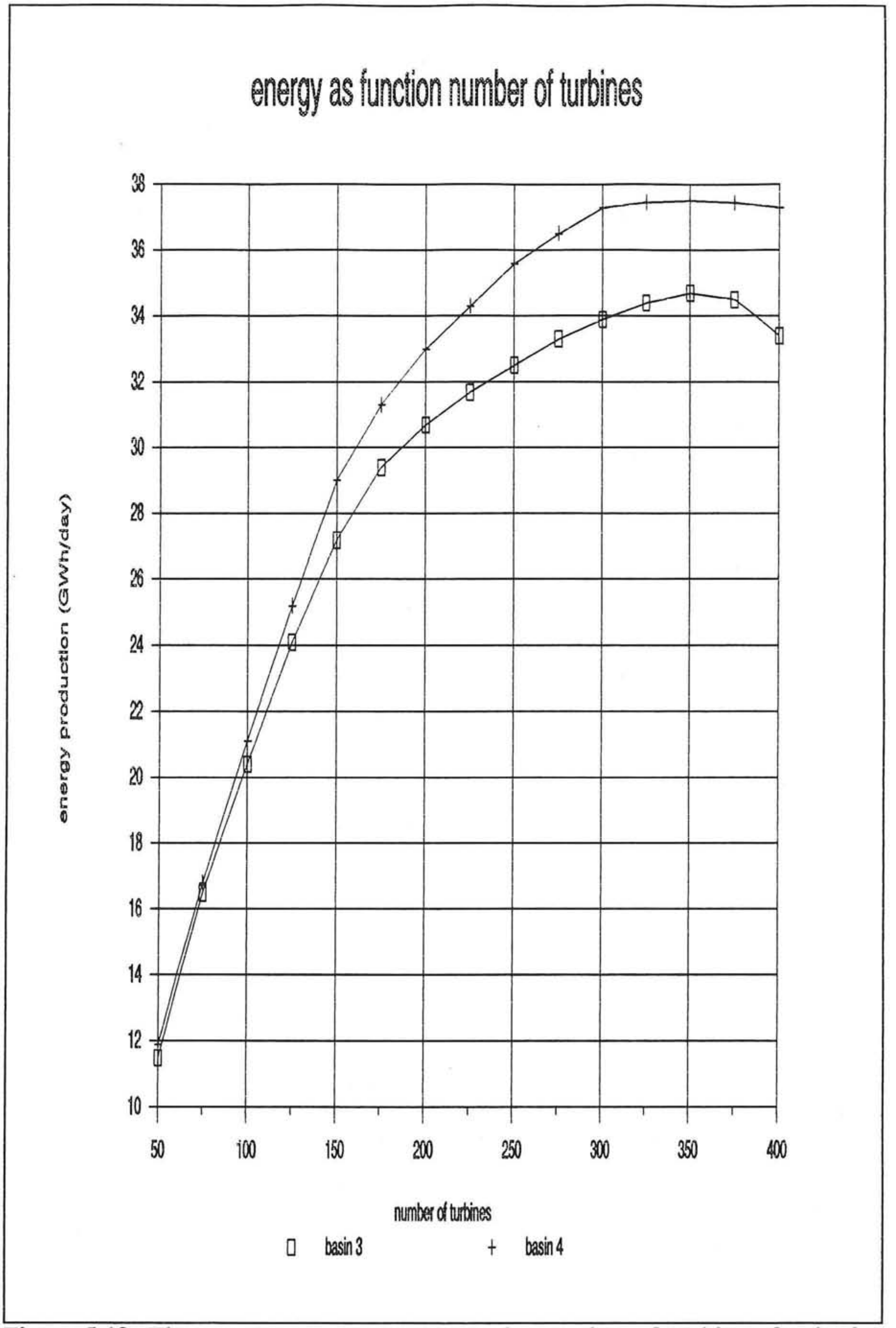


Figure 5.12 The net energy output versus the number of turbines for basins 3 and 4

the existence of reservoirs for the irrigation water. This condition beholds that extra investments will have to be made to make these reservoirs. Extra energy might be needed for pumping the irrigation water if the need for water is high during a neap tide period (low energy production by the tidal power station). The same (full price) holds for the energy used for drainage of possible polders.

An estimated value for the energy produced by the tidal power station as a total is Rp 0.75 per kWh. Other values for a kWh can be considered if a way is found to store the high energy peaks, and regain the energy in times when the power station has no production. A possible buffer might be the Shetrunji reservoir 50 kilometres South-West of Bhavnagar. This reservoir has a considerable surface of ci. 40 square kilometres at an altitude of 200 metres above the Gulf of Khambat. The distance between this reservoir and the Gulf is 30 kilometres. An other reason to consider higher kWh values is the possibility of higher oil prices in the next decennia, although this is pure speculation. For the sake of completeness kWh values will be taken of Rp 0.75, Rp 1.00 and Rp 1.25. In US$ this is US$ 0.028, US 0.037 an US$ 0.046 (with a rate of 27 rupees to 1 US$).

The costs of a turbine unit (including generator and powerhouse) have been determined with the cost equation given in an article in Water Power and Dam Construction (see lit [24]). In this article an equation is given which can be used to calculate the price of a water power station. The price is price of the station at the end of the building phase (inclusive interest losses and financial setbacks).

This equation is given below.

$$Cost = X * 10^{6} \left(\frac{MW}{H^{0.3}}\right)^{(0.82)} \qquad (US\$) \qquad \mathbf{(5.4)}$$

The value of X is given in the article but concerns the prices of the water power stations in the year of the article (1982). This given value of X is has 9 as a minimum, 15 as average and 21 as maximum. With a rate of inflation of 7.5 % annual these values have to be multiplied by a factor $(1.075)^{10}$ = 2.06 to get the prices in 1992. The range in which the value of X should than be found is between 18 and 43. The nature of the equation beholds that for a high value of the head a high value of X should be chosen. Since the head in the Gulf of Khambat is relatively low, a low value of X can be chosen. Other factors which tend to give a low value for X in this project are: the existing dam and the absence of costs of resettlement. Calculations have therefore been made with a X value of 18.0. The result of the cost calculation for the different number of turbines is given in Figure 5.13.

The line which represents the costs is a curved line, suggesting that the price of turbine unit is variable (getting lower the more turbine units are chosen). In fact the price of a turbine unit is more or less constant. The reason for this result is that the equation is not supposed to be used to calculate the turbine unit price but the price of the total structure including dam construction and transmission costs. This problem can be handled if a line is drawn into this figure giving a turbine unit price as tangent of this line and a constant cost for every number of turbine units.

Costs can then be written as: Costs = A + B * Y

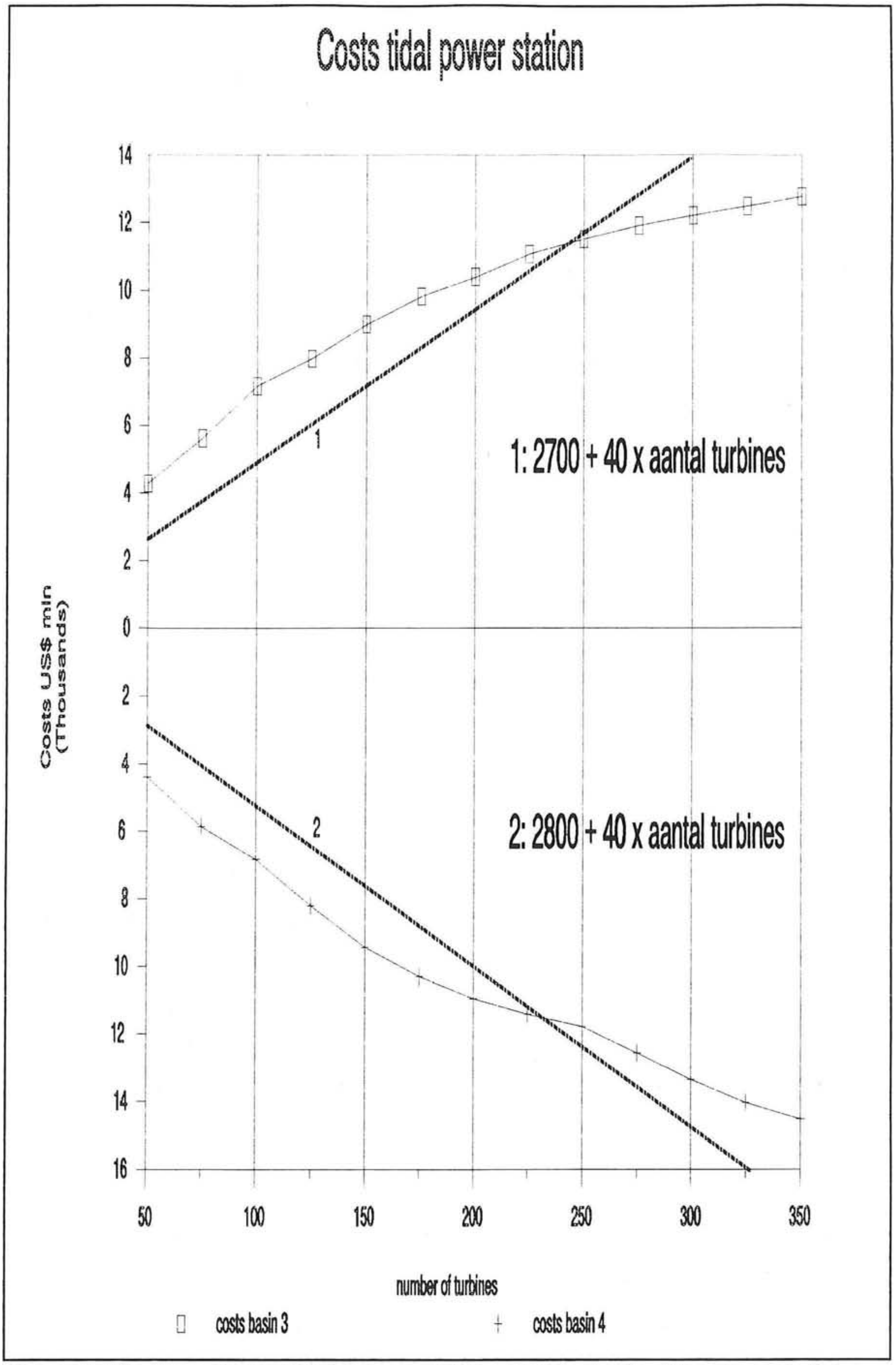


Figure 5.13 Cost of the tidal power station determined with equation 5.4

In which A is the constant costs such as for dam construction and transport lines. B is the cost per turbine unit and Y is the number of turbines. For both basins the value for the turbine unit is $40*10^6$ US$. This is a rather high figure. To test this value for a turbine unit two water power station of which the costs are known have been recalculated with the same equation. These two water power stations are both run of the river power stations build next to an existing dam in the Netherlands (see lit [25]). Both powerhouses were built inside cofferdams. The characteristics of these two water power stations are:

Water power station Maurik: costs = fl $60*10^6$ (1984)
head = 4 metres
capacity = 10 MW
Water power station Alphen: costs = fl $66*10^6$ (1986)
head = 4.5 metres
capacity = 13 MW

If equation (5.4) is used for Maurik the costs should be rewritten in US$ (1992). The costs in Dutch guilders 1992 is $(1.04)^8 * 60{,}000{,}000 =$ fl 82,000,000. The rate of inflation in the Netherlands in the years between 1984 and 1992 has not exceeded the 4 %, therefore the factor 1.04 has been used and not the factor 1.075 as was earlier used to determine the value of X, and the kWh price. With a rate of one US$ to two Dutch guilders this becomes US$ $41*10^6$.

Filling in equation (5.4):

$$Cost = X * 10^6 \left(\frac{MW}{H^{0.3}}\right)^{(0.82)}$$

$$41 * 10^6 = X * 10^6 \left(\frac{10}{4^{0.3}}\right)^{(0.82)}$$

$$X = 8.7$$

If the equation is used for the water power station in Alphen the cost can be set at US$ $42*10^6$.

$$Cost = X * 10^6 \left(\frac{MW}{H^{0.3}}\right)^{(0.82)}$$

$$42 * 10^6 = X * 10^6 \left(\frac{13}{4.5^{0.3}}\right)^{(0.82)}$$

$$X = 7.4$$

The result of these two tests show that the X value of 18 is much too high. Both Dutch projects have been built with an average X value of 8. Since the powerhouses in the Gulf of Khambat are planned to be build in caissons, which will result in higher costs, a multiplication factor of 1.25 is suggested for the X factor. This factor results in a used X factor of 10. This reduction of 45 % of X (compared to the earlier used 18) gives a 45 % reduction of the turbine unit costs, resulting in a new turbine unit cost of $22*10^6$ US$.

With both cost and profit known the optimum number of turbines can be determined for the two basins. A turbine unit is profitable if the annual result in produced kWh, is higher than the cost of the turbine as result of operation, depreciation and interest.

The operation costs are set at 1 % of the turbine unit cost. The depreciation period of the turbine unit is set at 30 years for the turbine and generator and 80 years for the powerhouse and other equipment. The turbine and generator costs for a project with this head can be estimated at 25 % of the total turbine unit costs (see lit 2) and are for this project 6,000,000 US$. To determine the depreciation costs a real interest (interest minus inflation) is taken of 4 %.

The cost of a turbine unit per year are:
- operation: 0.01 * 22,000,000 = 220,000 US$
- interest: 0.04 * 22,000,000 = 880,000 US$
- depreciation turbines and generator = 103,000 US$
- depreciation powerhouse = 28,000 US$

The sum of these four is 1,231,000 US$ (Rp 3.3 crores).

An extra turbine unit is profitable if the difference in production as result of this turbine is more than 1,231,000 US$. The extra production of the turbine unit should be 1,231,000 US$ / 0.028 US$/kWh per year = $44.3 * 10^6$ kWh. Since there are 353 tidal cycles a year, the "daily" production should be 126,000 kWh. For other (higher) values of a kWh lower minimum productions are allowed. If the price of a kWh is Rp 1.00 (US$ 0.037) a minimum production is allowed of 96,000 kWh a day. For a kWh price of Rp 1.25 the daily production should be 76,000 kWh.

In Table 5.5 the optimum number of turbines is given for the two different basins and for the three different prices of a kWh. The production of an extra turbine can be interpolated from the figures given in Table 5.4.

price kWh in Rp	basin 3		basin 4	
	number of turbines	annual production in GWh	number of turbines	annual production in GWh
0.75	125	8440	145	9670
1.00	150	9430	175	10,840
1.25	175	10,170	190	11,260

Table 5.5 Optimum number of turbines and annual production

5.7 Costs and profits of the two tidal power station alternatives

To make a decision between basin 3 or basin 4 all the costs of the tidal power station have to be known. The costs of the turbines and powerhouses are already known (US$ 22 mln per unit). For basin 3 the costs for the turbine units is US$ 2750 mln, for basin 4 the costs are US$ 3190 mln. The costs of the basin dams can easily be calculated if the cross section and materials are known, and this is done in Annex V. The costs of the basin dam for basin 3 are US$ 200 mln, and for basin 4 US$ 210 mln.

For the costs of the intake structures (the gates included) an preliminary estimation is made with the accepted price for the turbine unit as basis. The turbine unit has a width of 17 metres average. The cost of this turbine unit is US$ 22 mln, of which US$ 6 mln is spend on turbines and hydro electrical equipment. Remaining US$ 16 mln for the powerhouse caisson. The intake structure has a length of 30 metres, while the powerhouse has a width of 60 metres (see Annex VII). Together with the simplicity of the intake structure compared to the complicated powerhouse caisson a reduction factor of four can be accepted, resulting in a cost of US$ 4 mln per intake structure unit of 17 metres. Of this 17 metres only 15 metres can be used for opening. Two metres are reserved to serve as construction width.

In section 5.5 the width of the intake structure was determined if the intake structure had a depth of 5 metres below BM. If the turbine units and the intake units are not combined in one caisson the depth of the intake unit can be enlarged to 20 metres below BM. This will reduce the width of the intake structure considerable.

The width of the intake structure for basin 3 becomes 2300 metres and for basin 4 2500 metres. If the turbines are also used to take in water a reduction can be made. For basin 3 125 turbines are planned, these 125 turbines result in an opening of $125*\pi*(4 \text{ metres})^2 = 6300$ square metres. The reduced width of the intake structure becomes 2050 metres. For basin 4 the reduced width is 2200 metres. If a contraction coefficient of 1.1 is accepted the definitive width becomes for basin 3 and basin 4, respectively 2250 and 2450 metres. Every 15 metres of opening costing US$ 4 mln the costs of the intake structure becomes US$ 600 mln for basin 3, and US$ 650 mln for basin 4.

The estimated total costs of the two variants are :

BASIN 3:	turbine units	: US$ 2750 mln
	intake structure	: US$ 600 mln
	basin dam	: US$ 200 mln
	------------------------------ +	
	total	US$ 3550 mln

BASIN 4:	turbine units	: US$ 3190 mln
	intake structure	: US$ 640 mln
	basin dam	: US$ 210 mln
	------------------------------ +	
	total	US$ 4040 mln

With these costs and the profits of the tidal power station known a choice can be made about the two remaining alternatives, basin 3 and basin 4.
The basin dam, intake structure and 75 % of the turbine unit have a depreciation period of 80 years, 25 % of the turbine unit (the electro-mechanical part) has a depreciation period of 30 years. The operation costs of the turbine units and the intake structures are set at 1 % of the costs of these structures. The service costs of the basin dams are set at 0.5 % of the cost of the dam. The interest corrected for inflation is set at 4 %.

Annual costs and profit for the tidal power station of basin 3:

cost	: operation: 0.01 * (US$ 2750 mln + US$ 600 mln)	= US$ 33.5 mln
	service costs basin dam : 0.005 * US$ 200 mln	= US$ 1.0 mln
	interest : 0.04 * US$ 3550 mln	= US$ 142.0 mln
	depreciation turbines and generator :	= US$ 13.0 mln
	depreciation powerhouse, intake structure and basin dam :	= US$ 5.0 mln
	total	= US$ 194.5 mln
profit	: annual production in kWh * price per kWh $8440 * 10^6 * 0.028$ US$	= US$ 236.5 mln

result : US$ 236.5 mln - US$ 194.5 mln = US$ 42 mln = Rp 113 crores

Annual costs and profit for the tidal power station of basin 4:

cost	: operation: 0.01 * (US$ 3190 mln + US$ 640 mln)	= US$ 38.0 mln
	service costs basin dam : 0.005 * US$ 210 mln	= US$ 1.0 mln
	interest : 0.04 * US$ 4040 mln	= US$ 162.0 mln
	depreciation turbines and generator :	= US$ 15.0 mln
	depreciation powerhouse, intake structure and basin dam :	= US$ 5.5 mln
	total	= US$ 221.5 mln
profit	: annual production in kWh * price per kWh $9670 * 10^6 * 0.028$ US$	= US$ 271.0 mln

result : US$ 271.0 mln - US$ 221.5 mln = US$ 49.5 mln = Rp 134 crores

If a kWh price of Rp 1.00 can be used, and the same number of turbines are used the results would improve dramatic. For basin 3 the result would become US$ 121 mln (Rp 327 crores), the result of basin 4 would be US$ 140 mln (Rp 377 crores). If the optimum number of turbines for this scenario is used (150 instead of 125 turbines for basin 3, and 175 instead of turbines for basin 4) results would improve only slightly, US$ 124 mln (Rp 335 crores) for basin 3 and US$ 142 mln (Rp 382 crores) for basin 4.

In case of a kWh price of Rp 1.25, the results for basin 3 and 4 with an unchanged number of turbines would be respectively US$ 200 mln (Rp 540 crores) and US$ 230 mln (Rp 621 crores). Again the result with the optimum number of turbines for the accepted kWh price is only slightly higher, US$ 215 mln (Rp 580 crores) for basin 3 and US$ 243 mln (Rp 656 crores) for basin 4.

The conclusion of this evaluation is that basin 4 scores better for the three possible kWh prices than basin 3, and should be preferred above basin 3. Under the given preconditions of costs and profits (kWh price) the tidal power station is profitable as an extra construction connected to the closure dam. The feasibility of the project as a whole can only be calculated if the costs of the closure dam, shipping lock, spillway and compensation for loss of income is known. Benefits of the closure dam are availability of drinking and irrigation water, a short connection between the Saurastra peninsula and Bombay and the possibility of land reclamation. In chapter 6 a cost and profit survey will be given.

6 Total overview

In the previous chapters the design of both the closure and the tidal power station has been discussed separately. In this chapter the final design will be presented. Out of the study that has been carried out several conclusions can be drawn. These are presented in this chapter too, completed with some recommendations for further detailed study, experiments and measurements.

6.1 View on the total design

The total design of the Gulf of Khambat development works consists of several parts that have to be constructed in a certain sequence. Here the different parts and the construction sequence are discussed simultaneously.

Structures in the dam

Prior to the closure of the Gulf of Khambat the spillway is constructed in a temporary reclaimed polder. This in order to be able to establish a rather water-tight connection between the spillway-bottom and the foundation. The same counts for the ship-lock. After completion of this structures the bottom-protection is placed and the foundation-sill of the caissons, belonging to the tidal power station, is built by means of dump-barges. On this sill the caissons are positioned.

Closure

Now the final closure can be accomplished. First a sill is dumped till a level of 5 [m] below B.M.. On this sill a horizontal constriction is carried out. The rock is transported by means of a temporary railway on top of the closure dam. During closure the flow area of the intake structures of the tidal power station stay open in order to limit the flow-velocities and the required stone-mass. The alignment of the closure dam and the locations of the spillway and the ship-lock are shown in Figure 6.1.

Final dam

The final dam is constructed against the northern (basin-side) slope of the closure dam by means of hydraulic sand-fill. At the sea-side, above the berm a slope of 1 : 6 will be made. Together with a rough slope-protection the wave run-up will be limited. On the basin-side, above the water-level (during construction a water-level of 2 [m] above B.M. can be maintained) a slope of 1 : 3 is applied. Under water this is 1 : 15. The sea-side slope of the final dam is protected against wave-attack by rock. On the basin-side a revetment of concrete blocks has been chosen. The profile of the final dam is shown in Figure 6.2. On the dam crest a road and a double-tracked railway will be constructed.

In section 4.5 it was supposed that the lighterage port of Dahej is moved to the southern side of the final dam. So, here some provisions must be made (Figure 6.1).

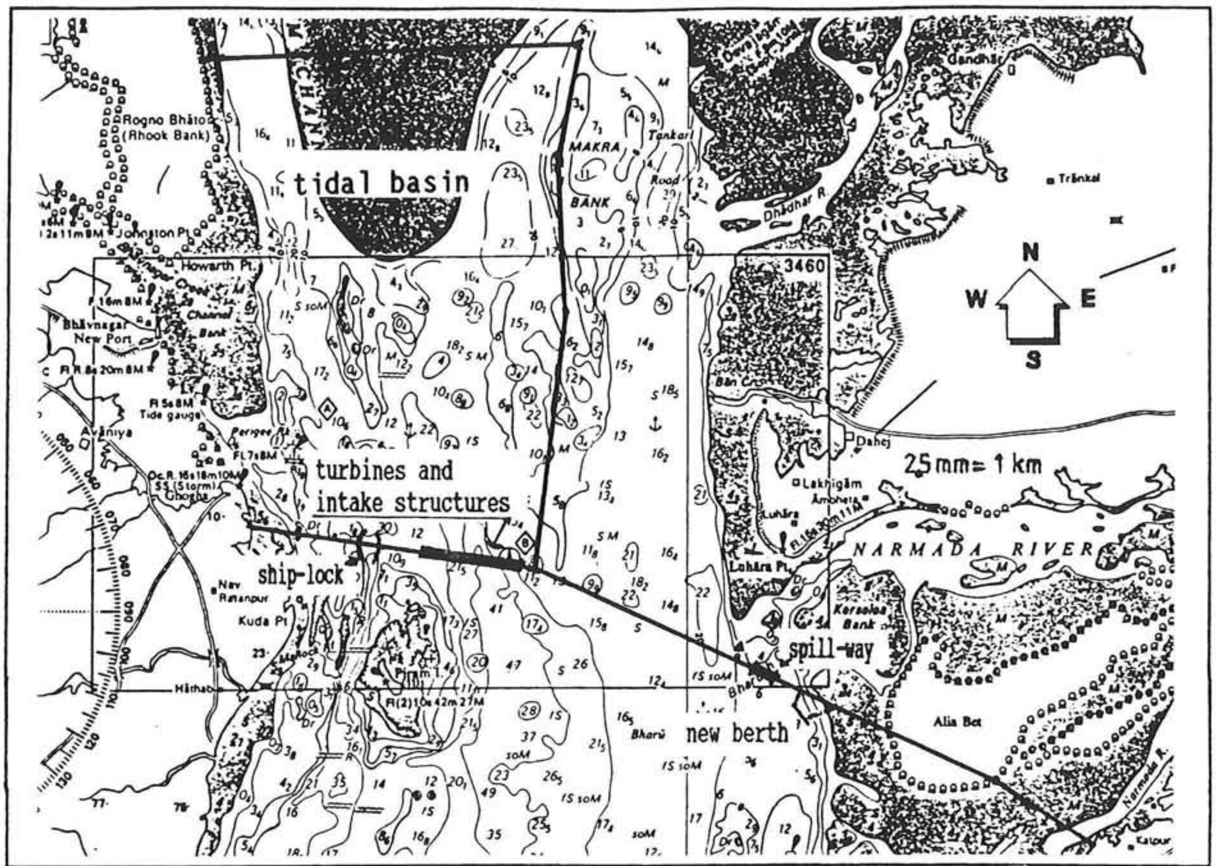


Figure 6.1 Alignment of the dams and locations of structures

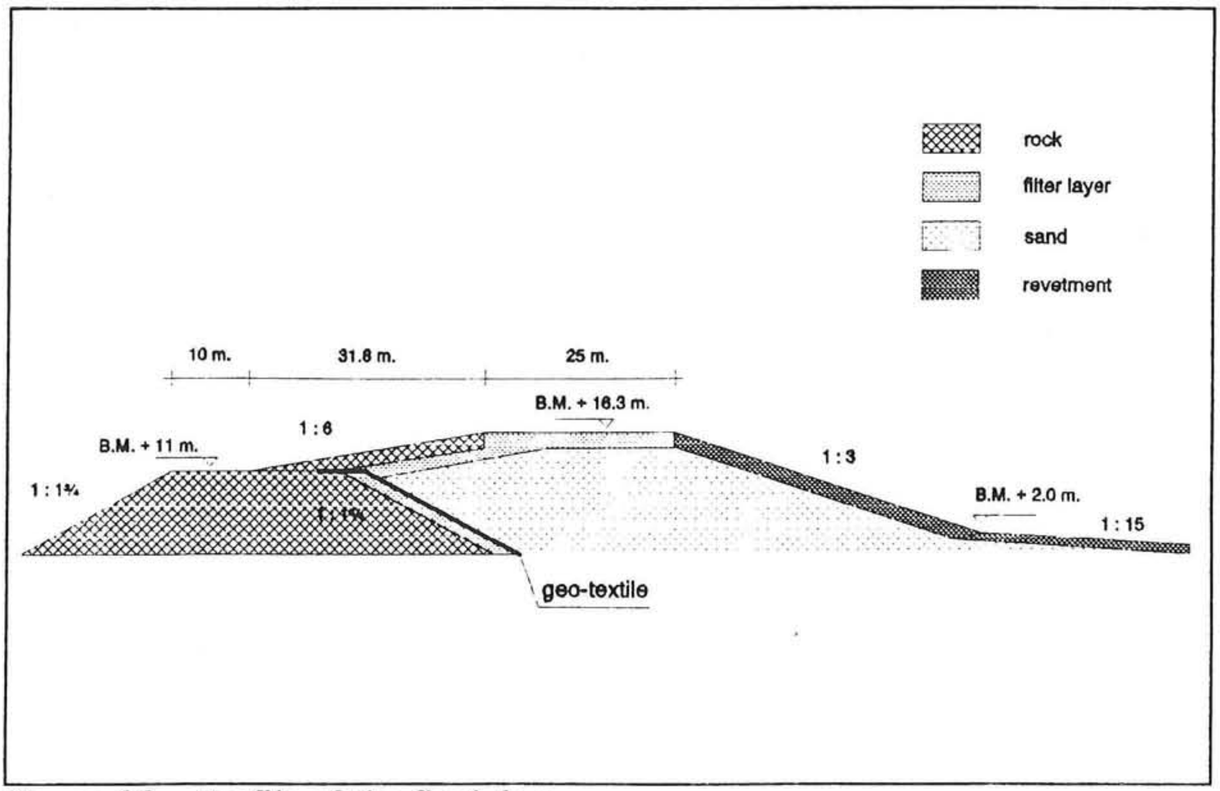


Figure 6.2 Profile of the final dam

As a second approach the waterlevel of the Highest Astronomical Tide (HAT) can be taken: 10.9 m. + B.M. (see Appendix A, section A5). This is higher than the former value of 10.3 m. The chance that a combination of HAT and high waves occurs is that low that for a temporary construction this is far too safe. With a margin for small waves the level of the top of the caisson becomes 11.5 m. This value has been chosen as the design level.

When the caisson is manoeuvred above the sill a keel-clearance of 1.0 m. is necessary above the highest point of the sill. This means that before sinking the upper level of the caisson will be 12.5 m. above B.M. while it is positioned when the waterlevel is approximately 1.5 m. above B.M.. When the draught of the caisson is estimated roughly as half the caisson-height H, H becomes (12.5 - 1.5)* 2 = 22 m.
This implies a maximum sill-height of 11.5 - 22 = 10.5 m. below B.M..

Based on the bottom-profile of the alignment first a closure-gap of 10 km. is chosen, represented by the sections 24 and 25, to be closed by means of sluice-caissons (Figure 4.29).

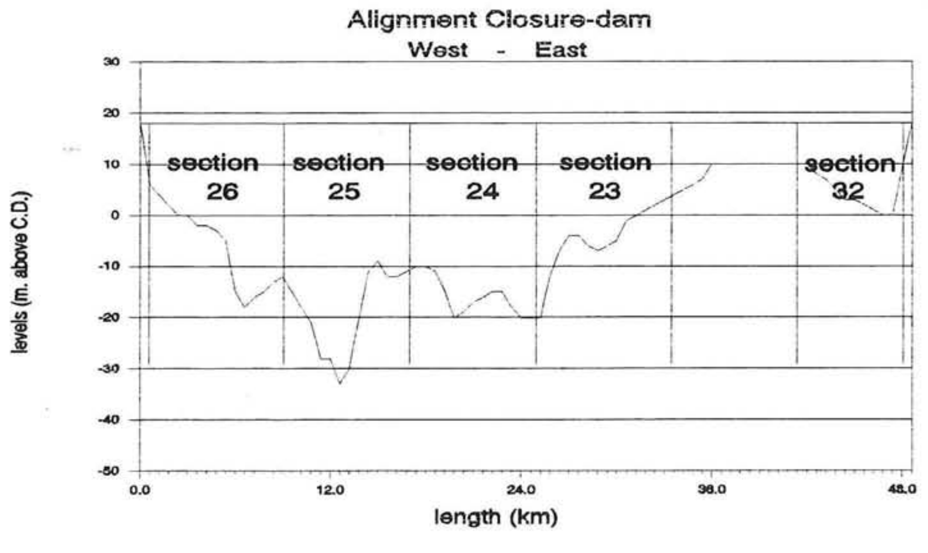


Figure 4.29 Longitudinal section of the alignment

The desired window of an hour between the velocities 2.0 and 0.5 m/s in the opposite direction must at least be present during a neap-tide in the situation of the positioning of the last caisson. The short period available for sinking a caisson asks for special equipment. Accurate positioning will be possible, provided that anchored pontoons equipped with tension winches or fixed anchor points are used.

The narrowing, caused by the walls of the caissons, is estimated to be 15 % . If 10 km. of caissons has to be placed it will be an enormous operation which asks for the sinking of caissons during spring-tide and at least mean-tide too. For this reason the sill-level is determined in a way that the desired window is present at a mean-tide in a totally narrowed situation. In the beginning the narrowing is less and the placement even can take place during spring-tide.

With a sill-level of 14 m. below B.M. and the application of a mean-tide and a narrowed cross-section (of 0.85 * 10 = 8.5 km.) the available window is 57 minutes (Figure 4.30). The value of 14 m. below B.M. is the internal sill of the caisson. When we estimate the height of construction of the caisson-floor, including the ribs necessary for the stiffness of the floor, at 4 m. the sill-level becomes 18 m. below B.M. and the caisson-height 29.5 m. which is rather large.

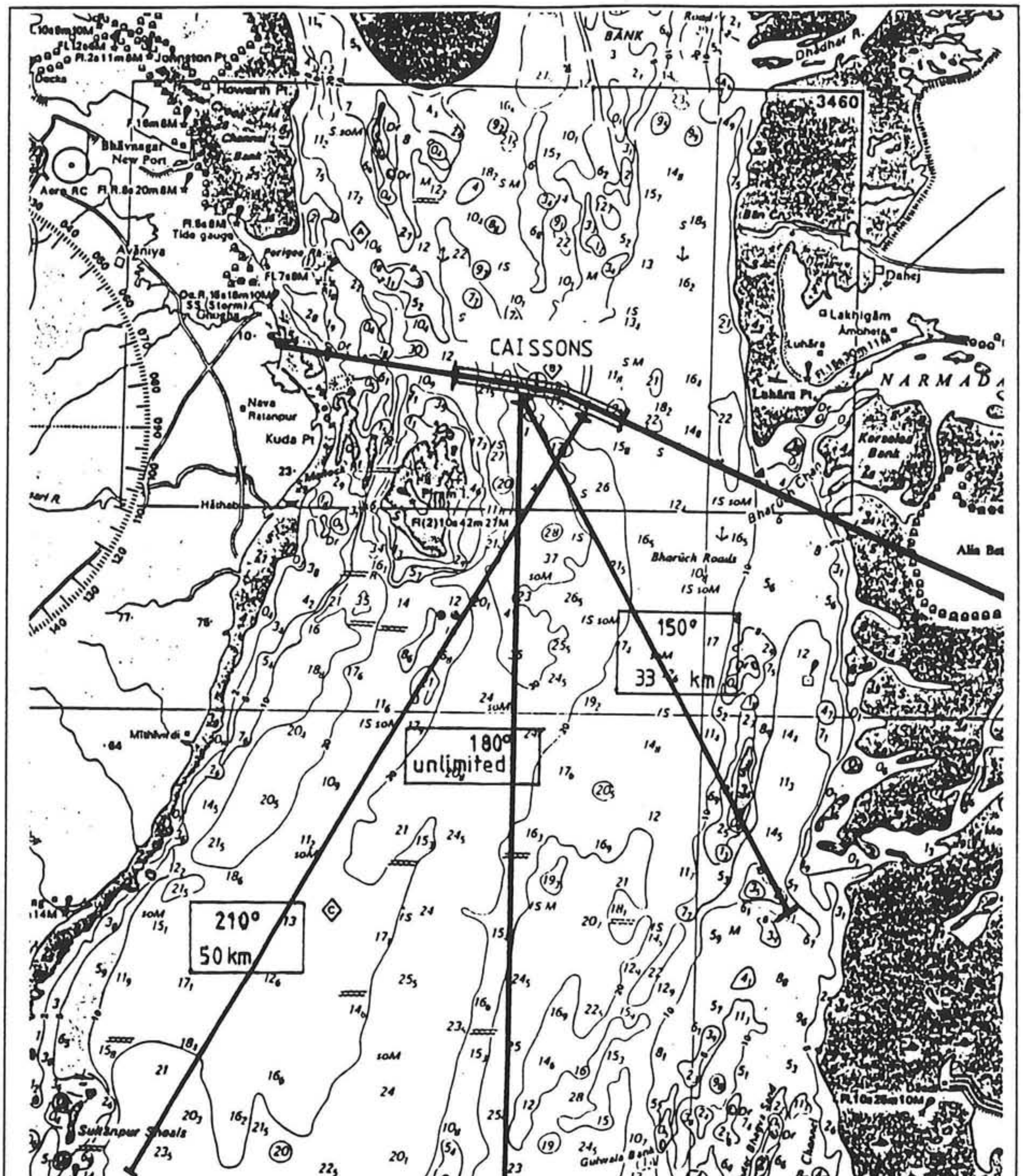


Figure 4.28 Possible fetches

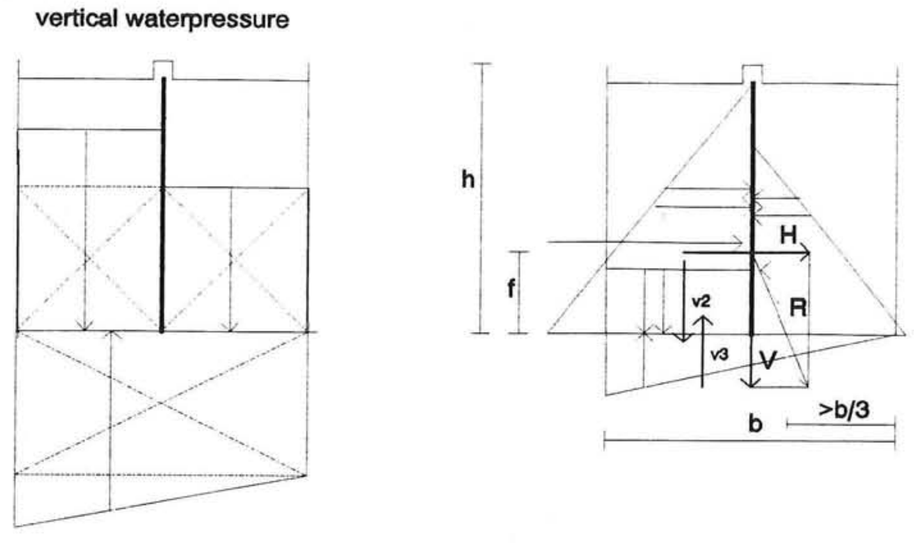


Figure 4.31 Hydrostatic loads on the caisson

In horizontal direction the resulting forces of the hydrostatic loads on both the basin- and the sea-side can be calculated (Eq 4.10). Out of this values the resulting force H and the distance f can be computed. The ratio between H and V must be such that the resulting force R stays within 1/3 of the width of the caisson. In this calculation the following values were used:

γ_{water} = 10.2 kN/m^3

$\gamma_{caisson}$ = 5.0 kN/m^3

(The value of $\gamma_{caisson}$ is based on experience in the Dutch Delta Works. When the draught of a floating caisson is approximately half the height a "$\gamma_{caisson}$" can be estimated as half γ_{water})

seaside

$$H_{sea} = \frac{1}{2}\gamma_{water}h_s^2 = \frac{1}{2}\cdot 10.2\cdot 29^2 = 4289\ kN/m \qquad \textbf{(4.10)}$$

basin-side

$$H_{basin} = \frac{1}{2}\cdot 10.2\cdot 22^2 = 2468\ kN/m$$

The resulting force **H** is the difference of the two values above: 1821 kN/m. The distance f can be calculated out of the balance of moments. The two vertical resultant forces V2 and V3 are equal in value (0.5*10.2*b*Δh) and play no important role in the balance of moments because they are close to each other. Because V2 and V3 depend on the (unknown) caisson-width they are neglected for this first calculation. Then f is calculated as follows (Eq. 4.11):

$$f = \frac{\frac{1}{3}\cdot h_s\cdot H_s - \frac{1}{3}\cdot h_b\cdot H_b}{H} = 12.8\ m \qquad \textbf{(4.11)}$$

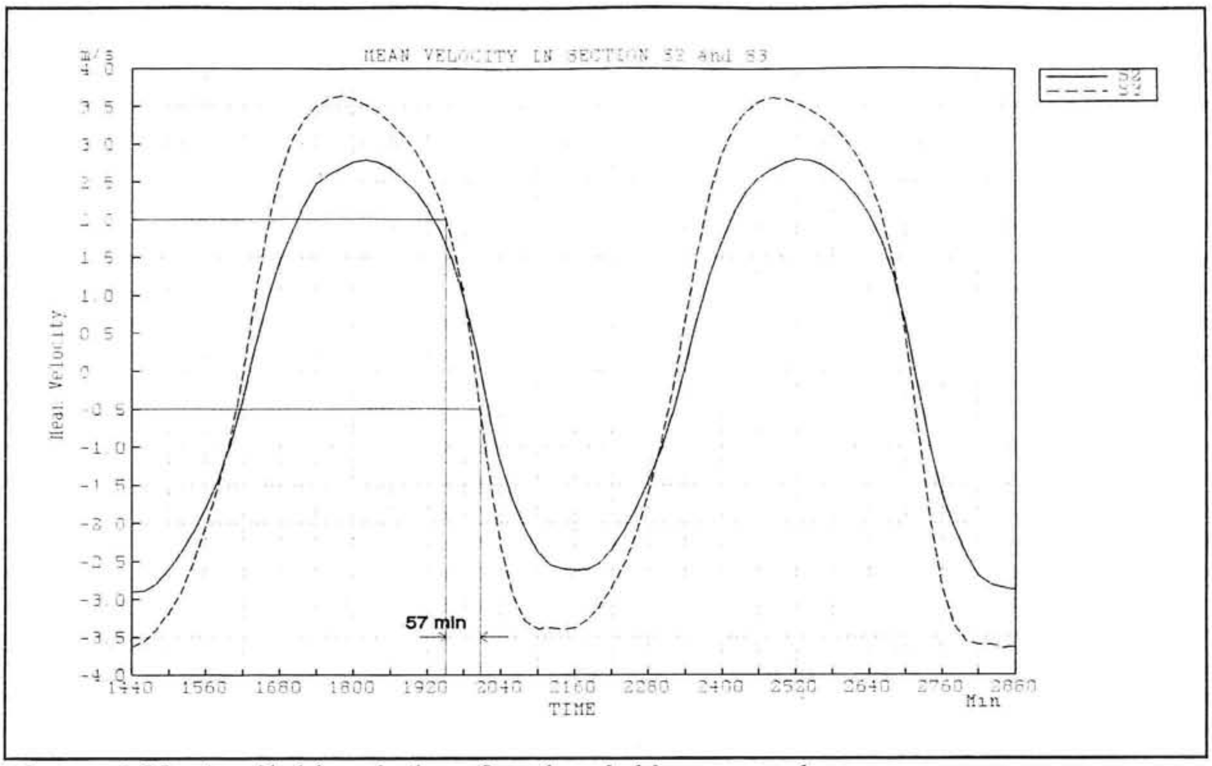


Figure 4.30 Available window for the sinking procedure

The application of even higher caissons may be technically possible but the problems you can expect with transport do increase and the construction will be more difficult. On the other hand the application of lower caissons can be achieved by taking a wider closure-gap. However, the biggest tidal channels are already included in the gap of 10 km. Using a closure-gap of 22 km. can restrict the sill-height till 12 m. below B.M. The caisson-height still is 23.5 m. but then 120 % more caissons are necessary.

