STUDY INTO MULTI-PURPOSE OPERATION OF FENI RESERVOIR, BANGLADESH

Low head hydro power in an irrigation project

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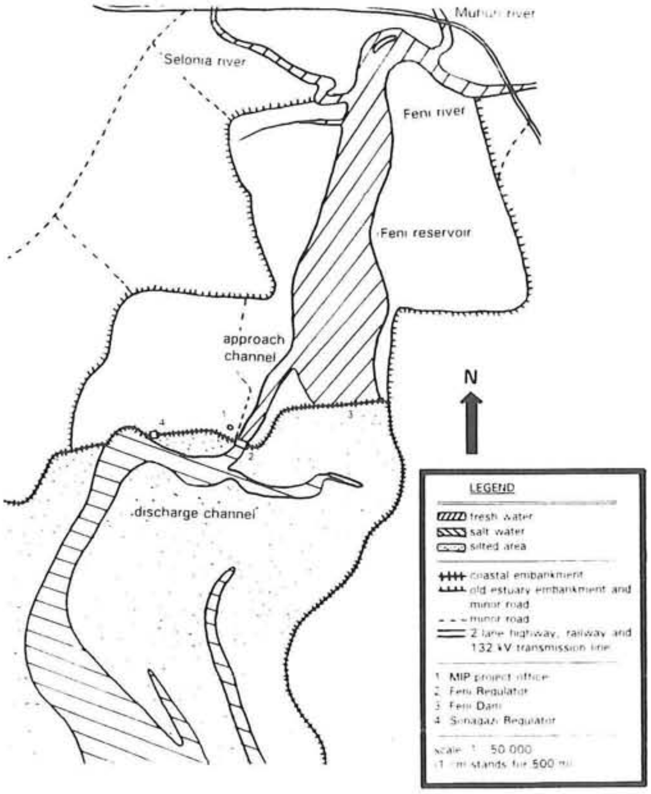
A Master thesis for Delft University of Technology,
Faculty of Civil Engineering

Summary

This report is a Master thesis for the Faculty of Civil Engineering of Delft University of Technology. The subject of this thesis is to find out how the surplus water of an irrigation project can be used for the production of hydro-electricity.



The irrigation project in question is an existing irrigation project in Bangladesh, the Muhuri Irrigation Project. It is located in the lower reaches of Feni River, near Sandwip Channel, a part of the Bay of Bengal; the main relevant features of the project are a reservoir, a dam and a large control structure (the Regulator), through which surplus water can be spilled toward the sea. A schematised view of the situation is presented in the accompanying figure.

The objectives of the Muhuri Irrigation Project are the provision of irrigation water in the dry season (December to April) and the protection against flooding in the wet season (May to October). The initial problem definition was as follows:

What is the optimum water use for the Muhuri Irrigation Project from an economic point of view, and what is the best type and size of hydro power plant for the water that is available for hydro power, following this optimum distribution of water.

The study started with a reconnaissance of the situation. In that first phase, as much insight and knowledge as possible was gathered, mainly with regard to three aspects:
- the functioning of the Muhuri Irrigation Project;
- the natural circumstances with regard to the project, as far as they influence the subject of the study;
- the state of electricity production and consumption in Bangladesh.

An important aspect, that will have its impact on this study, is the strong tidal influence at the downstream end of the project. The high tide deposits large quantities of silt in front of the Regulator. In the wet season, this silt is consequently washed away during the low tide, by the outflow through the Regulator gates. In the dry season however, the silt fills

up the discharge channel and the stilling basin of the Regulator. The results of this reconnaissance phase are presented in Chapter A2 to A5.
In Chapter A6, some important conclusions are drawn from the exploratory study. In short, these conclusions are the following:

- Hydro electricity production is only possible in the wet season, when the water is not needed for irrigation. In the wet season, its priority is second, after the first objective of the Muhuri Irrigation Project in that season, which is the protection against flooding. Still, the production of hydro power is possible with the help of an advanced management system and reliable forecasts of inflow.
- The characteristics of the projected power plant are a combination of a run-of-river plant and a tidal power plant. The plant is comparable to a run-of-river plant because the outflow is directly determined by the inflow; there is no possibility for long-term storage in the reservoir. On the other hand, the water level downstream from the plant is determined by the tide, which means that production is only possible during certain hours of each tidal cycle.
- Because of this downstream influence, the moment of production cannot be adapted to the electricity demand curve. However, the plant at Feni can be connected to the base load hydro plant at Kaptai; in such a situation, production at the Kaptai plant can be stopped during the time that there is production at Feni. In that case, production is useful at any time of the day.

With the information from the reconnaissance phase, the problem definition can be revised and specified. Determination of the optimum water use is irrelevant to the study, because of the different priorities in the two seasons: in the appropriate season for hydro power there is no competing requirement of the water. Boundary conditions, assumptions and objectives can be defined as well. This program of requirements is included in the report as Phase B; it is recapitulated integrally in this summary.

- <u>Problem definition</u>

> Taking into account the special character of the natural circumstances, what is the optimum application of hydro power from Feni Reservoir, more precisely:
> - annual production period;
> - size of production;
> - location of the plant.

- <u>Boundary conditions</u>

- The natural availability of water;
- Downstream water levels;
- Geology (including topography and morphology) of the region;
- Technical standard in Bangladesh;
- Economic situation of Bangladesh;
- Social situation of Bangladesh;
- State of the infrastructure in the region.

- <u>Assumptions</u>

- The goal of this study is educational in the first place; a boundary condition can be neglected if it reduces the educational value of the study;

- The available information is correct;
- Hydro power production is only the third priority of the MIP, after irrigation and flood control;
- Future upstream developments (e.g. in India) are not taken into account;

■ <u>Objectives</u>

The main objective of this study is to find the economically optimum application of hydro power for the water of the Muhuri Irrigation Project, that does not impair the initial objectives (irrigation and flood control) of the MIP.

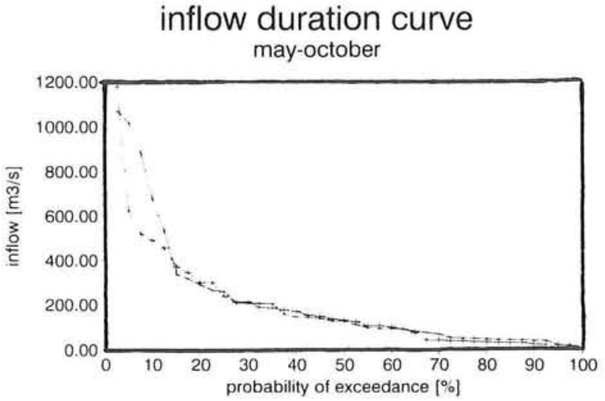
For this, an extensive analysis of available data regarding discharge and hydrostatic head has to be performed. With the results of this data analysis, the following aspects of this application will be studied and determined:
- design values;
- structural characteristics of the plant;
- capacity [MW] and output [GWh];
- location;

The economic value of the solutions will be assessed mainly on a qualitative basis, because detailed economic information is either unavailable or unreliable.

This program of requirements serves as the basis for the remaining part of the study. The mentioned data analysis concentrates on the availability of water and on the hydrostatic head over the plant, because these factors determine the nature, size and production of a hydro power plant. The data analysis is treated in Phase C, the resulting design process in Phase D.

The availability of water in the wet season can be presented by the following inflow duration curve, in which the reservoir inflow is plotted against the probability of exceedance.

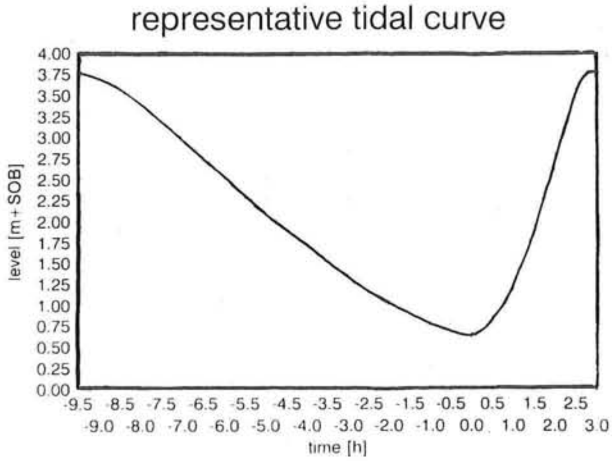
The inflow that is featured in this figure is the inflow into the reservoir.



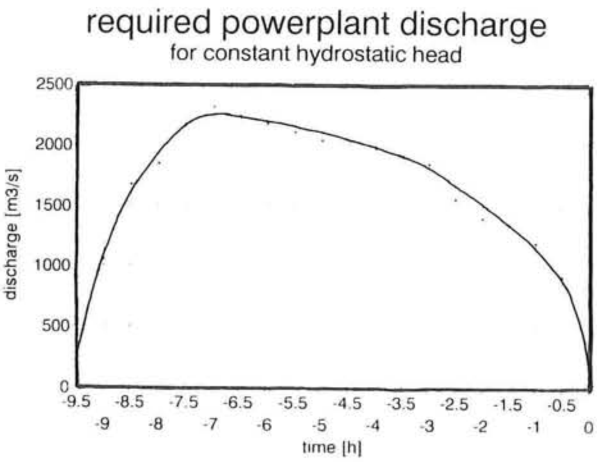
This inflow is not equal to the discharge through the plant: the tidal influence downstream from the plant means that production is only possible during a limited time per tidal cycle. In case of a certain reservoir inflow, a certain volume of water flows into the reservoir in the 12h24 of a tidal cycle. For the solution with the selected probability of exceedance, this is the volume of water that the power plant must be able to handle within the restricted production period.

For design, a *probability of exceedance* of *30%* has been selected on the basis of data from literature and existing projects. The *inflow* with that probability of exceedance is *200* m^3/s, according to the inflow duration curve. The design *outflow volume* then is $8.928 \times 10^6\ m^3$.

The production period per tidal cycle has to follow from the downstream tidal curve. The following curve can be derived from the available data:



Production in a hydro power plant is most effective if the hydrostatic head during production is constant. This is the basis for the selection of the production period: the design production period is the period in which the design volume of 8.928×10^6 m^3 is spilled, with such a rate that the reservoir water level falls by the same distance as the tidal level within that period.
At any moment the outflow that is required to keep the hydrostatic head constant is equal to the gradient dh/dt of the tidal curve multiplied by the reservoir area. Following this reasoning, a curve of the required discharge at each time during the falling tide can be constructed. The result is the following figure:



In this figure, the cumulative outflow between two points in time is equal to the area below the figure between these points. The end of the production period logically occurs at low tide, so the start of production can be calculated with an integration calculation.

In the following integral, t_1 is the start and t_0 the end of production.

$$8.928 \times 10^6 = \int_{t_0}^{t_1} Q_R(t)\,dt$$

$Q_R(t)$ stands for the required discharge.