Out of this results a final conclusion can not be drawn yet. A further optimisation, also based on total costs of the closure, should be carried out, but on forehand a maximum width of the gap to close with caissons of 10 km. seems appropriate.

rough determination of the other dimensions of the caisson

When the height of the caisson is known, the other dimensions, length and width, can be determined. The width of the caisson must be that large that the caisson can stand stable on its foundation under governing conditions. These occur when the gates in the caisson are closed and the head-difference over the caisson is extreme. Here the following values have been chosen:

* sealevel 11 m. above B.M. : $h_s = 29.0$ m.

* basinlevel 4 m. above B.M. : $h_b = 22.0$ m.
  (approximately M.S.L., which is rather safe because the porosity of the sill will decrease the head-difference)

The hydrostatic loads on the caisson are shown in Figure 4.31. On the left side the vertical components of the water-pressure are shown. The areas with the crosses balance each other, so only the remaining part of the vertical water-pressure with the resultant forces v2 and v3 returns in the drawing on the right.

Tidal power station

The tidal power station needs its own tidal basin. In the Gulf of Khambat a basin with an area of 510 [km^2] has been chosen. It is situated on the western side of the Gulf and encloses Bhavnagar port and a part of the Mal Bank. The basin dams are 49.5 [km] long and constructed by means of hydraulic sand-fill. On the slopes a protection of concrete blocks has been applied.

In order to produce electricity, turbines with a diameter of 8 [m] will be applied. Based on a price of electricity of 0.75 [Rp/kWh] the optimum number of turbines is found to be 145 (for the chosen basin). The length of the caissons in which the turbines are placed, is 2300 [m]. The number of turbines is rather sensitive to the price of a kWh. The intake-works have an optimised width of 2800 [m] combined with a sill-level of 20 [m] below B.M.. The construction and placement of separated caissons containing either turbines or intake works brings the total width of the tidal power structures on 5100.

Costs

For all parts of the Gulf of Khambat development scheme a rough cost estimation have been made. For the final dam, including the closure-operation, ship-lock and spillway the estimated costs amounts to 2,714 crores Rp (= 27.14 *10^9 Rp). The tidal power station is much more expensive. The costs of the basin dams and the necessary structures are estimated at 10,900 crores Rp. The construction costs of the total design become approximately 13,600 crores Rp which corresponds with 5.04 * 10^9 US$.

The annual cost and profit can now be calculated for the total project.

cost	: operation turbine unit and intake structure:	
	0.01 * (US$ 3190 mln + US$ 640 mln)	= US$ 38.0 mln
	service costs basin dam and closure dam:	
	0.005 * (US$ 210 mln + US$ 1000 mln)	= US$ 6.1 mln
	interest : 0.04 * US$ 5040 mln	= US$ 201.6 mln
	depreciation turbines and generator :	= US$ 15.0 mln
	depreciation powerhouse, intake structure, basin dam and closure dam:	= US$ 7.2 mln
		---------------
	total	= US$ 267.9 mln
profit	: annual production in kWh * price per kWh	
	9670 * 10^6 * 0.028 US$	= US$ 271.0 mln

result : US$ 271.0 mln - US$ 267.9 mln = US$ 3.1 mln = Rp 8.4 crores

A remark should be made concerning the result of the total project. The profit of the project is wider than the production of energy alone. Additional benefits are availability of drinking and irrigation water, a short connection between the Saurastra peninsula and Bombay and the possibility of land reclamation. Costs not taken into account concern compensation for loss of income and damage to the environment (which is almost impossible to give a value to).

6.2 Conclusions

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

- The closure of the Gulf of Khambat is considered technically feasible. The proposed closure-method limits the stone-mass to a grading of 10 - 15 tons. If rock this heavy can not be found the elements can be made of concrete.

- The closure operation asks for enormous amounts of rock. This is transported to the dumping-site by rail. The time-scheme of the closure and the logistics of the operation will be very important. This holds for the sinking operation of the tidal power caissons too.

- A subject of attention is the underseepage that occurs under the caissons of the tidal power station. No solution is found yet.

- The accessibility of the ports of Bhavnagar becomes little less. The Dahej lighterage port will be closed. As a replacement a new jetty connected to the Khambat dam will be constructed. This will improve the cargo-handling facilities in this area.

- The profitability of the tidal power station is very sensitive to the price of a kWh. No reliable estimate of the feasibility of the station can be made in this stage although a preliminary calculation expects the project to be just feasible.

6.3 Recommendations

Based on the results of this study the realization of the Gulf of Khambat is considered to be technical feasible. However, in order to make a final design many aspects must be verified of studied more in detail. Also the environmental impact and the consequences for the local inhabitants must be taken into account. The following recommendations are made:

- The impact of the closure and the change of a fresh into a salt water regime in the fresh water basin must be studied. The absence of the tide and the fresh water will cause a change in flora and fauna.

- The effects for the local economy and inhabitants must be investigated. The profits gained in bigger crops should be weighed against the losses in e.g. fisheries.

- The necessity of the accessibility of both Bhavnagar and Dahej port must be studied. In an over-all transport study also the profits of the new railway and road connection must be taken into account.

- The impact of the closure dam on the coastal morphology must be studied just as the sediment transport of the rivers.

- The process and speed of the desalinization must be estimated. Then it will become clear how long it will take before the water can be used for irrigation purposes.

In order to make the technical design more reliable the following measurements and studies will have to be carried out.

- The tidal model was based on data gathered from tide tables. Improvement of the model will be possible when tide measurements are carried out on several locations on the sea-ward boundary of the model. Measurements on locations in the Gulf of Khambat itself will allow more accurate calibration of the model.

- An extensive survey of the bottom-levels in the Gulf of Khambat opens the way to application of a, more detailed, two-dimensional tidal model.

- Based on the survey of the bottom-levels a more accurate estimation of the required quantities of construction materials can be made.

- In order to design the spillway and estimate the yearly available quantity of irrigation water the discharges of the Narmada, and the other rivers that debouch in the fresh water basin, must be measured. Especially the hydrographs of the Narmada will have changed due to the construction of the Narmada dams.

- A point of interest are the waves that can be expected on both sides of the dam. The HISWA-computer model contradicts with a simple fetch-calculation. A more advanced computer model (if available) can be used. Also the application of scale model study can be considered.

- Scale model study is certainly recommended when the closure operation is considered. A scale model of the estuary must be built. Then both the tidal flow and the closure elements must be modelled accurately applying the similitude criteria.

- Geological and geo-technical investigations must be carried out along the dam alignment in order to estimate the settlement of the dam. Also on the possible quarry sites this will be necessary. Then the maximum stone dimensions and the production of a certain quarry can be determined.

- A detailed construction scheme of the closure operation, including the sinking of the caissons, must be made. Especially the logistic process and the workability of above all the monsoon-periods must be worked out.

- The costs of the powerhouse and intake caissons have been estimated on basis of a simple equation. To be able to conduct a more reliable economic evaluation the exact design of the powerhouse and intake caisson must be determined. With the precise design a price based on the structure itself can be made.

- The DUFLOW model can be extended with a new option for the Duflow-structure pump. If the pump can be regulated by difference in water level over the structure the basis characteristics of each turbine can be simulated more reliable. If DUFLOW can be extended with a turbine as a new structure of which diameter, maximum discharge and efficiency can be given, the simulation of a tidal power station would give even more accurate figures.

References - Literature

[1] The Closure of Tidal Basins;
Closing of Estuaries, Tidal Inlets and Dike Breaches.
J.C. Huis in 't Veld e.a., Delft 1987

[2] Admiralty Tide Tables, vol 2: The atlantic and indian oceans,
D.W. Haslam, Taunton, Somerset, January 1981.

[3] Een halve eeuw zuiderzeewerken 1920 - 1970.
J. Th. Thijsse, H.D. Tjeenk Willink bv, Groningen 1972.

[4] Duflow, A Micro-computer Package for the Simulation of One-Dimensional Unsteady Flow in Channel Systems, (user's-manual),
W. Spaans e.a., Delft, June 1989

[5] Khambat Gulf Development Reconnaissance Report,
Haskoning & Ministry of transport and public works & Development consultants, Nijmegen, May 1989

[6] Bhavnagar ports siltation study; report on field investigation,
Nedeco report Ind 35/r 367, The Hague, June 1967

[7] Bhavnagar ports;
report on the investigation of the entrance channels,
Delft Hydraulics report M. 361, Delft, March 1952

[8] Tidal computations in rivers and coastal waters,
J.J. Dronkers, NHPC, Amsterdam 1964.

[9] Oceanographical Engineering,
R. L. Wiegel, Berkeley 1964

[10] Ethylene unloading terminal at Hazira, India,
Delft Hydraulics, report H 942, Delft, February 1989.

[11] Het Deltaplan, voor veiligheid en milieu,
publication of the department of 'Verkeer en Waterstaat', 1990

[12] De Lauwerszeewerken als afsluiting in het waddengebied
C. van der Burgt, Vereniging voor Bitumineuze werken,
Den Haag 1970

[13] Drie eilanden één,
H.J. Stuvel, Scheltema & Holkema N.V., Amsterdam 1963

[14] Kwaliteit en kwaliteitscontrole van breuksteen voor de waterbouw,
MAW-R-81054 / WKE-R-82002,
G.J. Laan, RWS-DWW, Delft 1982

[15] Shore Protection Manual (vol I and II),
Coastal Engineering Research Center (CERC), Washington 1984

[16] Afsluitdammen, Regels voor het ontwerp,
J.L.M. Konter e.a., Bouwdienst RWS-DWW, Utrecht 1992

[17] Kan het debiet door een blokkendam worden bepaald als functie van de vorm en afmeting van de dam, de blokafmeting en de waterstanden ter weerszijden van de dam, Nota W-69.093, W.A. Venis, RWS-Deltadienst-WA, Den Haag 1970

[18] Manual on the use of rock in coastal and shoreline engineering, M.E. Bramley e.a, CIRIA special publication 83 / CUR report 154, London/-Gouda 1991

[19] Leidraad voor het ontwerpen van rivierdijken - deel 2 benedenrivierengebied, TAW

[20] Handboek voor dimensionering van gezette taludbekledingen, CUR / TAW rapport 155, Gouda 1992

[21] Leidraad keuzemethoden Dijk- en Oeverbekledingen - deel 1, TAW

[22] Tidal Power Generation,
E.M. Wilson and M. Balls

[23] Aspects of a tidal power station in the Wyre Estuary,
S. Delfgaauw - Delft 1991

[24] Water power and Dam construction (International), Hydropower cost estimates, J.L. Gordon - November 1983

[25] HISWA, User manuel for the program,
N. Booy and L.H. Holthuijsen, Delft University of Technology, Departement of Civil Engineering - March 1992

[26] Meteorological and oceonographical conditions in the Gulf of Cambay and adjecent region,
Royal Netherlands Meteorological Institute - January 1968

[27] Cement, Waterkrachtprojecten in de Nederrijn en Maas - 1987 nr. 11 Waterkrachtcentrale Maurik, een van de drie centrales in aanbouw - 1988 nr. 7/8

[28] Global Ocean Tides, part II: The semidiurnal principal Lunar Tide (M2), Atlas of Tidal Charts and Maps. NSWC TR 79-414
E.W. Schwiderski, December 1979

[a] Waterbouwkunde, lecture notes F3
A. Glerum, Delft 1984

[b] Flexibele waterbouwkundige constructies, lecture notes F4
J. Bouwmeester, Delft 1989

[c] Coastal engineering, lecture notes F5
W.W. Massie a.o., Delft 1986

[d] Lange golven, lecture notes B73
C. Verspuy, Delft 1987

[e] Energiewaterbouwkunde, lecture notes F 20
Ir. J. van Duivendijk, Delft 1990

[f] Afsluiting van waterlopen, lecture notes F 11C
Ir F.C. van Roode, Delft, April 1988

References - Persons

Committee of supervision

chairman

Prof. drs. ir. J.K. Vrijling	-	TU Delft / Bouwdienst RWS

members

Ir. J. van Duivendijk	-	TU Delft / Haskoning
Ir. F.C. van Roode	-	TU Delft / Boskalis
Ir. C. Verspuy	-	TU Delft
Ir. K.G. Bezuyen	-	TU Delft

Others

Ir. G.J. Akkerman	-	Delft Hydraulics
Prof ir. K. d'Angremond	-	TU Delft
Dr. ir. N. Booy	-	TU Delft
Dhr. van de Bunt	-	Delft Hydraulics
Ir. G.J. Florian	-	TU Delft
Dr. ir. L.H. Holthuijsen	-	TU Delft
Ir. R.E. Jorissen	-	Bouwdienst RWS
Ir. F. Remery	-	Bouwdienst RWS
Ir. G.J. Schiereck	-	TU Delft
Dhr. F.T. Spanheim	-	ABB Lummus Crest B.V.
Ir. R.M. Sutmuller	-	Haskoning
Prof. dr. ir. M. de Vries	-	TU Delft

Annex I Borings in the Gulf of Khambat

Bottom information was taken from two different studies. The first study gave only information about the upper layers of the soil. The second study consisted of several borings near the proposed dam site (proposed in the reconnaissance study). Two boring have been made to a depth of 30 metres below mean sea level. The borelogs are shown in Figures I.1 and I.2.
The location of the borings is shown in Figure I.3.

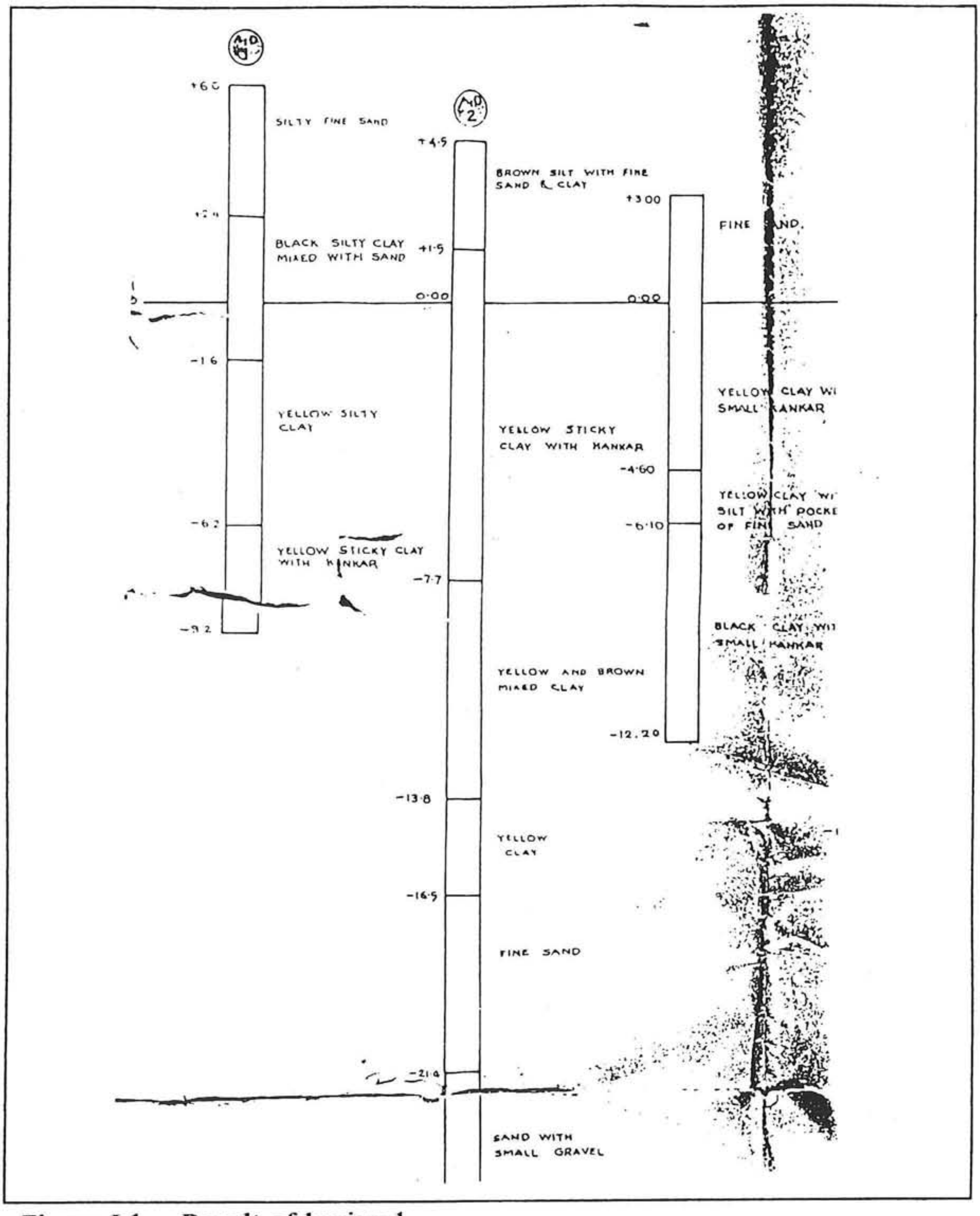


Figure I.1 Result of boring 1

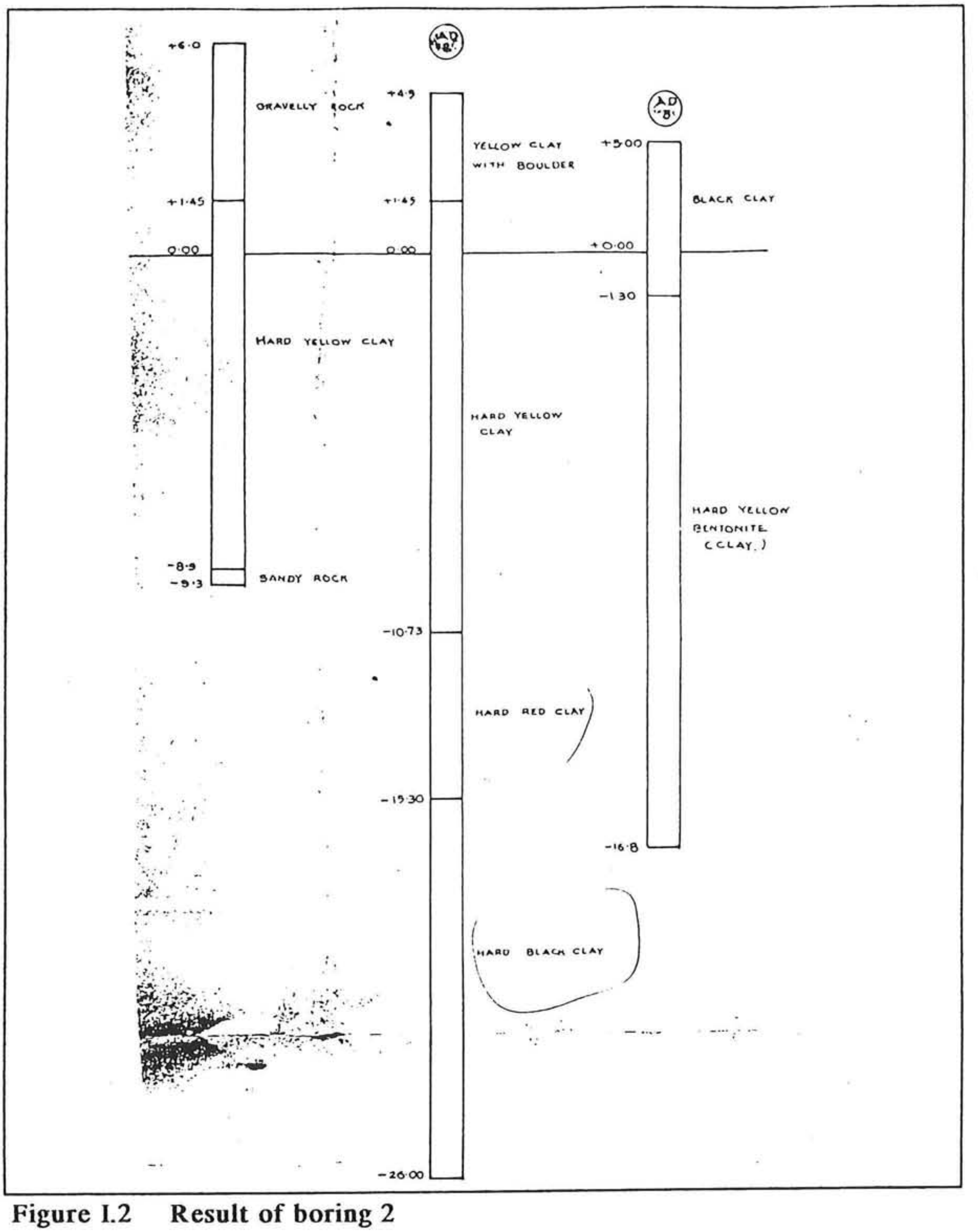


Figure I.2 Result of boring 2

These two borings were used as basis to describe the sub soil for the dam. Three bore-logs were drawn for different sections of the dam. One for the (deep) section in the middle of the Gulf of Khambat, one for the shallower parts of the dam location and one for the sections above Bench Mark (during ebb these sections run dry). The precise location and extend of these sections can be seen in Figure I.3. The bore-logs for the three sections are shown in figures I.4, I.5 and I.6. The reason for drawing the new borings is in the limited validity of the borings (only near the coasts of the Gulf).

The presence of sand with small gravel makes it possible to use this sand for the final profile of the closure dam, and the basin dams. On top of these dams a layer of clay is planed on which concrete blocks are placed. The thick layers of clay near the coasts should provide enough clay of sufficient quality to do so. The characteristics of the sand and clay is given in the figures with the borings.

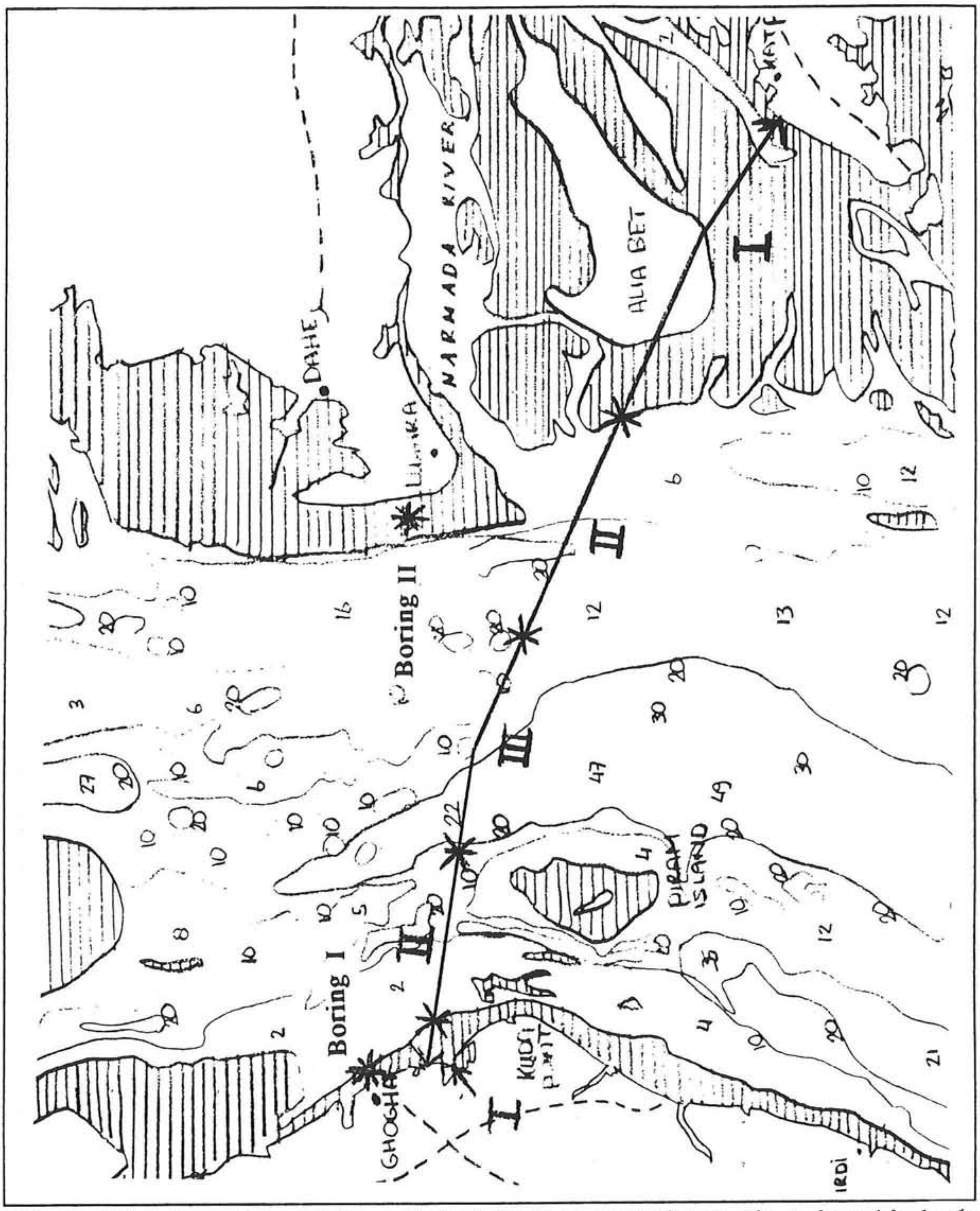


Figure I.3 Location of the original borings and the sections in whitch the new borings are valid

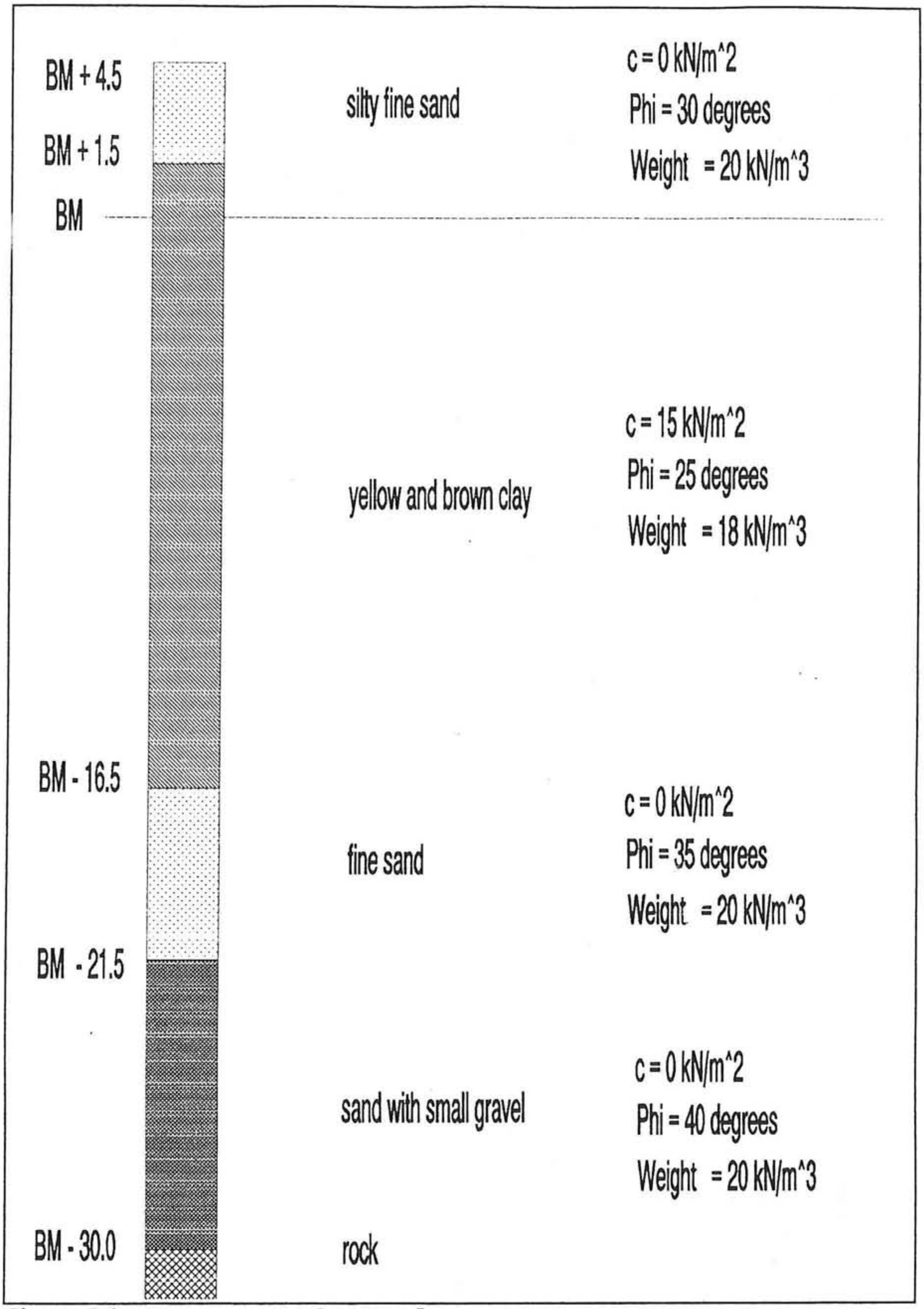


Figure I.4 Borelog valid for area I

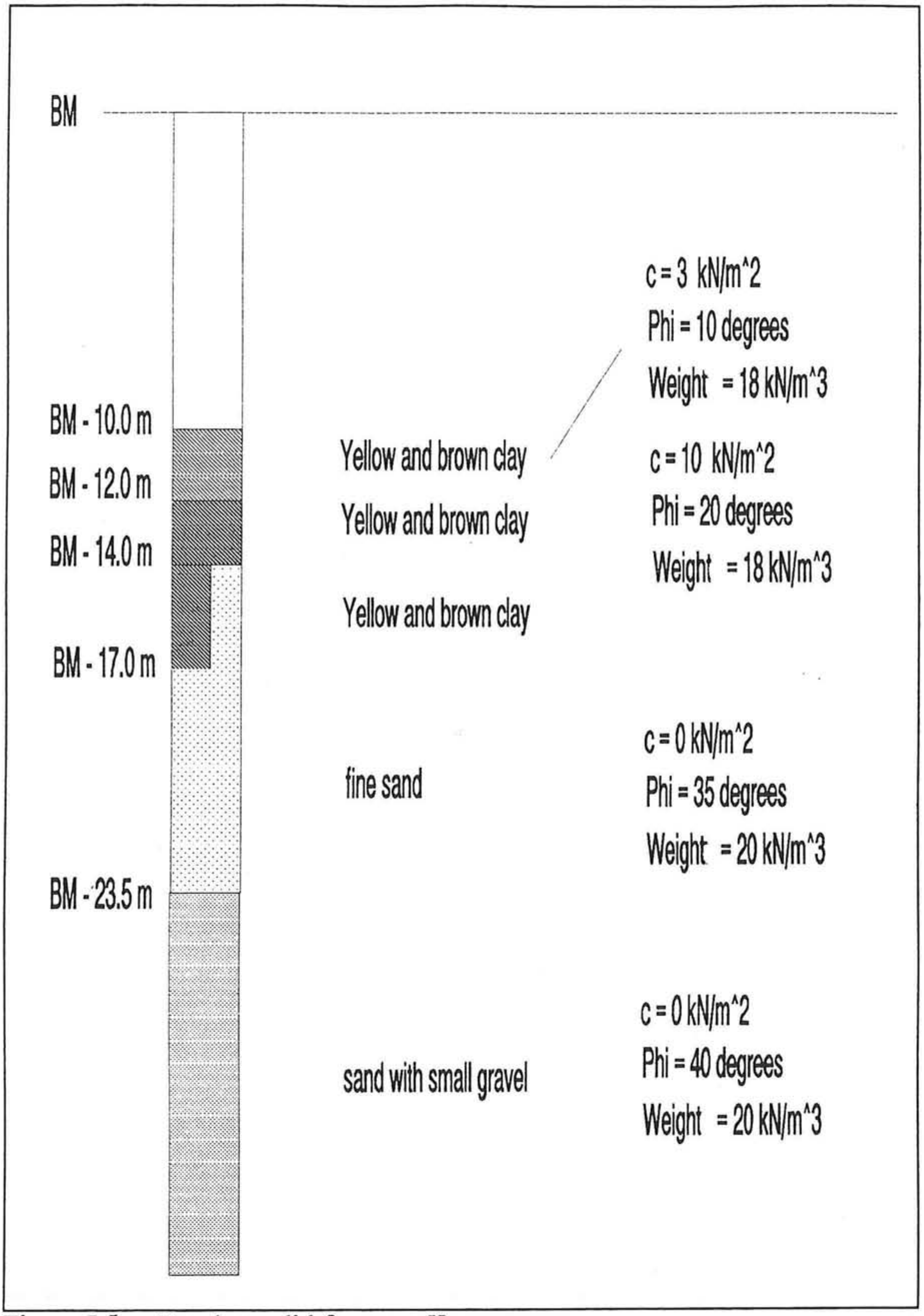


Figure I.5 Borelog valid for area II

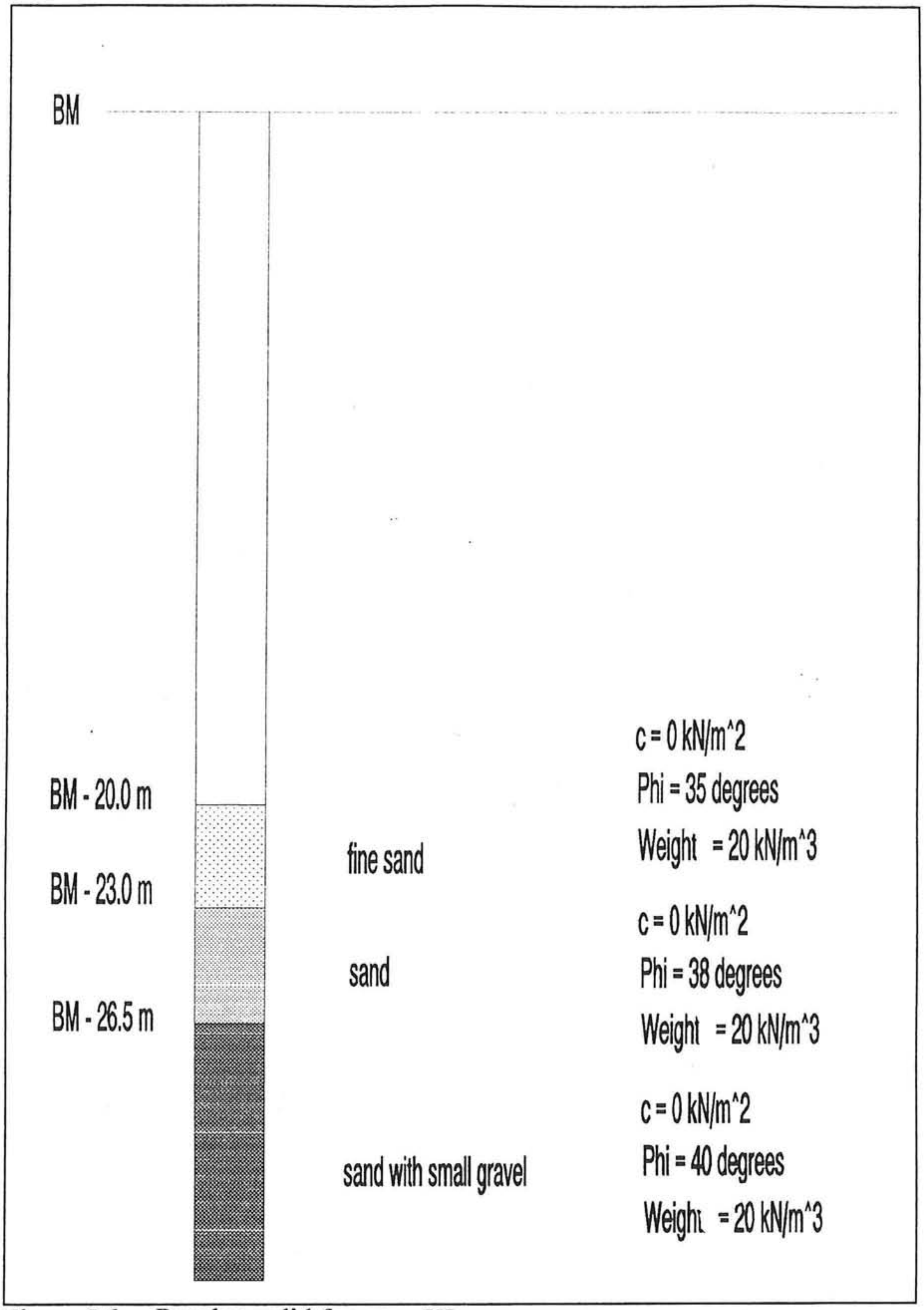


Figure I.6 Borelog valid for area III

Annex II Calculation Wave Prediction for Gulf of Khambat

In chapter two the wind speed prediction for the Khambat region is given. The figures were:

50 % chance of exceedance: 17 m/s.
10 % chance of exceedance: 24 m/s.
2 % chance of exceedance: 31 m/s.
1 % chance of exceedance: 35 m/s.

Extreme wave heights to be exceeded on the average with a chance of 10 %, 2 %, or 1 % only are expected to be always associated with the extreme wind speeds of tropical cyclones. It must be stated beforehand that the estimation of probabilities of extreme wave heights associated with tropical cyclones is still a more speculative matter than the estimation of the probabilities of extreme wind speeds, because reliable observations of high waves in tropical cyclones are very difficult thus rare.

To translate the given extreme wind speeds in extreme wave heights, two calculations have been made. For the first calculation equations given in the Shore Protection Manual (pp 3-77, 3-88)(lit [15]) are used. These equations can be applied for wave prediction in deep water as a result of cyclones. The second calculation is made with the computer program HISWA (HIndcast Shallow water WAves).

The calculation with the equations of the Shore Protection Manual

The equations in this annex are numbered, but not in the same way as in the Shore Protection Manual (SPM). A list of the equations is given at the end of this annex, with the corresponding SPM equations numbers.

The equations for the wave height and the wave period are:

$$H_o = 5.03\ e^{\frac{R\Delta p}{4700}} \left[1 + \frac{0.29\ \alpha\ V_F}{\sqrt{U_R}} \right] \qquad \textbf{(II.1)}$$

$$T_s = 8.6\ e^{\frac{R\Delta p}{9400}} \left[1 + \frac{0.145\ \alpha\ V_F}{\sqrt{U_R}} \right] \qquad \textbf{(II.2)}$$

In the equations for the wave height prediction (II.1) and the wave period prediction (II.2) variables are used such as R, the size of the eye of the cyclone, Δp, the difference in air pressure between the central pressure of the cyclone and the normal pressure (p_n) of 760 millimetres of mercury, V_F , the forward speed of the cyclone, and U_R , a combination of the maximum wind speed and the forward speed of the cyclone ($0.865*U_{max} +0.5*V_F$). These variables are not given with the wind speed. To be able to use the equations, the variables were determined from the wind speed prediction equation in the SPM (II.3). In this equation the number of unknowns is more then explicitly can be determined. To get results average (realistic) values were taken for R, the size of the eye of the cyclone, and V_F , the speed of the cyclone.

These assumed values are 80 kilometres for the eye of the cyclone and 15 meters per second for the speed of the cyclone.

$$U_{\max} = 0.447\ [14.5\ (p_n - p_o)^{\frac{1}{2}} - R\ (0.31f)] \qquad \textbf{(II.3)}$$

F is a Coriolis parameter and stands for the variation of the wind speed as result of the change in latitude. F is 2 ω sin(ϕ), in this case ϕ (the latitude) is 20° ,thus f is 0.179. For U_{max} = 35 m/s the equation becomes:

$$35 = 0.447\ [14.5\ (p_n - p_o)^{\frac{1}{2}} - 80\ (0.31*0.179)]$$

$$(p_n - p_o)^{\frac{1}{2}} = 5.74$$

$$p_o = 727\ \textit{millimetres of mercury}$$

For U_{max} is 31 m/s and 24 m/s p_o becomes 734 millimetres respectively 744 millimetres of mercury.