This integral can be solved for a parabolic approximation of $Q_R(t)$, and the result is a *production period* per tidal cycle of *2 hours and 15 minutes*, the end of production coinciding with low tide.
The downstream level during production directly follows from the tidal curve; the reservoir level depends on the level at the start of production. It is assumed that, with the help of the mentioned management system, a maximum level of SOB +3.81 m (which is 3.81 m above the sill of the Regulator) should not cause an excessive risk of flooding. If the reservoir level at the start of production is SOB +3.81 m, the *hydrostatic head* during the design production period is *2.74 m*.
The relatively low hydrostatic head and the varying production circumstances lead to a choice for *bulb turbines* for this project. The size of the turbines depends on the hydrostatic head as well as on the design discharge.
It appears that if the design volume of 8.928×10^6 m^3 is spilled with a constant discharge of 1100 m^3/s during the design production period, the variation of the hydrostatic head is not so large that it would reduce the efficiency too much. Therefore, a *power plant discharge capacity* of *1100 m^3/s* is selected for design.
Finally, the *number and size of the turbines* has to be selected. Qualitative economic considerations lead to the selection of *6 turbines* with a *discharge capacity* of *183 m^3/s*. From this, a *turbine diameter* of *4.96 m* can be calculated, with the help of a calculation method based on regression analysis of data from existing projects.

With these data, indications of the structural dimensions of the power plant have been determined. The maximum excavation depth, which follows from the requirement to prevent cavitation inside the turbines, is 12.7 m below the reference level. The length of the structure (perpendicular to the flow) would be about 87 m. For a power plant of these dimensions, and with the requirements that follow from the boundary conditions, the best location has been selected on the basis of qualitative economic considerations.
The main problem with regard to the implantation of the power plant in this environment is, that some sort of protection against downstream influences is required; the plant has to be protected from cyclones, but more importantly, it has to be protected from the siltation that is brought in by the high tide. It appears to be impossible to place the power plant behind the Regulator (on the upstream side), because the elevation of the sill of the Regulator is too high. Some other solution, that may have to be structural, will have to be found for this problem. This is however not treated in detail in this study.
The selected location for the plant is inside the Feni Dam proper, as much to the western end as possible.

The last phase of this study is the evaluation. In this final phase, the feasibility of the designed solution is assessed; furthermore, the design process is evaluated. The conclusions are, that the resulting design is probably not feasible in the present situation. The following reasons are given:

- The fact that most of the water for production originates in India means, that there is uncertainty about the future availability of water;
- Flood control and hydro power production have conflicting interests. As a result, hydro power production would only be possible with the help of an accurate management system. It is unlikely that such a system is possible in the present situation.
- Large scale siltation would occur in the downstream basin. This problem can be solved either by a structural protection or by dredging. This would however increase the investment costs to a large extent.
- Even without these extra costs, it is doubtful whether the investment costs would not be too high to be gained back by the relatively low output of the plant.

It is therefore questionable whether the optimum application of hydro power has been found. If less strict requirements of efficiency had been posed to the design, the result would have been a plant with a smaller output and smaller investment costs. The cost/benefit ratio might have been better for such a plant. It may even be necessary to resort to mini hydro power, which would result in a principally different solution, without major structural changes. The efficiency of production and the output would then be very low compared with the selected solution, but so would the investment costs.

However, it seems that if hydro power production is possible at the site, it would be because of the large possible discharges: the only way to use this feature to its full extent is with a solution similar to the one designed in this thesis. Furthermore, the educational value of designing a smaller-scale solution would have been much lower.

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VOORWOORD

Een voorwoord hoort een persoonlijk getinte tekst te zijn, die niet in direkt verband staat met de inhoud van het onderzoeksrapport. Vandaar dus in het Nederlands en niet in het Engels.

Als gevolg van mijn afstuderen is het afgelopen half jaar nogal veelbewogen geweest. Eerst al de reis naar Bangladesh, van 24 januari tot 4 maart. Een onmisbare periode in het kader van dit onderzoek, maar vooral ook een periode vol met nieuwe indrukken op allerlei gebieden.
De rest van de maand maart en de eerste helft van april werden gevuld met de rapportage van de eerste fase, en met de verhuizing naar Nijmegen. Vanaf 18 april heb ik op kantoor bij Haskoning kunnen werken. De ruime aanwezigheid van voorzieningen was vaak erg nuttig voor de voortgang van het projekt, en bovendien erg leerzaam; allerlei computertoepassingen en WP-functies waar ik voorheen nog nauwelijks van had gehoord heb ik intussen wel onder de knie gekregen door ze te gebruiken voor mijn projekt.

Al met al beschouw ik de afgelopen periode als geslaagd; ik heb me vaak afgevraagd wat ik zou kunnen produceren als ik ergens lange tijd full-time mee bezig zou zijn, en dat ben ik nu dus te weten gekomen.
Het is in bepaalde opzichten wel jammer dat we ervoor gekozen hebben om de studie af te ronden voor het eind van het academisch jaar. Hierdoor was de totale duur van het afstuderen weliswaar ruim zes maanden, maar eigenlijk waren alleen de laatste viereneenhalf intensief. Bepaalde aspecten van het onderwerp heb ik daardoor niet de aandacht kunnen geven die ik ze had willen geven (bijvoorbeeld een wat meer uitgebreide economische analyse en een wat meer gedetailleerd ontwerp). Maar goed, er zullen nog wel meer 'projektjes' komen.

Tenslotte dan de bedankjes, in willekeurige volgorde:
De commissie (de heren D'Angremond, Bezuijen en in het bijzonder Van Duivendijk, zonder wie dit projekt zeker niet mogelijk was geweest); Haskoning, voor het verblijf in Bangladesh en voor de geboden faciliteiten in Nijmegen; het Lamminga Fonds voor de tickets; Md.Atiqur Rahman, voor de onmisbare hulp bij de informatieverzameling in Dhaka en Feni; Dhr. M.J.W.Hennus, door wiens eindeloze gezeur ik toch dat stukje scherpte voor de goal wist te behouden (Zwijg!); Tess; Ronnie, Chans en toestel negen voor de dagelijkse inspiratie; Dirk Frans voor Ami Bangla bolte parina, etc.; verschillende Haskoning-medewerkers voor hun hulp en raad en familie en vrienden voor de mentale steun.

INTRODUCTION

This report is the description of a study into multi-purpose operation of an irrigation project in Bangladesh. This study was performed as the final part of the Master program in hydraulic engineering at the Faculty of Civil Engineering of Delft University of Technology.

The irrigation project in question is the Muhuri Irrigation Project (MIP). It is situated in the south-east of Bangladesh, near the coast of the Bay of Bengal, on the Feni river. The MIP has been operational from 1985, when the Feni estuary was closed by an embankment dam. This closure created a small reservoir; water can be spilled to the Bay of Bengal through a regulator and a discharge channel.

Since that time, the project protects the area from cyclones and extreme springtides, and provides water for irrigation in the dry season. The climate of Bangladesh is determined by the monsoon; this causes large differences between the wet and the dry season. From November to April, the weather is so dry that irrigation is required for agriculture; from May to October, rainfall is so abundant that the natural drainage capacity of the land is often insufficient. For the MIP this means, that the reservoir inflow is stored as much as possible in the dry season, and spilled toward the sea as much as possible in the wet season.

The initial definition of the problem was, what the optimum water use would be in this situation from an economic point of view, following from a detailed study of inflow, outflow and water use.
After initial research into the situation, the problem definition of the study was revised and specified, and became: in what way could hydro power be applied for the MIP in an economically optimum way, without endangering the other objectives of the project (dry season irrigation and safety against flooding).

For the solution of this problem, a complete description of the input values was needed. Especially the availability of water and the levels of the water inside the reservoir and downstream from the reservoir are important, as these three factors determine the production of a hydro-electric plant. For both these aspects, representative values were derived from the available data, taking into account the inflow from the rivers, rainfall and evaporation data and irrigation requirements.

From the values thus determined, the type and dimensions of the plant have been determined, using qualitative economic considerations for the selection. Finally, a location has been selected, based on the selected plant and the natural circumstances.

This report is divided into four phases. Phases are divided into chapters and chapters into sections. Tables and figures are named after the phase that they are part of and the chapter in which they occur. All tables are included in the text itself. All figures (graphs and illustrations) are presented at the end of the report; some figures that are essential to the build-up of the thesis are included in the main text as well. Lists of both the tables and the figures are included in general Appendices 1 and 2. Appendix 3 is a list of abbreviations. References are placed in square brackets, and a list of references is included at the end of the main text. The appendices are named after the phase that they refer to.
All levels are related to the Survey of Bangladesh (SOB) - level.

Phase A reports on the reconnaissance of the situation, in Phase B the program of requirements of the study is defined, in Phase C the available data are analysed and in Phase D the design of some aspects of the project is described. Finally, the results of the study are evaluated in Phase E.

Phase A: Reconnaissance of the situation

1.INTRODUCTION

The initial problem definition had to be derived from the formal description of the subject of the project, which was the starting point of this study. It reads:

> Multi-purpose operation of Feni Reservoir, Bangladesh
>
> The Feni estuary was closed in 1985. The freshwater reservoir behind the dam is providing water for irrigation in the Muhuri Irrigation Project.
> Surplus water (i.e. water which cannot be stored in the reservoir because it is full and which originated from the river Feni and local rain on the reservoir) is spilled through the Feni Regulator into the Bay of Bengal.
> Spilling is at present done on a more or less continuous basis during the wet season in order to counteract siltation of the channel between Regulator and Bay of Bengal.
> It has been proposed to construct a small hydropower plant which would generate energy by guiding the surplus inflow through the turbine(s).
>
> Aspects to be studied:
>
> - inflow into reservoir (average, minimum, peak) as it varies during a season and also during the past 8 years;
> - outflow as spilled through the reservoir and used for irrigation;
> - optimum water use as follows from agricultural practices and related costs and benefits of irrigation water supply and hydropower energy;
> - design of hydropower plant.

This description is also presented in Appendix A1.

The following problem definition can be derived from this description:

> **What is the optimum water use for the Muhuri Irrigation Project from an economic point of view, and what is the best type and size of hydro power plant for the water that is available for hydro power, following this optimum distribution of water.**

Very little was known about the project at this stage, and information about practically every aspect was still missing. For example:

- the distribution of river inflow over the year;
- irrigation use, and costs and profits of irrigation;
- outflow data;
- tidal curves;
- functioning of the MIP;
- siltation and erosion downstream;
- topography of the region;
- state of the electricity grid in Bangladesh and more specific in the Feni region;
- costs and profits of electricity in Bangladesh;
- experiences with similar projects.

Some information was available in the Netherlands, from former DUT students who had performed their Master thesis on Feni Dam, and from Haskoning. This information mainly concentrated on Feni Dam itself, and it was clear that many data that were necessary for this study would have to be retrieved in Bangladesh, either at the project site itself or in Dhaka.

For this particular study, there was an opportunity for a six-week stay in Dhaka, to be financed partly by the university and partly by Haskoning. This stay took place from the last week of January to the first week of March. During that time, the Dhaka offices of many companies and organisations that could possess relevant information were visited, including:

- Bangladesh Water Development Board (BWDB);
- Power Board;
- Systems Rehabilitation Project (SRP);
- Canadian International Development Agency (CIDA);
- World Bank and International Development Association (IDA);
- Bangladesh Inland Waterways Transport Authority (BIWTA).

The project site was visited as well on one occasion. During that same trip, the project office of BWDB in Feni and at the Regulator, and the SRP office in Feni were visited.

This stay in Bangladesh was very fruitful, and resulted in:

- much factual information, of which the relevant aspects are described in the following sections;
- insight into the functioning of the project and into the organisational structures in Bangladesh;
- some sets of photographs, that are included in Appendix A2. The first set was taken out of the airplane on the flight from Bangkok to Dhaka on January 23rd; the others were taken during the field visit on February 2nd.

The first chapter following this Introduction is a general introduction into the geography of the project. After that, the data and insight that were gained during the reconnaissance phase are described; the information is divided into three subjects:

- the functioning of the MIP (Chapter 3);
- the natural circumstances regarding the project (Chapter 4);
- electricity in Bangladesh (Chapter 5).

After that, reiteration to the problem definition has to occur. This will result in a revised definition, and a more elaborate program of requirements, including boundary conditions, assumptions and objectives.

2.GENERAL DESCRIPTION OF THE REGION

The location of the project is specified in Figures A2.1 to A2.4, on four different scales: 1 : 20 000 000, 1 : 2 500 000, 1 : 1 000 000 and 1 : 90 000.

The People's Republic of Bangladesh is situated on the eastern end of the Indian subcontinent. Almost all of its borders are with India; only at the south-eastern end it has Myanmar (formerly Birma) as its neighbour. The country extends between 88^0 en 93^0 E and between 20^0 and 27^0 N, so the tropic of Cancer passes through the middle of the country. To the south lies the Bay of Bengal.

Bangladesh is divided into 5 divisions: Dhaka, Chittagong, Khulna, Rajshahi and Barisal. Dhaka is the capital and the principal city, with more than 5 million inhabitants; Chittagong is the largest sea port and the second city with 1.5 million inhabitants.

The clearest physical features of Bangladesh are its flatness and its abundance of water; most of the country has been created by the delta of the Ganges-Brahmaputra system, which annually causes large-scale inundations and deposits its sediments onto the land and into the Bay of Bengal. The country is divided into two as yet unconnected parts by the widest branch of the delta (from upstream to downstream called Jamuna, Padma, Meghna and Shahbazpur). In the south-east the region of the Chittagong Hill Tracts, along the Indian border, is an exception: it consists of steep, jungle-covered hills.

The area of Bangladesh is about 144,000 km^2, which is roughly 3½ times the size of the Netherlands. It has a population of about 115 million, or 750 people per km^2. This is the highest population density of all countries in the world larger than 1200 km^2. Moreover, the population grows rapidly, by about 2% each year. The population is even more concentrated along the rivers; presently, there is a constant flow of landless people to the city of Dhaka.

The climate of Bangladesh is tropical. The dominating factor is the monsoon, which is from the north-east (over land) in winter and from the south-west (over sea) in summer. Annual rainfall is already huge (about 2500 mm compared to about 750 mm for the Netherlands), but it is moreover very concentrated from June to September - large parts of the country are often inundated just as a result of the rainfall. From time to time the low-lying areas near the coast and along the rivers are attacked by cyclones from the Bay of Bengal, sometimes with catastrophical effects; not only can there be a large direct loss of life, but the intrusion of salt water damages soil and crops as well.

The MIP is situated near the town of Feni, in the Chittagong Division, at the narrowest point of Bangladesh. Feni is located on the Dhaka to Chittagong highway and railroad, about 90 km north-north-west of Chittagong.
The reservoir is located just downstream from the confluence of the Feni, Muhuri and Selonia rivers, in a stretch of the river that used to be within the tidal influence. The lower reaches of the rivers and the project area itself are flat, coastal plains; a large portion of the catchment area is in the hills of the Indian state of Tripura.

3.FUNCTIONING OF THE MIP

3.1.Introduction

The information for this chapter has been derived from [Brown'86] and [FAO'89].

Two objectives were served by the creation of the MIP:
- improvement of irrigation possibilities by
  - creation of a head from river to irrigable land;
  - obstruction of the salt tidal flow that used to intrude into the region;
- mitigation of the effects of flooding from the river and from the Bay of Bengal by
  - defence against tidal and storm surges;
  - improvement of wet season drainage around high tide.

The last item requires some explanation. The most frequent cause of inundation in the region is the insufficient drainage of direct rainfall. Usually, the lands are drained via flapgate structures in the embankments; around high tide therefore, drainage used to be impossible. The fact that the tide does not intrude beyond the Dam anymore has improved the drainage of the land that drains onto that section of the river.

In this chapter, the background and the structures of the MIP are described.

3.2.History and institutions

3.2.1.Preparation

The first plan for the MIP was conceived in the early 1960s by the East Pakistan Water and Power Development Authority (EPWAPDA) together with an American consultant, the International Engineering Company (IECo). Bangladesh gained independence from Pakistan in 1971 and, after having acquired financial support from the International Development Association (IDA) of the World Bank, the Canadian International Development Agency (CIDA) and the European Economic Community (EEC), BWDB started implementation of the project in co-operation with IECo and a local contractor, in 1978. Costs were estimated at $52 million, and completion of the project was scheduled for 1983. At this stage, World Bank predicted an economic rate of return (e.r.r.) of 16%.

3.2.2.Completion

After many delays in construction and replacement of consultant IECo by Haskoning for the closure dam, the main features of the project were finally completed in 1985. A lot of the work on irrigation facilities still had to be performed (completion of the system of canals, installation of the pumps), but in 1986 the project was declared complete.

As a result of the delays and some mistakes in the estimates, project construction costs had risen to $66.5 million; an increase of 27%. Taking into account these larger costs and the other effects of the project, World Bank calculated a final e.r.r. of 12%.

3.2.3. Evaluation

From the first stages of the project, there had been growing concerns about possible difficulties with regard to erosion and siltation of the discharge channel. Experience with other, smaller regulators in the same area showed, that considerable siltation was likely to occur downstream of the structures during the dry season. And problems did arise: dry season siltation levels in front of the Regulator (at the seaside) reached elevations above the sill level; wet season outflow through the Regulator caused large scouring holes, that threatened to attack the downstream block apron; and finally the meandering of the wet season outflow had such force that a one km stretch of the coastal embankments of the right bank of the discharge channel was eroded away. Some research was done with regard to these three problems, resulting in solutions that will be treated in Section A4.4.

By 1994, it seems the objectives of MIP have partly been achieved: the full irrigation potential has not been reached, mainly due to lack of co-operation and maintenance. However, a considerable rise in dry season rice production has occurred. This increase in rice production has mainly been achieved by the substitution of different crops, like wheat, by rice. Ironically, the sharp decline in the price of rice has made growing of other crops more profitable, but the high ground water level in the project area, caused by the dam and needed for rice production, makes this diversification very difficult.

The project's other objective, prevention of flooding, has also partly been achieved: Feni Dam has protected the project area from some cyclones that devastated large parts of the country. On the other hand, because the structures do represent a certain obstruction of the discharge, many minor floodings due to flash river floods have occurred. Of course, these floods are not as disastrous as cyclone floods.

3.2.4. Institutions

The reservoir is being managed by BWDB from its office in Feni, following an operation and maintenance manual that was written by consultant IECo, and revised in 1987. During the implementation and the first years of operation (until 1990), both World Bank and CIDA staged some inspection missions that offered their recommendations to BWDB. With respect to irrigation, Systems Rehabilitation Project (SRP), a consortium of Dutch consulting firms, is hired to offer its advice. The regulator is operated by a BWDB engineer from an office at the spot.

3.2.5. Reservoir management

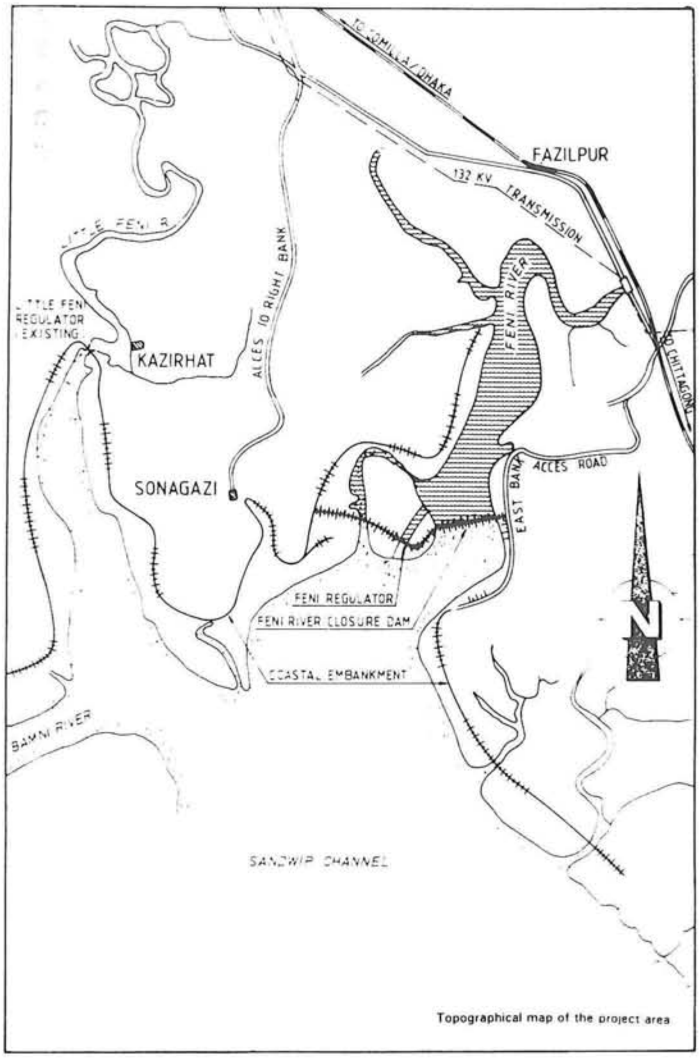
The reservoir is operated in accordance with recommendations from the World Bank. These recommendations are based on the principle that the water level be kept as high as possible until the harvest of the dry season crops, and after that as low as possible to diminish the risk of floods from large river discharges. A description of the practical implementation of these recommendations will be given in Section A3.3.3.3.

3.3.Structures

3.3.1.Introduction

Two structures are relevant to the research into the possibilities of water power generation at Feni: Feni Dam and Feni Regulator. Both these structures will be discussed in this section. Many more structures are part of the MIP (drainage structures, roads and bridges, etc.) but these are considered to be less important yet. Some statistics can be found on photo 6 in Appendix A2. The location of the main structures can be found on the following figure (similar to Figure A2.4).