With these data the equation (II.1) can be solved. For the maximum wind speed of 35 m/s the wave height becomes:

$$H_o = 5.03\ e^{\frac{R\Delta p}{4700}}\ [\ 1 + \frac{0.29\ \alpha\ V_F}{\sqrt{U_R}}\ ]$$

$$H_o = 5.03\ e^{\frac{80(760-727)}{4700}}\ [\ 1 + \frac{0.29 \cdot 1 \cdot 15}{\sqrt{(0.865 \cdot U_{\max} + 0.5 \cdot 15)}}\ ]$$

$$H_o = 15.0\ m.$$

The value of the wave height for the wind speed of 31 m/s and 24 m/s is 13.6 m respectively 12 m.

Since the waves in the Gulf of Khambat are not generated in nor travelling through deep water, the calculated waves are too high.

Equations for wave prediction as result of tropical cyclones in shallow water have not been developed. To get a more realistic value for the wave height, and the wave period in the Gulf, the wave heights, and the wave periods calculated from the deep water equations are converted to the shallow water conditions (depth is 25 meters average).

The conversion takes place as follows:
1 With the calculated wave height for the deep water situation and the wind speed an equivalent fetch can be determined (the fetch combined with the wind speed gives the calculated wave heights).
2 With this equivalent fetch and with the given wind speed a wave prediction can be made for shallow water. The wave prediction consists of a prediction for the wave height and for the wave period.

The equivalent fetch can be found in figure II.1.

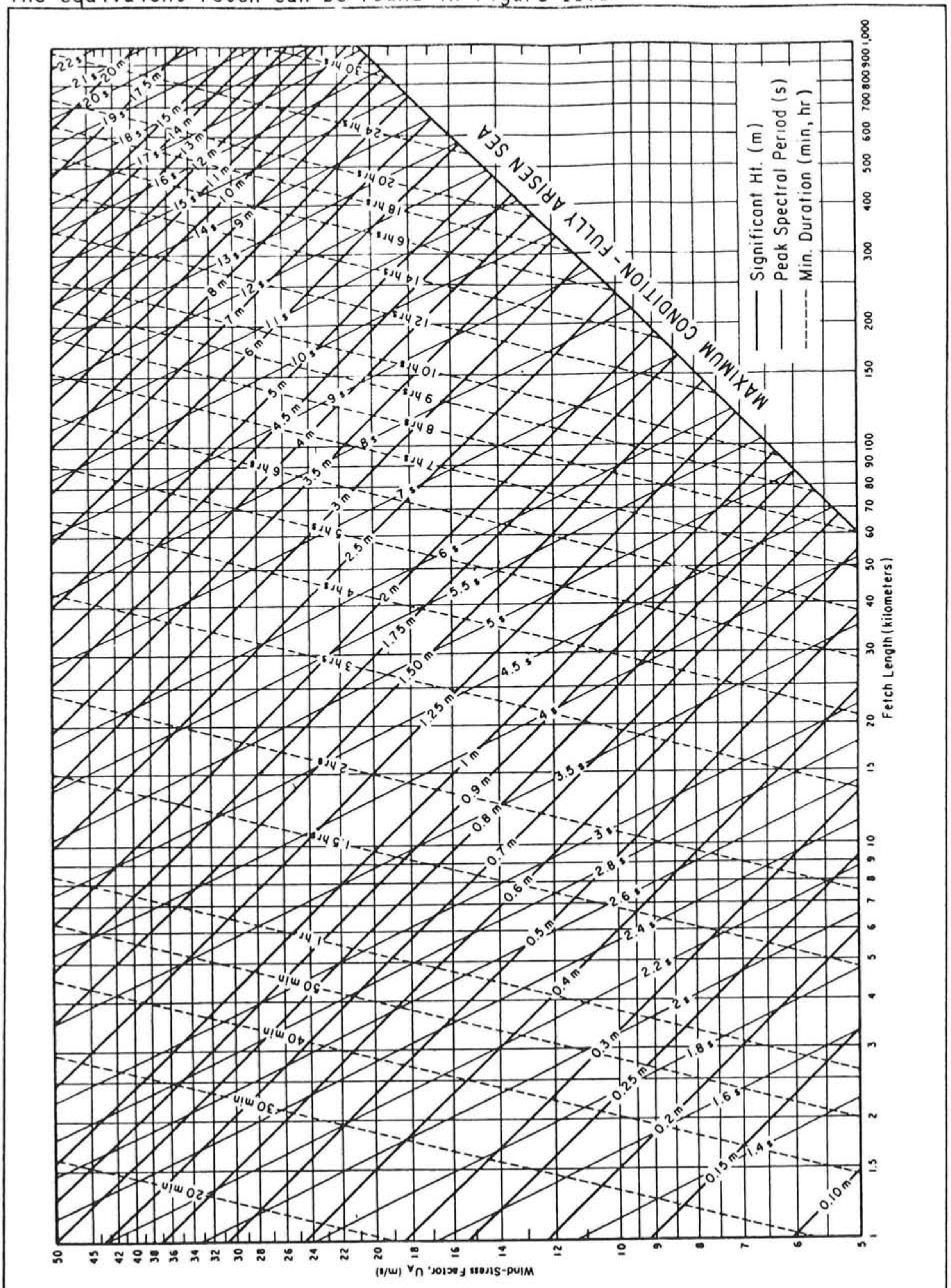


Figure II.1 Nomogram of deep-water significant wave prediction curves as functions of windspeed, fetch length and wind duration

The equivalent fetch for the given situation is:

change of exceedance	windspeed	wave height in deep water	equivalent fetch
10 %	24 m/s	12.0 m	700 km
2 %	31 m/s	13.6 m	700 km
1 %	35 m/s	15.0 m	700 km

The wave height for the shallow water conditions can now be calculated with the following Bretschneider wave prediction equations (II.4) and (II.5), in these equations U stands for the wind speed:

$$\frac{gH}{U^2} = 0.283 \tanh\left[0.530 \left(\frac{gd}{U^2}\right)^{3/4}\right] \tanh\left[\frac{0.00565 \left(\frac{gF}{U^2}\right)^{1/2}}{\tanh\left[0.530 \left(\frac{gd}{U^2}\right)^{3/4}\right]}\right] \tag{II.4}$$

$$\frac{gT}{U} = 7.54 \tanh\left[0.833 \left(\frac{gd}{U^2}\right)^{3/8}\right] \tanh\left[\frac{0.0379 \left(\frac{gF}{U^2}\right)^{1/3}}{\tanh\left[0.833 \left(\frac{gd}{U^2}\right)^{3/8}\right]}\right] \tag{II.5}$$

Filling in g=10 m/s², U=35 m/s, d=25 m. and F=700,000 m. (II.4) becomes:

$$\frac{10H}{35^2} = 0.283 \tanh\left[0.530 \left(\frac{250}{35^2}\right)^{3/4}\right] \tanh\left[\frac{0.00565 \left(\frac{7,000,000}{35^2}\right)^{1}}{\tanh\left[0.530 \left(\frac{250}{35^2}\right)^{3/4}\right.}\right.$$

$$0.00816H = 0.283 \tanh [0.161] \tanh \left[\frac{0.428}{\tanh [0.161]}\right]$$

$$H = 5.48\ m.$$

And (II.5):

$$\frac{10T}{35} = 7.54 \tanh\left[0.833 \left(\frac{250}{35^2}\right)^{3/8}\right] \tanh\left[\frac{0.0379 \left(\frac{7,000,000}{35^2}\right)^{1/3}}{\tanh\left[0.833 \left(\frac{250}{35^2}\right)^{3/8}\right]}\right]$$

$$0.286T = 7.54 \tanh [0.459] \tanh \left[\frac{0.678}{\tanh [0.459]}\right]$$

$$T = 10.4\ s.$$

The same can be done for the other windspeeds of 31 m/s and 24 m/s. The results are:

31 m/s --> H = 5.12 m, T = 10.0 s
24 m/s --> H = 4.46 m, T = 9.5 s

These values for the maximum wave height are given with much reserve. Especially the wave heights near the dam site can vary because of limiting factors as:

- limited fetches,
- shallowness,
- turbulent currents.

Individual Waves

So far all considerations and conclusions refer to the significant wave heights. It is well-known that some individual waves are much higher than the significant wave height and that the probable highest individual waves increases with the duration considered. In order to estimate the probable highest individual waves associated with the extreme significant wave heights as given earlier in this chapter by applying the raleigh distribution the following should be done.

1 Determine the duration of the maximum wind speed. The duration of this wind speed is the size of eye of the cyclone divided by the speed of the travelling cyclone. In the former calculations the size of the eye of the cyclone is assumed as 80 kilometres, and the travelling speed of the cyclone as 15 metres per second. The duration of the storm then becomes 5400 seconds.
2 Divide this duration time by the most probable wave period to get the total number of waves N.
3 The multiplication factor for the significant wave height to get the maximum wave height can be determined from equation II.6 (n = 1 for the highest wave n = 2 for the second highest wave etc.).

$$H_n = 0.707\ H_{sig} \sqrt{(\log \frac{N}{n})} \qquad \textbf{(II.6)}$$

In case of the windspeed of 35 m/s the significant period is 10.5 seconds. The matching value of N is 5400/10.5 = 514. For windspeeds of 31 and 24 m/s the value of N is 540 respectively 568.

Filling equation II.6 in with these values gives :

For 35 m/s : $$H_n = 0.707 \cdot 5.48 \sqrt{(\log \frac{514}{1})} = 9.68\ m.$$

For 31 m/s : $$H_n = 0.707 \cdot 5.12 \sqrt{(\log \frac{540}{1})} = 8.56\ m.$$

For 24 m/s : $$H_n = 0.707 \cdot 4.44 \sqrt{(\log \frac{568}{1})} = 7.49\ m.$$

The corresponding Shore Protection Manual numbering for the used equations is in the edition of 1984:

(II.1) = (3-59) page 3-83
(II.1) = (3-60) page 3-84
(II.3) = (3-63) page 3-84
(II.4) = (3-39) page 3-55
(II.5) = (3-40) page 3-55
(II.6) = (3-67) page 3-87

The calculation with the HISWA computer model

The HISWA computer model is a model made by N. Booij and L. H. Holthuijsen of the Delft University of Technology, Department of Civil Engineering. The model is very useful for the determination of the kind of waves present in the Gulf of Khambat. The Gulf of Khambat is characterised by many sand banks stretching out from the northern part of the Gulf to more than 100 kilometres southern of Bhavnagar (and the location of the proposed dam-site). To determine the wave height in the region where the dam is proposed ordinary equations such as made earlier in this annex cannot satisfy completely, since they work with an average depth of the water. Waves on the other hand are very much determined by the shoals present in the region where they travel through. In HISWA these shoals can be introduced in the bottom grid of the model. An extra reason for taking the computer model is that it takes account of refraction and diffraction (see computer model manual lit [25]), also very important for the local wave climate.

In the HISWA model two different runs have been made. The first one was for a large area, 135 kilometres by 135 kilometres. The second run was a nested run, for a smaller area of 70 kilometres by 70 kilometres. A part of the first model was recalculated with a finer grid. In this way waves can be followed from a far distance (where only a few sand banks exist) and the waves calculated near the proposed dam site are more accurate since they are calculated in a finer grid. In figures II.2 and II.3 the iso lines of the bottom for the two models are shown.

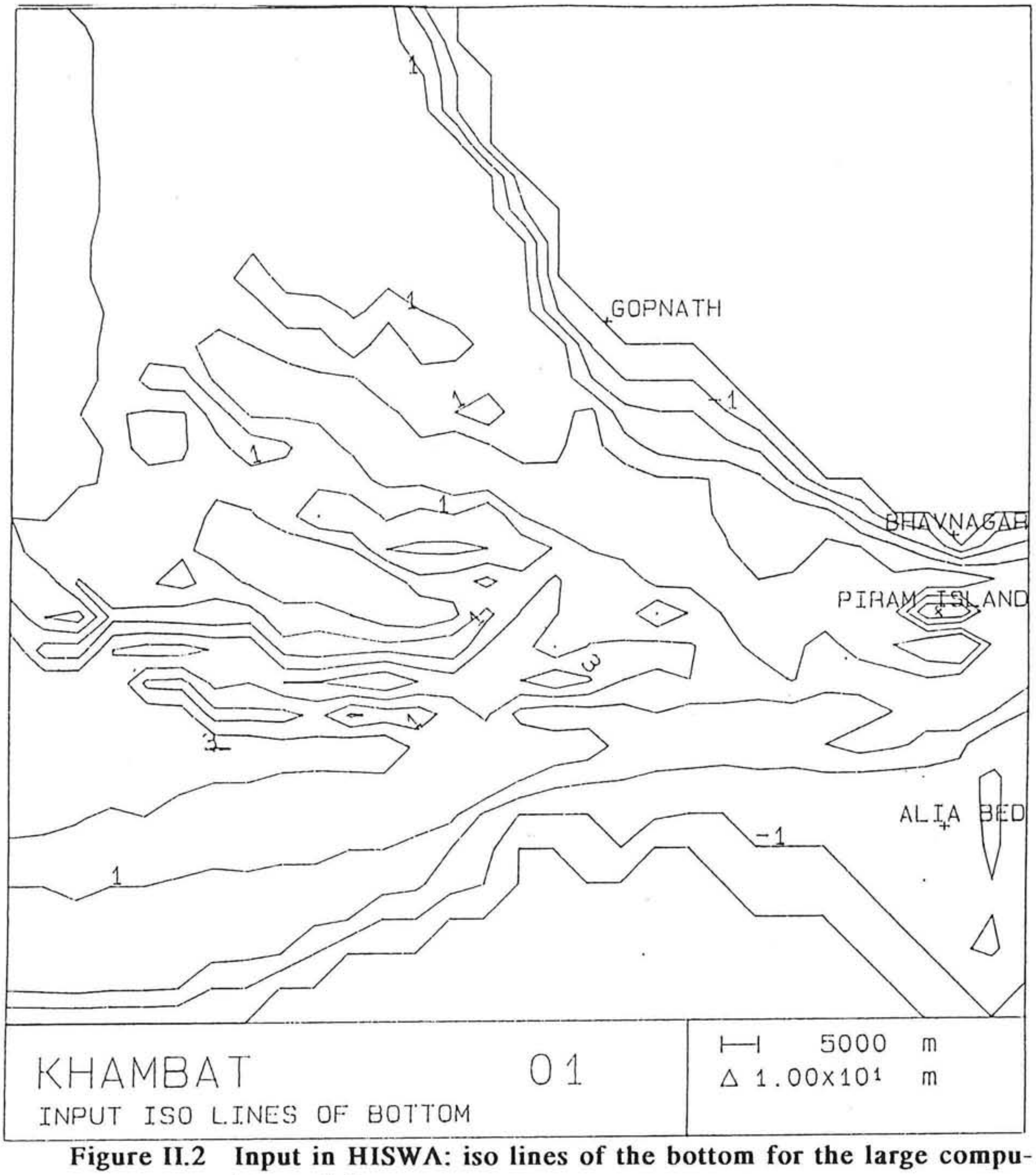


Figure II.2 Input in HISWA: iso lines of the bottom for the large computer model

The wind speed used for the calculations in HISWA are the same as used for the equations in the previous paragraph. The maximum wind velocity is 31 m/s from southern directions with a chance of exceedance of 1 %. In the HISWA model this wind has been taken with the wind direction varying between - 30 and + 30 degrees from the south. The maximum wave hight occurs when the wind direction is - 25 degrees (measured counterclockwise from the south).

The fetch in the HISWA model can not be much longer than 135 kilometres since the model is not larger than 135 by 135 kilometres. Waves generated in this small area can never be maximum waves possible in the region. Therefore an incoming wave field is inserted in the model. The direction of this wave field has been taken in the same direction as the wind.

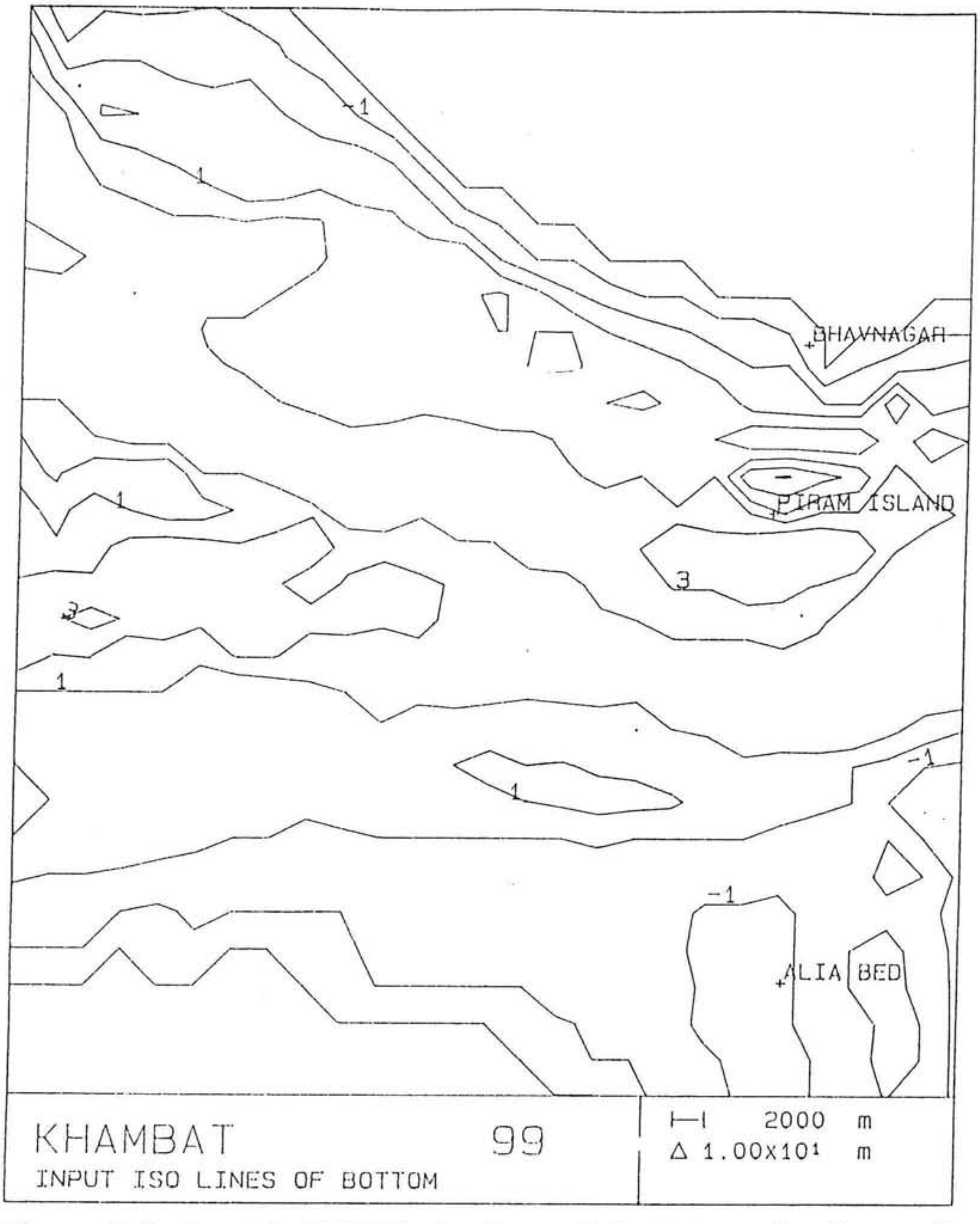


Figure II.3 Input in HISWA: iso lines of the bottom for the small computer model

The wave height of the incoming wave field has been varied between 5 and 10 metres. 10 metres, the upper limit, has been taken since the waves have travelled through water with a dept of less than 30 metres over a distance of more than 200 kilometres to get in this part of the Gulf of Khambat. 5 metres, the lower limit has been taken since these waves require, with a dept of 25 metres, only 300 kilometres of fetch to be generated (with a wind speed of 31 m/s). The difference in wave height near the probable dam location for the different incoming wave fields was negligible (maximum between 3.3 metres and 3.8 metres). This maximum wave height of 3.8 metres (also taking place if the incoming wave field has a significant wave height of 6 metres), applies only for a small part of the dam location. The iso lines of the wave hight for the total region of the Gulf of Khambat are shown in Figure II.4. The wave height near Bhavnagar is considerable lower

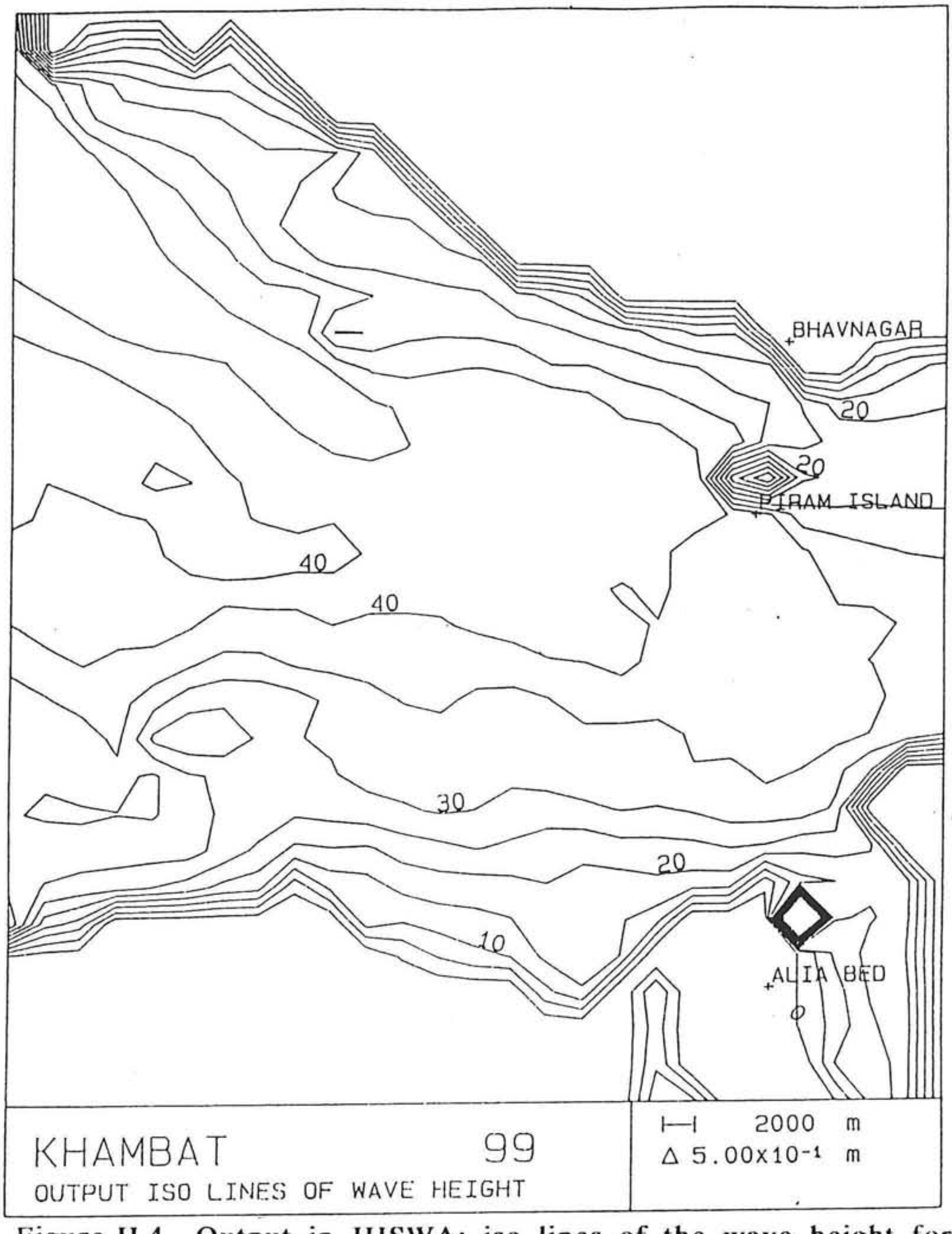


Figure II.4 Output in HISWA: iso lines of the wave height for a wind speed of 31 m/s.

due to the sheltering effect of Piram Island. Maximum wave heights are reduced to 2.0 metres and lower. Near Alia Bet the wave height is also considerable lower than 3.8 metres due to the shallowness of this region.

The wave heights mentioned before and shown in Figure II.4 occur only if the water level set at 8 metres above B.M. (for B.M. see section 3.4). This is the maximum water level during a mean tide. If the water level is set at 0 metres (the minimum water level during a mean tide) the maximum wave height is reduced to 2.7 metres. The water level of 8 metres is chosen since this water level can be maintained for more than 2 hours if a wind set up takes place of 2.0 metres. Higher water levels (resulting in slightly higher waves) have not be taken into account since the chance of high water levels (such as during a spring tide) occurring at the same time as the maximum wind speed is very small.

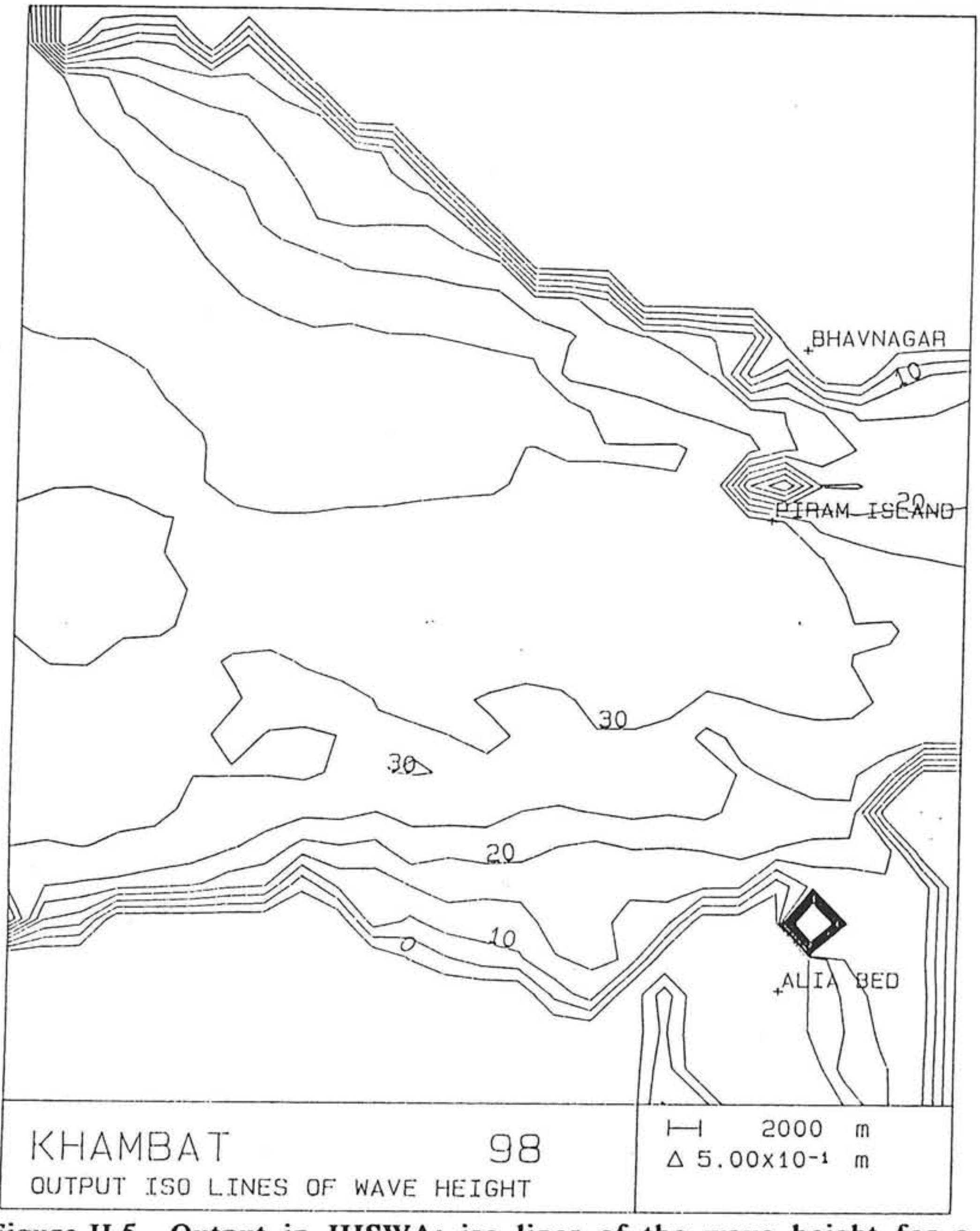


Figure II.5 Output in HISWA: iso lines of the wave height for a wind speed of 24 m/s.

With the HISWA computer model the wave heights in the enclosed basin have also been calculated. The basin level has been set at the maximum water level of B.M. + 10 metres. The waves have been calculated for wind speeds of 31 metres per second and 24 metres per second. The waves as result of a wind speed of 31 metres per second have a significant wave hight of 3 metres and a significant wave period of 5.5 seconds. A wind speed of 24 metres per second gives waves with a significant wave height of 2.3 metres and a wave period of 5.0 seconds. The wind direction has been chosen from the east and from the west. The given values are the maximum waves in the middle of the basin, there where the basin dams are planned. The waves as result of a wind from the north (breaking on the north side of the closure dam) are assumed to be the same.

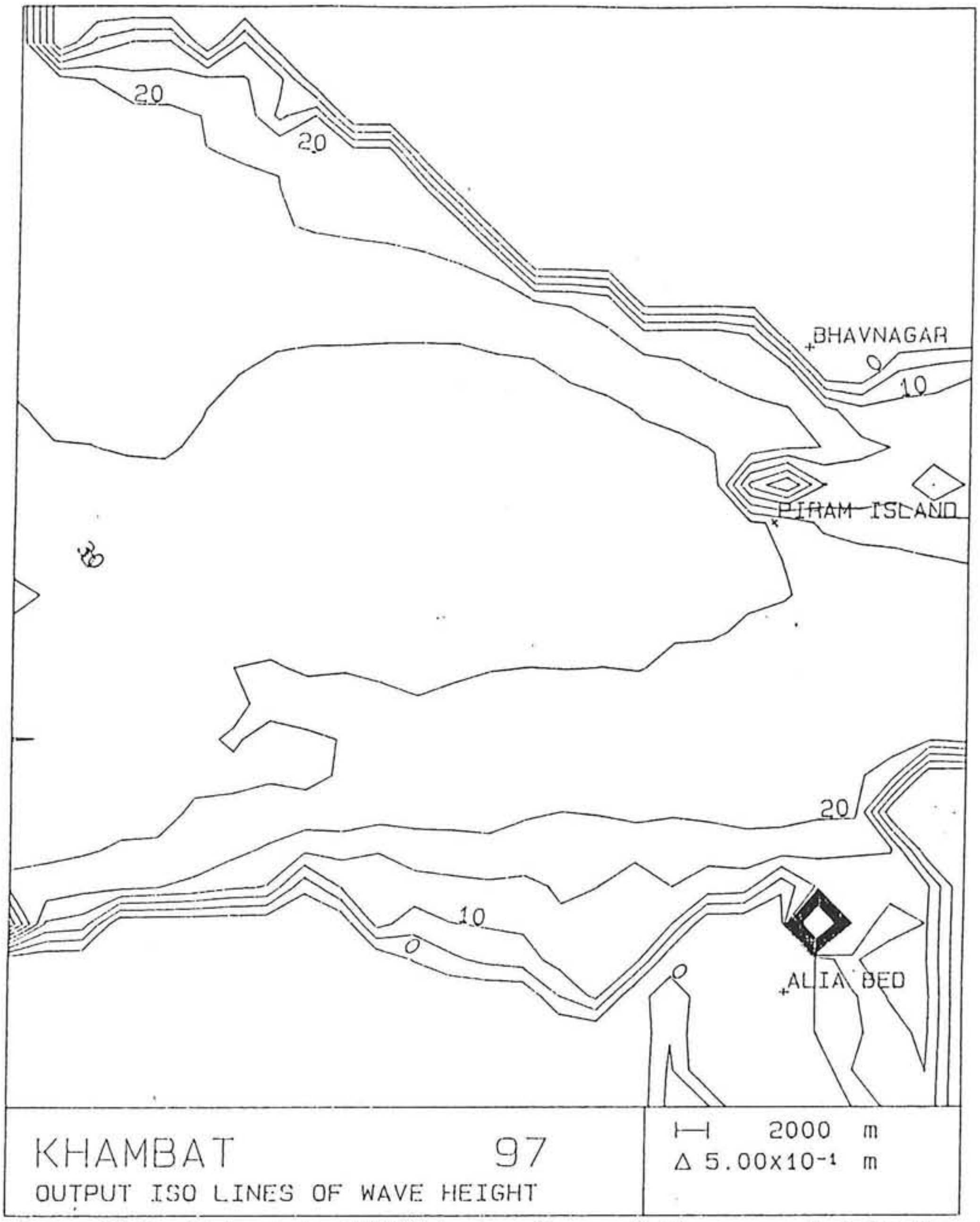


Figure II.6 Output in HISWA: iso lines of the wave height for a wind speed of 17 m/s.

Annex III The availability and properties of rock

On both the east and west-side of the Gulf of Khambat large areas of so called **Deccan Trap** can be found. Deccan Trap consists mainly of **basalt** with different density. Basalt is an igneous rock. This means that it is formed by the crystallisation and solidification of a molten silicate magma. This process gives strong rock with interlocking crystals. Basalt is greenish-grey to black in colour and commonly used as a building stone. The massive variety is good as a rock.

At the west side of the gulf, in the Amreli and Bhavnagar district (Figure III.1), the range in density of the basalt is 2.6 till $2.9*10^3$ [kg/m^3] (Table III.1).

location	rock type	density [$*10^3$ kg/m^3]	percentage water-absorption	compressive strength [N/mm^2]
Amreli district				
Darbargadh	Basalt	2.62	2.92	34.0
Quarry Dhari	Basalt	2.78	0.41	104.9
Kotadapitha	Basalt	2.68	0.27	128.2
Rajula	Rhyolite	2.42	0.23	82.0
Bhavnagar district				
Bhavnagar	Basalt	2.70	0.40	157.7
Botad	Basalt	2.81	0.55	101.1
Chamardi	Gronophyre	2.57	0.57	106.4
Gariyadhar	Basalt	2.91	1.33	64.4
Shihor	Rhyolite	5.61	2.50	92.2

Table III.1 Physical properties of quarry-stone

No density-data are available of the basalt found in the Bharuch district, at the east-side of the Gulf of Khambat. The geological map (Figure III.2) however shows a large area of Deccan Trap and many existing quarries. The range of basalt-weights that can be found here is assumed to be the same as in Amreli and Bhavnagar district.

According to an unknown source, in Bhavnagar district, near Shihor, Rhyolite (also an igneous rock) with a density of approximately $5.6 * 10^3$ [kg/m^3] can be found (Table III.1). Unfortunately this high value of density is not likely. For a quarry in Amreli district a value of $2.42 * 10^3$ [kg/m^3] is mentioned which is more realistic compared to the values given in the **Manual on the use of rock** [lit. (18)]. See Table III.2.

Rock group name	Rock mass density (t/m^3)	Unconfined compressive strength (MPa = N/mm^2)	Water absorption (%)	Porosity (%)
Igneous				
Rhyolite	2.3 - 2.8	100 - 260	0.2 - 5.0	0.4 - 6.0
Basalt*	2.5 - 3.1	160 - 280	0.1 - 1.0	0.1 - 1.0
Sedimentary				
Limestone	2.3 - 2.7	30 - 120	0.2 - 5.0	0.2 - 20.0

* Dense basalt excluding vesicular basalt

Table III.2 Generalised engineering characteristics of un-weathered common rocks

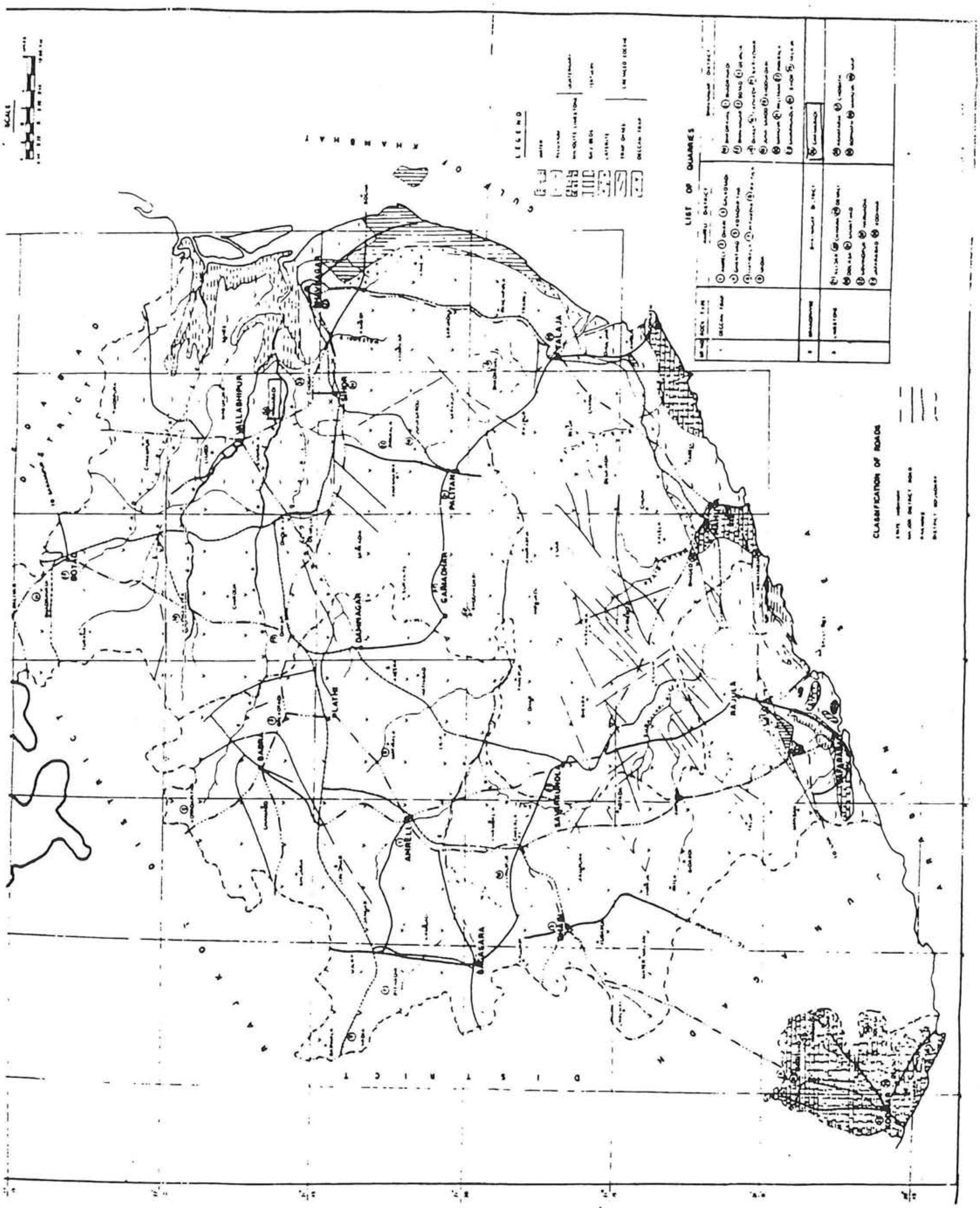
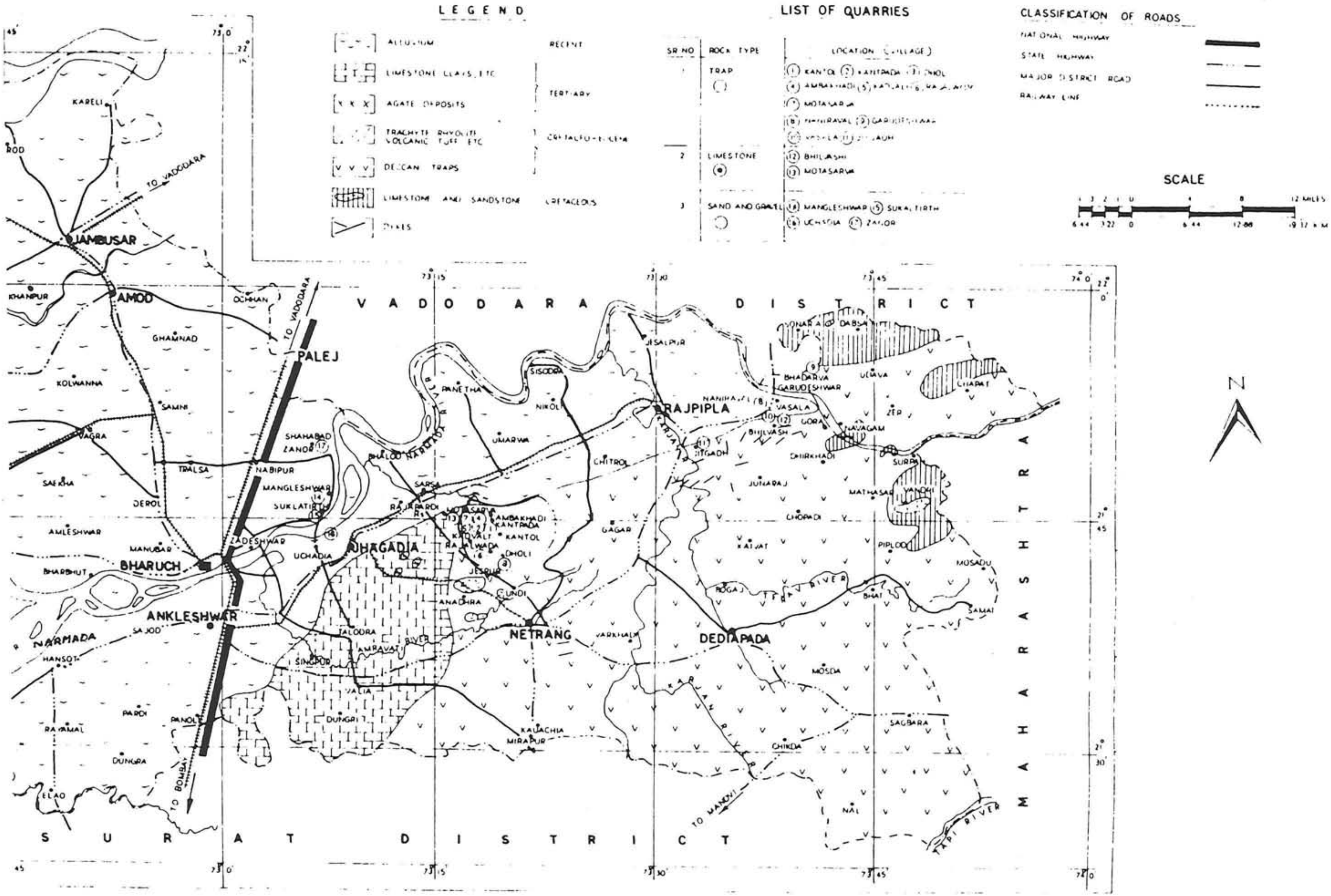


Figure III.1 Geological map of Amreli and Bhavnagar district

Figure III.2 Geological map of Bharuch district



GEOLOGICAL MAP OF BHARUCH DISTRICT

AFTER GEOLOGICAL MAP OF G.S.I.

Annex III

The porosity and the value for the absorption of water are a measure for the weathering of rock. When the absorption stay below 5 % the risk of weathering is minor.

Rock group name	Typical grain size range (mm)	Visible voids	Typical texture	Typical rock mass appearance	Typical basic fragment shape	Typical geological distribution
Igneous						
Rhyolite	Grains not visible to unaided eye	Rare	Locally variable micro-fractures common	Irregular	Cubic/ prismatic	Localized areas
Basalt	Grains not visible to unaided eye	Common, large and small	Isotropic uniform	Irregular/blocky columnar	Cubic/prismatic elongate	Extensive sheets
Sedimentary						
Limestone	2 - 0.01	Common, large and small	Narrow grain size ranges or cemented fragments	Blocky/flaggy	Cubic/tabular	Extensive areas

Table III.3 Geological properties of the available rock types [lit. (18)]

A third rock type present in the mentioned districts is **limestone**. Limestone is a sedimentary rock (formed by the sedimentation and subsequent lithification of mineral grains).

The choice between the three kinds of rock is based on the following points:

- The availability is good for basalt and, to a certain extent, also for limestone. Rhyolite is only found locally and in relatively small amounts (see Table III.3). Blocks of rhyolite are generally rather small.

- On their way from the quarry to their definite position the rock will be loaded during different block-handling phases (Figure III.3). A high breakage resistance limits the mass reduction of the armour blocks during handling. Basalt has a higher breakage resistance than limestone and, on the average, rhyolite.

These features lead to the choice of using basalt for construction purposes in the Gulf of Khambat. It is supposed that two kinds of rock are available:

- A large amount of basalt with a density of 2700 [kg/m^3]

- A more limited amount of basalt with a density of 2900 [kg/m^3] for use at places were the loads on the stones will be higher.

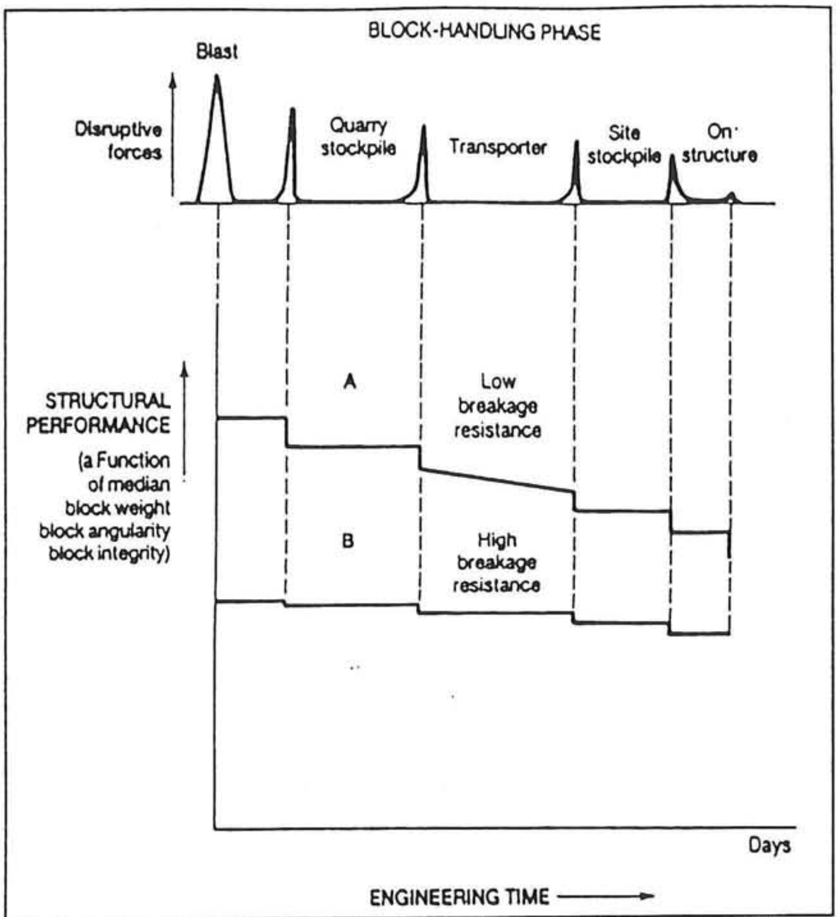


Figure III.3 Weight reduction of stones during handling

Stability of rock

The stability of rock under flow attack depends on the product of their nominal diameter D_n and their relative density Δ.

$$\Delta = \frac{\rho_{stone} - \rho_{water}}{\rho_{water}} \qquad D_n = (volume_{50\%})^{1/3} = (\frac{M_{50\%}}{\gamma_s})^{1/3}$$

D_n represents in this way the linear dimension of a cube of which the mass is exceeded by 50 % of the total number of stones.

When the density of the rock increases the effect is twofold:

- For a stone with equal mass the necessary diameter can become smaller;
- The diameter D_n becoming smaller results in smaller loads which decreases the necessary stone-mass.

When the value of ΔD_n is known the stone-mass depends strongly on the density of the rock available. This is illustrated in Figure III.4 for the two available kinds of basalt and the unlikely heavy rhyolite.

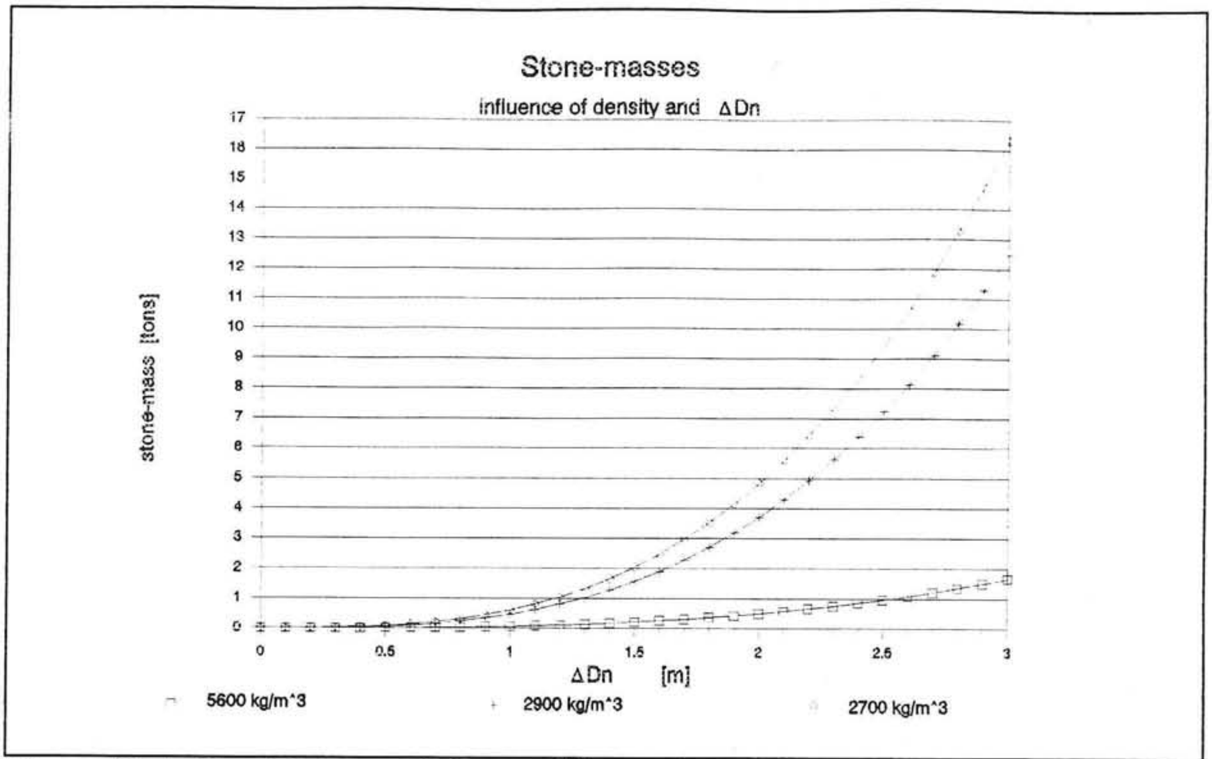


Figure III.4 The influence of the specific gravity on the necessary stone-mass

Gradings

In the quarry the rock will be selected. Different gradings will be distinguished. It is assumed that by means of sieving (till 300 [kg]), weighing and visual selection, the gradings of Table III.4 can be distinguished.

Based on a uniform diameter (D) distribution an imaginary M_{50} can be determined in the way as shown below:

grading: 60 - 300 [kg]

assume
- the stones are cubical-shaped (a sphere gives the same result for the value of M_{50})
- the density is 2700 kg/m^3 (the M50 is indifferent for the value of the density)

60 [kg] $D = (60/2700)^{1/3} = 0.281$ [m]

300 [kg] $D = (300/2700)^{1/3} = 0.481$ [m]

$D_{average} = 0.381$ m.

Now the imaginary M_{50} becomes:

$$M_{50} = 0.381^3 * 2700 = 149 \text{ [kg]}$$

A less difficult approach is taking the average of the grading limits:

$M_{50} = (60 + 300) / 2 = 180$ [kg]

According to information in the **Manual on the use of rock** [lit. (18)], the second, simple, approach is better (Table III.4). This values are approximately in the middle of the expected range. The values determined based on a uniform diameter distribution seems to small. Thus the average of the class limits is taken as a measure for the M_{50}.

grading	approached M_{50}	Average of limits	Expected range M_{50} [lit. (18)]
10 - 60 kg	28 kg	35 kg	26 - 46 kg
60 - 300 kg	149 kg	180 kg	150 - 220 kg
300 - 1 000 kg	582 kg	650 kg	595 - 760 kg
1 000 - 3 000 kg	1 821 kg	2 000 kg	1 800 - 2 200 kg
3 000 - 6 000 kg	4 328 kg	4 500 kg	4 200 - 4 800 kg
6 000 - 10 000 kg	7 830 kg	8 000 kg	7 500 - 8 500 kg
10 000 - 15 000 kg	12 332 kg	12 500 kg	- - -
15 000 - 20 000 kg	17 380 kg	17 500 kg	- - -
20 000 - 25 000 kg	22 407 kg	22 500 kg	- - -

Table III.4 Estimated values of the M_{50}

For the two available kinds of rock the limits of the ΔD_n-value can be determined for each grading. D_n represents the linear dimension of a cube of the material considered of which the individual mass is exceeded by 50 % of the total number of stones. Thus when the stone mass computed out of the ΔD_n exceeds the M_{50} of a certain grading a heavier grading must be used. The way of determination is shown in Figure III.5, III.6 and III.7 and the results are summarized in Table III.5.

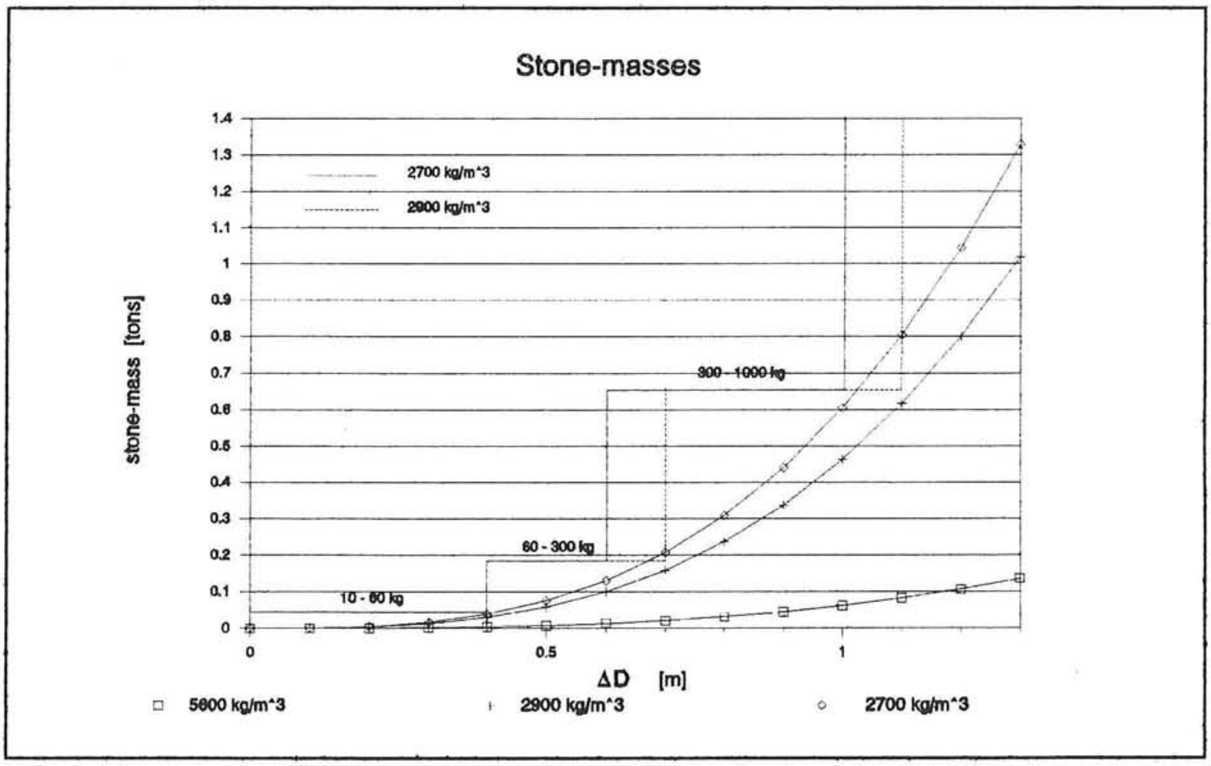


Figure III.5 Determination of class limits of rock lighter than 1000 kg

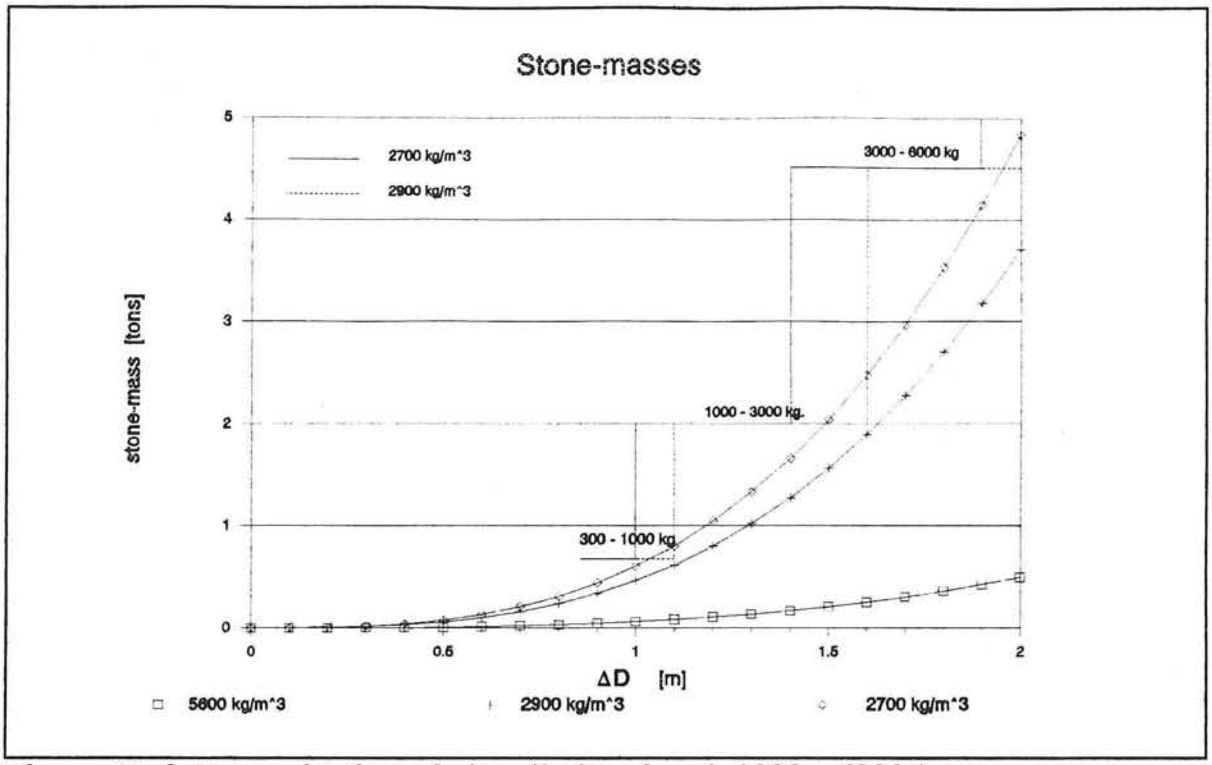


Figure III.6 Determination of class limits of rock 1000 - 6000 kg

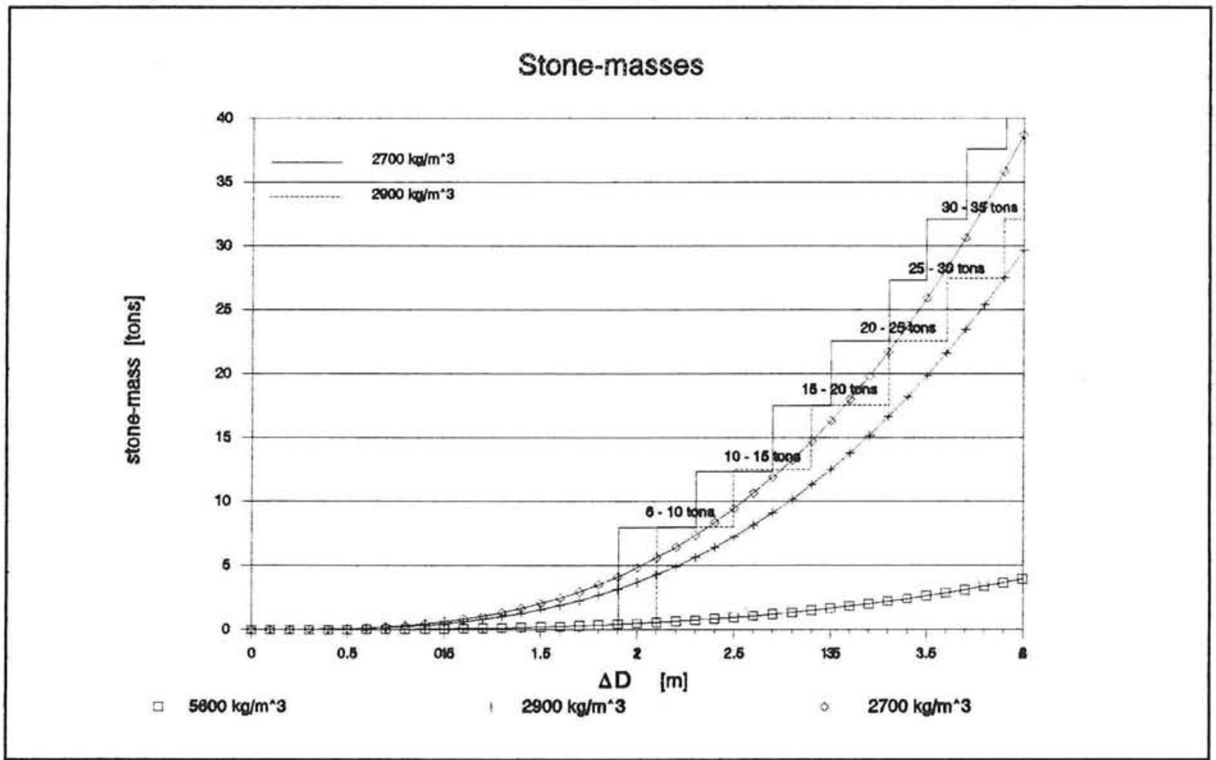


Figure III.7 Determination of class limits of rock 6 - 35 tons

grading	M_{50}	average D_n [m]	ΔD_n limits [m] 2700 kg/m³	2900 kg/m³
10 - 60 kg	35 kg	0.23	0.0 - 0.4	0.0 - 0.4
60 - 300 kg	180 kg	0.41	0.4 - 0.6	0.4 - 0.7
300 - 1 000 kg	650 kg	0.62	0.6 - 1.0	0.7 - 1.1
1 000 - 3 000 kg	2 000 kg	0.90	1.0 - 1.4	1.1 - 1.6
3 000 - 6 000 kg	4 500 kg	1.18	1.4 - 1.9	1.6 - 2.1
6 000 - 10 000 kg	8 000 kg	1.42	1.9 - 2.3	2.1 - 2.5
10 000 - 15 000 kg	12 500 kg	1.67	2.3 - 2.7	2.5 - 2.9
15 000 - 20 000 kg	17 500 kg	1.84	2.7 - 3.0	2.9 - 3.3
20 000 - 25 000 kg	22 500 kg	2.00	3.0 - 3.3	3.3 - 3.6

Table III.5 ΔD_n limits for different gradings

Extreme stone-masses

Here it is assumed that the largest necessary stones can be produced by a quarry in one of the districts close to the Gulf of Khambat. Further examination on the site of possible quarries, new or existing ones, is necessary to determine the maximum available stone-mass and diameter. This is determined by the existing cracks in the rock and the way in which the rock is layered. When the largest required stones are not available they can be made of concrete, using heavy basalt as an aggregate.

Annex IV Rock-fill structures

IV.1 Stability of stones on a sill

The stability of rock-fill dams has been investigated by **Delft Hydraulics Laboratory** in the Netherlands. Many data gained in experiments have been plotted in the graph below (Figure IV.1) [lit. (1)].

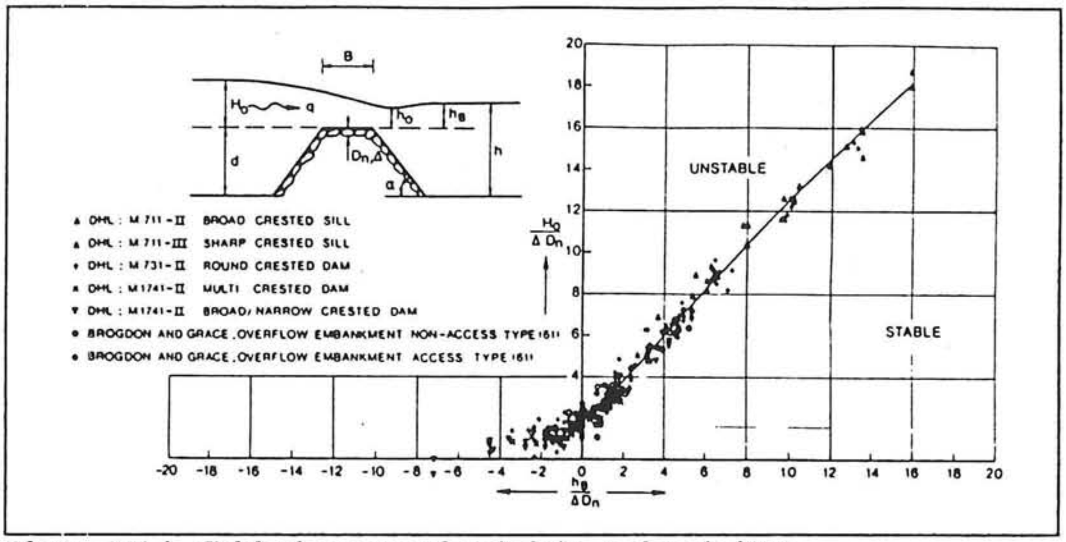


Figure IV.1 Critical overtopping height against $h_B/\Delta D_n$

$$\Delta = \frac{\rho_{stone} - \rho_{water}}{\rho_{water}} \qquad \textbf{(IV.1)}$$

$$D_n = (volume_{50\%})^{1/3} = (\frac{M_{50\%}}{\gamma_s})^{1/3} \qquad \textbf{(IV.2)}$$

D_n in this way represents the linear dimension of a cube of the material considered of which the individual weight is exceeded by 50 % of the total number of stones.

The graph can be schematized as a combination of two branches:

<u>left branch</u>

Left of the point $\frac{h_B}{\Delta D_n} = -1$

the graph can be approached by a line were the value of $\frac{H_o}{\Delta D_n} = 1$

The stone-parameter ΔD is then equal to H_o $\qquad \Delta D_n = H_o \qquad$ **(IV.3)** right branch

Right of the point $\frac{h_B}{\Delta D_n} = -1$

the graph can be approached by a line that crosses the vertical axis at a value of 2 and has a gradient of 1. This leads to the following equations:

$$\frac{H_o}{\Delta D_n} = \frac{h_B}{\Delta D_n} + 2 \qquad\qquad \Delta D_n = \frac{H_o - h_B}{2} \qquad \textbf{(IV.4)}$$

In order to determine the ΔD-value necessary during the closure of the Gulf of Khambat these equations (Eq. IV.3 and IV.4) have been used.

IV.2 The influence of the head effect on the stone-diameter

Horizontal constriction of the dam causes contracted flow through the closure-gap bordered by vortex-streets (Figure IV.2). This vortex streets attack the bottom protection or the upper layer of the sill on which the constriction takes place. This effect is not taken into account using Figure IV.1 to determine the ΔD_n-value.

In order to reckon with the effect of the vortex street the application of a factor on the velocities in the closure-gap seems most appropriate. Then (Eq IV.5):

$$V = \overline{V} + \xi\overline{V} \qquad \textbf{(IV.5)}$$

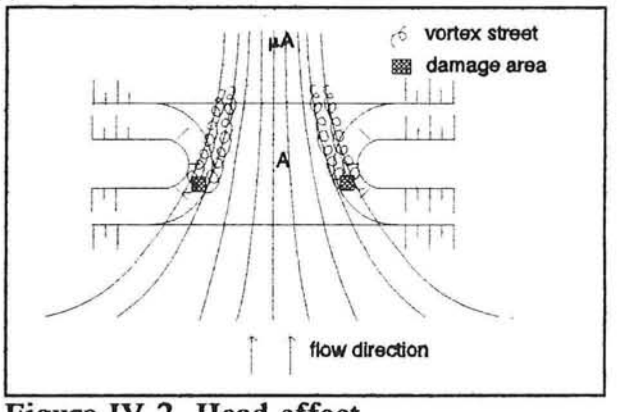


Figure IV.2 Head effect

The way of determination of the ΔD_n-value used so far is not directly based on velocities but on head-differences. Rewriting the results is necessary. The equation for the velocity in the closure-gap is (Eq. IV.6 see also section 4.2.1):

$$V_{gap} = \mu\sqrt{2\cdot g\cdot(H-h)} \qquad \textbf{(IV.6)}$$

with $0.8 <= \mu <= 1.0$

which can be written as:

$$V_{gap} = \mu\sqrt{2\cdot g\cdot\frac{(H-h)}{\Delta D_n}\,\Delta D_n}$$

together with Eq. IV.4 this becomes:

$$V_{gap} = \mu\sqrt{4\cdot g\cdot\Delta D_n} \qquad <=> \qquad \Delta D_n = \frac{1}{2\cdot\mu^2}\,\frac{V_{gap}^2}{2\cdot g}$$

Including turbulent flow:

$$\Delta D_n = \frac{(1+\xi)^2}{2\cdot\mu^2}\frac{V_{gap}^2}{2\cdot g} \qquad \textbf{(IV.7)}$$

Equation IV.7 shows resemblance to the Izbash-equation (Eq IV.8):

$$\Delta D_{50} = \frac{b}{k_\alpha\cdot k_\beta}\frac{V_{gap}^2}{2\cdot g} \quad <=> \quad \Delta D_n = \frac{1}{1.24}\frac{b}{k_\alpha\cdot k_\beta}\frac{V_{gap}^2}{2\cdot g} \qquad \textbf{(IV.8)}$$

Izbash defines a D_{50}, which is the diameter of a sphere. The ratio between the diameter of a sphere and a cube of the same volume is 1.24.

The stability-parameter b varies between 0.7 and 1.4 depending on the degree of turbulence and the bottom-roughness.

b = 0.7 for a broad-crested sill or bottom-protection loaded by moderate turbulent flow
b = 1.4 for a sharp-crested sill or bottom-protection loaded by strong turbulent flow, e.g. an hydraulic jump.

The coefficient k_α is a correction for the decrease in stability of the stones when they are placed on a slope perpendicular to the flow-direction. It depends on (Eq. IV.9):

-the angle of internal friction ϕ, for rock $\phi = 40°$

-the angle of the slope α, in case of a rock-fill dam 1 : 1½, $\alpha = 33.7°$

$$k_\alpha = \cos\alpha\sqrt{1-\left(\frac{\tan\alpha}{\tan\phi}\right)^2} = 0.505 \qquad \textbf{(IV.9)}$$

The coefficient k_β is a correction when the direction of flow does not correspond with the horizontal plane: here $k_\beta = 1$.

example

As an example to illustrate the comparison of the different calculation-methods we take the following case:

- a horizontal constriction on a sill with a level of 20 [m] below B.M., the width of the closure-gap is 100 [m]
- the contraction coefficient $\mu = 0.85$
- the maximum velocity during a tidal cycle at average spring-tide is 8.20 [m/s]
- the maximum head difference then amounts to 4.74 [m]
- at first no turbulence or slope effects are taken into account so $\xi = 0$ and $k_\alpha = 1$.

The value of ΔD_n, using equation IV.4 becomes 4.74/2 = 2.37 [m] Equation IV.7 leads to the same result:

$$\Delta D_n = \frac{(1+0)^2}{2 \cdot 0.85^2} \frac{8.20^2}{2 \cdot 9.8} = 2.37$$

(These values have to be the same if DUFLOW and the derivation of Eq. IV.7 are both correct)

When we assume that the Izbash-equation (Eq IV.8) gives the same result, the value of b can be determined:

$$\frac{1}{1.24} \frac{b}{k_\alpha \cdot k_\beta} = \frac{(1+\xi)^2}{2 \cdot \mu^2} \qquad <=> \qquad b = 1.24 \frac{1}{2 \cdot 0.85^2} = 0.86$$

This value of b represents a broad-crested sill with moderate turbulence. The sill at 20 [m] below B.M. indeed has to be rather wide so the result is acceptable. The value of b is indifferent for the velocity in the closure gap.

The stability of the stones on the dam-head slope is described by equation IV.8, using k_α = 0.505. This almost doubles the ΔD_n-value: ΔD_n = 4.47 [m], which means that the stones have to be eight times (2^3) as heavy.

The stone-weights determined this way are supposed to be safe for a permanent construction. During stages of a closure operation instability of some stones will result in a decreasing slope-angle of the dam-heads. Only few stones will move to a place outside the profile of the closure dam. The nett loss will be limited. The occurring loss will "only" retard the closure operation.

More critical are the stones on the top of the sill (Figure IV.2). The sill consists of layers of stones with different weights. The heaviest upper layer will have a limited thickness of three or four times D_n. When dump-barges are used locally the thickness can be even less. When (only few) stones of the upper layer move and disappear the risk of the second layer being attacked by the flow increases. The stability of this layer is less, so severe damage can be expected.

When a strong turbulent flow is expected the value of b in Eq. IV.8 becomes 1.4. When the contraction in the closure-gap is maximum the value of μ = 0.8. The ξ-value can be estimated now:

$$(1+\xi)^2 = \frac{2\mu^2}{1.24} \frac{b}{k_\alpha k_\beta} = \frac{2 \cdot 0.8^2}{1.24} \frac{1.4}{1} = 1.45 \qquad <=> \qquad \xi = 0.2$$

Applying this ξ-value on the results of the example the value of ΔD_n becomes:

$$\Delta D_n = (1+0.2)^2 * 2.37 = 1.44 * 2.37 = 3.41 \text{ [m]}$$

This still results in stones with a weight of 1.44^3 = 3 times the weight following from eq. IV.4.

IV.3 The permeability of the dam

A rock-fill dam is pervious to water. The discharge through the dam will decrease the head-difference over the dam in the final stages of the closure-operation. Thus the necessary stone-weight can be reduced. An estimation of the discharge can be made using equation IV.10 [lit. (17)].

$$q^2 = \frac{2gD_n\varepsilon^5}{3C}\cdot\frac{H^3-h^3}{L} \qquad \textbf{(IV.10)}$$

in which:

q	= discharge through the dam per running meter		[(m³/s)/m]
g	= acceleration of gravity	= 9.8	[m/s²]
D_n	= nominal stone-diameter		[m]
ε	= porosity of the dam	= 0.40	[-]
C	= f(Re) for turbulent flow	C = 0.3	[-]
H	= upstream water depth		[m]
h	= downstream water depth		[m]
L	= representative length of the flow path		[m]

L is determined as shown in Equation IV.11 using the parameters of Figure IV.3

$$L = 2ad + b - \frac{2}{3}a\,(H+h) \qquad \textbf{(IV.11)}$$

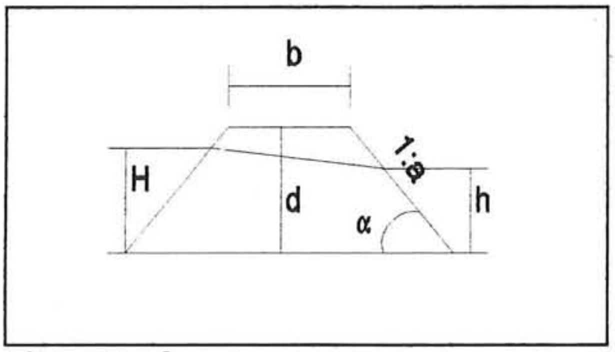


Figure IV.3

Here, for the whole closure dam the following values have been taken:

D_n	=	1.5 [m] (stones in range 6 - 10 ton)
b	=	24 [m]
a	=	1.5 [-]
d		depends on the depth of the section, top on a level of 12 [m] above B.M.
H,h		depend on both the bottom-level of the section and the tidal amplitude

The permeability of the dam can be taken into account in the DUFLOW-model using culverts. The dimensions and discharge-coefficient of the culverts have been calibrated using the results of Eq. IV.10. The situation of average spring-tide combined with a finished closure-dam has been taken as a starting point to determine the dimensions. In that case the extreme waterlevels in front of the sea side of the (impervious) dam will be:

low tide: 0.58 [m] - B.M. high tide: 8.68 [m] + B.M.

The maximum discharge through the dam occurs when both the head-difference and the flow-area are maximum. The maximum head-difference is taken as half the tidal difference, hence 4.63 [m] Thus:

up-stream water-level: 8.68 [m] + B.M.
down-stream water-level: 8.68 - 4.63 = 4.05 [m] + B.M.

In the DUFLOW-network the Gulf of Khambat is subdivided in five sections along the alignment of the closure-dam. The section south-east of Alia Bet is neglected here because of its small depth. In the four remaining sections (23, 24, 25 and 26 in Figure IV.4) a significant flow through the dam is supposed to take place.

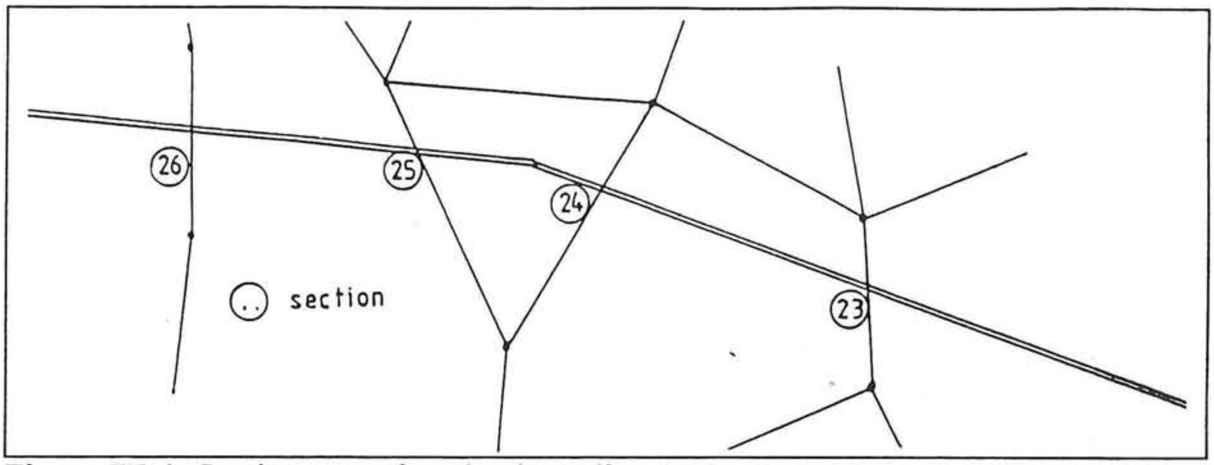


Figure IV.4 Sections crossing the dam-alignment

The discharge q is computed for each of this sections using Eq. IV.10 and IV.11.. The results are shown in Table IV.1

section	bottom-level [m - B.M.]	H [m]	h [m]	L [m]	q [m^3/s/m]
23	25	33.68	29.05	78.27	7.65
24	20	28.68	24.05	73.27	6.65
25	45	53.68	49.05	98.27	11.17
26	35	43.68	39.05	88.27	9.50

Table IV.1 Determination of the discharge through the closure-dam

This results may be regarded as a rough estimation of the discharge through the closure-dam. Some remarks on the accuracy are made below:

- The porosity of the dam is taken as 0.4. This is not an accurate value but just an estimation. Because of properties of the rock and the construction method the porosity can approximately vary between the values 0.37 and 0.43.

- The value of D_n is taken as 1.5 [m] here. Stones with a diameter of 1.5 [m] will be used during stages of the closure were the loads are already severe. When e.g. stones with a D_n of 0.5 [m] are used the discharge will be (Eq. IV.10) $\sqrt{1.5} / \sqrt{0.5}$ = 1.7 times smaller.

- The porosity of the dam, ε, can change in course of time. For instance growth of marine life in the pores can reduce the porosity as well as washing in of fines.

The culverts in the DUFLOW-model have been calibrated using the computed values of Table IV.1. For highest astronomical tide the results of Eq. IV.10 and the DUFLOW-calculation have been compared (see Appendix A section A6.3). The discharges did not differ more than 1.5 percent.

As an example the influence of the dam-porosity on the value of ΔD_n is shown in Figure IV.5 for a horizontal closure with highest astronomical tide used as a boundary condition.