3.3.2.Feni Dam



Topographical map of the project area

Feni Dam is an earthfill structure with a maximum crest elevation of SOB +10.5 m, a height from foundation mattresses to crest of 14.5 m, and a length of 3.4 km. More details about the dimensions can be found in the cross-section of Figure A3.1 and the site plan of Figure A3.2.

The most important design values (like the crest elevation) were derived from the MIP's objective to shield the region from cyclones and extreme springtides from the Bay of Bengal. It can be noted in Figure A3.1 that both the slope protection and the angle of the slope are designed for more severe conditions on the sea side than on the reservoir side.

Other features, such as the bed protection and the build-up, are determined by the nature of the subsoil and by the extra-ordinary character of the closure operation, which had to be completed up to a certain level within the timespan of one low tide.

The other objective, amelioration of irrigation possibilities, logically posed less strict demands on the structure. Therefore, the crest elevation of the dam is much higher than the crest elevation of the lateral embankments of the reservoir.

The subsoil at the site of the Dam consists mainly of loosely packed silty sand or sandy silt, with occasional occurrence of clay. Bearing capacity is considered generally low [Haskoning'85/2].

3.3.3.Feni Regulator

3.3.3.1.Introduction

Feni Regulator is a reinforced concrete structure that has been designed to control water levels inside the reservoir by regulating the outflow and blocking the incoming tide. It is the largest structure of its kind in Bangladesh. It consists of 40 gates, that each measure 3.66 m by 3.66 m (12' by 12'). The gates are separated by 94 cm (3') wide piers. The sill level of the gates is at SOB 0.0 m. All gate openings are provided with flap gates and radial gates. A transverse section of the Regulator is presented in Figure A3.3. Longitudinal sections are not available; an impression can however be gained from photo 5.

In this section, characteristic and relevant features of the structure, the operation and the environment of the Regulator are described.

3.3.3.2.Gates

Flapgates are doors of steel that hang down from their hinges, and that only open in case of pressure from upstream to downstream. They are located on the downstream side of the Regulator, facing the sea side. As a result, these gates allow water to flow out of the reservoir, but prevent the intrusion of salt water into the reservoir around high tide.[1]

The radial gates are on the upstream side of the Regulator, right in front of the flap gates. With the radial gates, the rate of discharge can be adjusted. When closed, they form a spillway at a level of 3.81 m (12.5') above the sill of the Regulator (see photo 4). The radial gates can each be operated separately with their own 440 volts, 3 phase motors. In case there is a power failure, there is an emergency 200 kW generator; the gates can be operated manually as well.

3.3.3.3.Operation

The principles governing the management of the reservoir levels were mentioned in Section A3.2.5: the water level has to be kept as high as possible until the harvest of the irrigated dry season crops, and after that as low as possible to diminish the risk of floods from large river discharges.

[1]Some intrusion can still be expected, if the downstream water level exceeds the sill level, but the reservoir level is even higher. In that case there will probably be an underflow into the reservoir.

In practice, keeping the water level as high as possible during the dry season means, that all radial gates have to be kept closed all the time; in that way, the reservoir level is maintained at the spillway level of the radial gates (SOB +3.81 m)[2]. Exception is made for a specially designed program to counteract siltation downstream. This will be described in more detail later on.
In the wet season, keeping the reservoir level as low as possible means, that the radial gates are kept open permanently.

A special point of interest is the guideline from World Bank to always open all gates simultaneously. This point was only added when it appeared that there was excessive downstream erosion in the wet season; it was hoped that equal spreading of the flow would lead to less erosion. It seems this measure has produced the effects that were intended.

3.3.3.4.Hydraulic characteristics

The following two tables, that were derived from the [IECo'83], give an indication of the discharges through the Regulator. It is difficult to calculate the discharges through the gates with theoretical hydraulics formulas, as the regularity of the outflow is strongly disturbed by the flapgates. As explained earlier, the momentum of the flow opens the flapgates.

Table A3.1 shows the discharges through one gate at a time and through all gates of the Regulator in case of free outflow with a reservoir level of SOB +3.81 m (the spillway level, that normally occurs during the dry season).

GATE OPENING [m] ([ft])	DISCHARGE THROUGH 1 GATE [m^3/s]	REGULATOR DISCHARGE [m^3/s]
0.61 (2)	11.5	459
1.22 (4)	21.9	877
1.83 (6)	31.3	1250
2.44 (8)	39.4	1577
fully open	45.4	1815

Table A3.1: Regulator discharges at spillway level (SOB +3.81 m)

Table A3.2 shows the discharge through the Regulator as a function of reservoir water levels, with all gates fully open.

[2]Obviously only as long as the inflow is larger than irrigation use plus evaporation plus outflow.

RESERVOIR ELEVATION [m]\[ft]	MAXIMUM DISCHARGE [m^3/s]	AVERAGE DISCHARGE [m^3/s]
3.35\11	1497	629
3.66\12	1702	838
3.81\12.5	1815	949
3.96\13	1915	1056
4.27\14	2136	1269
4.57\15	2364	1484
4.88\16	2598	1699
5.18\17	2840	1900
5.49\18	3087	2166

Table A3.2: Regulator discharges with all gates open

The 'maximum discharge'-value in Table A3.2 is the discharge that occurs when the downstream level is low enough for free outflow to occur. The 'average discharge'-value reflects the average discharge through the gates during a 12h24 design tidal cycle. This value differs from the maximum discharge because outflow is impeded partly as long as the sea water level is high enough to disturb the critical flow through the gates, and even completely as long as the flapgates are closed.
An elevation of 4.90 m (16.1') is considered to be a threshold level for flooding of the lateral embankments of the reservoir (see Section A4.4). Therefore, levels much higher than this (hopefully) are not reached normally.

Some more datasets that relate measured levels to calculated discharges are available [Halcrow'91]. These could be used to construct a more detailed and calibrated rating curve for the Regulator. However, these sets do not seem to be altogether consistent. They will be analysed more profoundly later on.

3.3.3.5.Foundation

The materials constituting the subsoil of the Regulator are comparable to those of the Dam. However, the layer structure is probably different: the Dam is located in the middle of the estuary bed, while the site of the Regulator used to be a sandbank in most recent history.
For the construction of the Regulator, the subsoil was not excavated much beyond the lowest point of the structure; the substructure consists of sheet piling down to an elevation of SOB -21.3 m (-70') below the structure itself and down to SOB -11 m (-36') at the downstream end of the stilling basin. The substructure is also included in Figure A3.3.

For design of any structure near the Regulator, the influence on the subsoil and substructure of the Regulator itself would have to be studied carefully; because of the loosely packed soil, an event such as a small earthquake would be enough to cause liquefaction, resulting in large deformations of the soil.

4.NATURAL CIRCUMSTANCES

4.1.Introduction

The amount of power that is generated in a hydro power plant depends essentially on two values: discharge and hydrostatic head. A more detailed description of the formulas that determine this possible capacity will be given later on.

The power plant discharge depends on the water availability, which is equal to inflow minus outflow. Sources of inflow and causes of outflow are described in Section A4.2. The hydrostatic head is the difference between the water levels upstream and downstream from the plant. The data that are available on levels are described in Section A4.3. Geographical circumstances can influence the discharge and the hydrostatic head to a large extent as well; apart from that, they determine the location of the power plant. Relevant aspects of the geography of the project area are given in Section A4.4.

4.2.Water availability

4.2.1.Introduction

Firstly, the components that make up the reservoir inflow and outflow are presented and discussed; after that, there's a description of the different sources of information and methods of calculation with regard to the inflow into Feni reservoir and the discharge through the Regulator.
The actual design values will be determined later on.

4.2.2.Components of inflow

4.2.2.1.Introduction

The inflow into the reservoir is made up of two factors: inflow into the reservoir by means of rivers, and rainfall onto the reservoir area itself. Both these factors are of course mainly determined by the climate of the region. In this section, a short recapitulation will be given of this climate, after which the resulting input parameters will be discussed.

4.2.2.2.Climate

The climate of the project area is mainly determined by the monsoons: during part of the year, winds are negligible and come from the north, while during another part strong south-western winds prevail. The air of this south-western monsoon has had to pass over the Bay of Bengal before reaching Bangladesh (see Figure A2.1), is therefore very moist, and brings a lot of rain.

As a result of the monsoon, there's a clear distinction between a dry and a wet season: a dry season from December to April and a wet season from May to November. March is the driest month, while July is the wettest.

Annual rainfall near Feni is about 3 m. To put this value into perspective: the weather station of Cherrapunji in India, about 250 km to the north of Feni, is well known for recording record rainfall values, up to 2.6 m per month in the wet season and more than 13 meters annually. On the other hand, annual rainfall in the Netherlands is about 0.75 m, which is spread out nicely over the year, contrary to the situation in Bangladesh. The annual distribution of the rainfall at Feni will be treated later on.

Bangladesh is in the northern hemisphere, right on the tropic of Cancer. For Bangladesh, this means that summer coincides with the wet season and winter with the dry season. As a result of that, the values of the temperature, and therefore of the evaporation, are (even) higher in the wet season than in the dry season.

4.2.2.3. River inflow

Feni reservoir is fed by four sources: Feni, Muhuri, Selonia (on some maps called Khalidas Paharia Khal) and local inflow. The following table presents the size of the catchment areas of these rivers, together with Bangladesh's and India's respective approximate shares of this area.

NAME	AREA [km^2]	BANGLADESH [%]	INDIA [%]
Feni	1466	30	70
Muhuri	1103	62	38
Selonia	267	40	60
local inflow	75	100	0
Total	2911	49	51

Table A4.1: Catchment sizes [SRP'91]

The consequences of the magnitude of India's share of the catchment area are important. This will be discussed in Section A4.4.

4.2.2.4. Rainfall

The second source of inflow into the reservoir is direct rainfall. For the MIP, direct rainfall probably is not very important compared to river inflow: the area of the reservoir is approximately 17 km^2 (see Figure A2.4), while the total catchment area is almost 3000 km^2 (Table A4.1): that's half a percent. Obviously, there is also some groundwater flow into the reservoir from the directly neighbouring land, but this is probably negligible compared to total inflow, as this land is very flat, and is used for intensive agriculture.

4.2.3.Components of outflow

4.2.3.1.Introduction

In the present situation, by far the largest part of the annual outflow is the wet season spilling through the Regulator. It is quite useless to try to quantify this amount directly: presently, it is the amount that remains after quantification of the other terms.

The main quantifiable causes of outflow out of Feni reservoir are irrigation use and evaporation.

4.2.3.2.Irrigation use

There is only need for irrigation in the dry season, from mid-December to the end of April. The required amount of irrigation water depends on the type of crop and on the state of the crops; it therefore varies per user, but it also varies in time.

The area of irrigable land in the MIP is about 23000 ha, according to the official figures. It seems however, that the available amount of water does not suffice for more than 14000 ha [SRP'92/2].