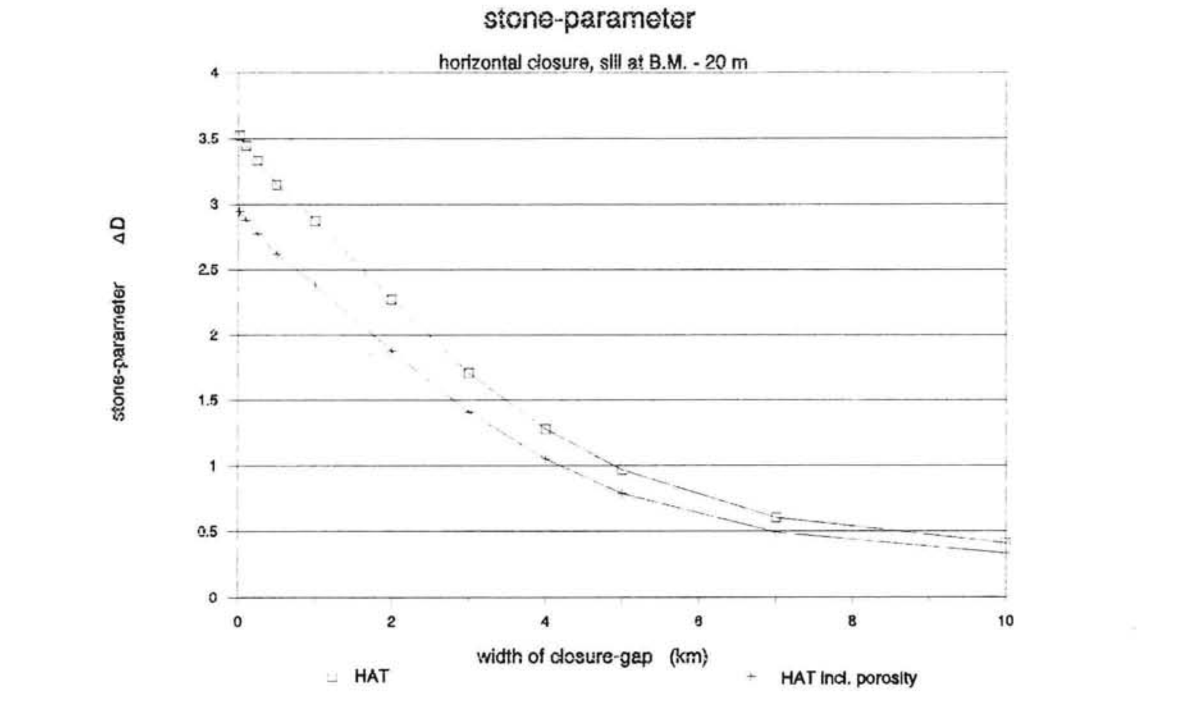


Figure IV.5 Influence of the dam-porosity on the stone parameter ΔD_n

The difference in the necessary stone-mass is considerable. Assuming a density of 2900 [kg/m^3] and Δ = 1.84 the maximum masses M_{max} become:

HAT

ΔD_n = 3.5 [m] D_n = 1.90 [m] M_{max} = 20.0 * 10^3 [kg]

HAT including dam-porosity

ΔD_n = 2.9 [m] D_n = 1.58 [m] M_{max} = 11.4 * 10^3 [kg]

The difference in the results is large. However, for safety reasons and keeping in mind that the majority of stones will be smaller than assumed, it is advised to take only half the reduction of ΔD_n into account. M_{max} in this case is still 15.2 * 10^3 [kg].

Annex V Design and costs of the basin dams

The basin dams are planned in the closed part of the Gulf of Khambat protecting them from waves and water levels outside the closure dam. In the closed part of the Gulf the water levels and waves are very different from the unprotected part of the Gulf.

Water level

The water level in the tidal basin varies between BM + 4 metres and BM + 9 metres, and this variation takes place two times every 24 hours and 50 minutes. The water level in the fresh water basin varies between BM + 2 metres and BM + 10 metres, both extremes occurring very seldom. The water in the fresh water basin can rise 4 metres in a very short time if the Narmada has an extreme discharge. The water level only drops relative rapidly if the overflows are used in case of a basin level threatening to exceed
BM + 10 metres. This will only happen when the level in the basin is already high (more than BM + 8 metres) and a high water wave in the Narmada is on the way to the Gulf of Khambat. In normal conditions the water level in the fresh water basin is very stabile between BM + 4 metres and BM + 8 metres.

Waves

The waves in the two basins are always limited by the possible fetches of the wind. The fetch over water with a depth of more than 10 metres are never longer than 20 km. The wind speed in the closed part of the Gulf are also limited. Wind speeds of 35 metres per second blowing from other directions than the South can never be expected due to the sheltering effect of the main land. A wind speed of 31 metres per second blowing from east north and west can be expected with a change of 1 % of exceedance. The wind speed with a change of 2 % of exceedance is set at 24 metres per second.

These more comfortable conditions for the basin dam and north side of the closure dam, make it possible to design a less high, less defended, and less costing basin dam compared to the closure dam.
Since there is no current of any significance during the construction of the basin dam (the closure dam has already been build) the dam can easily be made with sand. In the Gulf of Khambat thick layers of sand can be found beginning at the bottom of the Gulf. These layers contain sand with a grain size suitable for dam construction. Suction dredges with pipe lines can be used to transport the sand of place of extraction to building place.

Slope of the dam

Dumping the sand on the building place the sand will have a slope of 1 : 15 if it is dumped under water. If the sand is dumped above water level a slope of 1 : 1.5 can be realised (an angle of 34 degrees with the horizontal). The faint slope of the sand under water costs enormous quantities of sand. If the water level in the closed part of the Gulf is lowered as much as possible the quantity of sand needed for the constructi-on can be kept as low as possible. The water level in the basin can be lowered by opening the intake structure if the water level is low at the south side of the dam. The intake structure are closed if the water level in the unclosed part of the Gulf rises again. By constantly letting water out during ebb the water level in the basins should be able to drop to BM + 2.5 metres. The minimum water levels as result of ebb vary between BM - 0.5 metres and BM + 2 metres. An extra advantage of the low basin level is the reduction in the quantity of salt water present in the fresh water basin to be.

Height of the basin dam

The height of the dam of the dam is determined by two factors; the water level and the wave height. In the design of the dam a maximum overflow of the dam is accepted of 10 litres of water per second per meter dam. This quantity of water is the discharge as result of wave run over.
The equations to calculate the height of the dam for this discharge are given below in equations V.1, V.2 and V.3:

$$Y = \frac{q\ T_{1/3}\ \sqrt{cotg(\alpha)}\ 1.6 * 10^{-3} * 2\pi}{0.1 * 1.15\ H_{1/3}\ g\ (\frac{T_{1/3}}{1.15})^2} \qquad \textbf{(V.1)}$$

$$\log(Y) = -0.214\ X^2 - 0.787\ X + 0.103 \qquad \textbf{(V.2)}$$

$$z = \frac{X\sqrt{[(H_{1/3}/1.6)\ g\ (T_{1/3}/1.15)^2/2\pi]}}{cotg(\alpha)} \qquad \textbf{(V.3)}$$

in which :

q = average discharge over dam in l/s per metre[1]
$H_{1/3}$ = significant wave height in metres
$T_{1/3}$ = significant wave period in seconds
tg (α) = the slope of the dam
g = acceleration of gravity
z = height of the dam above mean water level in metres

The unknown factors in the equations are the wave height, the wave period and the slope. The average discharge after all has been set at ten litres per second per metre. This discharge may seem very high, but has to be seen in the light of the occurrence of once in the 100 years. If the height of the dam is calculated with the waves generated by the wind speed of 31 metres per second (significant wave height is 3.0 metres, significant wave period is 5.5 seconds) and a slope of 1 : 3, z is 4.8 metres. If a slope of 1 : 6 is used z is 2.4 metres. The width of the dam and the quantity of sand required for the dam does not change much by this change in slope. For a windspeed of 24 metres per second (significant wave height is 2.3 metres, significant wave period is 5.0 seconds) and a slope of 1 : 3, z is 3.6 metres. Again if the slope is set at 1 : 6, z halves to 1.8 metres.

Quantity of sand

If the wind speed of 31 metres per second is taken the value of height of the dam becomes BM + 10 metres + z = BM + 15 metres. With the slope of the dam known (1 : 15 from bottom to BM + 3 metres and 1 : 3 above BM + 3) the quantities of sand required for the four basin dams can be calculated. The result is given in Table V.1.

Alignment	Length (km)	Surface (km^2)	Quantity of sand (10^6 m^3)
1	30.0	256	115
2	34.5	363	116
3	43.5	473	111
4	49.5	510	114

Table V.1 Characteristics for the four tidal basin dams

From the table it follows surprisingly that the quantity of sand required for the alignments does not vary much. For alignment four less sand is required than for alignment one, but the basin surface area is more than double. The reason for this unexpected result lies in the Makra bank. The alignments of basins 3 and 4 follow this bank with depths of less than 5 metres below BM for more than half of their length, while the alignments for the basin dams of basins 1 and 2 have to cross depths of more than 15 metres below BM.

The chance of a maximum water level combined with the maximum wind speed and accompanying waves is however very slim. Therefore the waves as result of a wind speed of 24 metres will be taken in combination with the maximum basin level. If a wind speed of 31 metres per second can be expected and the water level in the basin is higher than BM + 9 metres water can be let out through the spillway. The reduction in sand required for the dam by construction more than equals the costs of loss of water in such a case. The reduced quantity of sand is:

basin 1 : 115 10^6 m^3 - 4 10^6 m^3 = 111 10^6 m^3
basin 2 : 116 10^6 m^3 - 5 10^6 m^3 = 111 10^6 m^3
basin 3 : 111 10^6 m^3 - 5 10^6 m^3 = 106 10^6 m^3
basin 4 : 114 10^6 m^3 - 6 10^6 m^3 = 108 10^6 m^3

The revetment of the dam however does have to be calculated with the maximal calculated wave height of 3.0 metres.

Reduction of seepage

The difference in water level between the two basins (the tidal basin and the fresh water basin) can be as much as 6 metres in case of a full fresh water basin and a low tidal basin. This difference would result in a considerable seepage through the dam if no precautions were taken. There are two principles of defence possible against this seepage: a impermeable core (for instance made of clay) or a impermeable revetment (for instance a geotextile with blocks on it or a layer of clay with blocks). Both principles have been looked at and the costs of both solutions have been calculated.

An impermeable revetment has the advantage that it can be placed after the sand has been dumped. A core made of clay has to be made during the dumping of sand. A second advantage an impermeable revetment is that it has a higher efficiency than an impermeable core. If the revetment is made impermeable from BM + 3, where the dam has a slope of 1 : 3, to the crest of the dam the length through the dam for the seepage water is more than 70 metres. If the core is made impermeable from BM + 3 to the crest hight the seepage length is only reduced to 45 metres. If the core is tried to be made impermeable to a depth lower than BM + 3 a difficulty in constructing arises. The clay will have to placed below the water level in the basin, this implies dumping the clay on the sand dam while it is still below the water surface.

An alternative for the clay core is a core made of bentonite mixed with cement. This core can be made when the sand profile has been finished. A gully is made in the dam, during excavation the bentonite is pumped in the gully to prevent it from collapsing. Bentonite is a liquid substance of clay particles with water, the substance has a volemic weight of 11 kN/m^3. This volemic weight makes it possible to make stabile gullies with slopes of 90 degrees. The depth of these gullies can increase to 40 metres and more. The cement is added to the bentonite to make the liquid impermeable after hydration. The most important disadvantage of this alternative are the costs. One cubic metre of bentonite/cement placed in the dam will cost US$ 75 (Rp 2000) making this alternative much more expensive than the clay alternative.

Since the quantities of clay required for reduction of the seepage length do not vary much for the two alternatives the impermeable revetment is chosen for its feasibility.

Slope protection

The slope protection must be stable under the design conditions and must prevent for loss of base material. In the protection a layer of clay must be integrated to reduce the seepage length. The thickness of this layer of clay is partly determined by the way of construction. The minimum thickness is 0.80 metres. In the design the thickness will be set at 1.0 metres leaving some reserve for construction deviations. The concrete blocks can not be placed directly on the layer of clay, since this layer may not be exposed to daily occurring hydraulic wave loads. Therefore a geo textile and a thin filter layer will be placed between the clay and the concrete blocks. The thin filter reduces the requested dimensions of the stones of the upper layer.

First a simplified design method [lit. (20)] will be applied. When pitched stones on a granular filter are used a subdivision can be made between: good, moderate and poor designs. Here we strive for a good design which implies the following demands (see also the flow-chart of Figure V.1):

- The filter layer is thin: $b/D < 0.5$, where b and D are the thicknesses of respectively the filter layer and the stones.
- The filter material is fine: $D_{f15} < 10$ [mm], where D_{f15} is the particle size of the filter that is not exceeded by 15 % (by weight) of the particles.

- The outer layer is open, the value of Ω (relative opening surface) must be bigger than 3%, and the openings are not filled. For details about the holes see the flow chart.

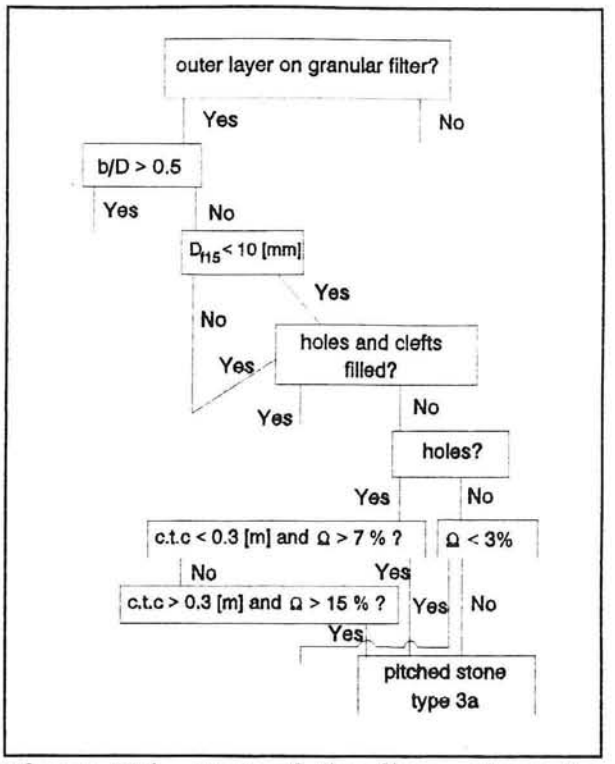


Figure V.1 Part of the flow chart for the construction type

The design can start under the assumption that a good construction will be possible. The simplified design method establishes a relation between the breaker parameter ξ_m and the parameter $H_s/\Delta D$.

First the design-wave must be known in order to compute ξ_m. For the check on the wave run-up $H_{1/3}$ = 2.3 [m] has been used, having a recurrence interval of 50 years. For the loads on the revetment however, the wave with a chance of occurrence of 1/100 per year is chosen:

$H_{1/3}$ = 3.0 [m]
T = 5.5 [s]

The mechanism of the uplifting of a block is ruled by the wave attack and specifically by the way the wave breaks. This process is described by the breaker parameter ξ_m (Eq. V.4).

$$\xi_m = \frac{\tan\alpha}{\sqrt{H_s/L_o}} \qquad \textbf{(V.4)}$$

First the wave-length L_o must be computed. For this relatively short wave the deep-water equation can be used:

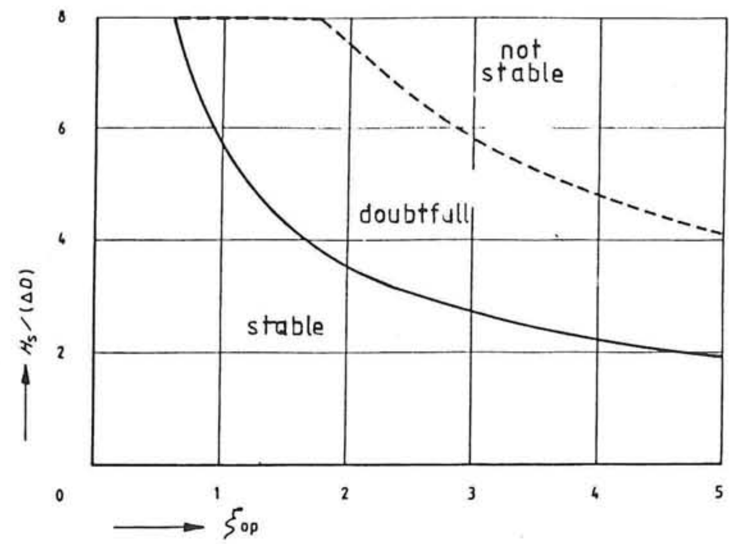
$$L_o = 1.56\ T^2 = 47\ [m]$$

ξ_m becomes:

$$\xi_m = \frac{1/3}{\sqrt{3.0/47}} = 1.32$$

Now the required value of $H_s/\Delta D$ can be read in the graph of Figure V.2. In this case:

$$H_s/\Delta D = 4.6$$



Stones on a granular filter, favourable construction (type 3a) (check the result with the analytical method).

Figure V.2 Design graph for a good construction of stones on a granular filter

The value of Δ is determined using a specific gravity of concrete of 2500 [kg/m³]. Then Δ becomes (2500 - 1020)/1020 = 1.45. A first estimate of the necessary thickness of the blocks is:

$$D = \frac{H_s}{4.6\,\Delta} = \frac{3}{4.6 \cdot 1.45} = 0.45 \quad [m]$$

In a further stage this value must be checked with the analytical method [lit. (20)].

The layer thickness found for the stones implies that the filter thickness is maximum 0.22 [m]. In the design a layer of 0.20 [m] thick is proposed. Stones without holes are chosen here, which implies that the relative open surface Ω must be larger than 3 %. Clefts of 10 [mm] between the blocks satisfy this requirement when blocks are applied with dimensions of 0.60 * 0.60 * 0.45 [m³]. A view of the design is presented in Figure V.3.

As already mentioned the basin-level can be lowered till a level of BM + 2.5 m. for construction purposes. Below this level the supposed constructi- on can not be applied. This asks for a toe-construction. An example of a possible toe-construction is shown in Figure V.4. At the tidal basin side of the basin dam the minimum water level is BM + 4 metres. The layer of rock will have a minimum length of 10 metres ending at BM + 2.3. At the fresh water basin side of the basin dam the minimum water level is
BM + 2 m. With a water level of BM + 2 metres the waves generated in the Gulf of Khambat will have less depth available, the design wave height of 3 metres will never be reached near the dam. But to reduce the danger of sand flowing away with incoming waves at low water-levels, the layer of rock will be carried through over a distance of 30 metres ending at BM + 1 m.

A figure of the cross section of the dam is presented in Figure V.5.

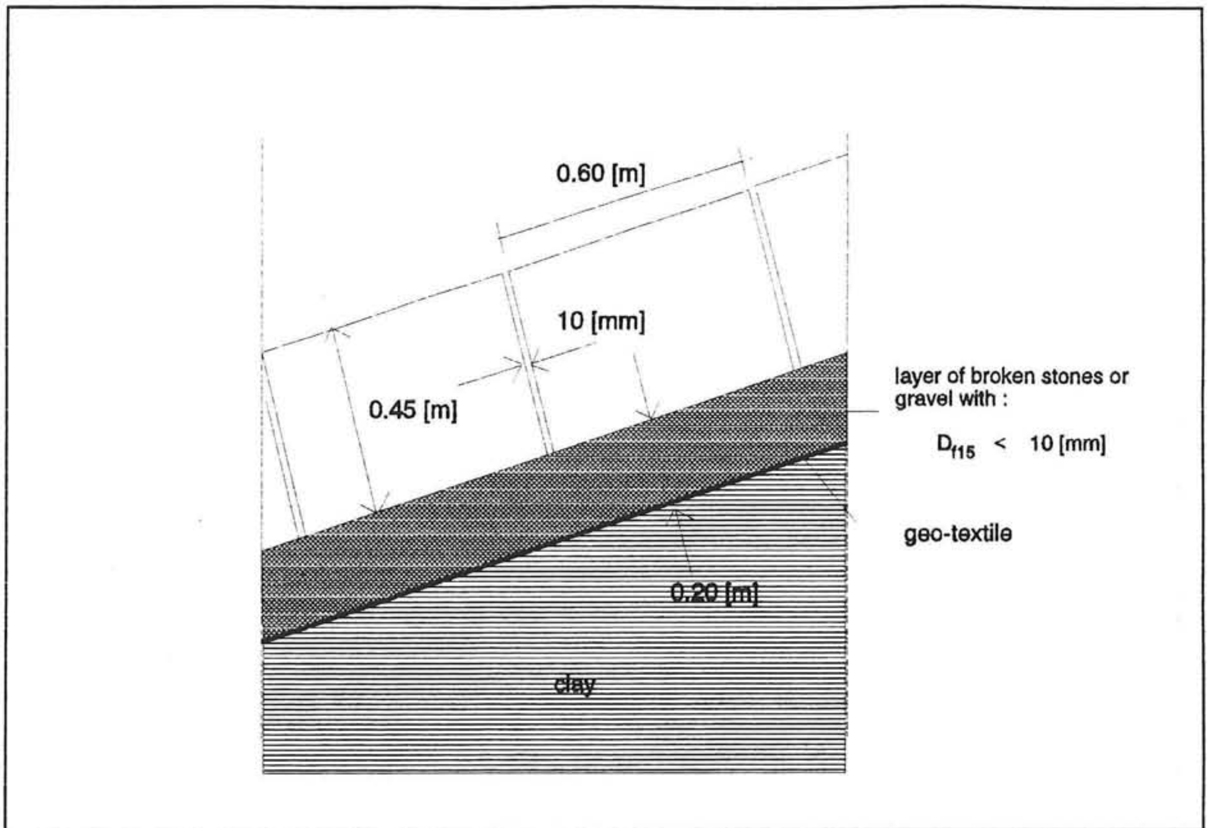


Figure V.3 Slope protection with concrete blocks

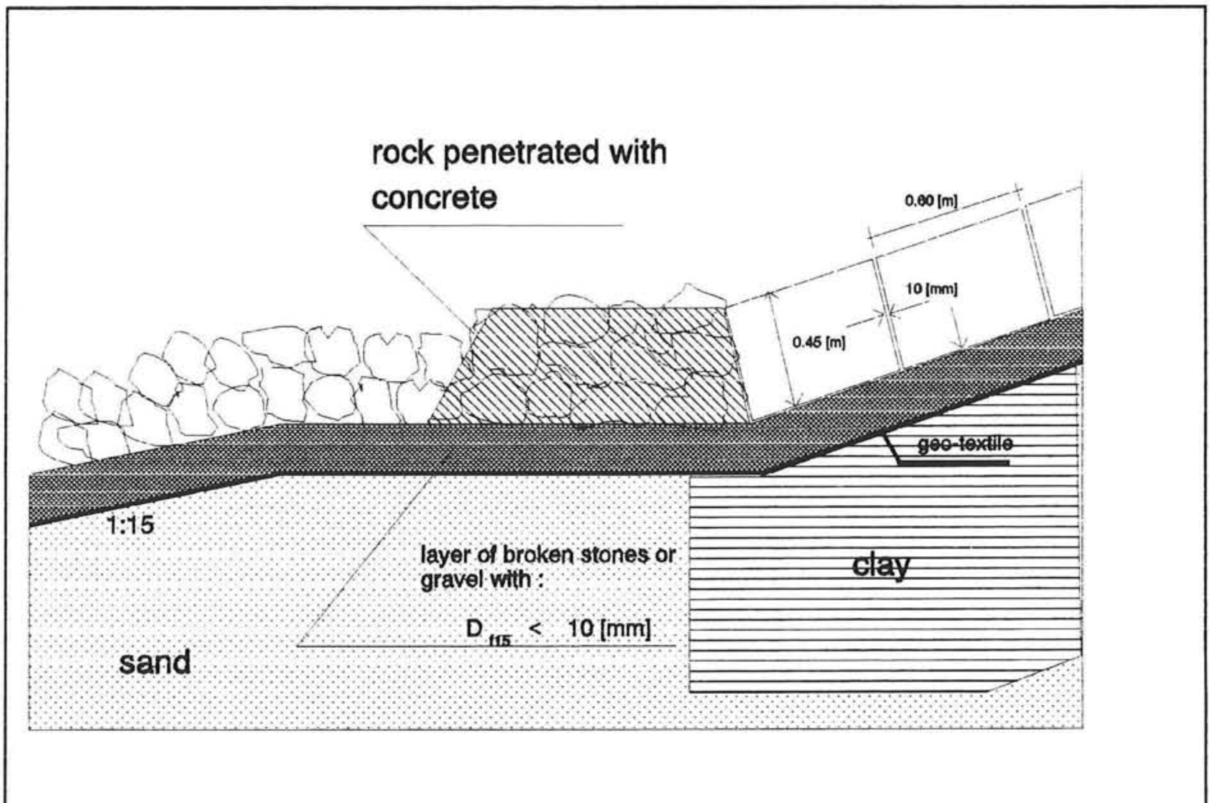


Figure V.4 Example of toe-construction

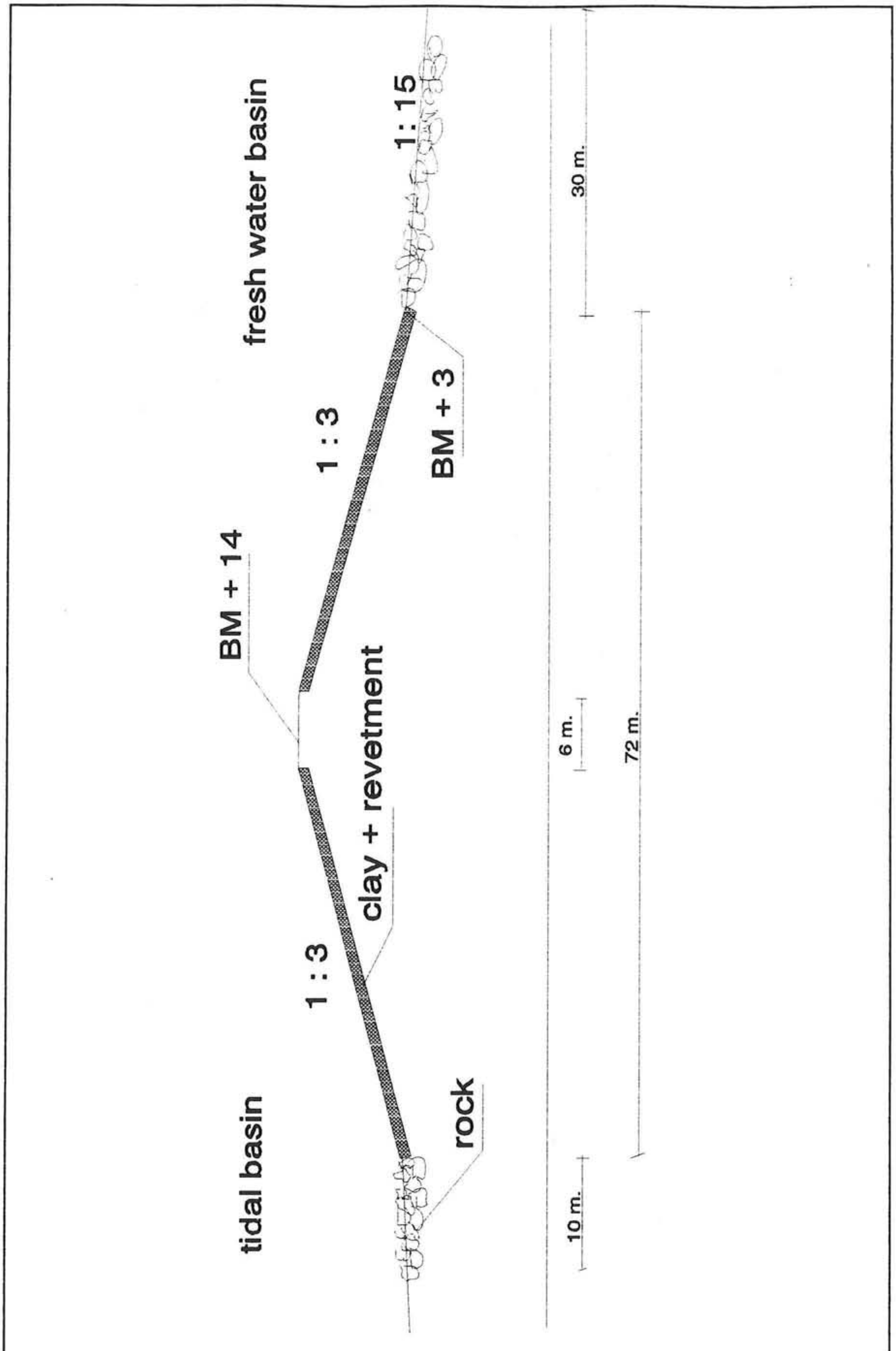


Figure V.5 The cross section of the basin dam

Costs of the basin dam

For the basin dam five different materials are used, the price of these materials is given in Table V.2.

material	price per unit (Rp)	unit
sand	34	[m^3]
clay	62	[m^3]
concrete blocks	110	piece
geo textile	80	[m^2]
layer of broken stones or gravel	100	[m^3]
rock	150	[m^3]

Table V.2 Prices of materials used for construction of the basin dams

The prices of sand and clay have been taken from the Reconnaissance study (lit [5]) with a rate of inflation of 7.5 % annual. The prices of broken stones and of rock have also been taken from the reconnaissance study. The price of rock has been taken as example price, a reduction has been made since these stones have smaller diameters and are easier placeable. The price of a concrete block have been calculated with an average price for one cubic metre concrete of Rp 1080 (US$ 25). The price of the geo textile has been set with the prices known from recent projects.

The costs of the basin dams for basins 1 and 2 will not be calculated since these basins will not be further evaluated for economical feasibility. The costs made for the sand is a simple multiplication of quantity of sand (given in Table V.1) and price of the sand (given in Table V.2). The costs made for the other materials are costs depending only on the length of the basin dam. The quantities of material used for one metre of basin dam is given in the paragraph below.

In one metre of basin dam the length of the revetment including the crest is 76 metres (6 metres of crest and 35 metres of slope on both sides).

Clay:	76 m^2 times 1.0 metres is 76 m^3.
Broken stone or gravel:	76 m^2 times 0.2 metre is 15.2 m^3.
Geo textile:	76 m^2 plus 2 times 5 m^2 extension is 86 m^2.
Concrete blocks:	76 m^2 divided by $(0.6\ m)^2$ is 211 blocks.
Rock:	(10 metres (tidal basin side) + 30 metres (fresh water basin side)) times 1.0 metre thickness is 40 m^3.

The costs of clay and broken stone may be reduced with the price of sand since these materials take the place of sand already accounted for.
The price of one metre of basin dam becomes:

Clay:	76 * (62-34)	=	2130 Rp
Broken stones:	15.2 * (100-34)	=	1000 Rp
Geo textile:	86 * 80	=	6880 Rp
Concrete blocks:	211 * 110	=	23210 Rp
Rock:	40 * 150	=	6000 Rp
Total price per metre[1]		=	39220 Rp (US$ 1450)

The price of the basin dam according alignment 3 is :
43,500 m * 39220 Rp/m + 106 * 10^6 m^3 * 34 Rp/m^3 =
Rp 170 crores + Rp 360 crores = Rp 530 crores (US$ 200 mln).

The price of the basin dam according alignment 4 is :
49,500 m * 39220 Rp/m + 108 * 10^6 m^3 * 34 Rp/m^3 =
Rp 195 crores + Rp 365 crores = Rp 560 crores (US$ 210 mln).

Annex VI Turbine characteristics

The turbine is the most essential part of the tidal power station. It translates the energy of the water in the basin to electric energy. For a turbine many characteristics have to be known to determine what the output of the turbine will be when it is placed. To obtain these characteristics calculations and tests are made in the research centra of the manufacturers. These tests and calculations are so important for the market value of a turbine that these figures are kept secret for as long as is possible. Only when a concrete plan for a structure with turbines is presented by a known company or institute, the turbine characteristics are given with the restriction of secrecy to outsiders.

Therefore the characteristics of a turbine used by S. Delfgaauw for a tidal power scheme in the Wyre Estuary (England) are used as basis model for a turbine in the Gulf of Khambat.The characteristics of this basis turbine are presented in figure VI.1. The exact efficiency curves of this turbine are not known. In the turbine diagram an efficiency is presented of 85 % average. In the Wyre project the turbine was also used as pump when the basin was filled. In the project in the Gulf of Khambat the turbines are not used as pumps, therefore a higher efficiency of the turbines is suggested: 90 % average. The possibility of using detailed efficiency curves in the Duflow model is very limited

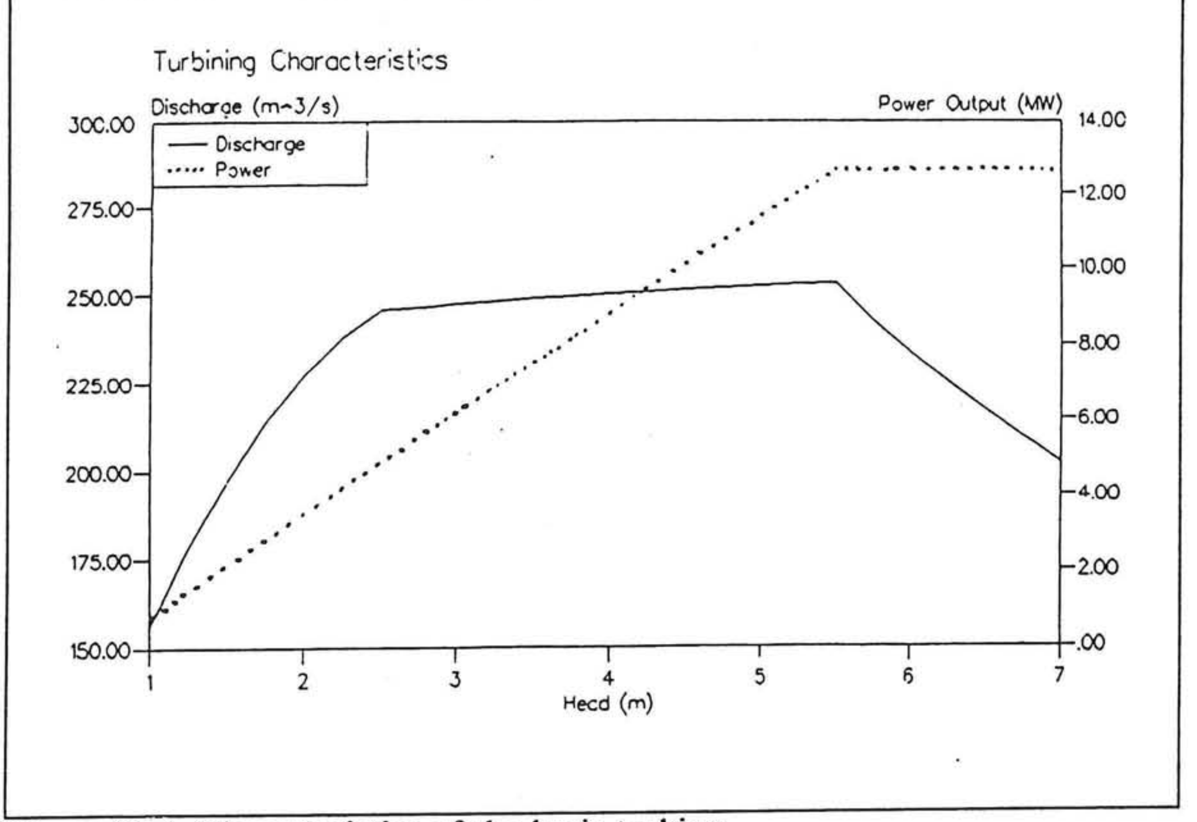


Figure VI.1 Characteristics of the basis turbine

This basis turbine has a diameter of 6.0 metres, and a design head of 5.5 metres. With these dimensions a maximum discharge can be processed of 250 m^3/s, resulting in a maximum net power output of 12,5 MW. The rotational speed of the turbine is 58.9 rpm. In the Gulf of Khambat the working conditions of the turbines differ much from the conditions in the Wyre. In the Gulf of Khambat the number of turbines is much higher than in the Wyre, therefore the diameter of the turbines can be enlarged to the maximum possible diameter. In the present technology the maximum diameter for turbine with a horizontal axis is 8 metres. This diameter is chosen for this project in order to reduce the number of turbines. Disadvantages of such large diametered turbines is the sensitivity of the structure for vibration, (this sensitivity determines the maximum attainable diameter), and the depth of the total structure. When a larger diameter is used for the turbine, the foundation of the power-house has to go down the difference in diameter. One half times the difference because the axis of the turbine has to be located on the same hight, and a second half times the difference because the powerhouse structure below the turbine equals half the size of the turbine diameter. Since the subsoil of the Gulf of Khambat consists mostly of sand a deeper foundation of the power house does not form a major constraint. It would become a problem if the subsoil would consist of rock.

With this new diameter for the turbine a new set of characteristics for the turbine can be calculated with the help of the two expressions given below in equations VI.1 and VI.2. These equations give the connection between similar turbines for different diameters (D), different heads (H), different rotation speeds (n) and different maximum discharges (Q).

$$\frac{Q}{n\,D^3} = constant \qquad \textbf{(VI.1)}$$

$$\frac{n^2\,D^2}{H} = constant \qquad \textbf{(VI.2)}$$

If the value for the diameter of the basis turbine is changed, the values of the rotational speed and the maximum discharge will change too. Using equation VI.2 the rotational speed becomes:

$$\frac{(58.9)^2\,(6.0)^2}{5.5} = \frac{n^2\,(8.0)^2}{5.5}$$

$$n^2 = (58.9)^2\,\frac{(6.0)^2}{(8.0)^2}$$

$$n = 44.2\ rpm$$

And the discharge can be calculated with equation VI.1:

$$\frac{250}{(58.9)\ (6.0)^3} = \frac{Q}{(44.2)\ (8.0)^3}$$

$$Q = \frac{(44.2)\ (8.0)^3}{(58.9)\ (6.0)^3}\ 250$$

$$Q = 445\ m^3/s$$

If the design head over the turbine also changes and is set as a variable, a new equation can be made. The equations VI.1 and VI.2 first are rewritten as:

$$\frac{Q}{n} = constant \quad \wedge \quad \frac{(n)^2}{H} = constant$$

Then the two equations can be combined to

$$Q_x = Q_{5.5\ m}\ \frac{n_x}{n_{5.5\ m}}$$

$$in\ which\ Q_{5.5\ m} = 445\ m^3/s$$

$$and\ \frac{n_x}{n_{5.5\ m}} = \sqrt{\frac{H_x}{H_{5.5\ m}}} \qquad \textbf{(VI.3)}$$

$$Q_x = 445\sqrt{\frac{H_x}{5.5\ m}}\ \ m^3/s$$

$$or\ Q_x = 190\ \sqrt{H_x}\ \ m^3/s$$

The gross power of a turbine is a product of the discharge, the head and the specific gravity of the water flowing through the turbine.

$$P_{turbine} = \rho * g * H * Q(H)$$

$$\rho = 1020\ kg/m^3$$

$$g = 9.8\ m/s^2$$

$$Q(H) = 190\ \sqrt{H}\ \ m^3/s$$

$$P_{turbine} = 1.9 * 10^5\ \sqrt{H^3}\ \ W = 0.19\ \sqrt{H^3}\ \ MW$$

Both equations for power of and discharge through the turbine can be set out in a graph, this is done so in Figures VI.1 and VI.2

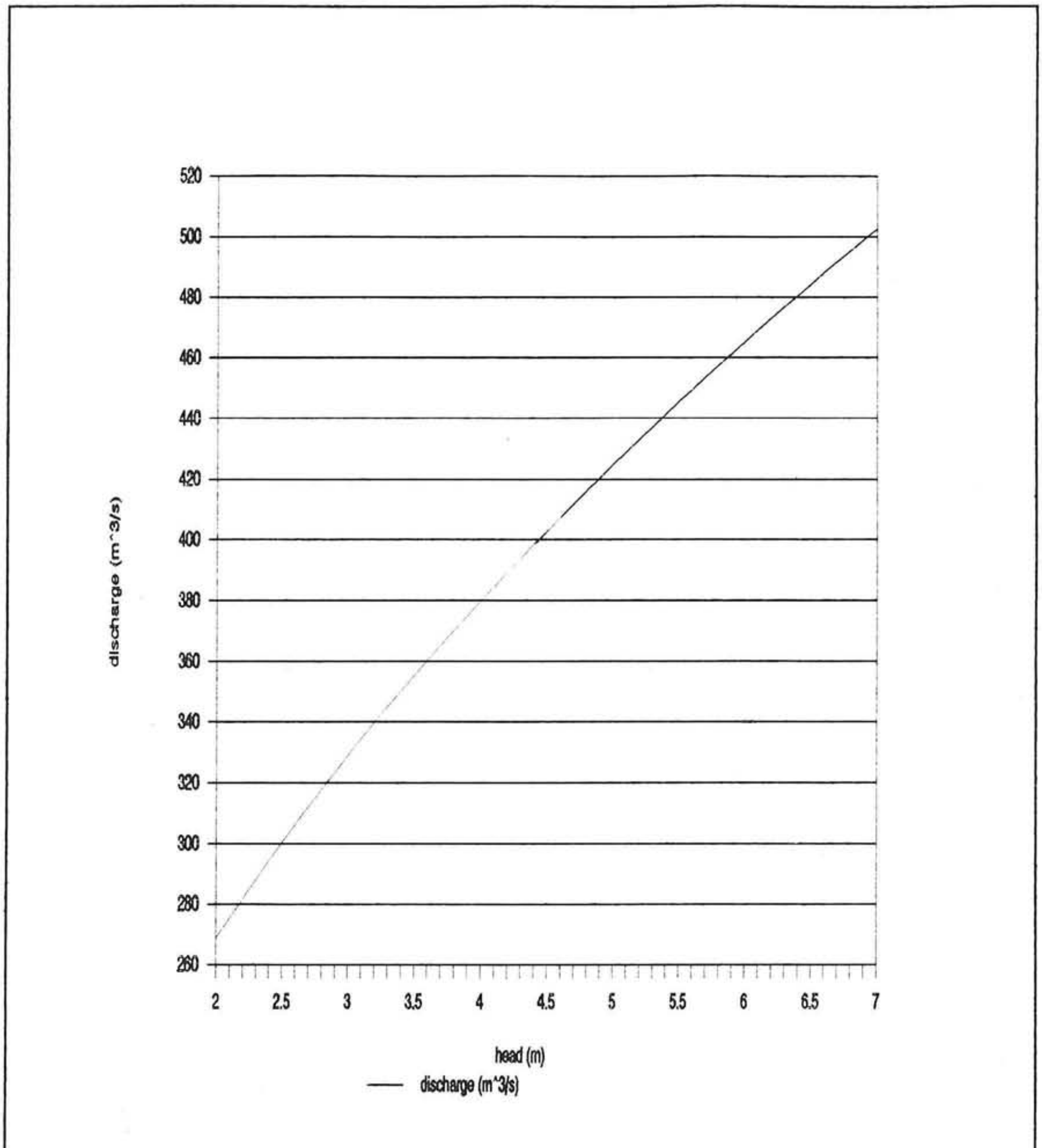


Figure VI.2 Maximum discharge as variable of the design head

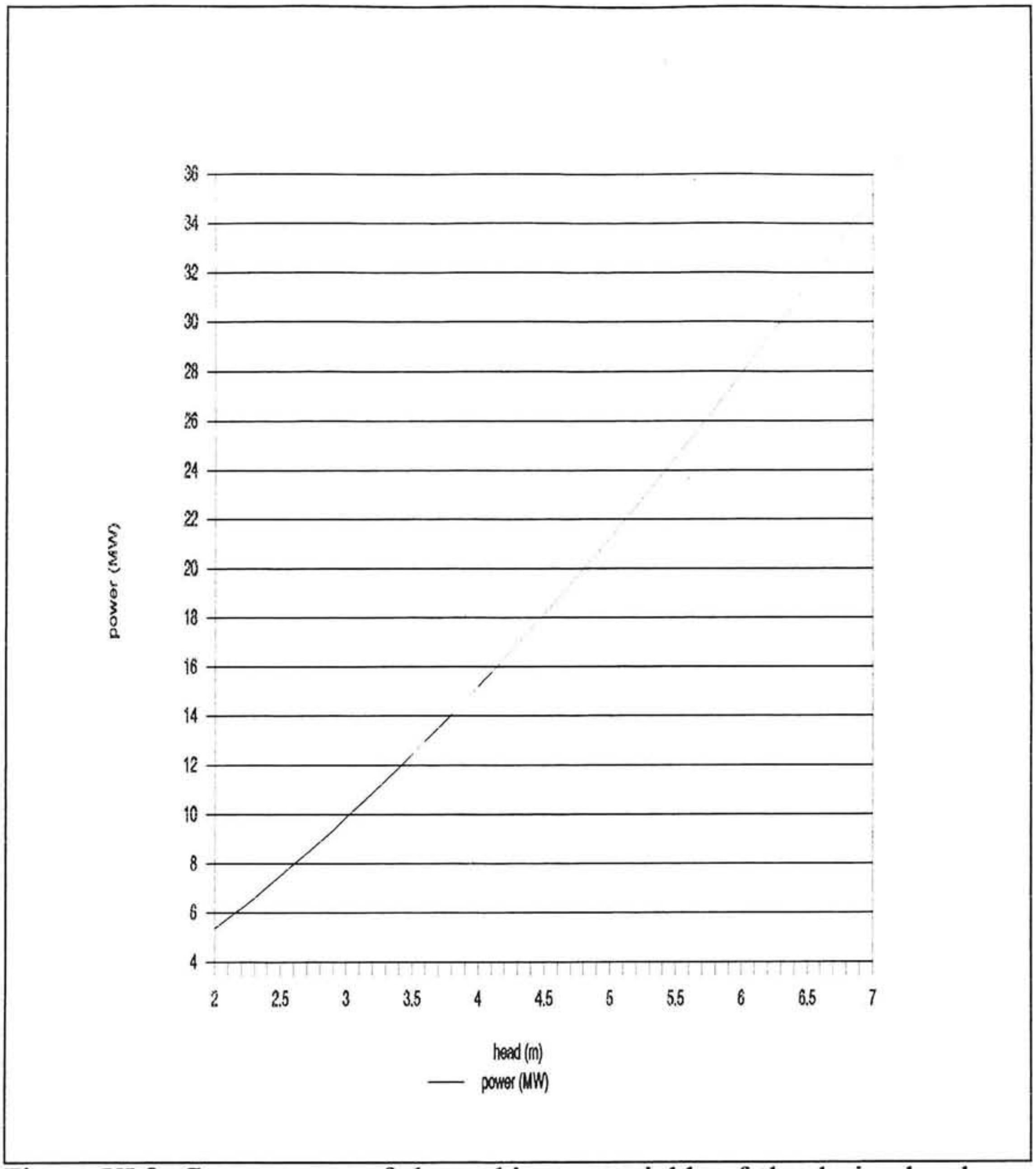


Figure VI.3 Gross output of the turbine as variable of the design head

Annex VII Aspects of the design of the powerhouse caisson

The dimensions of the powerhouse caisson are very much determined by the dimensions needed for the turbines placed in it. The turbine used has a diameter of 8 metres. This turbine requires an draft tube with a length of 7.5 times the diameter size which amounts to a length of 60 metres. Three times the diameter is needed for the entrance tube. Four and a half times the diameter of the turbine is needed for the exit tube.