4.2.3.3.Evaporation

Evaporation seems to influence the dry season irrigation capacity of the project to a large extent. According to some sources, evaporation consumes about a third of the water that is available in the dry season. Even though, apparently, this is a common value for projects like this, design values of the irrigation capacity were too high partly as a result of underestimation of the evaporation.

4.2.4.Sources of information

4.2.4.1.Introduction

From the reservoir management principles as described in Section A3.3.3.3, and from indications of the dry season situation of the MIP, it seems to follow that the attention in this study will probably have to be focused on the wet season as the season for hydro-electricity production: in the dry season the main part of the water will be needed for irrigation.

No direct data of inflow rates are available: the river discharges into the reservoir have to be calculated from discharges at some gauging stations more upstream, along both the Feni and the Muhuri rivers.
Over the years, there have been several programmes of research: into inflows during the whole year by IECo, for the design of the structures and the irrigation capacity of MIP; and later on into dry season inflows to reassess the irrigation capacity of the MIP. This section presents the sources of information for this subject and surveys the possibilities of obtaining more wet season data with the help of dry season calculation methods.

4.2.4.2.IECo 1973

The first calculations of inflow into Feni reservoir were done by IECo in their 1973 feasibility study for the MIP, and later on, for the final design of the Regulator. The results of this research were presented in the operation and maintenance manual [IECo'83]. They are presented here in Table A4.2.

MONTH	AVERAGE INFLOW [m^3/s]	RAINFALL [mm]	EVAPORATION [mm]
January	22.9	10.2	58.4
February	20.3	25.4	66.0
March	19.3	63.5	94.0
April	31.0	177.8	119.4
May	72.2	302.3	132.1
June	241.1	566.4	106.7
July	285.9	630.0	83.8
August	234.2	630.0	73.7
September	145.4	411.5	94.0
October	118.3	193.0	88.9
November	66.3	35.6	94.0
December	31.0	12.7	58.4
Total	107.3	3058.4	1069.4

Table A4.2: Inflow components [IECo'83]

The values in Table A4.2 are based on the following datasets:
inflow: monthly averages from 1960 to 1973.
rainfall: monthly totals for 54 years of record.
evaporation: monthly totals at Noakhali for 1979-1980

The 'total' value of the inflow in the table is actually the total monthly average, while the other two totals are the real cumulative totals.[3]

It has to be noted that the 'inflow'-values were presented in the source report as 'discharge at Feni Regulator site'. This is incorrect: the given values have been calculated directly from the river discharge of the Muhuri river; irrigation use and other interactions have not been taken into account.

The values in this table were used for the design of the Regulator and of the irrigation section of the MIP.

4.2.4.3. Dry season inflow

In more recent years there has been more research into the inflow of the MIP, after doubts had risen concerning the correctness of the numbers of Table A4.2. In the meantime, more gauging data had become available and calculation methods had become more sophisticated. Companies involved were Delft Hydraulics in 1983 and SRP from the late eighties onward [SRP'91].

This new research was only performed for dry season discharges, to assess irrigation water availability for MIP. For these calculations, data of the gauging station at Parshuram, along the Muhuri river, have been used. The catchment area of the Muhuri at Parshuram is estimated at 648 km^2, or 22% of the total catchment area (see Table A4.1). This station has been operational since 1959. Among hydrologists, it is common practice to calculate the discharge of a river as a linear function of the size of its catchment area, if geological and climatological circumstances can be considered similar.

For the region draining onto MIP however, this was not thought to be the case, because the catchment areas of the lower Feni, the lower Muhuri and of the local inflow are of a very different nature from the more upstream regions. While the upper regions consist of hills, the lower ones are flat floodplains. This difference has important consequences for the drainage of the land: hilly terrains characteristically drain fast and do not allow much time for evaporation, while flat terrains retain rainfall for a longer time, with more seepage and more evaporation as a result. Apart from this, in the lower regions more dry season rainfall will be retained by farmers for purposes of irrigation, because the lower regions are more intensely populated and farmed.
In the first place, this effect influences the calculations of the inflow into the reservoir, as will be explained hereafter. Apart from this, it affects the ratio between India's and Bangladesh's share in the inflow as well. The consequences of this will be discussed in Section A4.4.

For their dry season calculations therefore, both Delft Hydraulics and SRP-hydrologists considered these lower regions to contribute less to the total inflow. Delft Hydraulics still allowed for some dry season discharge from the floodplains, determining a total inflow of 4 × Parshuram flow, but SRP did not: in fact, they determined the total dry season inflow

[3] A graphical representation of this table has been made in a later phase; it can be found in phase C, as Figure C2.1.

to be 3.2 × Parshuram flow, while the total catchment area is 4.5 times the one at Parshuram. Apparently, some 840 km^2 were considered not to add to the inflow. Indicative calculations and measurements have shown, that SRP's formula is very accurate for the dry season.

4.2.4.4.Wet season inflow

Since IECo's study in 1973, no more wet season calculations are known to have been performed. Clearly, IECo's dry season estimates were too high: whereas their report gives an average March inflow of 20 m^3/s, in reality the long term average inflow value of the driest month is about 13 m^3/s [SRP'91]. IECo's value is apparently some 50% too high. The question is: does the same go for the wet season values?

The considerations that caused hydrologists to annul the floodplains' contribution to the inflow are not valid to the same extent for the wet season as well. It seems that there will be at least some wet season inflow from the flood plains, even though there is still more evaporation there than in the hills. Firstly, relatively smaller quantities of water will be retained by farmers for irrigation purposes; secondly, the quantity of rainfall is so much larger that not all of it could disappear into evaporation (even though temperatures are higher in the wet season (Section A4.2.2.2)), and seepage would only add to the groundwater flow into the reservoir. Clearly, wet season inflows can be calculated by multiplying Parshuram flow with a factor somewhere between 3.2 and 4.5. Design values will be determined in a later phase of this study. The datasets that will be used for this (among which data on levels and discharges at Parshuram) will be presented then as well.

4.2.5.Floods

Extreme discharge values are not needed for the calculation of the hydro-electric potential at Feni; depending on the location of the power plant, they are needed for the structural design of such a plant though. Table A4.3 was included in the final design report of the Regulator.

Comparison to Table A3.2 shows that for the Regulator to be able to cope with this 100-year flood, reservoir levels of about SOB +4.5 m, or almost 15' would be needed. This level should not cause flooding, as will be explained later on.

More research into flash floods has been done by SRP [SRP'92]. One conclusion was, that recorded gauging data are very unreliable. A Gumbel analysis of floods in the relevant gauging stations was performed. A flood with a 10 year return period was analysed more profoundly; for this flood a peak inflow rate of 2450 m^3/s, a peak Regulator discharge of 2160 m^3/s and a duration of 6 to 10 days were determined, resulting in a reservoir level of SOB +4.8 m. Comparison of these data with design data from [IECo'83] (Tables A3.2 and A4.3) does not produce a consistent picture; apparently, IECo underestimated flood inflow and overestimated dry season inflow.

RECURRENCE INTERVAL [years]	DISCHARGE [m^3/s]
1.01	375.2
2.33	856.8
5	1097.6
10	1338.4
30	1780.8
50	2013.2
100	2329.6

Table A4.3: Flood frequency at Regulator site [IECo'83]

4.3.Levels

4.3.1.Introduction

As stated in the introduction to this chapter, the power that is generated in a hydro power plant depends on the hydrostatic head over the plant as much as on the discharge through the plant. Hydrostatic head simply means the difference in water level between the upstream and downstream side of the plant. As a result of the geographical and climatological circumstances, and of the management of the MIP, both upstream and downstream levels vary very much through the year.

In this section, firstly a summary of historic measurements of upstream and downstream levels will be presented in separate subsections. These two datasets will be analysed more profoundly later on; the resulting distributions will then be used for the design of the power plant.

The given data have been measured at the Regulator site, with an electrical gauge, according to instructions derived from the design of the MIP; they have been recorded by a BWDB employee.

Obviously, for research into the hydro-electric capacity of a site, the extreme values are not essential: average or representative values are needed. Most datasets however contain extreme values, per month or per day. From the daily extremes though, statistical patterns, such as tidal curves, can easily be derived.

4.3.2.Upstream levels

4.3.2.1.Introduction

As stated in Section A3.2.5, reservoir levels are kept as high as possible during the dry season and as low as possible during the wet season. As a result, two clearly distinct periods occur, with very different statistical characteristics.

4.3.2.2.Dry season

In the dry season the radial gates are closed all the time[4]. Reservoir levels vary only little, and are around SOB +4 m until well into March; this is some 20 cm above the spillway level of the Regulator (see Section A3.3.3). Apparently, there is a surplus of inflow until that time, and there is a practically constant flow over the radial gates. After about March 15, the level does drop to about SOB +3.5 m around April 15. That is the point when the harvest of the rice starts, when irrigation is no longer required. At that moment the gates can be opened, and for the MIP the wet season starts.

The stability of the water level (no sudden quick rise or fall) is caused by the relatively low inflow and outflow rates in the dry season (see Table A4.2).

4.3.2.3.Wet season

In the wet season, it is much more difficult to give statistical characteristics of the reservoir levels: the levels are determined directly by the inflow into the reservoir and by the downstream water level. Combined with the fact that inflow and outflow rates are high compared to the volume of the reservoir (see Section A4.3), the reservoir level varies much more than in the dry season.

Measurements indicate, that the minimum value can be as low as the sill level (SOB 0, see Section A3.3.1); maximum values can be as high as SOB +4.5 m, at the top of a flood from the rivers, with all gates opened. Some sort of average value seems to be between SOB +2.5 and SOB +3 m. As already stated, a more profound analysis of these data will be performed later on, as this first part of the study is only exploratory.

It has to be noted, that the mentioned values are very characteristic for the present state of the project, with the present operation recommendations. In a situation with a power plant, the guidelines will probably be different; therefore, the mentioned values are probably irrelevant for design. (The same cannot be said about dry season reservoir levels and downstream water levels in general: they will probably be similar in a situation with a power plant)

[4]not taking into account the operation of the anti-siltation program, see Section A4.4.

4.3.3.Downstream values

4.3.3.1.Introduction

There is no sea level gauging station in or near the MIP: the nearest station is at Sandwip Island, operated by the Bangladesh Inland Water Transport Authority (BIWTA). Calculation of tidal levels at Feni from there is very complicated, but it has been done. Obviously, the closure of the river in 1985 has caused changes to the situation.
A year round characteristic of the tidal curve at the site is its asymmetry: the flood flow only lasts some 3 hours, which leaves almost 9½ hours for the ebb flow.

Just like the water level inside the reservoir, tidal levels are heavily influenced by the time of the year: continuous high pressure areas over the Bay of Bengal in the dry season influence sea levels on the coast of Bangladesh as well. As a result, the mean sea level is substantially higher in the wet season than in the dry season [Haskoning'85/2].

The mean sea level over the whole year is SOB 0.0 m [Brown'86], and the mean tidal range is 3 m (see Figure A4.1).

4.3.3.2.Dry season

During the dry season, downstream water levels at the Regulator vary about 2 m: maximum levels are usually between SOB +2 and SOB +3 m, minimum levels range between SOB 0 and SOB +1 m.

It has to be noted that low tide levels in the stilling basin, just downstream from the Regulator, are often lower than the bed level in this season, as a result of the heavy siltation; there is usually some outflow over the radial gates though, so that's the water level that is measured in that case.

4.3.3.3.Wet season

Measured levels in the wet season show a greater variance between high and low tide: high tide levels usually range between 3 and 4 m, low tides between SOB -1 and SOB +1 m.

These levels might often be higher than the astronomical high tide because of large outflow through the Regulator in the wet season. According to some research [SRP'92], the hydraulic condition downstream from the Regulator is even more determined by the capacity of the discharge channel than by the tide, as a result of the siltation of the discharge channel. As a result, 'drowned' outflow[5] through the Regulator can be

[5]A situation that occurs when the downstream water level is too high for a hydraulic jump to be formed; the outflow is then overlaid by a turbulent mass of water. This results in a reduced discharge [Henderson'66].

expected, even at low tide. However, this seems less probable in the wet season, when the capacity of the discharge channel is much larger than in the dry season.

It is surprising to note, that the low waters are apparently lower in the wet season than in the dry season; this contradicts the remark in Section 4.3.3.1 concerning the higher mean sea level in the wet season. This difference is mainly caused by the fact that in the wet season, low waters below SOB can be measured because the deposited silt has been washed away then, and the bed level of the stilling basin reaches SOB -4 or -5 m (see Section A4.4).

4.4. Geography

4.4.1. Introduction

Various aspects of the geographical surroundings of the project are relevant to the subject of hydro-electricity production from Feni reservoir. These aspects will be treated in the direction of the flow, from upstream to downstream.

4.4.2. Upstream developments

As stated in Section A4.2.2, of the area that drains onto the Feni reservoir, 49% is in Bangladesh and 51 % is in India, in Tripura State (see Table A4.1). In India, this catchment area consists of hills; in Bangladesh though, only part of the area is hilly, while another part is low, flat, alluvial terrain. Due to this difference in terrain, India's share in the amount of water is larger than 51%. Especially in the dry season, the amount of water that evaporates from the alluvial plains in Bangladesh is relatively larger than from the hills. India's share in the amount of inflow into the reservoir is estimated at about 60%.
This is a matter of great concern, as relations between India and Bangladesh are not that good traditionally, so that predictions of future developments in India are very unclear; this especially causes uncertainty as far as irrigation is concerned, because irrigation possibilities depend on dry season inflow. During the planning stage of the project, the amount of irrigable land was even reduced by some 10% for this reason, as World Bank inspection reports indicate [FAO'89]. Contrary to irrigation, hydro-electricity production would mainly be determined by wet season discharges, which are less vulnerable to developments in India because wet season inflow

- originates from India for a relatively smaller part because in this time of the year, the alluvial plains in Bangladesh do produce part of the inflow;
- will be influenced to a smaller extent by developments in India because there is less need for irrigation in the wet season.

Apart from possible irrigation schemes in the future, the main concern in this respect is caused by the deforestation that is happening in Tripura; this development changes the distribution of the inflow in time, making the rivers even flashier than they already are. This could have a negative impact on the prospects of hydro-electricity at Feni, as there is practically no room for storage in the reservoir (see Section A4.4.3); electricity production would thus become flashier as well.

Finally, the possibility of hydro-electricity development in the hills of Tripura cannot be excluded either. This would clearly influence a possible Feni power plant, but the Muhuri Irrigation Project to a much greater extent.

A second matter of concern is the fact, that more and more farmers upstream, whose lands strictly do not belong to the project area, are using water from the contributing rivers for irrigation. Again, this development will mainly affect dry season inflows into the reservoir, so that it will not have a seriously detrimental influence on hydro-electricity production downstream.

4.4.3.Reservoir

Upon flowing into the reservoir, water velocities drop radically, which causes siltation just downstream from the point of inflow. Until now, this siltation has not caused problems, as a reduction of the volume of the reservoir does not interfere with its objective to maintain a certain water level.

The reservoir is a very shallow one: it pairs an area of 17 km^2 with a volume of $2.9*10^7$ m^3 [Brown'86]. The average depth then is 1.7 m. Because of this, the storage capacity of the reservoir is very low (and getting even lower because of the siltation). To illustrate this: it takes an average July river discharge only 28 hours to replace all the water in the reservoir. Of course, floods would take an even shorter time; a time that could coincide with high tide, and therefore with closed flap gates. This means, that in the wet season water levels have to be kept at as low a level as possible, to prevent flooding through large inflows.

4.4.4.Irrigation

The project area covers about 40000 ha, of which some 25000 ha are irrigable (some 16 by 16 kilometre). Water is used for irrigation in the dry season; this enables farmers to grow High Yielding Varieties (hyv) of rice, even then. The irrigation season starts around December 15 and finishes when the harvest starts, around April 15. From the reservoir, irrigation water flows into supply canals, and from there it is pumped onto the farmland [Brown'90].
Logically, the extent to which the water can flow into the region depends on the water level and on the bed level of the canals. For irrigation water to reach a certain point in the canal, a minimum water level is required; however, due to lack of maintenance of the supply canals, siltation has caused these minimum levels to rise significantly. In fact, at the end of the dry season, considerable areas of farmland cannot be irrigated because of this.

In some cases, wet season irrigation would be beneficial as well, but so far, costs of installation of pumps have blocked this development. Also, adaptation of reservoir management for this cause would probably be too difficult to implement: this would increase the risk of flooding from the rivers to such an extent that any extra income will not be enough of an argument in favour.

4.4.5.Floods

The crest level of the embankments surrounding the reservoir is not very high. In the design of structures of the project, it was estimated that a reservoir level of SOB +4.90 m (16.1') would probably be a maximum level, because above this level outflow over the embankments onto the land would become so large that it would balance inflows.
This low embankment level logically poses strict limitations to the maximum water levels that can be allowed inside the reservoir. The elevation of the project area is estimated to be between SOB +4 and +5.5 m (13'-18'). It is clear that large parts of this area could be inundated even with slightly higher levels, depending on the duration of the flood. In the wet season of 1993 there was some flooding from the reservoir, for the first time since the closure of the river. No data are available on the size of this flood.

More common are small size floods, that occur as a result of so-called impeded drainage: a combination of large rainfall and high river water levels with poor drainage facilities of particular areas of farmland. These floods usually are not caused by high reservoir water levels.

4.4.6.Outflow

4.4.6.1.Introduction

The reservoir is connected to the Regulator by an approach channel which is designed for a flow of 840 m^3/s [IECo'83]. Surplus water flows through the Regulator, either through the opened radial gates when spilling is considered necessary or over the closed radial gates when the reservoir has reached its spillway level of SOB +3.81 m (12.5').

Downstream from the Regulator, a stilling basin was constructed, in order to reduce the energy of the outflow through the gates. The bed level of this basin is at SOB -6.71 m (-22'). It is protected by a block apron (see Figure A3.3).

The water then flows into the discharge channel. This discharge channel is very active in a morphological sense, which has important effects in both seasons. These effects will be described in the following sections.

4.4.6.2.Dry season

In the dry season, sediments that are carried into the discharge channel by the flood flow are not transported back into the Bay of Bengal, as in that period there is hardly any discharge from the reservoir. As a result, enormous quantities of silt are deposited in the channel, up to the Dam and the Regulator, filling the stilling basin and reaching an elevation of SOB +1 m (see Figure A4.2 and photo 3). Consequently, sediments are deposited against the flap gates of the Regulator as well, blocking them in the process. In this situation, if a flash flood from the rivers would occur, it would take a large force to open the flap gates up against the sand; this might cause flooding of the banks of the reservoir. In order to prevent such an occasion, a program of flushing has been designed: every five to seven days, all radial gates are opened for five to ten minutes (all gates at once). The pressure of the water then opens the flap gates, and the deposited sediments are washed away.

One effect of this program is a loss of irrigation water. Indicative calculations in [Brown'86] suggest however, that the lost amount of water 'is not excessive and does not detract from the reservoir's major objective of providing water for irrigation'. A total annual value of $6.2*10^5$ m^3 is mentioned. This is less than 2.5% of the volume of the reservoir.

As data of reservoir levels indicate, there is a surplus of water well into the dry season; extra water flows over the top of the closed gates in that situation. However, around March, the critical time at the end of the dry season, water levels do drop below levels that are necessary for irrigation of some parts of the project area. Even though the main cause for this seems to be a lack of maintenance of the canals, it might prove useful to review the need for the flushing program: it seems a maximum level of silt around SOB + 1 m is reached each time, depending on sea water levels at high tide. It might be that without flushing, silt levels would not rise above this level; the program of flushing could then be reduced to flushing just once, before the beginning of the wet season, maybe even after completion of the harvest. In that case, there would not be any unnecessary water loss at all.
However, the following reasons plead for retaining the program as it is: the fact that there is a surplus in the beginning of the wet season; the need for testing of the radial gates during the dry season; the fact that flushing away dried and compacted silt is more difficult; and the possible coincidence of water shortage and the need for flushing at the end of the dry season in case the program is abolished.
This issue however is not relevant to the subject of this study, as it only concerns the dry season situation. Therefore, it will not be discussed further.

4.4.6.3.Wet season

From the moment the gates are opened when the dry season harvest is completed, there is no need for flushing anymore: in fact, the dimensions of the discharge are so large that the flow scours the stilling basin up to a depth of four or five meters below SOB; the block apron of the stilling basin was even under threat from a scouring hole further downstream. As this became apparent, World Bank added an extra point to its operation regulations, namely that all gates should only be opened simultaneously. After this, physical model studies have been performed at BUET in Dhaka, to find constructive solutions to this problem [Halcrow'91]. It was concluded that, indeed, the main cause for the scour had probably been incorrect gate operation rather than large discharges.
Improvement of inspection and repair in case of failure were proposed rather than reinforcing the construction of the apron; obviously, it is very difficult to carry out any maintenance on the block apron in either dry or wet season (due to either sediments or flow) without high costs. A 'wait and see'-attitude was therefore considered more economic.

From the stilling basin, in the downstream direction, the discharge channel turns into a meandering river, attacking, and even conquering, the embankments of the channel (see Figure A4.3 and photos). In February 1994, a set of groynes was built in order to protect the restored embankments against further attacks.

4.4.7.Land accretion

In front of the dam, large areas of deposited silt have consolidated and are already being used for grazing of cattle, even though high springtides still reach the dam through a small tidal channel (see photo 7). Research has shown, that from the Dam to the discharge channel, a landlevel of about SOB +3.4 m has been reached in about 3 years time [Barua'90]. This is 1 m above mean high water, which is a good practical level for empoldering. In the mentioned study, it was predicted that empoldering could start in 1991 or 1992. However, according to sources a quick implementation of this project has to be considered unlikely.

Empoldering of this area could be very useful from this study's point of view; more about that later.