The prescribed entrance has a width of 14 metres, and a hight of 18 metres resulting in an entrance area of 252 m^2. The prescribed exit has a width of 14 metres, and a hight of 15 metres resulting in an entrance area of 210 m^2. In the design of the powerhouse caisson these dimensions have been slightly changed in able to get more turbines in one caisson. The entrance has been given a width of 13 metres and a height of 19 metres, the entrance area almost remaining the prescribed 252 m^2. The exit has been given a width of 13 metres and a height of 16 metres, the exit area remaining the prescribed 210 m^2. With this reduction in width the width of the caisson can be decreased offering place to the same number of turbines.

The axis level of the turbine is set at BM - 12 metres to avoid cavitation. This level can only be calculated if all dimensions and all characteristics of the turbine and the powerhouse are known. In this case an estimation has been made from information given in the report of S. Delfgaauw (lit [23]). With this axis level the place and size of the entrance and exit area are known and a first sketch can be made of the front and back of the caisson. These sketches are given in Figure VII.1 and Figure VII.2.

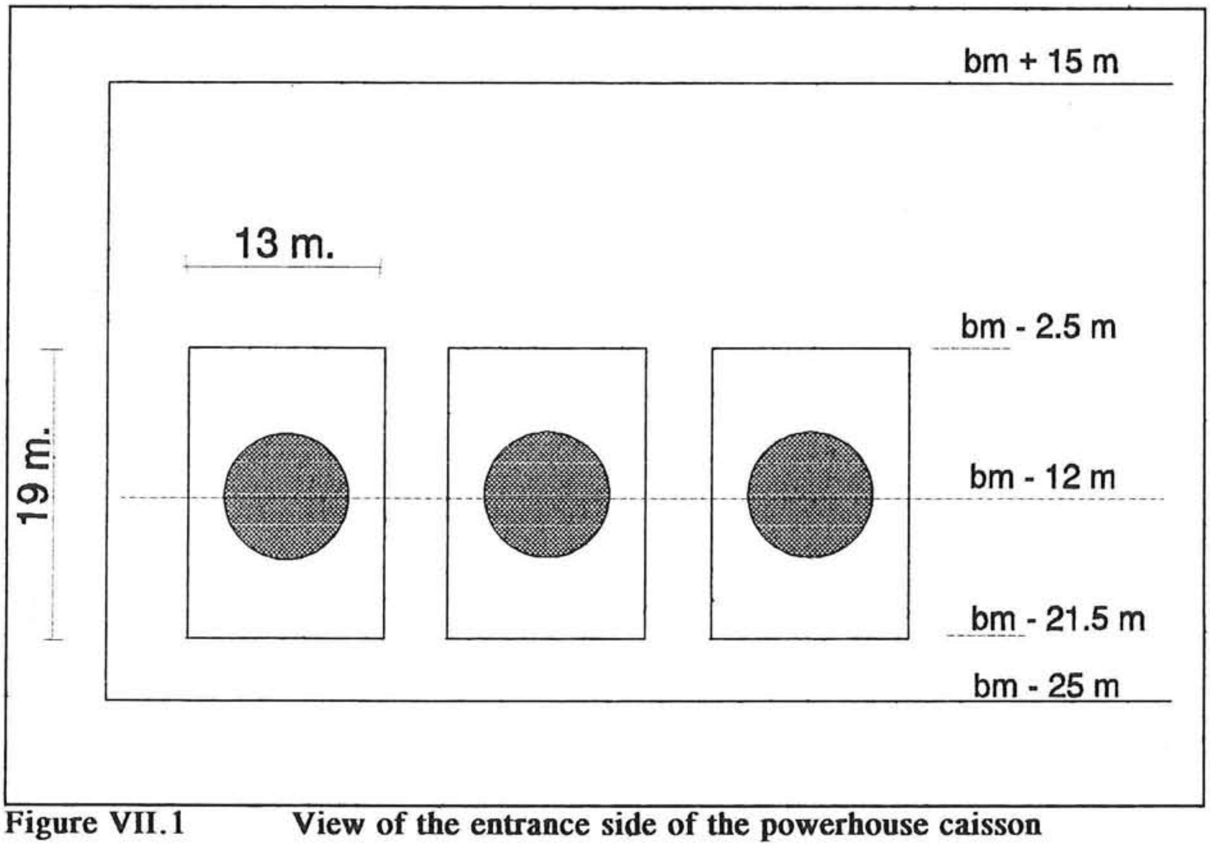


Figure VII.1 View of the entrance side of the powerhouse caisson

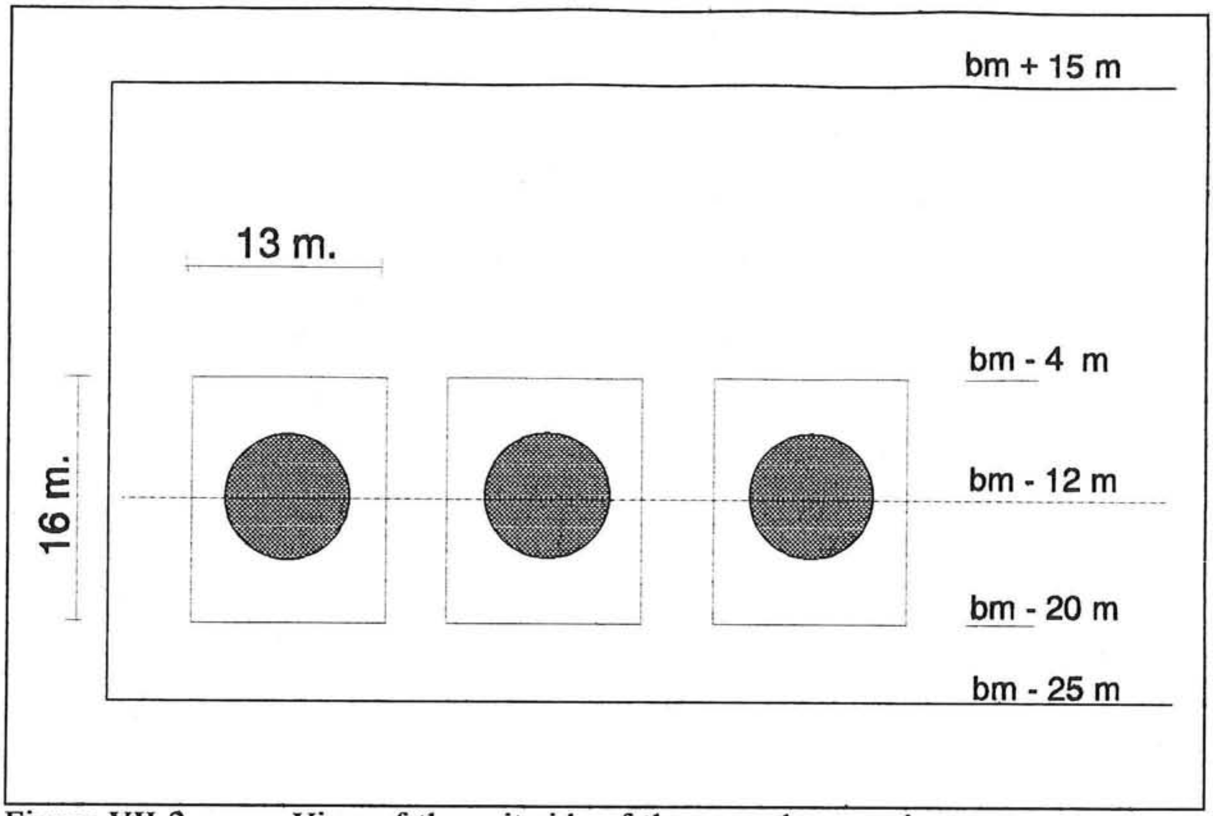


Figure VII.2 **View of the exit side of the powerhouse caisson**

The dimensions of the draft tube are given in two cross sections of the power house caisson, Figures VII.3 and VII.4.

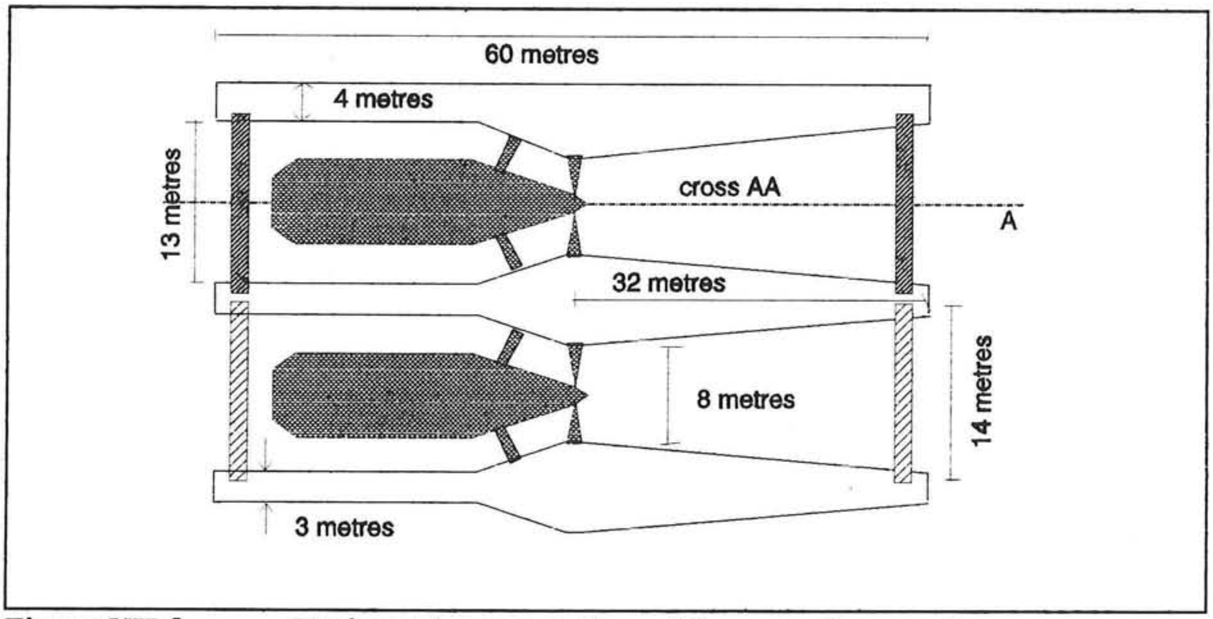


Figure VII.3 **Horizontal cross section of the powerhouse caisson**

The vertical cross section A-A is given in Figure VII.4.

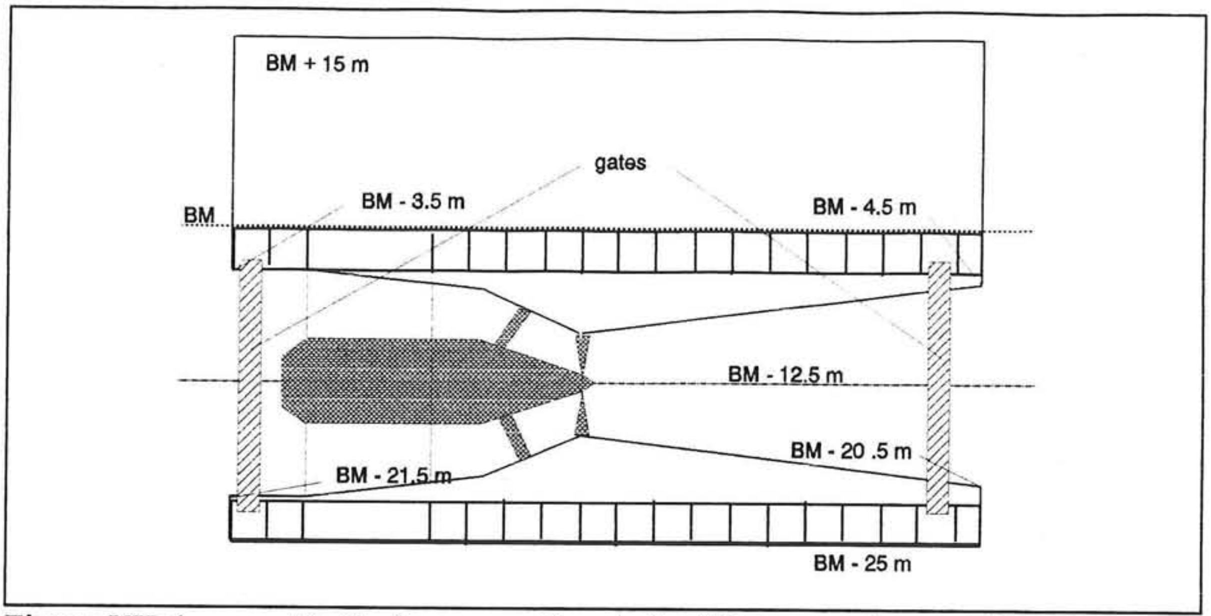


Figure VII.4 **Vertical cross section A-A of the power house caisson**

The roof of the power house caissons has a width of 60 metres. This width has to be used for the entrance to the generator and turbines, and for the double track railway and the road. In section 4.6 of the main report the dimensions for these railway and road are estimated with a width of 25 metres. This leaves 35 metres for the entrance to the generator and turbines, and a crane used for placing the generator and turbines. A possible division of the available space is given in Figure VII.5.

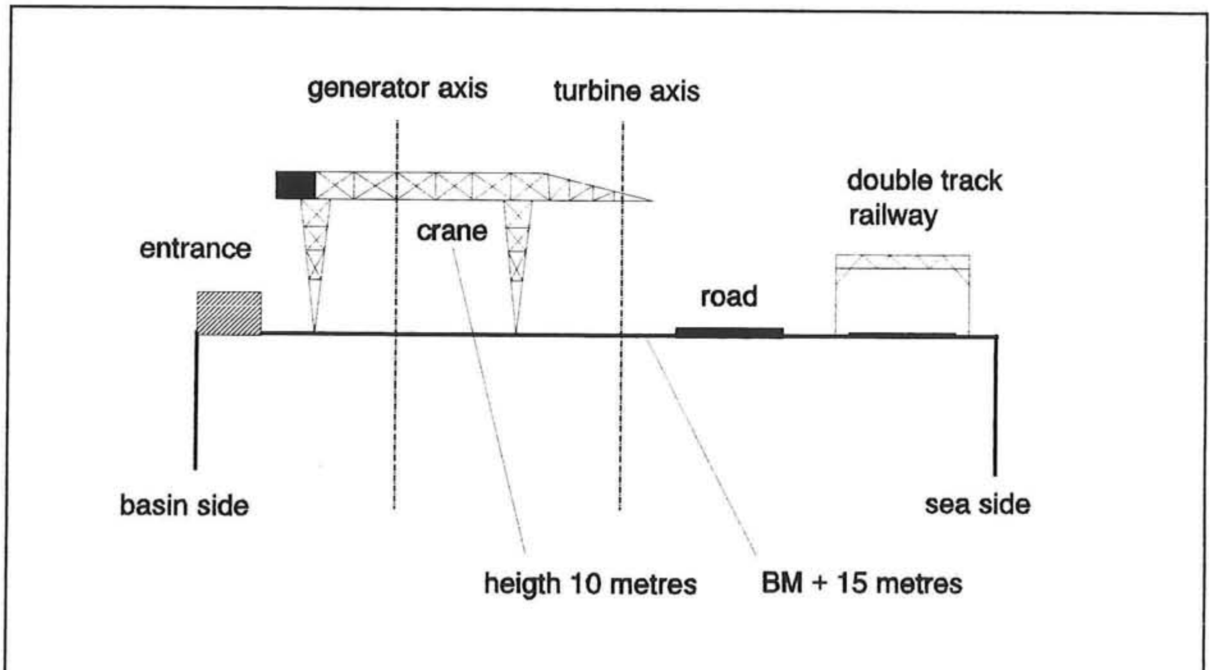


Figure VII.5 **Division roof of power house caisson**

For the rails of the crane and the rails of the railway a special construction has to be made in the caisson since the pressure of the axles of the train and the crane can be considerable. In Figures VII.3 and VII.4 assumptions have been made for the gates and the thickness of the wall at the bottom of the caisson and the walls between the draft tubes. In the next paragraphs these assumptions will be roughly checked.

The load on the walls has its extremes for the different parts of the structure for different load combinations. The possible extreme loads are:

1 a high or a low water level at the sea side of the turbine caisson
2 a high or a low water level at the basin side of the turbine caisson
3 a draft tube set dry for maintenance
4 the transport and placing of the caisson
5 the load on the roof of the caisson (as result of trains or the crane)

The minimum water level at the sea side of the caisson is BM - 2 metres, the minimum water level at the basin side of the caisson is BM + 2 metres. The maximum water level at the sea side of the caisson is BM + 12 metres, the maximum water level at the basin side of the caisson is also BM + 12 metres.

The combinations of high and low water at both sides of the turbine caisson are highly correlated. A maximum water level at the sea side of the caisson is always correlated to a high basin water level. A reduction in the basin water level to BM + 11 metres (see design of the basin dams) will not be accepted for the design of the power house caisson. A water level reduction to BM + 11 metres only is realised if a high basin level is expected in combination with extreme wind speeds.

Design of gates

The maximum load on the gates occurs if the draft tube is set dry for maintenance on the turbine, and the water levels at either side of the caisson reach their maximum. The load on the gates is transferred to the walls between the draft tube. This is the shortest route to the two possible set of lines of support. The first possible set of support-lines are the horizontal lines at the bottom and the top of the draft tube. The distance between these two lines is 19 metres. The second possible and chosen set of support-lines are the support lines at both sides of the draft tube. The distance between these to lines is only 14 metres. A scheme of the gate structure is given in Figure VII.6.

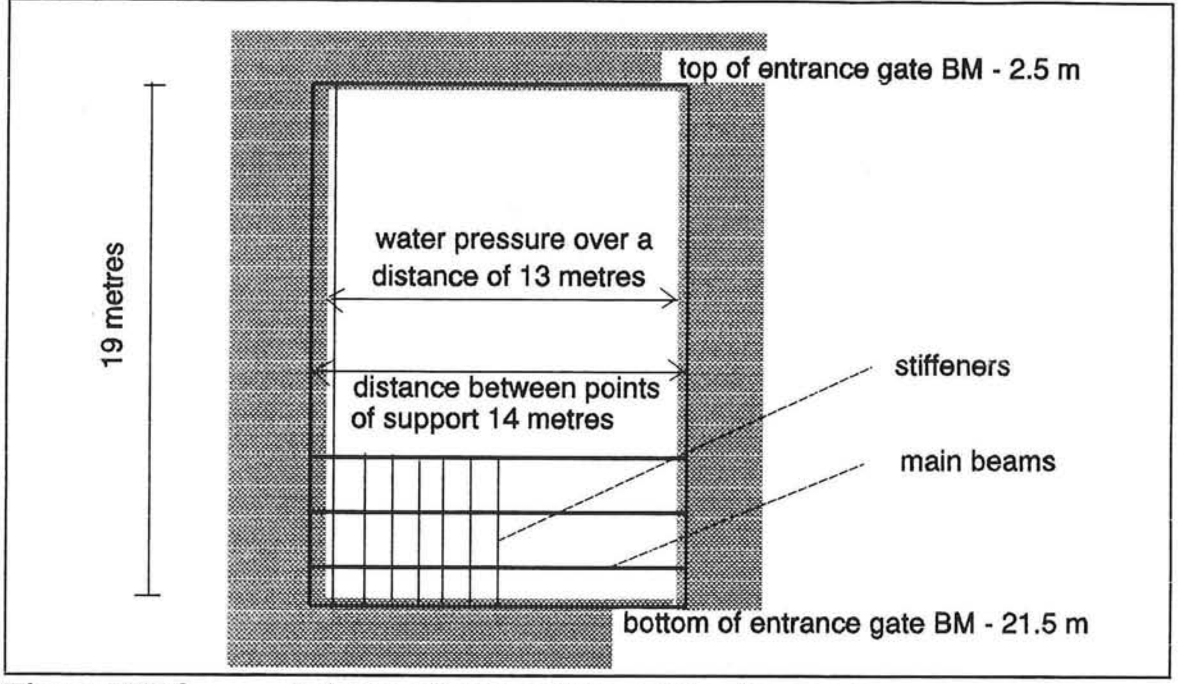


Figure VII.6 Scheme of gate and its points of support

The place of the draft tube gate in the draft tube can be changed only slightly in case of the entrance gate. In case of the exit gate more variety is possible. The gate can be placed at the end of the draft tube. The distance between the support lines remains the earlier mentioned 14 metres. The exit gate however can also be placed nearer to the turbine reducing the height as well as the span of the gate. The result is a smaller and much less heavy gate.

Disadvantages of the location nearer to the turbine are a more disturbed flow pattern of the water leaving the turbine, and the necessity of using temporary gates closing of the rest of the draft tube during transport. The result of the disturbance in the flow pattern of the water is a less efficient turbine. Extra advantages of the location nearer to the turbine are a higher horizontal pressure on the bottom, and a lighter structure around the part of the draft tube which will not be set dry. Around the draft tube a heavy comb structure in needed to take the load of the water in the neighbouring draft tubes. There where the draft tube is still filled with water a lighter structure can be taken. The horizontal pressure of the water in the not closed part of the draft tube reduces the necessary friction coefficient to transfer eventual horizontal forces to the bottom. The stability of the caisson becomes better as result of a location of the exit gate nearer to the turbine.

A more detailed study should give answers to what location of the gate is preferable. In this annex the location of the gate is set at 10 metres distance of the turbine. The height of the gate becomes 12 metres instead of 17 metres at the end of the draft tube. The span of the gate becomes 11 metres instead of 14 metres. This reduction in span leads to a reduction in moment of 38 %.

The maximum load on the gate occurs when the gate is closed and the draft tube behind it is set dry. The water level outside has a maximum of BM + 12 metres. For the design of the gate a lower water level could be accepted since the chance of this water level is very small. A regulation should then be made to not set dry the draft tube in case of an expected extreme water level. In this Annex the extreme water level is taken to rule out the chance on mistakes or to have a reserve to a possible water level rise.

If the water level rises to BM + 12 metres (the water level during an average spring is BM + 8.7 metres) the water pressure at the lowest point of the entrance door is 33.5 metres. If the span of the door is 14 metres (a entrance width of 13 metres and 0.5 metres distance to the point of support) the moment in the middle of the span is:

$$M = 1/8\ q\ l^2$$

$$l = 14\ metres$$

$$q = 32.75\ m * 9.8\ m/s^2 * 1020 kg/m^3$$

$$q = 327.5\ kN/m^2$$

$$M = 1/8 * 327.5\ kN/m^2 * (14m)^2$$

$$M = 8023\ kNm$$

The maximum moment in the door changes with the height. At the top of the entrance gate the maximum moment is 3553 kNm/m^1. The course of the load and the maximum moment and the maximum shear force on the entrance gate is given in Figure VII.7.

The maximum moment in the much smaller exit gate is limited to 4436 kNm at the bottom of the gate, and 2823 kNm at the top of the gate. To withstand that kind of moment the closure door has to be made from steel. The maximum stress steel can have is dependable of the quality of the steel. If a quality Fe 510 is taken, the maximum stress is 360 N/mm^2.
The W of the entrance gate must have a value of $1.5 * 8023 * 10^6$ Nmm divided by 360 N/mm^2 is $3.34 * 10^7$ mm^3 per metre at the lowest point of the door. The W required for the doors can not be provided by normal profiles. The maximal W of standard profiles is $1.43 * 10^7$. These profiles would have to lie almost side to side to satisfy. A profile will have to be made by welding plates of steel.

If the profile is given a width of 1.5 metres the moment in the lowest profile would become $1.5 * 1.5 * 8023$ kNm = $18.1 * 10^3$ kNm (inclusive safety factor of 1.5). The required W for the profile is $5.0 * 10^7$ mm^3. The profile given in Figure VII.8 has a W of $5.20 * 10^7$ mm^3. The upper part of the profile is a plate with a thickness of 35 mm. The tension in the lower plate (40 mm) is $18{,}051 * 10^6$ Nmm / $5.21 * 10^7$ mm^3 = +347 N/mm^2. The tension in the upper plate, the plate holding back the water, is $18{,}051 * 10^6$ Nmm / $7.99 * 10^7$ mm^3 = -226 N/mm^2.

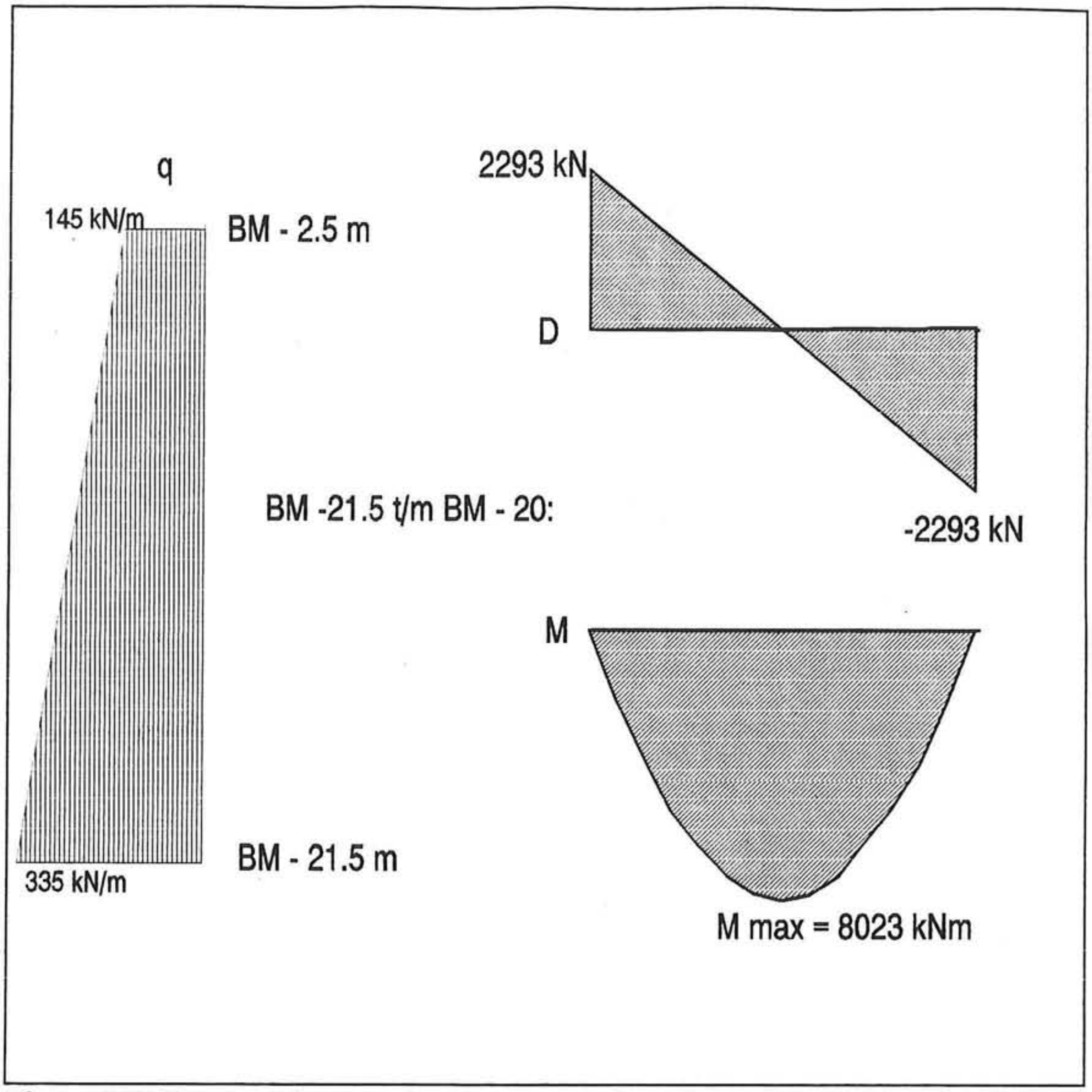


Figure VII.7 **Course of moment and shear force**

The shear force gives a shear stress in the z-direction. The force only gives a shear stress in the vertical plate of the beam. The surface of this plate is 1500 * 40 mm^2. The shear stress is maximal near the points of support and has a size of 1.5 * 2293 * 10^3 N/ 60,000 mm^2 = 57 N/mm. The profile has enough surface to resist this shear force. Even the combination of the maximum shear stress and the maximum stress caused by the moment in the beam (not taking place in the same part of the beam) could be absorbed by the beam given in Figure VII.8. The maximum combined stress is:

$$\sigma_{e,\max} = 347\ N/mm^2$$

$$\sigma_{s,\max} = 57\ N/mm^2$$

$$\sigma_{c,\max} = \sqrt{\sigma_{e,\max}{}^2 + 2 * \sigma_{s,\max}{}^2} = \sqrt{(347\ N/mm^2)^2 + 2 * (57\ N/mm^2)^2}$$

$$\sigma_{c,\max} = 356\ N/mm^2 < \sigma_{\max} = 360\ N/mm^2$$

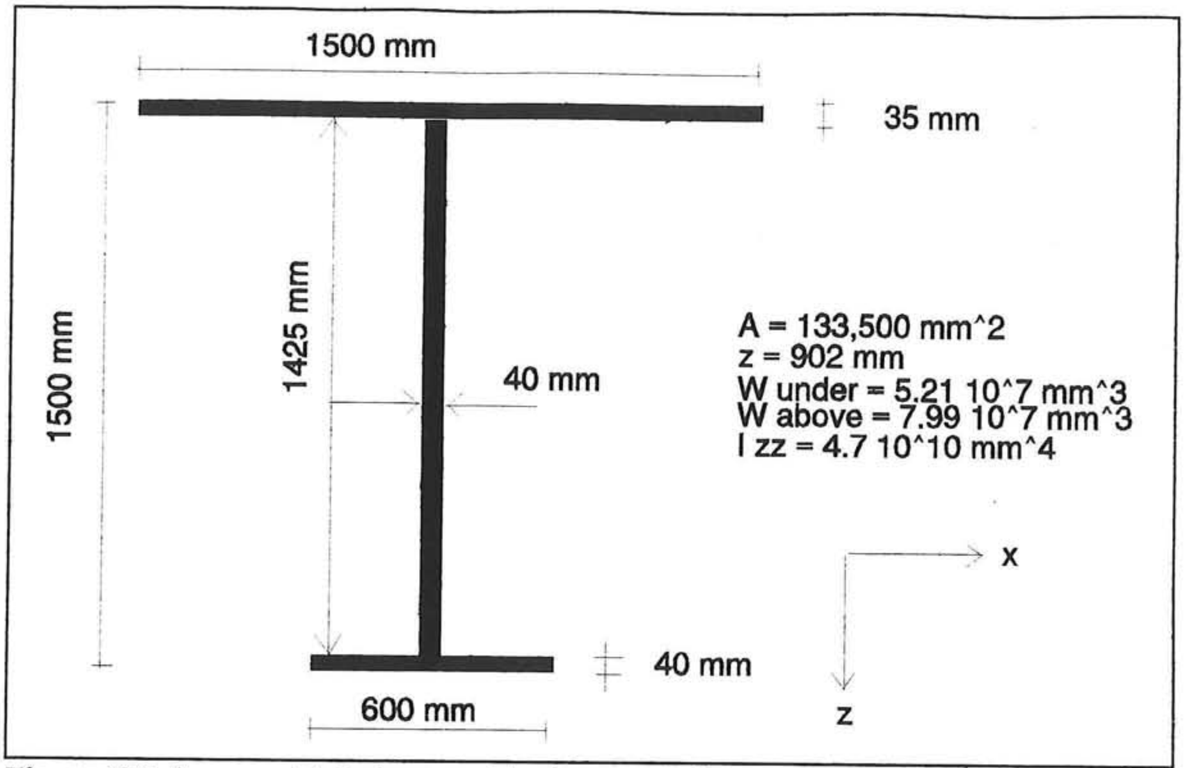
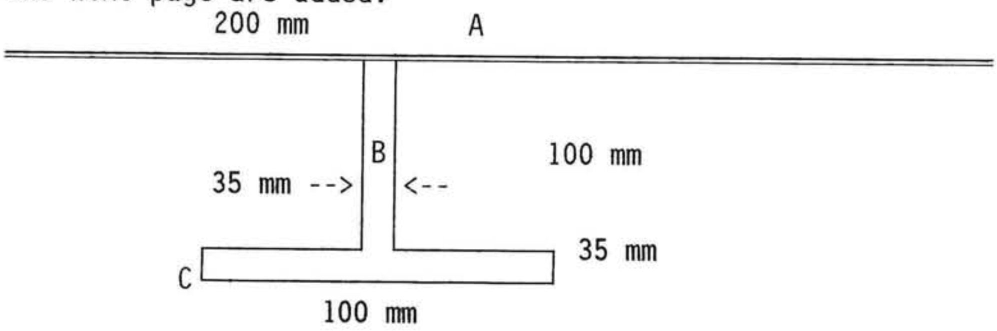


Figure VII.8 **Main beam with characteristics**

To resist the water pressure on the plates between the beams extra strength and stiffness is required. Perpendicular to the main beam profiles as drawn on the next page are added.



These stiffeners are placed at 0.75 metres from each other reducing the span of the plate holding back the water. The necessary dimensions of the stiffeners is dependable of the depth on which they are placed. At the lowest point of the door the length of the stiffeners is 1.5 metres and the load 0.75 metres times the load of average 30 metres water pressure.

$q = 1.5 * 0.75 \text{ m} * 1020 \text{ kg/m}^3 * 9{,}8 \text{ m/s}^2 * 30 \text{ m}$

$q = 337 \text{ kN/m}$

The maximum moment in these stiffeners is:

$M = 1/8 * q * l^2$

$M = 0.125 * 337 * 1.5^2$

$M = 95 \text{ kNm}$

If the thickness of the stiffener is 35 mm (the same as the plate on which it is attached) and the hight of the stiffener is 100 mm, the W of the stiffener is $417 * 10^3$ mm^3, resulting in a tension in the outer plates of 227 N/mm^2.

At the top of the gate the load on the door is only 50 % of the load at the bottom of the gate. The main beams can be placed at the double distance compared to the bottom of the gate. The resulting moment on the stiffeners increases to:

$$q = 1.5 * 0.75 \text{ m} * 1020 \text{ kg/m}^3 * 9{,}8 \text{ m/s}^2 * 15 \text{ m}$$
$$q = 169 \text{ kN/m}$$

$$M = 1/8 * q * l^2$$
$$M = 0.125 * 169 * 3.0^2$$
$$M = 190 \text{ kNm}$$

The stiffeners with a height of 100 mm do not comply with the moment of 190 kNm. If a height of 110 mm is chosen the value of W increases to 5.5 10^5 mm^3. The tension in the outer plate C rise to 318 N/mm^2. The tension in the gate plate A is maximal in the middle of the span and is 212 N/mm^2. The shear force in the stiffeners causes no problems.

To use only one sort of stiffeners for one gate the stiffeners with a height of 110 mm will be used for the top as well as the bottom of the gate. The difference in use of steel is to little to make a changing size of the stiffeners. The structure scheme of the entrance gate is given in Figure VII.9

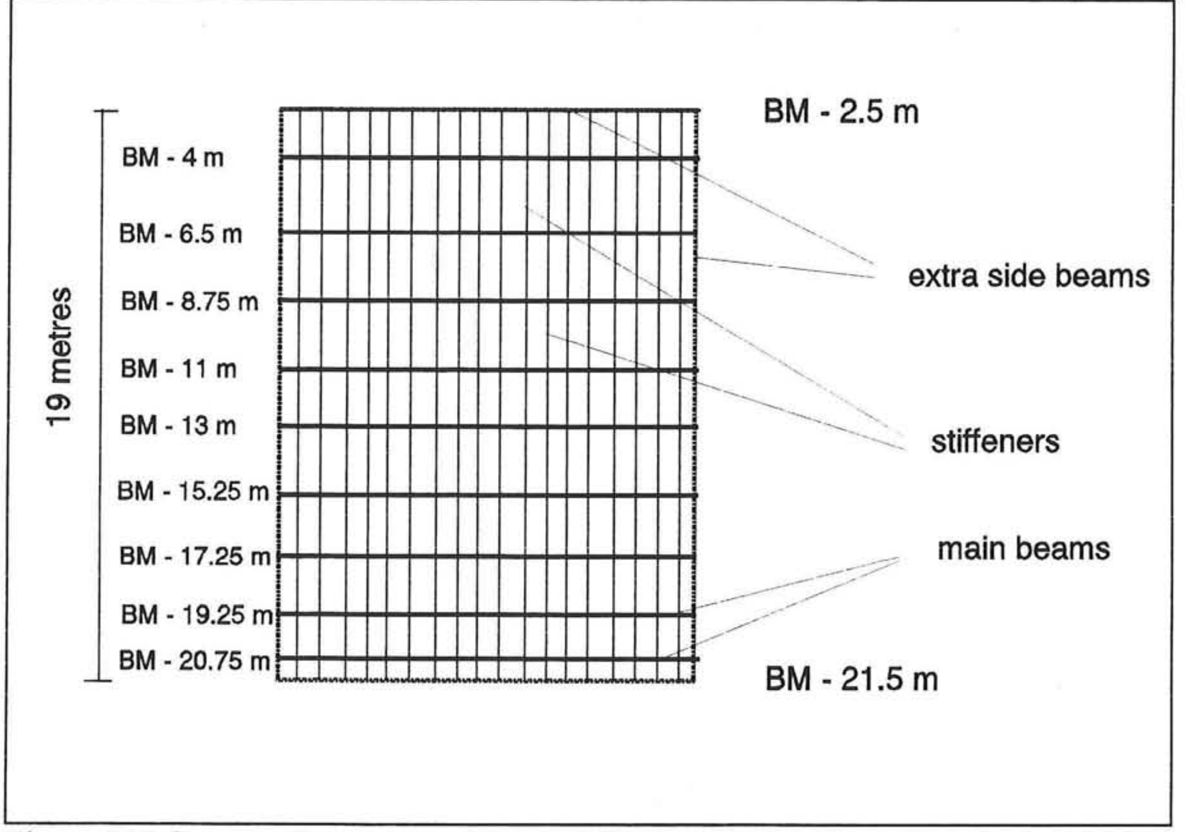


Figure VII.9 **Structure scheme of the entrance gate with main beams and stiffeners**

To secure a watertight closure special precautions have to be made at the sides of the gates, as well for the entrance gate as for the exit gate. These special precautions consist of an extra beam at the sides of the gate, making sure that the sides, the bottom and the top of the gates will not bend to much. If the bending is to much a watertight closure is not possible, with the beams small cracks however will still exist.

These cracks have to be closed with slabs at the side and the bottom of the gate. The slab at the top of the gate has to be attached to the draft tube. The position of the slabs on the gate is given in Figure VII.10.

The maximum moment in the much smaller exit gate is limited to 5220 kNm (compared to 8023 kNm for the entrance gate). The main beam can therefore be less high and less heavy. The dimensions of this beam:

top plate holding back the water	: 1500 mm * 35 mm
vertical plate	: 35 mm * 1130 mm
bottom plate	: 600 mm * 35 mm
total height of the door	: 1200 mm

total surface : 113,050 mm^2
$z = 761$ mm
$W_{top} = 5.95 * 10^7$ mm^3
$W_{bot} = 3.43 * 10^7$ mm^3
$I = 2.62 * 10^{10}$ mm^4

The stiffeners and the side beams can also be lighter (less load and less height). The result is a gate with a weight of 100,000 kg compared to 200,000 kg for the entrance gate. The costs of the exit gate is estimated at US$ 400,000 (US$ 4.00 per kg), the costs of the entrance gate is estimated at US$ 800,000.

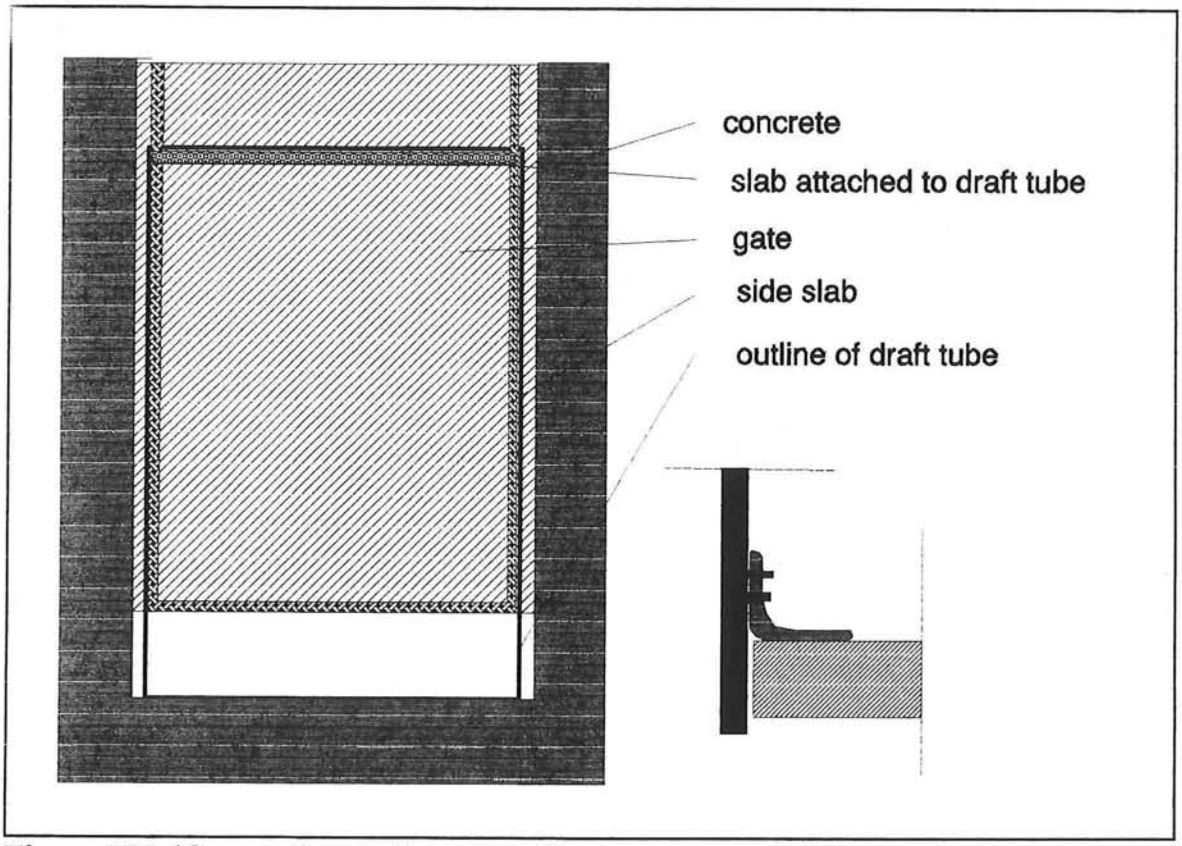


Figure VII.10 Precautions to make the gate watertight

Design of the vertical caisson walls between the draft tubes.

The draft tubes in the power-house caisson are surrounded by a comb-structure as showed in Figure VII.11. In the next pages a limited outline design of the verticals and bottom of this comb structure shall be made. The design shall be based on the load on the structure. In a more detailed calculation the consequances of the cracks and the salt water intrucion in the cocrete must be handled. The quality of the concrete is B35, the quality of the reinforcement steel is FeB 500.

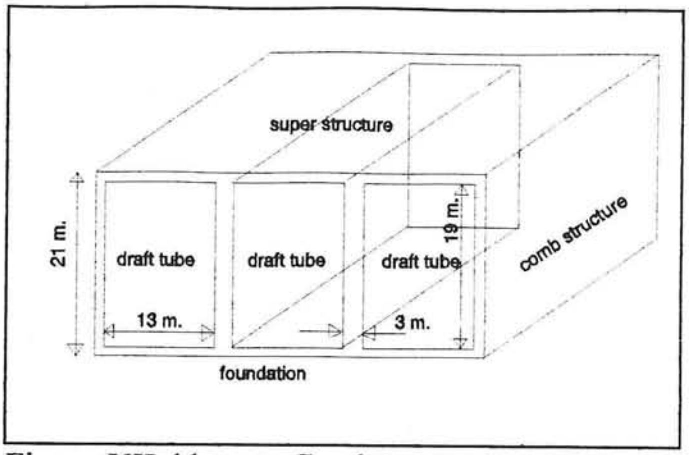


Figure VII.11 Comb structure of power house caisson

Inside the combs the exact dimensions of the draft tube are realised. Above the comb-structure the superstructure of the caisson is build. Below the comb-structure the foundation of the total caisson is realised. The verticals between the draft tubes have been given a thickness of 3 metres in the preliminary sketch of Figure VII.3. In the next pages the exact dimensions of these walls will be determined. The load scheme of the verticals is given in Figure VII.12.

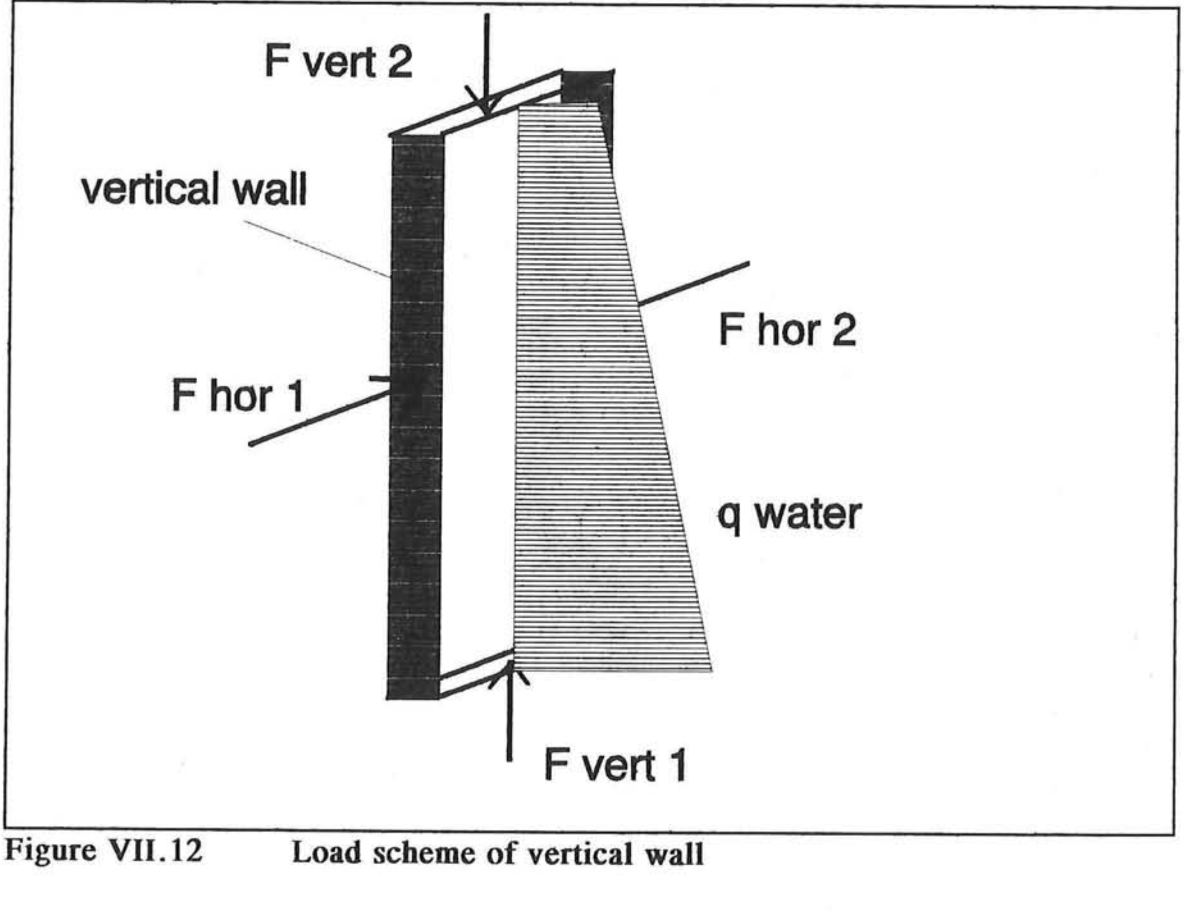


Figure VII.12 Load scheme of vertical wall

The maximum load on the walls occurs when one of the draft tubes is set dry. In that case at one side of the wall the full water pressure can be present while at the other side no water pressure is present at all. Other forces on the wall in the same load case are a vertical force F_v of the superstructure and a horizontal force F_h between the two gates holding back the water.

To determine the value of F_v the weight of the super structure must be known. Then the uplifting force of the water in the neighbouring draft tube (water pressure on the roof of the draft tube) should be subtracted from this value. The difference between $F_{v,1}$ and $F_{v,2}$ lies in the weight of the wall itself. The importance of this weight is dependable of the value of $F_{v,2}$, if this force is much larger than the force as result of the weight of the wall the contribution of this wall can be neglected.

The super structure has a hight of BM + 15 metres - BM - 2.5 metres = 17.5 metres. Between BM - 2.5 metres and BM - 0.5 metres the concrete comb structure closing off the draft tube at the top-side is planned. Of this two metres only 1 metre consists of concrete. The space inside the concrete ribs will not be filled in this design. However the possibility of filling this area exists. The weight of this structure can be estimated at 25 kN/m^2. At the roof of the caisson an other structure is needed to support the road, railway and crane. For the roof of the caisson therefore a similar structure is assumed with the same weight of 25 kN/m^2. Between the structures a space remains of 13.5 metres. If this space is divided over 3 floors, two extra floors with an estimated thickness of 0.5 metres are needed. The weight of these floors is again 25 kN/m^2. All the floors as well as the roof of the caissons have supports on walls placed right above the verticals of the comb-structure. The weight of these walls is estimated at 170 kN/m^1 (a thickness of 0.5 metres and a height of 13.5 metres). The outside walls holding back the water will not be taken into account since the force of these walls will be carried by only a small part of the verticals of the comb-structure (probably even only outside the closed part of the draft tube). The last factor increasing the value of F^v is the weight of the equipment placed on the floors. The weight of this equipment is set at 40 kN/m^2. The total value of F_{ss} becomes:

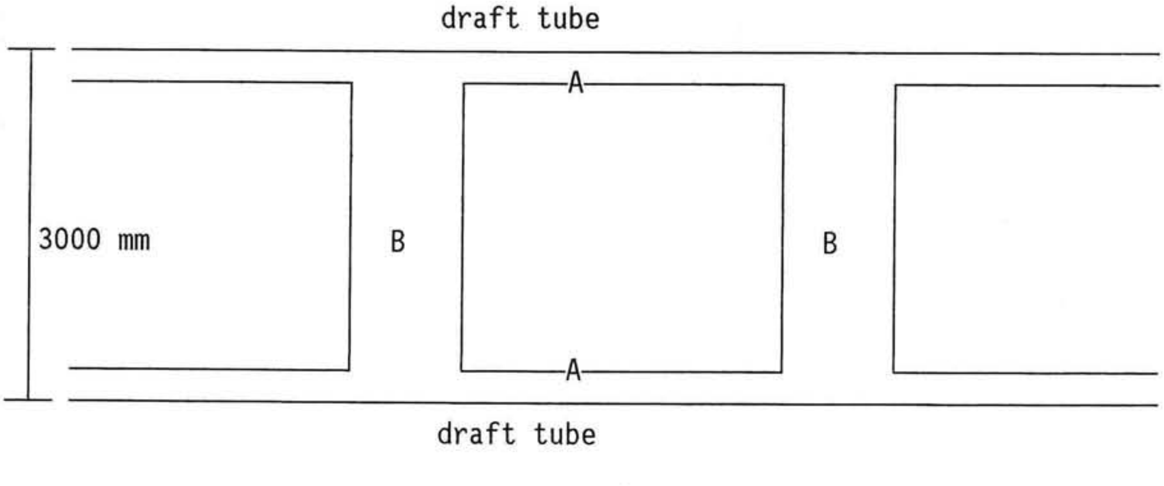
F_{ss} = 16 metres * (25 kN + 25 kN + 25 kN + 40 kN) + 170 kN = 1900 kN

The uplifting force of the water in the neighbouring draft tube is:

F_{uplift} = 6.5 metres * (BM + 12 m. - BM - 2.5 m.) * 9.8 m/s^2 * 1020 kg/m^3
= 945 kN

$F_{v,2} = F_{ss} - F_{uplift}$ = 1900 kN - 945 kN = 955 kN

For the verticals a structure as shown below is taken.



If the vertical consists of two ribs A with a thickness of 0.4 metres the and beams B with a thickness of 0.5 metres the value of $F_{v,1}$ becomes:

$F_{v,1} = F_{v,2}$ + 19 metres * 25 kN/m^3 * (2 * 0.4 metres * 1.00 metres + 1/2 * 0.5 metres * 2.2 metres))
$F_{v,1}$ = 955 kN + 641 kN = 1596 kN

The total load on the vertical and its results for moment , shear force and normal force is given in Figure VII.13.

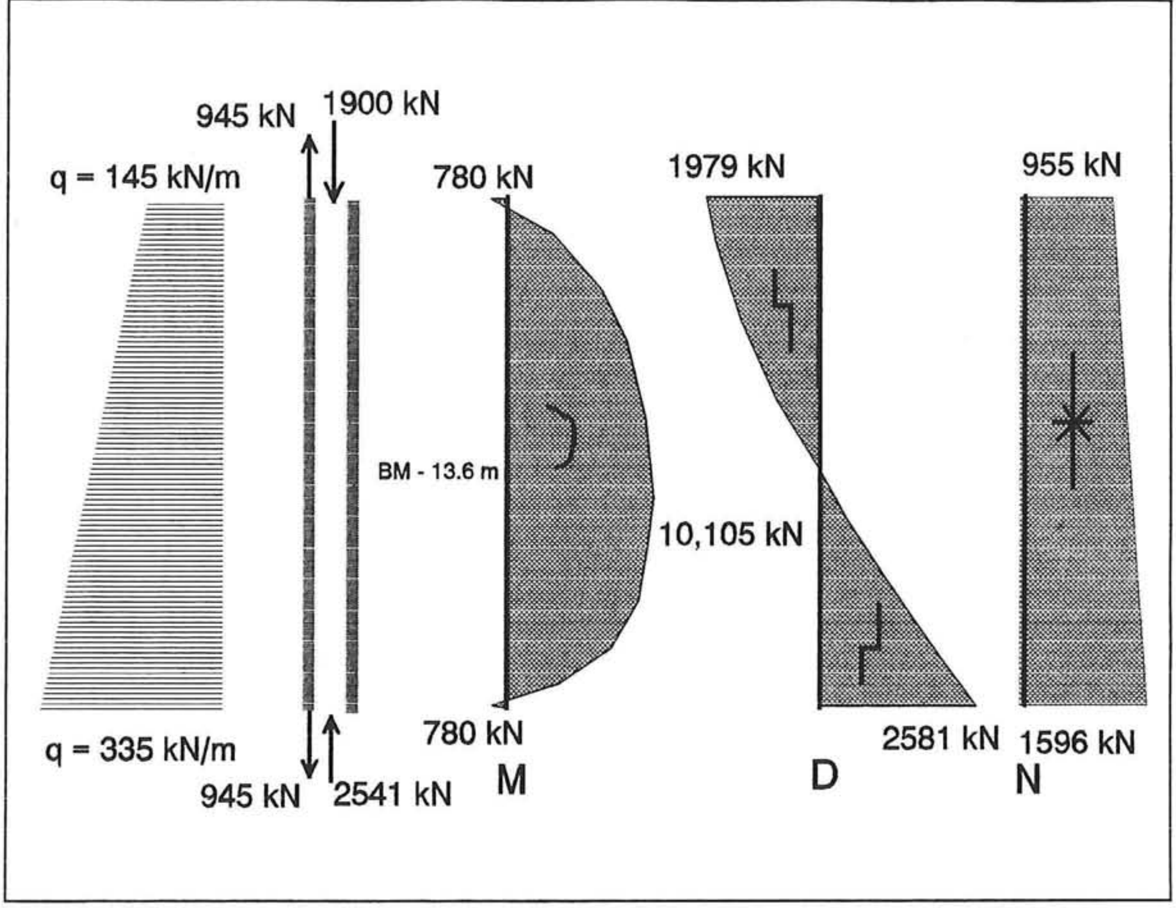


Figure VII.13 Load on the vertical and its results

The maximum moment in the vertical is at BM - 12.6 metres: 10,105 kNm/m. The shear force at this point is evidently 0. The normal force at this point is: N = 955 kN + 10.1/19 * (1596 kN - 955 kN) = 1296 kN/m

To determine the optimum cross section of the vertical, four elements should be checked:
1 the moment in the ribs A between B
2 the shear force in the ribs A close to B
3 the moment in the beams B at BM - 12.6 m
4 the shear force in the beams B at the point of support at BM - 21.5 m

The moment in ribs A:

If the distance between the beams B is set at 2 metres the maximum moment in the ribs A is:
$M_a = 1/16 * q * l^2$
in which $l = 2$ metres
and $q = (BM - 20.5\ m - BM + 12\ m) * 9.8\ m/s^2 * 1020\ kg/m^3$
$q = 325\ kN/m^2$
$M_a = 1/16 * 325\ kN * (2\ m)^2$
$M_a = 81\ kNm$

$M_{c,a} = \gamma * M_a = 1.7 * 80\ kNm = 138\ kNm$

The quality of the concrete is B 35, the steel used for reinforcement is FeB 500.

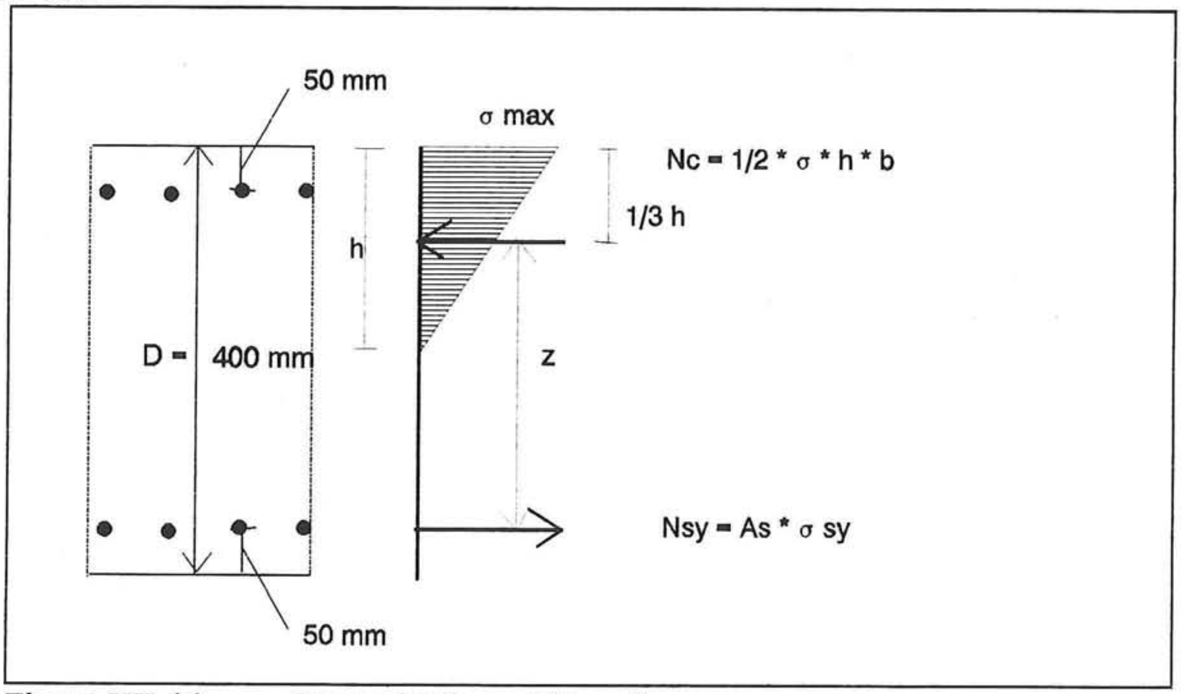


Figure VII.14 Determination of M_{max} rib

a $N_c + N_s = 0$
$- 1/2\ b\ h\ \sigma_{c1} = A_s\ \sigma_s$

b $(-\epsilon_{c1})/h = (\epsilon_s)/(d-h)$

c $\epsilon_{c1} * E_c = \sigma_{c1}$
$\epsilon_s * E_s = \sigma_s$

a + b + c with $\omega_d = (A_s)/(b * d)$ and $n = E_s/E_c$ determines h as:

$h = d\ \{-n\omega_d + \sqrt{((n\omega_d)^2 + 2n\omega_d)}\}$
$z = d - 1/3\ h$

$n = 205{,}000/32.000 = 6.4$
$d = 400\ mm - 50\ mm = 350\ mm$

if ω_d = 0.5 % (A_s = 1750 mm^2/m) then:

h = 78 mm
z = 324 mm
σ_s = 138 * 10^6 Nmm/(1750 mm^2 * 324 mm) = 243 N/mm^2
$_{cl}$ = - 138 * 10^6 Nmm/(1/2 * 1000 mm * 78 mm * 324 mm) = - 10.9 N/mm^2
(< 0.8 * 0.85 * 35 N/mm^2 = 23.8 N/mm^2)

if ω_d = 0.4 % (A_s = 1400 mm^2/m) then:

h = 71 mm
z = 326 mm
σ_s = 138 * 10^6 Nmm/(1400 mm^2 * 326 mm) = 302 N/mm^2
σ_{c1} = - 138 * 10^6 Nmm/(1/2 * 1000 mm * 71 mm * 326 mm) = - 11.9 N/mm^2
(< 0.8 * 0.85 * 35 N/mm^2 = 23.8 N/mm^2)

The horizontal force F_h in the vertials has a positive influece for the moment in the ribs. Since the moment in these ribs is very small and can easily be taken by a minimal reinforcement the force will be left aside

<u>The shear force in the ribs A:</u>

The maximum shear force in the ribs takes place at BM - 20.5 m. The load on the ribs below BM - 20.5 m. is partly distributed to the horizontals at the bottom of the draft tube. The maximum shear force in the ribs depends on the dimensions of beam B. If a width of this beam of 500 mm is taken, the distance over which the load is working on the ribs is 1500 mm. The maximum shear force than becomes:

F_{shear} = 0.75 m * 325 kN/m^2 * 1.000 m = 244 kN

The safety factor included the shear force is 414 kN.

A part of this shear force can be transferred by the concrete. The rest will have to be transferred by stirrup reinforcement.

concrete: $V_{cu} = \tau_1 * b * d$
τ_1 = 0.5 * 0.7 * $f_{ct,0}$ = 0.35 * 2.4 N/mm^2 = 0.84 N/mm^2
V_{cu} = 0.84 N/mm^2 * 1000 mm * 360 mm = 302 kN

steel : F = F_{shear} - V_{cu} = 112 kN/m^1
A_{ss}/t = 112,000 N/m/(335 mm * 400 N/mm^2)
A_{ss}/t = 0.835 mm^3/m
if t = 100 mm then A_{ss} is 83.5 mm^2/m

With a diameter of the stirrups of 8 mm, two stirrups have to be placed every meter. The cross section of the ribs is shown in Figure VII.15

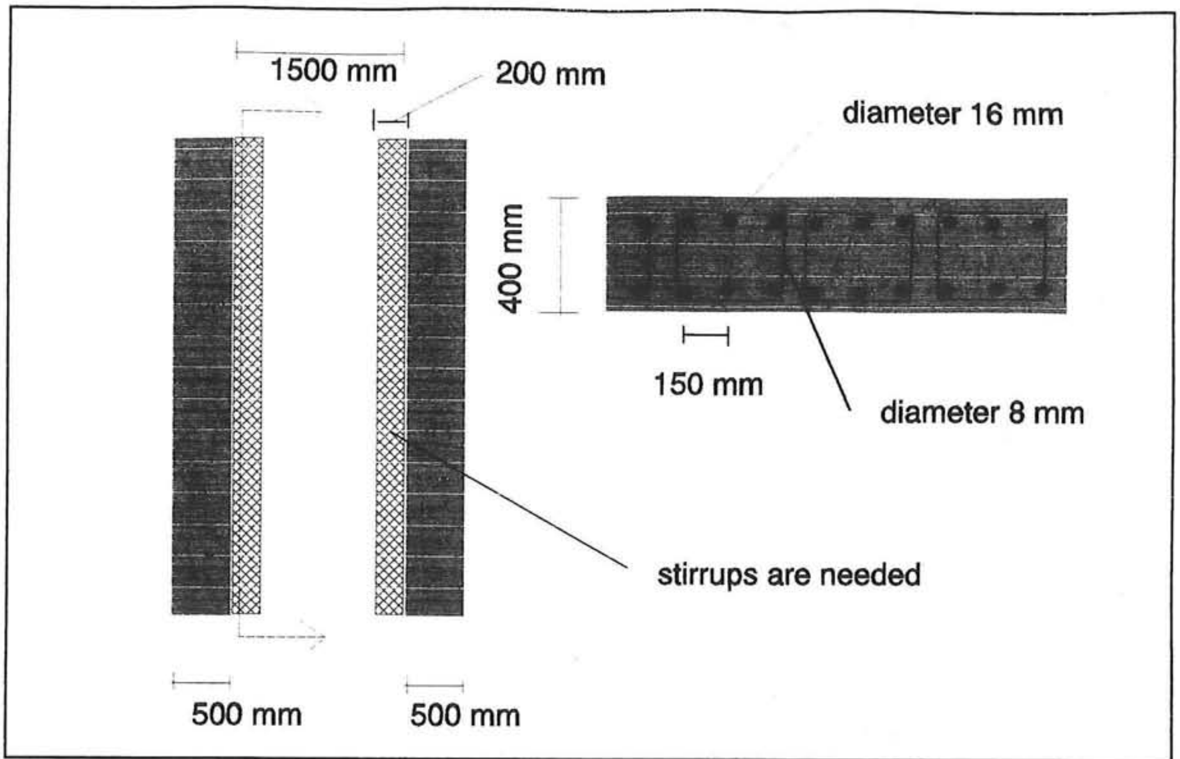


Figure VII.15 Cross section of ribs A

The moment in the beams B:

The moment in the beams B is 2.0 m * 1.7 * 10,105 kNm/m = 34,360 kNm at BM -12.6 metres. The cross section of the ribs between two beams can be added in the compressive zone of the beam at the place of the maximum moment. The cross section of the beam is as shown in Figure VII.16.

The moment in the beam can be reduced slightly as result of the normal force F_v. A part of this force (the uplifting force of the water in the neighbouring draft tube) has a small excetricity, this excentricity shall not be taken into account in the design. The effect of this excentricity is a small reduction (3 %) of the moment. This force results in a normal tension of:

$\sigma_c = F_v / A_c$

$F_v = 1.7 * 2 \text{ m} * 1296 \text{ kN/m}$
$= 4407 \text{ kN}$
$A_c = 3000 \text{ mm} * 500 \text{ mm} +$
$2 * 400 \text{ mm} * 1500 \text{ mm}$
$= 1,200,000 \text{ mm}$

$\sigma_c = 4407\ 10^3 \text{ N}/1.2\ 10^6 \text{ mm}^2$
$= 3.7 \text{ N/mm}^2$

The beam has a value of W of $1/6 * w * h^2$
$W = 7.5\ 10^8 \text{ mm}^3$

The moment the beam can withstand without tensile stress is:

$M = W * \sigma$
$= 7.5\ 10^8\ mm^3 * 3.7\ N/mm^2$
$= 2.8\ 10^9\ Nmm = 2800\ kNm$

The moment in the beam is reduced to 34,360 - 2,800 = 31,560 kNm

A first estimation gives a value for z of 3000 mm - 0.4 * 400 mm - covering of reinforcement (60 mm) = 2780 mm.

The compression zone is limited to the top 400 mm.

The stress in this zone is $M/(z * A) + 2 * 3.7\ N/mm^2 = 14.2 + 7.4 = 21.6\ N/mm^2$ ($< 23.8\ N/mm^2$)

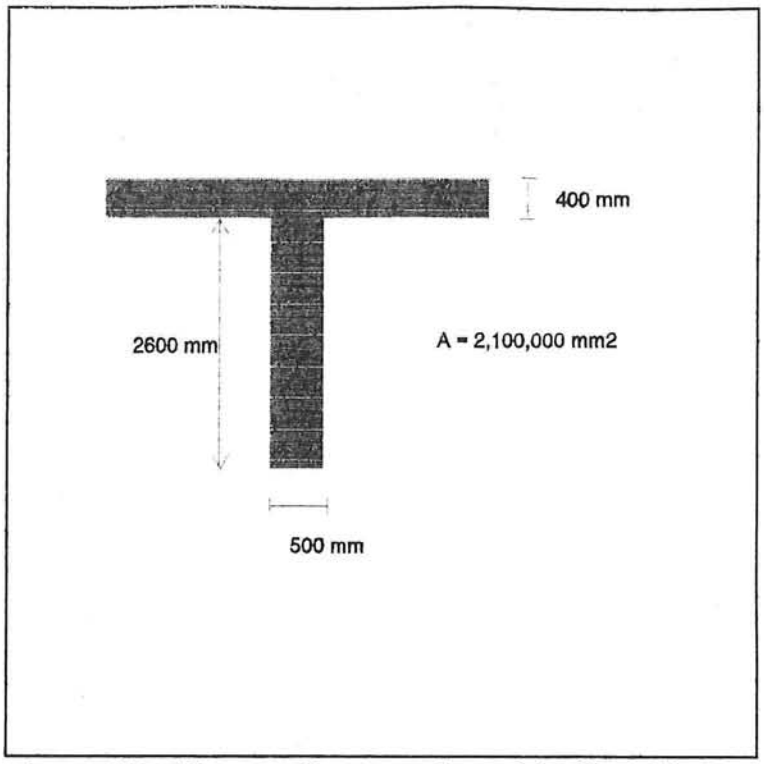


Figure VII.16 Cross section with ribs added

The amount of steel required for the reinforcement is $M/(z * \sigma_s) = 31,560 * 10^6/2780 * 500 = 22,700\ mm^2$.

19 bars of steel with a diameter of 40 mm have a total surface of 23,120 mm^2. To place this amount of steel the reinforcement has to be placed in three rows, considerably reducing the value of z.

The effect of the necessity of placing the reinforcement in three rows is a reduction of z from 2780 mm to 2580 mm. With this new value of z the tension in the concrete increases to 22.7 N/mm^2. The quantity of reinforcement steel becomes 24,470 mm^2 (20 bars).

In Figure VII.17 the cross section of the beam B is given with the calculated reinforcement.

<u>The shear force in beam B:</u>

The maximum shear force in the beams takes place at BM - 21.5 m.
$F_{shear} = 2581\ kN/m * 2\ m = 5162\ kN$

The safety factor included the shear force is 8775 kN.

A part of this shear force can be transferred by the concrete. The rest will have to be transferred by stirrup reinforcement. An alternative for this stirrup reinforcement is bending the reinforcement placed for the moment. This reinforcement is not needed at the part of the beam where the shear force is maximal. In this annex the stirrup reinforcent shall be calculated without using the option of bending the moment-reinforcement.

concrete: $V_{cu} = \tau_1 * b * d$
$\tau_1 = 0.5 * 0.7 * f_{ct,0} = 0.35 * 2.4\ N/mm^2 = 0.84\ N/mm^2$
$V_{cu} = 0.84\ N/mm^2 * 500\ mm * 3000\ mm = 1260\ kN$

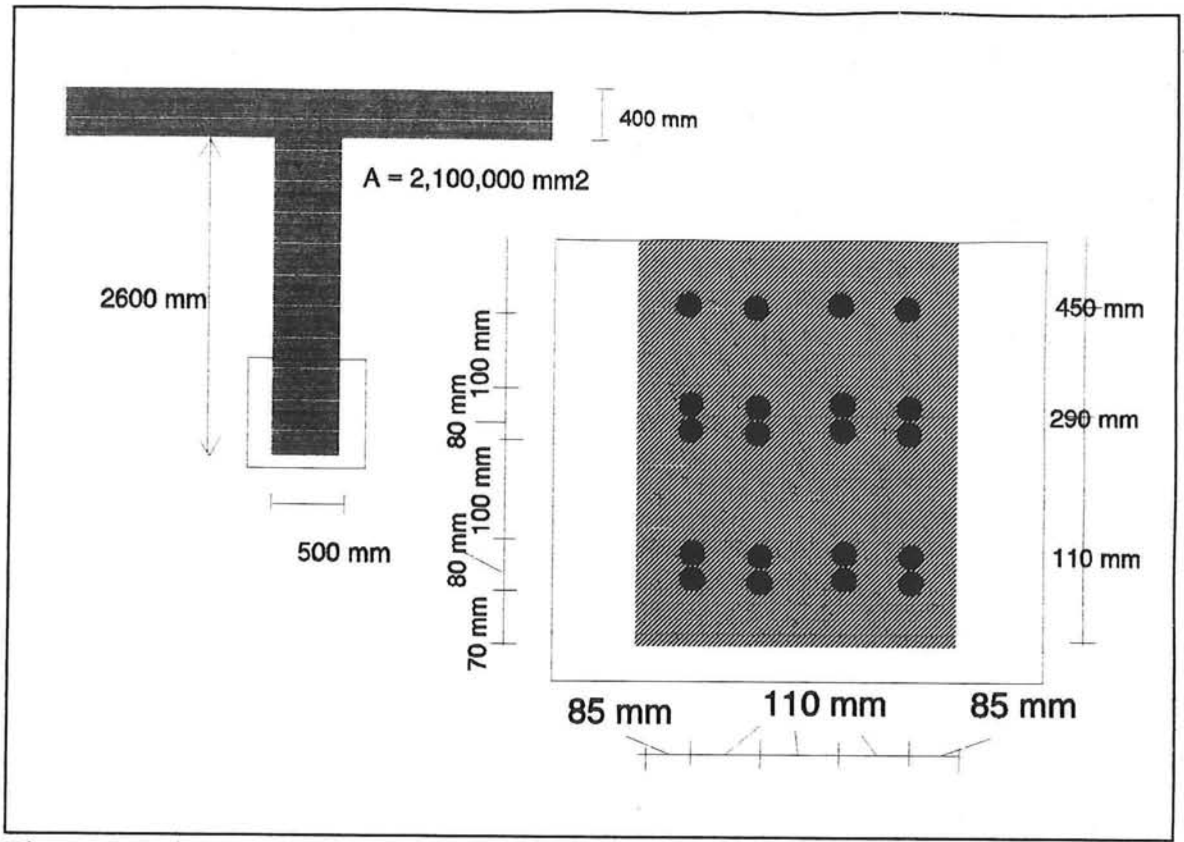


Figure VII.17 **Cross section of beam B**

steel : $F = F_{shear} - V_{cu} = 7525$ kN
$A_{ss}/t = 7525,000$ N/(2580 mm * 500 N/mm^2)
$A_{ss}/t = 5.8$ mm^3
if t = 100 mm then A_{ss} is 580 mm^2

The stirrups will have a diameter of 20 mm. The surface of a stirrup ϕ 20 mm is 2 * 314 mm^2 = 628 mm^2.

The concrete tension as result of the shear force also has to be checked. The scheme for transferring the shear force is given in Figure VII.18. The surface of the concrete diagonals between the vertical stirrups is:

$A = 1/2 * b * z * \sqrt{2} = 912,000$ mm^2

$N_c = 8775$ kN * $\sqrt{2}$ = 12,410 kN for the first diagonal and
$N_c = 7525$ kN * $\sqrt{2}$ = 10,642 kN for all the other diagonals.

$\sigma_{c,s} = N_c/A_c = 12.41 * 10^6$ N/ 912,000 mm^2 = 13.6 N/mm^2.
$\sigma_{c,s}$ should be lower than $0.65 * f_{c,0} = 0.65 * 28$ N/mm^2 = 18.2 N/mm^2.

With this design of the vertical of the comb structure only a basis is made on which further optimalisation is required. Optimalisation is possible for the ribs A and beam B. Possibilities for optimisation are:

- -pre-stressing of beam B to reduce the necessary reinforcement, the pre-stressing has to be centric since the moment in the beam can work in two directions
- -reduction of the span of the ribs from 1.5 metres to 1.1 metres with as result that no stirrups are needed
- -enlarging the thickness of the ribs with again as result that no stirrups are needed
- -changing the dimension of the ribs and the beam to get a minimum usage of concrete

A combination of all these points probably would give the best result.