5.ELECTRICITY IN BANGLADESH

5.1.Introduction

The preceding chapters describe the supply side of electricity production. In this chapter, the demand side, and the state of the electricity system in Bangladesh are treated.

First the present situation with regard to consumption and production is described. After that, the expectations with regard to some relevant aspects are mentioned.
Much of the information in this chapter was derived from [Halcrow'87].

5.2.Energy consumption

Bangladesh has one of the lowest levels of commercial energy use in the world: average use per capita per annum was 1.14 GJ in 1984. To illustrate this: 1.14 GJ is the amount of energy that is consumed if a person uses 145 W during six hours of each day of the year. This level is so low, that it is less than 15% of the average commercial energy use of the 34 World Bank-defined 'low income countries'. Actual energy use is higher because of extensive use of traditional fuels in rural area's; these traditional fuels make up about 70% of Bangladesh's total energy consumption.

No official data on the distribution of electricity consumption through the year are available yet, but this is expected to be reasonably constant, as it is always warm anyway. If there is a difference, summer consumption is probably higher as a result of more refrigeration and air-conditioning.
Obviously, a very small section of the society accounts for a large share of the totally consumed energy: only few people have such luxuries as refrigeration or airconditioning.

5.3.Power generation

5.3.1.Introduction

Despite this low level of energy use, demands still exceed supplies. In January 1994, total installed generation power was about 2500 MW, but between a third and half of this is out of order due to lack of maintenance funds [WPDCfeb'94]. Meanwhile, the peak load of the system was about 1700 MW. It has to be added that around a third of this demand is consumed by illegal connections to the grid. In an attempt to share the burden of the temporary shortages, a program of load shedding (temporary shut down of supply in turns for every region) has been implemented.

The majority of fuels has to be imported, mainly in the form of crude oil and coal. This import consumes a large part of Bangladesh's total export earnings: up to 87% in the beginning of the 1980s, but the share did decrease afterwards. In 1984 it was down to 45%.

5.3.2.Fossil natural resources

Fuel resources are limited in Bangladesh: the most important one is natural gas. This has an important share in power generation, but only in the eastern part of the country, where it is found as well; as yet, there is no transport capacity across the dividing Jamuna river.

There is a considerable amount of coal, but only in very deep layers, which makes exploitation expensive. Even so, exploitation is starting.

Thirdly, there are reserves of peat, mainly in the south-western part of the country, near Khulna. Here as well, technical problems make exploitation too expensive.

5.3.3.Hydro power resources

The second largest power source of the country, after natural gas, is hydro power. All hydro power in the country is generated by one hydro power plant: Kaptai, in the Chittagong Hill Tracts, with an installed capacity of 210 MW. It produces 850 GWh annually.

Contrary to thermal power, hydro power is very suitable for providing peak power, because starting up and closing down of production is simple and quick. In this way, it can be more efficient for the system to produce the same amount of energy in a shorter time, when energy demand is higher. However, because there's a shortage of base power as well, and because Kaptai's traditional function in the electricity grid of Bangladesh is to provide base load for the area of Chittagong, this useful feature of hydro power is not used.

In spite of an abundance of water in the country, it is difficult to extract more power from it because of the flatness of the country. More about this in Section A5.4.2.

5.3.4.Situation in Feni area

As a result of the fact that Bangladesh is only a few kilometres wide near Feni, one of the most important transmission lines of the country, from Comilla to Chittagong, passes trough Feni. Usually, the region gets its supplies from Chittagong through the national grid, sometimes it receives power from the north via Comilla (see Figure A2.2). The average demand at the city of Feni is about 32 MW, peaks are around 35 MW.

5.4.Relevant aspects

5.4.1.General

In 1994, the population of Bangladesh was 115 million, and growing by about 2% each year. Two developments cause an even larger increase in energy use:

- to cater for the growing population with the limited available agricultural area, agricultural methods will have to become more intensive (more irrigation: more use of pumps and fertilizers);

- due to a decrease in agricultural employment, the urbanisation and industrialisation process will continue even more vigorously.

These two developments will cause an increase in energy intensity of the economy of Bangladesh; it is evident that satisfying such growth in demand will become more and more difficult, if the already huge energy imports are taken into account. The government searches for new ways to generate power, and hydro power is one of those.
First, the experience and expectations in this respect are summarised. Then, a table of expected investment costs for power plants in Bangladesh is presented. Finally, the supply costs of energy in Bangladesh are treated.

5.4.2.Hydro power

5.4.2.1.Introduction

As stated before, hydro power development in Bangladesh is a very difficult thing because it's a practically flat country. Still, some research has been done, and not all of it has had only negative results. Some possible developments are described in this section.

5.4.2.2.Chittagong Hill Tracts

The most probable new project is an extension of Kaptai with an extra 2 × 50 MW for base load; location for these two turbines has already been provided for. Apart from this, research has been done into raising the reservoir level, which would possibly enable the installation of 200 MW more.
There has also been some research into hydro power development on two more rivers to the south of Kaptai, in the same area, namely at the Sangu river (300 GWh, either by a 150 MW long-peak power plant or by a 4 × 100 MW short-peak power plant) and at the Matamuhari river (200 GWh, by a 100 MW long-peak power plant).
Finally the region offers possibilities of pumped storage power plants, in combination with the reservoir that serves the plant at Kaptai.

However, implementation of any project that involves more submerged area is more than likely to be prevented by environmental concerns, by the possibility that the reservoir might stretch into India in that case, and most of all by concerns for the traditional way of life of the indigenous peoples, like the Chakma's, that live in the Hill Tracts.

5.4.2.3.River barrages

There are plans to construct a run of river power plant on the Brahmaputra river, in the framework of a larger river rehabilitation project. Generation capacity of both these plants would probably be in the order of hundreds of MW, but there are many doubts about the feasibility: investment costs would be so high that plants like these would not be competitive with natural gas power plants.

5.4.2.4. Existing irrigation dams

The advantage of constructing a hydro power plant at an existing dam is evident: investment costs are much lower, because dam and reservoir have already been created; only the plant itself remains to be paid for. However, there is also a clear disadvantage: usually, power generation interests will have to be subject to irrigation and flood control interests, so that firm power cannot easily be guaranteed for specific times. Some research has been done into hydro power potential at the Burri-Teesta Irrigation Project, in the north of the country, but so far no clearly positive results have been obtained.

This study is about the same type of project.

5.4.3. Investment costs of new plants

The following table displays predictions of the investment costs per unit generation capacity for newly to be constructed power plants. It must be noted that these data are a little bit out of date: they stem from [Halcrow'87] again, but the data that were used for that report are from even further back, namely from 1984. Still, it is believed that they can serve as indicative values.

PLANT TYPE	INST. POWER [MW]	CONSTR. COSTS [$/kW]	CONSTR. PERIOD [years]	FIXED O&M COSTS [$/kW]	VAR. O&M COSTS [$/MWh]	AVAIL-ABILITY [%]
gas	20	350	1	10	2	80
steam	300	870	5	6.5	1.5	85
combined	90	620	3	10	2	70
coal	300	1025	5	7.8	1.75	80
peat	25	1600	5	62.5	5	80
nuclear	300	2130	6	36	8.42	80
hydro	100	478	2	7	-	100

Table A5.1: Investment costs of new plants

The hydro plant that features in this table is the first extension of the Kaptai plant with 2×50 MW that is supposed to serve as a base load plant, as indicated by the availability factor of 100% (see Section A5.3.2.2).

It has to be clarified that this table has not been included to be used as some argument in favour of hydro power (although it does seem to compare favourably to the other sources in this case), but only to serve as an indication of investment costs per kW for power plants in Bangladesh, for later use to determine the economic feasibility of a hydro power plant at Feni.

5.4.4.Energy supply costs

More recent data are available on energy supply costs in Bangladesh; for the fiscal year 91/92 Table A5.2 is valid.

The numbers in Table A5.2 are average numbers, that do not take into account the differences in fuel costs per kWh between the East Zone (natural gas, therefore cheap) and the West Zone (imported oil products, therefore expensive).
The average tariff is Tk 2.00 per kWh (Tk 40 equals US$1).

CATEGORY	COST [Tk/kWh]	COST [$/kWh]
fuel	0.79	0.01975
repair & maint.	0.17	0.00425
depreciation	0.79	0.01975
salaries	0.20	0.005
administration	0.05	0.00125
interest	0.44	0.011
others	0.03	0.00075
total	2.47	0.06175

Table A5.2: Energy supply costs '91/'92

6.CONCLUSIONS

6.1.Introduction

From this reconnaissance, some conclusions can be drawn that should help to clarify the situation; the problem of the study could then be defined more exactly. To finish off this reconnaissance phase, these conclusions are presented in this section.

For further design it is essential to have a clear image of the role of the project in its environment. The position of power generation at Feni reservoir can be defined from three different points of view:

- the position of power generation within the Muhuri Irrigation Project;
- the interaction of the power plant with the natural circumstances;
- the position of the power plant within the electricity grid of Bangladesh.

These conclusions will then result in a formal program of demands for the remaining part of the study.

6.2.Priorities

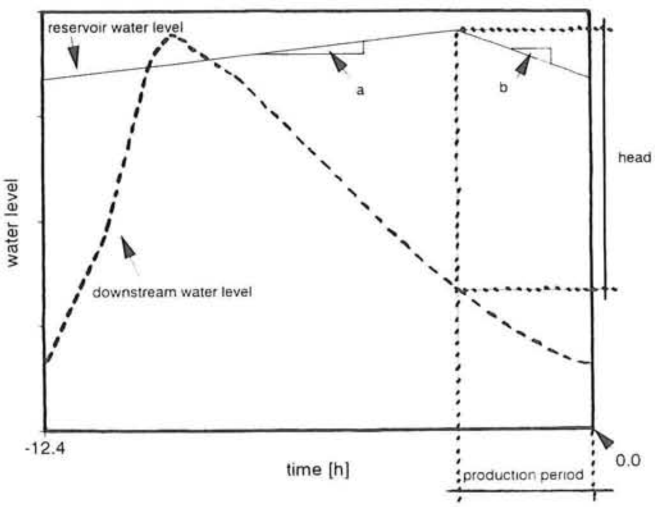
The first conclusion to be drawn is, what the function and position of hydro power production is in the system of the Muhuri Irrigation Project.

It can be assumed that production is only possible in the wet season. Presently, the first and only priority of the MIP in the wet season is, to spill enough water during each low tide to prevent flooding of the reservoir embankments during the following high tide. In the present situation, this goal is achieved by keeping the radial gates open permanently.
In the situation with a power plant, there will be two exits for the water, namely through the plant or through the Regulator. The first priority then will still be to spill enough water (either through the plant or the Regulator, or both), but the second priority will be to have an optimum electricity production with this water. This means that as much water as possible will have to be guided through the plant instead of the Regulator, with as much hydrostatic head as possible, all the while making sure that a sufficient total amount of water is spilled.