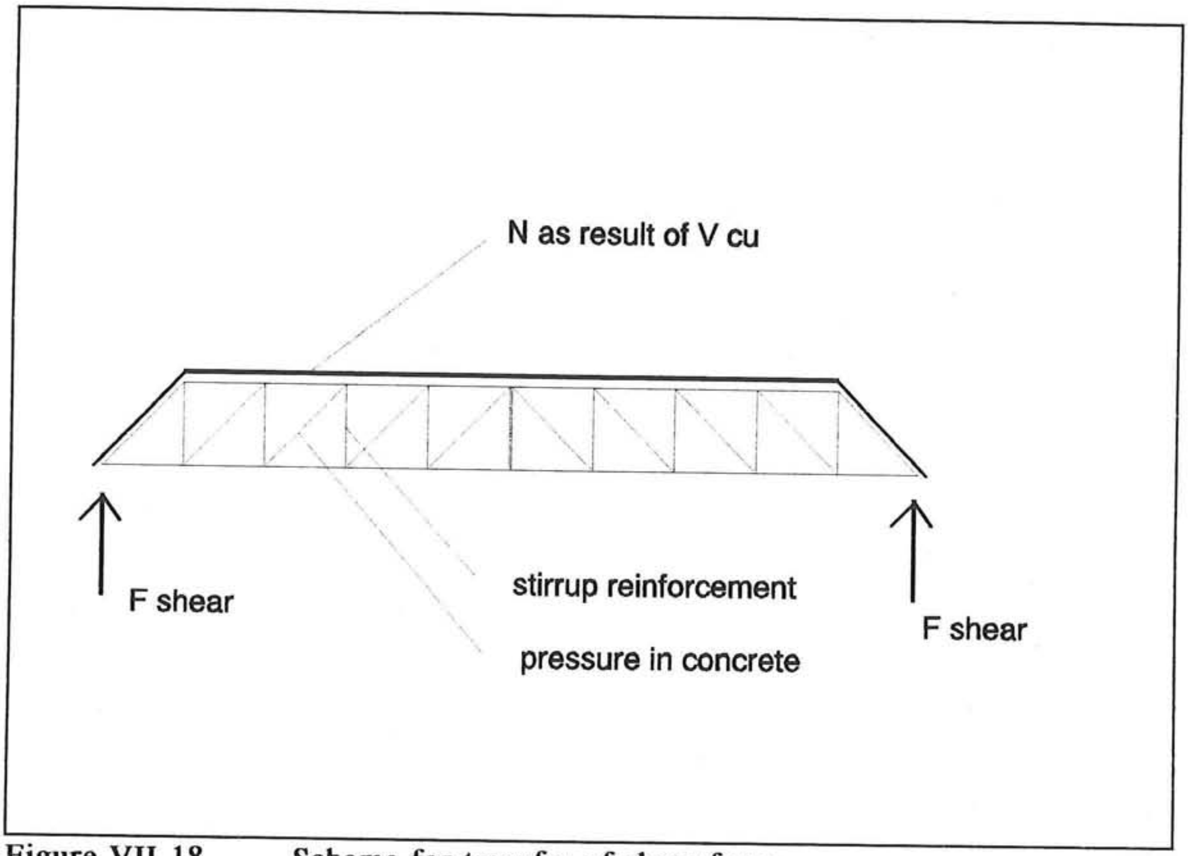


Figure VII.18 Scheme for transfer of shear force

Design of the bottom of the power house caisson.

For the design of the bottom of the caisson the grain pressure on the bottom of the caisson has to be known. This earth pressure depends on the weight of the caisson. Therefore an estimation of the weight of the caisson shall first be given. The power house caisson consists of the following elements:

- superstructure
- turbine and generator
- verticals
- draft tube
- water in the draft tubes
- gates
- bottom of the caisson

Superstructure

The super structure has a hight of BM + 15 metres - BM - 2.5 metres = 17.5 metres. Between BM - 2.5 metres and BM - 0.5 metres the concrete comb structure closing off the draft tube at the top-side is planned. Of this two metres only 1 metre consists of concrete. The weight of this structure can be estimated at 25 kN/m^2. At the roof of the caisson an other structure is needed to support the road, railway and crane. For the roof of the caisson therefore a similar structure is assumed with the same weight of 25 kN/m^2. Between the structures a space remains of 13.5 metres. If this space is divided over 3 floors, two extra floors with an estimated thickness of 0.5 metres are needed. The weight of these floors is again 25 kN/m^2. All the floors as well as the roof of the caissons have supports on walls placed right above the verticals of the comb-structure. The weight of these walls is estimated at 170 kN/m^1 = 170 kN/m^1/16 m = 10.6 kN $/m^2$ (a thickness of 0.5 metres and a height of 13.5 metres). The walls at the basin side and the sea side of the caissons both have a height of 17.5 metres. The weight of these walls is estimated at 20 $kN/m^1/m^1$. The contribution of these walls to the weight of the superstructure per square metre is 2 * 17.5 m * 20 kN/m^2 / 60 metres = 12 kN/m^2. An other factor increasing the weight of the superstructure is the weight of the equipment placed on the floors. The weight of this equipment is set at 40 kN/m^2.

The weight of the super structure is therefore:

$$W_{ss} = 25\ kN/m^2 + 25\ kN/m^2 + 25\ kN/m^2 + 10.6\ kN/m^2 + 12\ kN/m^2 + 40\ kN/m^2 = 138\ kN/m^2$$

Turbine and generator

The estimated weight of turbine and generator is 1,500,000 kg = 15,000 kN. With this weight the weight per square metre of caisson is:

$$W_{t\&g} = 15{,}000\ kN/(60\ metres * 16\ metres) = 15.6\ kN/m^2$$

Verticals

The verticals have a cross section per two metres of 3000 mm * 500 mm + 2 * 400 mm * 1500 mm = 2,700,000 mm^2. The weight of this cross section is 67.5 kN/m^1. The weight of the verticals per square metre is:

$$W^v = 19\ m * 67.5\ kN/m^1 / (2\ metres * 16\ metres) = 40\ kN/m^2$$

Draft tube

The draft tube is constructed in the comb structure described before with the design of the verticals. The draft tube is partly a concrete structure and partly a steel structure. Near the turbine axis the draft tube is made in steel to get a very smooth surface increasing the efficiency of the turbine. Near the entrance gate and the exit gate the draft tube is constructed in concrete to reduce costs. The weight of the draft tube is estimated at 5000 kN for the concrete part of the draft tube and 3000 kN for the steel part of the draft tube (length 20 metres, start 10 before and end 10 metres after the turbine axis). The total weight is 8000 kN. Per square metre of caisson:
W_{dt} = 8000 kN/(60 metres * 16 metres) = 8 kN/m^2

Water in draft tubes

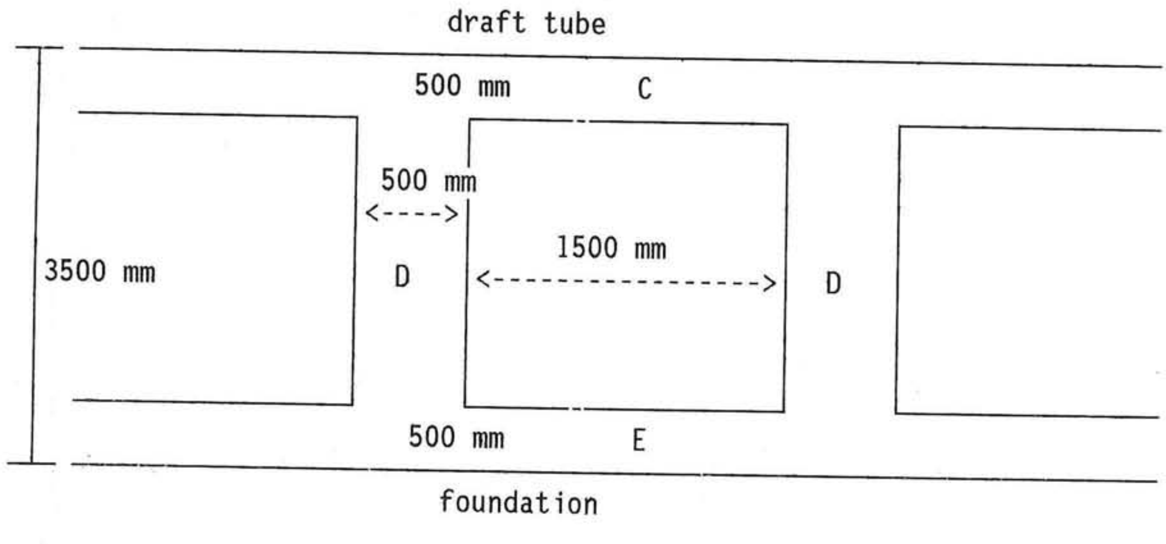
The quantity of water in the draft tube is 8,2000 m^3. The weight of this water is 82,000 kN
W_w = 82,000 kN/(60 metres * 16 metres) = 86 kN/m^2

Gates

The weight of the gates is 2,000 kN for the entrance gate and 1,000 kN for the exit gate
W_d = 3000 kN/(60 metres * 16 metres) = 3 kN/m^2

Bottom of caisson

The dimensions of the bottom of the caisson are estimated as shown below:



The bottom has a cross section per two metres of 3500 mm * 500 mm + 2 * 500 mm * 1500 mm = 3,250,000 mm^2. The weight of this cross section is 81.3 kN/m^1. The weight of bottom per square metre is:
W^b = 81.3 kN/m^1 / 2 metres = 40.6 kN/m^2

The maximum weight of the caisson per square metre is:

$$W_{max} = W_{ss} + W_{t\&g} + W_v + W_{dt} + W_w + W_g + W_b$$
$$= 138 + 15.6 + 40 + 8 + 86 + 3 + 40.6 = 330 \text{ kN/m}^2$$

A first conclusion of this value for W_{max} is that it insufficient to equalize the uplifting force of the water under the caisson if the water level is higher than BM + 8 metres. If one (of say eight) draft tube is set dry the caisson would even float at BM + 7 metres. Extra weight of at least 70 kN/m² should be added. A first option is ballasting of the caisson. An other option is to enlarge the thickness of the ribs A of the verticals to 500 mm, to avoid stirrup reinforcement. The same action can be made for the ribs in the bottom structure (enlargement to 600 mm). Both actions increase the weight of the structure with 6 kN/m². Other options are an enlargement of the width of the beams B and D. to 700 mm. Again 12 kN/m² is added, and less reinforcement is needed. Further enlargement of W_{max} has to be found in ballasting between the draft tube and the comb structure, or a more extensive use (more floors and/or more permanent load) in the superstructure.

All these options are with the precondition that the estimated weight for the different elements of the caisson is correct. A more detailed research on the characteristics of equipment and weight of the super structure is preferable.

The desired weight of the caisson without the water is 310 kN/m².With this weight the caisson would have a freeboard of 9 metres during transport. If placing of the caisson takes place during a mean spring tide the water level over the sill is BM + 8 metres - BM -25 metres is 33 metres. The keel clearance is 2 metres. If water is let into the draft tubes the caisson can be placed on the sill. Even if an extreme water level of BM + 12 metres is reached the caisson would still rest on the sill with a ground pressure of 310 kN/m² + 86 kN/m² - 37 metres * 10 kN/m² = 16 kN/m².

The maximum load on the bottom does not take place during an extreme height of the water level but during an extreme difference in water level in combination with a relative high water level. If the basin of the tidal power station is filled during an extreme spring tide the basin level is as high as BM + 12 metres. Energy production begins if the difference in water level is 3 metres. If the water level at the sea side of the caisson reaches its minimum of BM + 0 metres the water level in the basin is still BM + 7.5 metres. The load-scheme on the caisson is as shown in Figure VII.19.

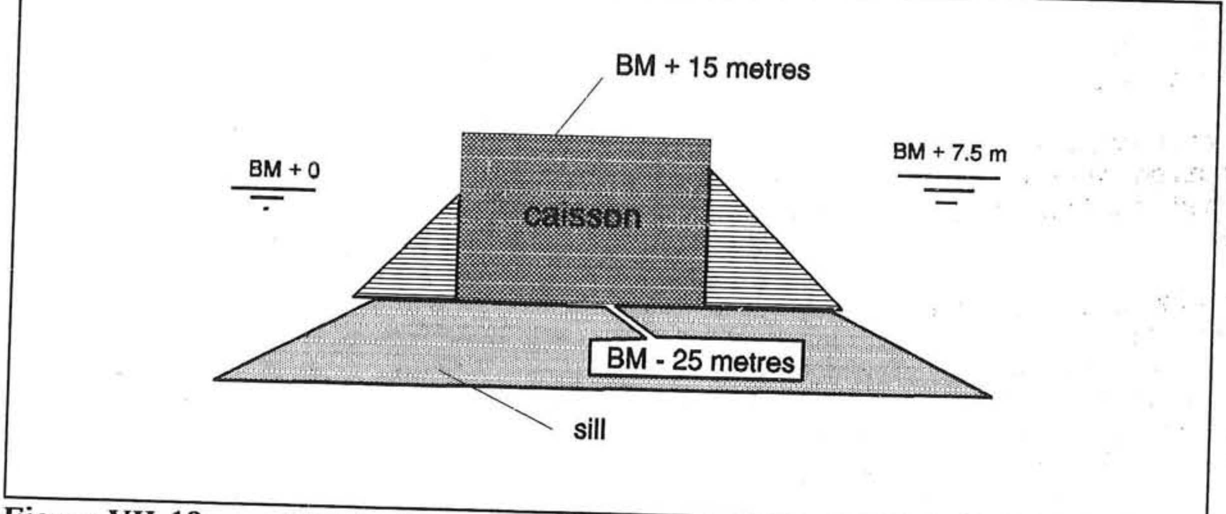


Figure VII.19 **Load scheme on caisson with extreme difference in water level**

The force of the water level at the sea side of the caisson is:

$F_{sea} = 1/2 * \rho_w * g * (H)^2 = 0.5 * 1020 \text{ kg/m}^3 * 9.8 \text{ m/s}^2 * (25 \text{ m})^2$
$= 3125 \text{ kN/m}^1$

The place were this resulting force acts on the caisson is at a height of BM - 16.7 metres.

The force of the water level at the basin side of the caisson is:

$F_{bas} = 1/2 * \rho_w * g * (H)^2 = 0.5 * 1020 \text{ kg/m}^3 * 9.8 \text{ m/s}^2 * (32.5 \text{ m})^2$
$= 5281 \text{ kN/m}^1$

The place were this resulting force acts on the caisson is at a height of BM - 14.2 metres.

The resulting force on the caisson is 2156 kN/m^1. The resulting moment on the caisson is 22,803 kNm. The resulting force on the caisson of 2156 kN/m is transferred to the sill by means of friction. In this load case the friction coefficient should be at least 0.3:

$\tau_F = 2156 \text{ kN/m}^1/60 \text{ metres} = 36 \text{ kN/m}^2$
$\sigma_N = W_{caisson} - F_{water,uplift} = 396 \text{ kN/m}^2 - (BM + 3.75 \text{ m} - BM - 25 \text{ m}) * 10 \text{ kN/m}^3$
$= 396 \text{ kN/m}^2 - 287.5 \text{ kN/m}^2 = 108.5 \text{ kN/m}^2$
$f = 108.5 \text{ kN/m}^2 / 36 \text{ kN/m}^2 = 0.3$

With the stones used for the sill a friction coefficient of 0.5 can be realised.

The moment on the caisson results in a changed ground pressure under the caisson. Without the difference in water level the ground pressure is 396 kN/m^2 - (av. BM + 3.75 m - BM - 25 m) * 10 kN/m^3 = 108.5 kN/m^2. The W of the power house caisson is $1/6 * b * h^2 = 1/6 * 1.00 \text{ m} * (60 \text{ m})^2 = 600 \text{ m}^3$. The difference in ground pressure as result of the moment = M / W = 22,803 kNm / 600 m^3 = 38 kN/m^2. The resulting forces and moment on the caisson is shown in Figure VII.20.

The maximum pressure on the bottom of the caisson is the maximum ground pressure plus the accompanying water pressure. An other way to calculate the maximum pressure on the caisson-bottom is to add the tension as result of the moment mentioned earlier, to the pressure caused by the dead weight of the caisson. The maximum load is 396 kN/m^2 + 38 kN/m^2 = 434 kN/m^2.

With this maximum load the bottom of the caisson can be determined. The bottom of the caisson consists of the same elements as the verticals designed earlier this annex. The height of the caisson bottom is 3.5 metres.

To determine the optimum cross section of the bottom, four elements should be checked:

1 the moment in the ribs E between D
2 the shear force in the ribs E close to D
3 the moment in the beams D
4 the shear force in the beams D

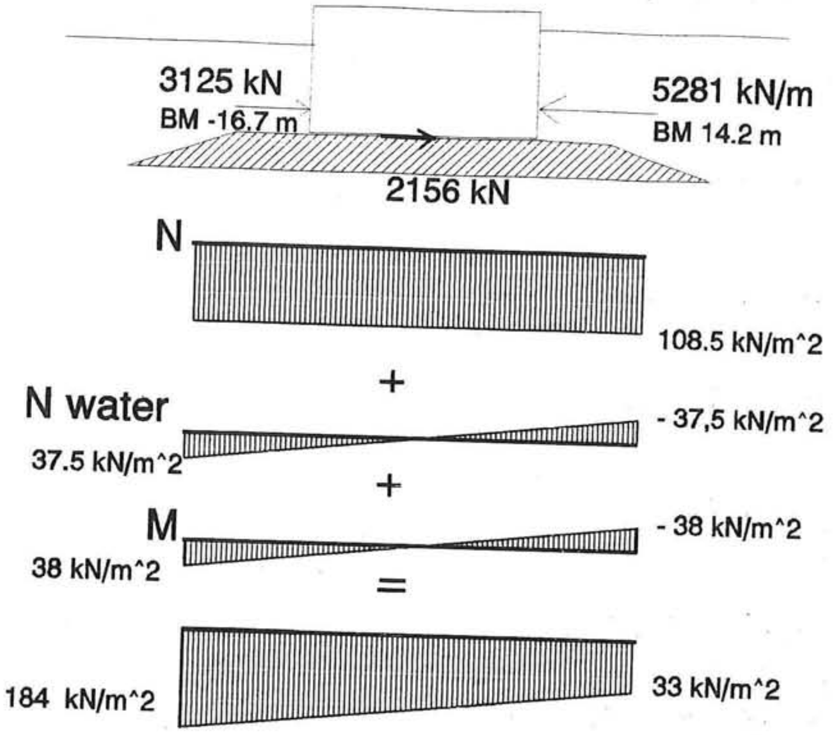


Figure VII.20 **Forces and resulting ground pressure as result of extreme difference in water levels**

The moment in ribs E:

The distance between the beams D is 2 metres, the maximum moment in the ribs C is:

$M^e = 1/16 * q * l^2$

in which l = 2 metres
and q = 434 kN/m^2

$M^e = 1/16 * 434 \text{ kN/m}^2 * (2 \text{ m})^2$

$M_e = 108.5$ kNm

$M_{c,e} = \gamma * M_a = 1.7 * 108.5 \text{ kNm} = 184 \text{ kNm}$

The quality of the concrete is B 35, the steel used for reinforcement is FeB 500.

a $\quad N_c + N_s = 0$

$\quad - 1/2\ b\ h\ \sigma_{c1} = A_s\ \sigma_s$

b $\quad (-\epsilon_{c1})/h = (\epsilon_s)/(d-h)$

c $\quad \epsilon_{c1} * E_c = \sigma_{c1}$

$\quad \epsilon_s * E_s = \sigma_s$

a + b + c with ω_d = $(A_s)/(b * d)$ and $n = E_s/E_c$ determines h as:

$$h = d \{-n\omega_d + \surd((n\omega_d)^2 + 2n\omega_d)\}$$
$$z = d - 1/3\ h$$

n = 205,000/32.000 = 6.4
d = 500 mm - 50 mm = 450 mm

if ω_d = 0.5 % (A_s = 2250 mm^2/m) then:

h = 100 mm
z = 417 mm
σ_s = 184 * 10^6 Nmm/(2250 mm^2 * 417 mm) = 196 N/mm^2
σ_{c1} = - 184 * 10^6 Nmm/(1/2 * 1000 mm * 100 mm * 417 mm) = - 8.8 N/mm^2

if ω_d = 0.4 % (A_s = 1800 mm^2/m) then:

h = 91 mm
z = 420 mm
σ_s = 184 * 10^6 Nmm/(1800 mm^2 * 420 mm) = 243 N/mm^2
σ_{c1} = - 184 * 10^6 Nmm/(1/2 * 1000 mm * 91 mm * 420 mm) = - 9.6 N/mm^2

<u>The shear force in the ribs E:</u>

The maximum shear force in the ribs takes place in the ribs E close to the beams D. The maximum shear force in the ribs depends on the dimensions of beam D. If a width of this beam of 700 mm is taken, the distance over which the load is working on the ribs is 1300 mm. The maximum shear force than becomes:

F_{shear} = 0.65 m * 434 kN/m^2 * 1.000 m = 282 kN

The safety factor included the shear force is 480 kN.

A part of this shear force can be transferred by the concrete. The rest will have to be transferred by stirrup reinforcement.

concrete: $V_{cu} = \tau_1 * b * d$
τ_1 = 0.5 * 0.7 * $f_{ct,0}$ = 0.35 * 2.4 N/mm^2 = 0.84 N/mm^2
V_{cu} = 0.84 N/mm^2 * 1000 mm * 460 mm = 386 kN

steel : $F = F_{shear} - V_{cu}$ = 94 kN/m^1
A_{ss}/t = 112,000 N/m/(429 mm * 500 N/mm^2)
A_{ss}/t = 0.52 mm^3/m
if t = 100 mm then A_{ss} is 52 mm^2/m

If stirrups with a diameter of 8 mm are used only one stirrups has to be placed every meter. To have a more evenly placed reinforcement the stirrups will be placed every 500 mm. The cross section of the ribs is shown in Figure VII.21. An alternative for the stirrup reinforcement is a local thickening of the ribs.

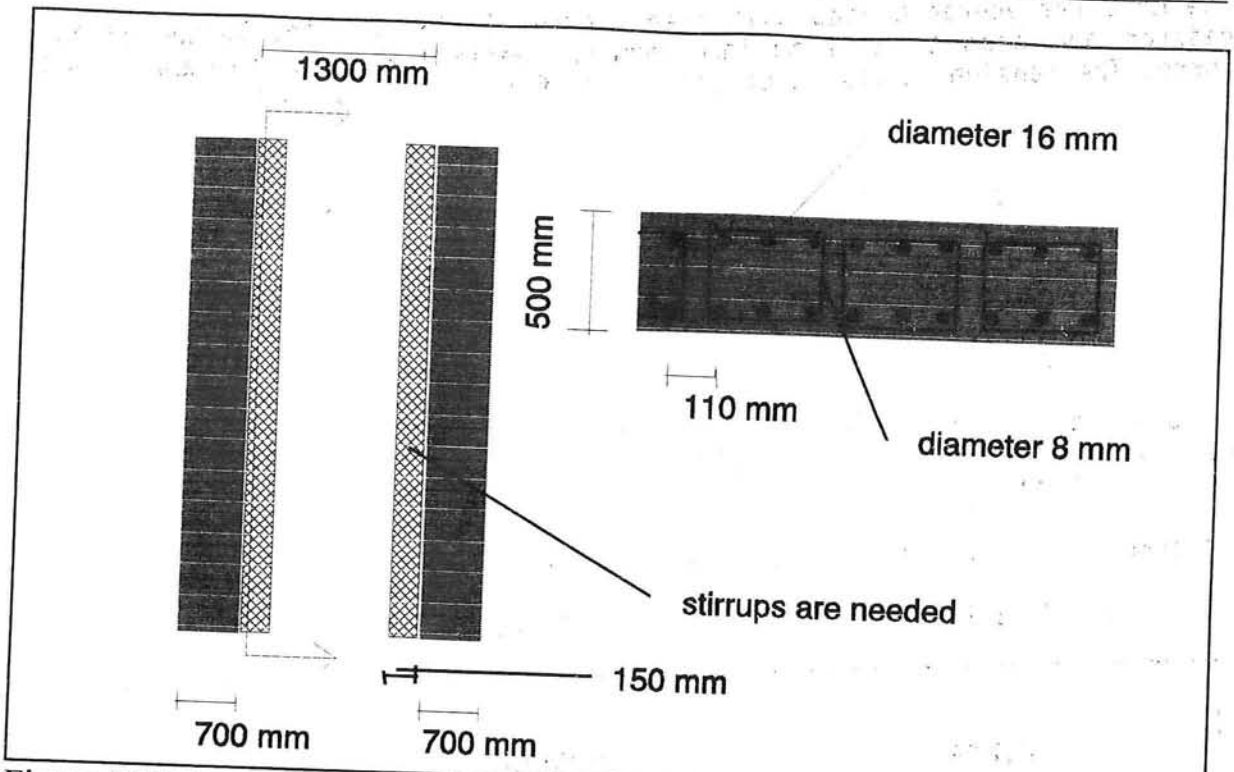


Figure VII.21 Cross section of ribs E

The moment in the beams D:

The moment in the beams B is 1/? * q * l^2. The question mark is placed in the equation since this factor is unknown unless a calculation of the total caisson is made. If the ends of the beams can rotate freely the maximum moment would be 1/8 q l^2. If the ends of the beams can not rotate at all the maximum moment would be 1/16 q l^2. For this design a factor of 1/10 will be taken.

The cross section of the ribs between two beams can be added in the compressive zone of the beam at the place of the maximum moment. The cross section of the beam is as shown in Figure VII.22.

The moment in the beam can be reduced slightly as result of the normal force N. This force is the result of the water pressure on the verticals.

The maximum value of this force is 8775 kN, the minimum value is 4900 kN (in case of a low water).

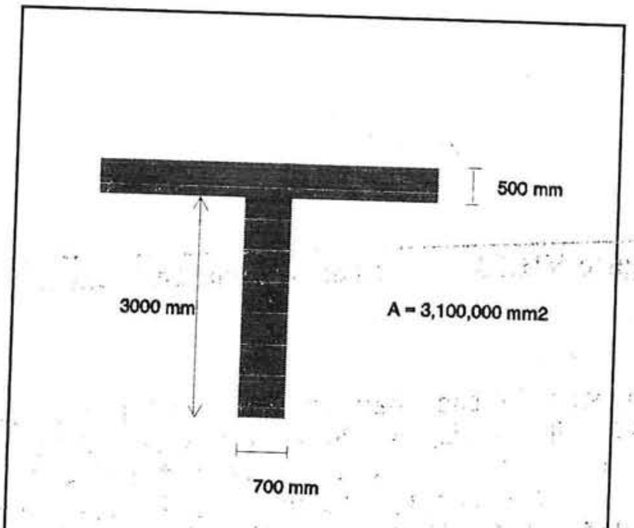


Figure VII.22 Cross section beam D with ribs added

The most unfavourable load case (see Figure VII.23) for the bottom of the caisson and beam D is a maximum moment combined with a minimum normal force. The tension as result of the normal force is as follows:

$\sigma_c = N / A_c$

A_c = 3500 mm * 700 mm +
2 * 500 mm * 1300 mm
= 3,750,000 mm (W = 94 kN/m^1)

$\sigma_c = 4900\ 10^3\ N/3.75\ 10^6\ mm^2$
$= 1.3\ N/mm^2$

The beam has a value of W of $1/6 * w_{tot} * h^2 - 1/6 * w * h^2$
$W = 1/6 * 2000 * 3500^2 - 1/6 * 1300 * 2500^2$
$W = 2.73\ 10^9\ mm^3$

The moment the beam can withstand without tensile stress is:

$M = W * \sigma = 2.73\ 10^9\ mm^3 * 1.3\ N/mm^2 = 3.5\ 10^9$ Nmm = 3500 kNm

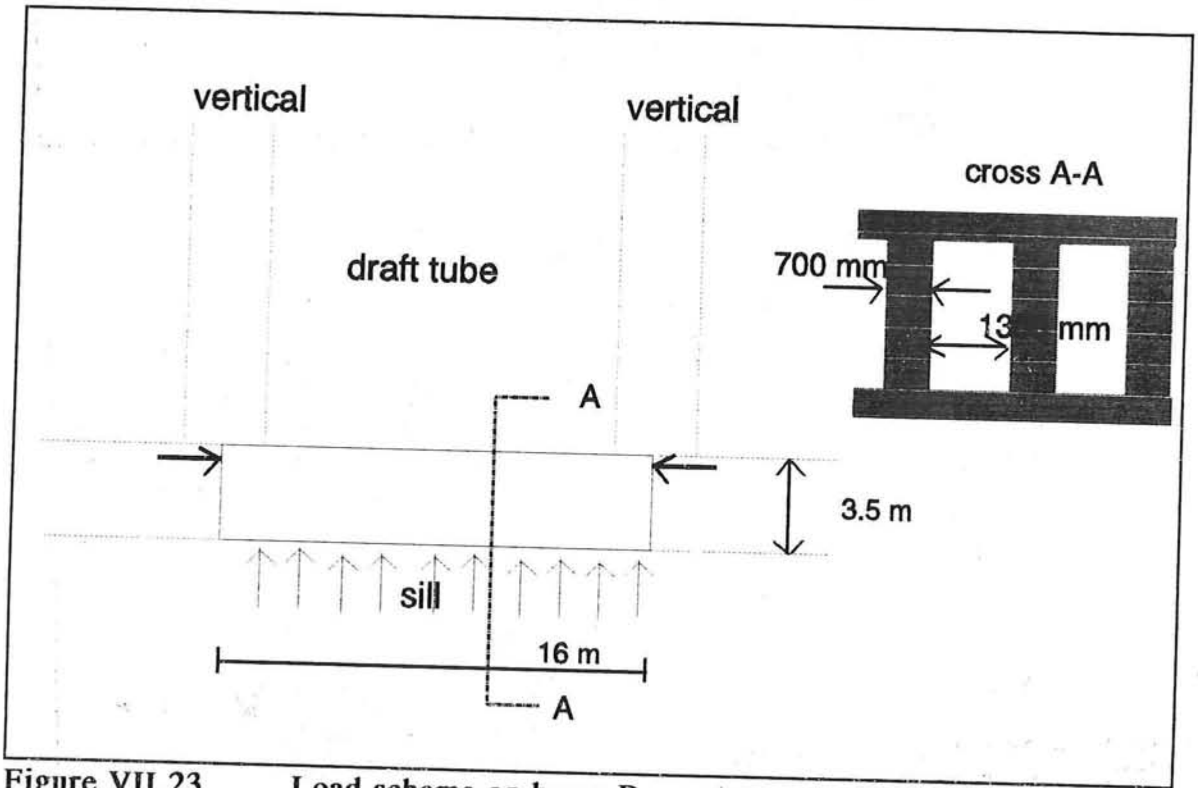


Figure VII.23 **Load scheme on beam D**

The moment in the beam as result of the q-load is as follows:
$q = 434$ kN/m - W_{beam} = 434 - 94/2 = 387 kN/m^2
$M = 1/10 * q * l^2 = 0.1 * 2.00\ m * 387\ kN/m^2 * (16\ m)^2$ = 19,814 kNm
Including the safety factor of 1.7 this becomes 33,684 kNm. This moment in the beam can be reduced to 33,684 - 3,500 = 30,180 kNm

A further reduction of this moment is possible since the normal force does not act in the hart of the beam. The normal force has an eccentricity of 1500 mm, resulting in a moment of 4900 kN * 1.5 m = 7350 kNm. The moment for which reinforcement is needed is 30,180 - 7350 = 22,830 kNm

A first estimation gives a value for z of 3500 mm - 0.4 * 500 mm - covering of reinforcement (100 mm) = 3200 mm
The compression zone is limited to the top 500 mm.

The stress in this zone is M/(z * A) + 2 * 1.3 N/mm^2 = 7.1 + 2.6 = 9.7 N/mm^2
The amount of steel required for the reinforcement is M/(z * σ_s) = 22,830 * 10^6/3200 * 500 = 14,270 mm^2.

12 bars of reibforcement with a diameter of 40 mm have a surface of 15,080 mm^2. This reinforcement also has to be placed in two rows.

The effect of the necessity of placing the reinforcement in two rows is a reduction of z from 3200 mm to 3170 mm. With this new value of z the tension in the concrete increases to 9.8 N/mm^2. The quantity of reinforcement steel (FeB 500) becomes 14,400 mm^2 (15 bars).

In Figure VII.24 the cross section of the beam D is given with the calculated reinforcement.

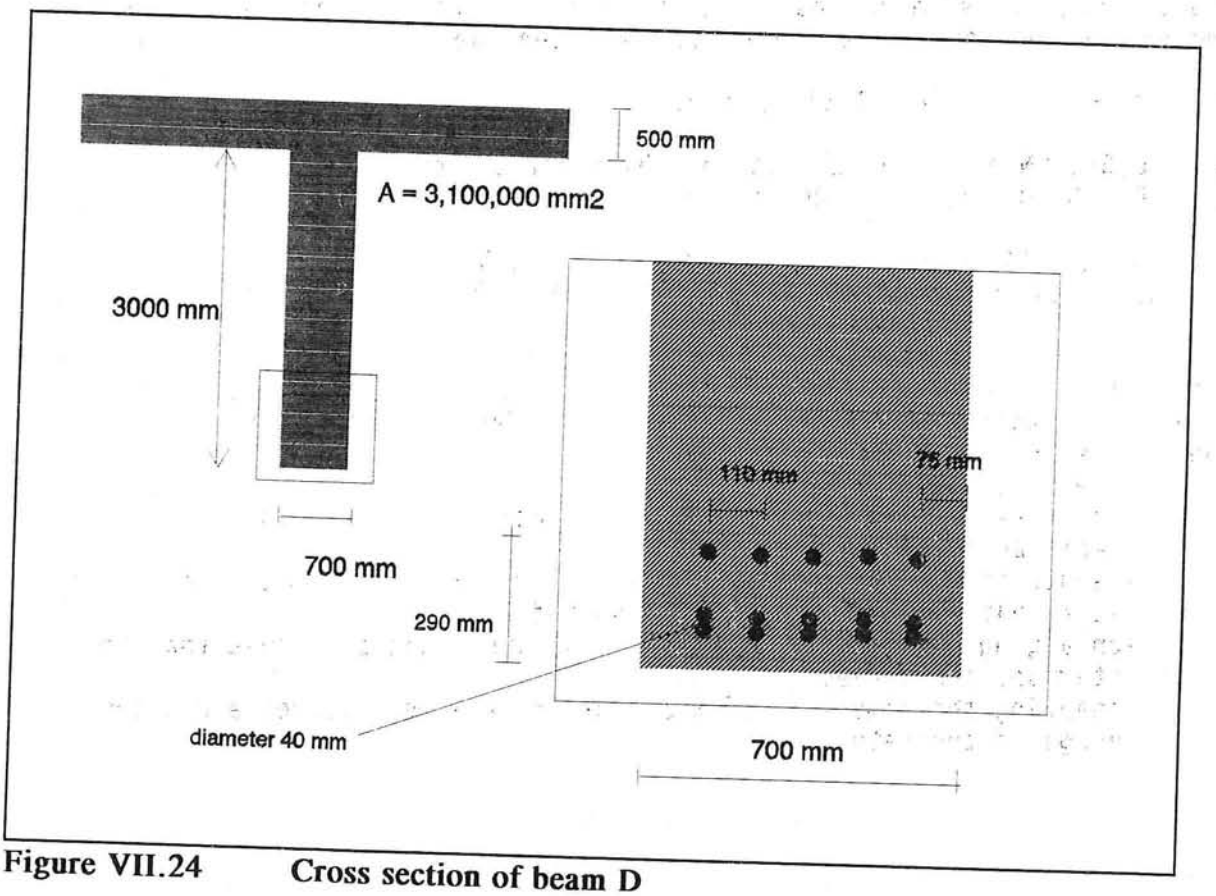


Figure VII.24 Cross section of beam D

The shear force in beam D:

$F_{shear} = q * (16\ m/2) * w$
$q = 434\ kN/m - W_{beam} = 434 - 94/2 = 387\ kN/m^2$
$F_{shear} = 387\ kN/m^2 * 8\ m * 2\ m = 6192\ kN$

The safety factor included the shear force is 10,530 kN.

A part of this shear force can be transferred by the concrete. The rest will have to be transferred by stirrup reinforcement.

concrete: $V_{cu} = \tau_1 * b * d$
$\tau_1 = 0.5 * 0.7 * f_{ct,0} = 0.35 * 2.4\ N/mm^2 = 0.84\ N/mm^2$
$V_{cu} = 0.84\ N/mm^2 * 700\ mm * 3500\ mm = 2060\ kN$

steel : $F = F_{shear} - V_{cu} = 8470\ kN$
$A_{ss}/t = 8470{,}000\ N/(3160\ mm * 500\ N/mm^2)$
$A_{ss}/t = 5.4\ mm^3$
if $t = 100\ mm$ then A_{ss} is $540\ mm^2$

The stirrups will have to get a diameter of 20 mm. The surface of a stirrup ϕ 20 mm is $2 * 314\ mm^2 = 628\ mm^2$.

The concrete tension as result of the shear force also has to be checked. The surface of the concrete diagonals between the vertical stirrups is:

$A = 1/2 * b * z * \sqrt{2} = 1{,}564{,}000\ mm^2$

$N_c = 10{,}530\ kN * \sqrt{2} = 14{,}890\ kN$ for the first diagonal and
$N_c = 8{,}470\ kN * \sqrt{2} = 11{,}980\ kN$ for all the other diagonals.

$\sigma_{c,s} = N_c/A_c = 14.89 * 10^6\ N/\ 1{,}564{,}000\ mm^2 = 9.5\ N/mm^2$.
$\sigma_{c,s}$ should be lower than $0.65 * f_{c,0} = 0.65 * 28\ N/mm^2 = 18.2\ N/mm^2$.

With this design of the vertical of the comb structure only a basis is made on which further optimalisation is required. Optimalisation is possible for the ribs A and beam B. Possibilities for optimisation are:

- pre-stressing of beam B to reduce the necessary reinforcement, the pre-stressing does not have to be centric since the moment in the beam can only work in one direction
- reduction of the span of the ribs from 1.3 metres to 1.0 metres with as result that no stirrups are needed
- enlarging the thickness of the ribs with again as result that no stirrups are needed
- changing the dimension of the ribs and the beam to get a minimum usage of concrete

A very schematic cross section of the power house caisson is given in Figure VII.25.

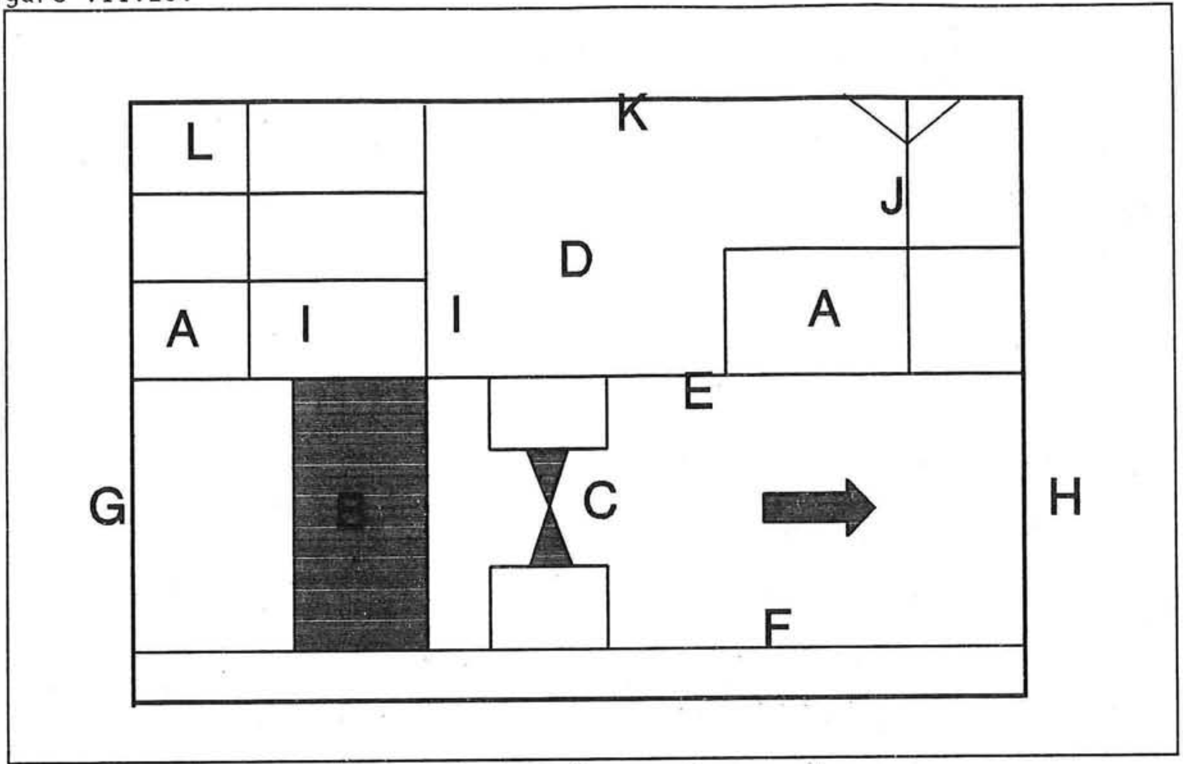


Figure VII.25 **Schematic design of power house caisson**

In this Figure the letters A to L stand for:

A: equipment to lift and lower the door
B: generator
C: turbine
D: transformation room and transport lines
E: top of draft tube structure
F: bottom of draft tube structure and of caisson
G: entrance draft tube
H: exit draft tube
I: support structure for crane
J: support for railway
K: roof caisson (also road)
L: entrance caisson for people

The length of the power house caisson is variable with the number of turbine units placed in one caisson. If 8 turbine units are placed in one caisson the length of the power house caisson is 130 metres. This length has been taken for the caisson for stability reasons and for feasibility reasons. The length of a caisson should be two or more times the width of the caisson to reassure a stabile transport. The stability of the caisson in the width has not been checked since the weight of the turbines and generator are not known. The feasibility of the caisson forces us to keep the length of the caisson limited. A long caisson is more amenable to resonance as result of waves.