Clearly, to handle a situation like this in an optimum way, a professional and accurate management system will be required, with the help of reliable forecasts of inflow and tidal curve at the site, both on the short and on the longer term. A comprehensive theoretical description of such a system is given in [WPDCnov'84]. It is doubtful whether such a system is feasible under the local circumstances. Still, because of the fact that this study is a university Master thesis (which means it has to be educational in the first place, and practical only in the second place), it will be assumed that operation along these lines is possible.

6.3. Production period

Secondly, the nature of the power plant and the way in which energy would be produced can now be clarified. The operation of the plant has to be adapted to the boundary conditions, mainly the inflow and the tidal level. This interaction is schematised in the following figure.



Explanation:
At t = -12.4 h, both the upstream and the downstream levels are at their minimum. The power plant is closed, so there is no outflow. From that point on, the downstream level rises with the tide, while the upstream level rises as a result of the river inflow, with a rate that is equal to tan (a) in the figure. The downstream level reaches its maximum around t = -9½ h, and starts to fall again. As soon as a certain hydrostatic head is reached, the power plant is opened, and outflow starts. This is the start of the production period.

Two conditions have to be fulfilled during production. In the first place, both the downstream and the reservoir water level have to fall at about the same rate (tan (b) in the figure), in order to keep the hydrostatic head about constant - this improves the efficiency of production.
Secondly, the outflow volume during this production period has to be about equal to the total inflow in the complete tidal cycle; this results in a water level at the end of production that is about equal to the level at the start.
These two conditions will have to determine the production period, the hydrostatic head and the discharge capacity of the power plant.

According to this description, the nature of the power plant is a combination of a run-of-river plant and a tidal power plant. Like in a run-of-river plant, there is no room for long-term storage, and outflow must balance inflow on a short term; like in a tidal plant, the downstream water level varies with the tide, and the reservoir water level varies during production.

6.4. Electricity grid

As far as the production schedule is concerned, the plant will be comparable to a tidal power plant: production is only possible during certain hours of the tidal cycle, when the downstream level is low. As the period of the tide is 12.4 h, the production period occurs 24 minutes later every day; it cannot be fixed to occur during peak demands. Apart from that, the values of both discharge and hydrostatic head during production may vary a lot per tidal cycle, so it would be difficult to guarantee a minimum level of power.

Because of these typical tidal plant-characteristics, linking of the production to a base load plant (a power plant that constantly produces power) is usually the only way to make a tidal power plant economically feasible; the power that is produced by the plant can then be saved at the base load plant[6]. In that case production would be useful at all times. A disadvantage of this system usually is, that thermal base load plants cannot be stopped quickly and easily. In Bangladesh though, about 10% of the base load (210 MW) is produced by the hydro power plant at Kaptai (Chapter 5): it is very easy to stop production in a hydro power plant, so linking the Feni plant with Kaptai would be convenient.

[6]The only operational tidal plant in the world (La Rance in France) is linked to a base load plant as well (nuclear, in this case).

Phase B: Program of requirements

■ Introduction

After the reconnaissance phase, a revised problem definition for the remaining part of this study can be formulated, the boundary conditions and assumptions that apply to this study can be summed up, and the objectives of the study can be summed up.

■ Problem definition

Taking into account the special character of the natural circumstances, what is the optimum application of hydro power from Feni Reservoir, more precisely:

- annual production period;
- size of production;
- location of the plant.

■ Boundary conditions

- The natural availability of water;
- Downstream water levels;
- Geology (including topography and morphology) of the region;
- Technical standard in Bangladesh;
- Economic situation of Bangladesh;
- Social situation of Bangladesh;
- State of the infrastructure in the region.

■ Assumptions

- The goal of this study is educational in the first place; a boundary condition can be neglected if it reduces the educational value of the study;
- The available information is correct;
- Hydro power production is only the third priority of the MIP, after irrigation and flood control;
- Future upstream developments (e.g. in India) are not taken into account;

■ Objectives

The main objective of this study is to find the economically optimum application of hydro power for the water of the Muhuri Irrigation Project, that does not impair the initial objectives (irrigation and flood control) of the MIP.

For this, an extensive analysis of available data regarding discharge and hydrostatic head has to be performed. With the results of this data analysis, the following aspects of this application will be studied and determined:

- design values;
- structural characteristics of the plant;
- capacity [MW] and output [GWh];
- location.

The economic value of the solutions will be assessed mainly on a qualitative basis, because detailed economic information is either unavailable or unreliable.

Phase C: Data analysis

1.INTRODUCTION

In this phase of the study, the boundary conditions with regard to the hydro-electric capacity of the site are analysed. The values that will result from this analysis are the basis for the design of the power plant: its production capacity, its size and its location.

In Chapter 2, the available data on reservoir inflow are analysed, leading to a statistical description of the water availability during the year, and to the selection of the period of the year for which the turbine capacity has to be designed.

In Chapter 3, water levels inside the reservoir and tidal levels just downstream from the MIP are analysed, leading to representative values for the hydrostatic head over the power plant in the selected period of the year.

In this phase, the selection of location of the power plant structure will not be looked into, even though sometimes the location influences the design values. The determined water availability is valid for the reservoir; the determined water levels, both upstream and downstream, are valid for the Regulator site (see Figure A2.4).

2.WATER AVAILABILITY

2.1.Introduction

In this chapter, the amount of surplus water of the MIP and the annual timespan when this water is available will be determined, from the datasets that are available.

First, in Section C2.2, the distribution of the monthly average reservoir inflow over the year, calculated from measured water levels in the Feni and Muhuri rivers, will be determined. Then the other interactions of the reservoir with its environment (precipitation, evaporation and irrigation) will be combined with the inflow data. This will result in a clear picture of how much water is available at which time of the year.

After that, in Section C2.3, the production period (annual scale, i.e. the months of the year when there is a surplus of water) is selected on the basis of the determined distribution of water availability. The available data on inflow values over the selected annual timespan will be analysed statistically, resulting in a duration curve of the reservoir inflow. This curve will be used for the design of the power plant.

2.2.Average water availability

2.2.1.Inflow

2.2.1.1.Introduction

The starting point of this analysis is the set of average reservoir inflows in the table 'Average monthly discharge, rainfall and evaporation data' as it was presented in [IECo'83], or Table A4.2 of this study. The set of inflow data is presented here as well, in Table C2.1. In this section, this dataset will be corrected.

2.2.1.2.Calculation methods

From [IECo'83] the conclusion can be drawn that the data in Table C2.1 have been determined from measured water levels of the Muhuri river at the gauging station at Parshuram (Figure A2.3). Water levels of a river at a certain station can be used to compute the discharge of the river, and with this discharge at a certain station, the total inflow into the reservoir can be calculated.
The procedure that is followed to compute the discharge of the river from the water level is described in [SRP'91]. A rating formula is used of the following type: $Q=Q_0 \times (H-H_0)^m$.
The coefficients of this formula are revised each year; the computed results from earlier years, however, are not revised to bring them in accordance with the new formula. As no data are available on the rating formulas that have been used to calculate the Parshuram discharges from 1960 to 1973, the discharge datasets that were used to compute Table C2.1 will be assumed to be correct. It is certain that they contain errors, but these errors

can be expected to neutralize each other because a different rating formula is used for each year, and the values in Table C2.1 are the result of the average of those formulas.The procedure that is used to calculate reservoir inflow from the discharge at Parshuram was already mentioned in Section A4.2.4: the discharge of the river is multiplied by a factor; this factor is equal to the total catchment area of the reservoir divided by the area that drains onto the station. For [IECo'83], a factor of 4.78 was used for all calculations (wet season and dry season). This value is too high.

MONTH	AVERAGE INFLOW [m^3/s]
January	22.9
February	20.3
March	19.3
April	31.0
May	72.2
June	241.1
July	285.9
August	234.2
September	145.4
October	118.3
November	66.3
December	31.0
Annual average	107.3

Table C2.1: Average monthly inflow [IECo'83][7]

In Table A4.1, the sizes of the catchment areas of all four contributing rivers were presented. In this phase, this table is extended to include a more detailed division of catchment areas, distinguishing hilly and flat areas. The data that are used are, again, extracted from [SRP'91].

CATCHMENT SECTION	AREA [km^2]	AREA [%]
Feni above Kaliachari	1264	43.4
lower Feni	202	6.9
Muhuri above Parshuram	648	22.3
hilly lower Muhuri	150	5.1
flat lower Muhuri	305	10.5
hilly Selonia	160	5.6
flat Selonia	107	3.5
local inflow	75	2.7
total	2911	100

Table C2.2: Size of distinct catchment areas

[7]The values in this table are monthly averages, based on records from 1960 to 1973.

As indicated in Section A.4.2.4 the contribution of every part of the catchment area of the MIP depends on the topographical circumstances (it is assumed that meteorological circumstances are constant for the region): hilly areas drain fast and do not allow the water to evaporate or to seep into the ground, and are less intensively used for agriculture; therefore, the relative contribution of flat lands is smaller, and in the dry season even negligible.

2.2.1.3.Dry season calculations

In [SRP'91], this calculation method was used to compute the dry season inflow into Feni reservoir. It was assumed that, apart from the discharges of the Feni at the Kaliachari gauging station and the Muhuri at Parshuram, only the hilly part of the lower Muhuri area and the hilly part of the Selonia area would contribute to the dry season inflow. Furthermore, taking into account that the flow from the Selonia was apparently being impeded by dams of local farmers, only the 150 km^2 of the hilly lower Muhuri were included in the calculation. As the records of the gauging station at Parshuram appeared the most reliable and dated back the longest, it was decided to calculate the inflow into the reservoir as follows:

$$\begin{aligned}\text{inflow} &= (\text{contributing area/Parshuram area}) \times \text{Parshuram flow}\\ &= \frac{1264+648+150}{648} \times \text{Parshuram flow}\\ &\approx 3.2 \times \text{Parshuram flow}\end{aligned}$$

Measurements have shown that this formula yields very accurate results for the dry season.

2.2.1.4.Wet season calculations

According to [SRP'91], the contribution of the annulled part of the catchment area is also small during the wet season. This contribution however is not completely negligible: in the wet season, relatively much less water will be consumed by irrigation, seepage or evaporation.

CATCHMENT SECTION	AREA [km^2]	FACTOR	CONTRIBUTING AREA [km^2]
Feni above Kaliachari	1264	1	1264
lower Feni	202	0.5	101
Muhuri above Parshuram	648	1	648
hilly lower Muhuri	150	1	150
flat lower Muhuri	305	0.5	152.5
hilly Selonia	160	1	160
flat Selonia	107	0.5	53.5
local inflow	75	0.5	37.5
total	2911		2566.5

Table C2.3: Wet season contributing areas

To determine the correct wet season factor, the same procedure as in the preceding section has been followed; the result is Table C2.3.
In the wet season calculation, the amount of 'lost' water on the flat areas is estimated at 50%. In reality, it is extremely difficult to determine this value, as it depends on factors such as the amount of rain, the succession of rainy periods, the amount of sunshine, etc. The error that is caused by this estimation is not very large: it follows from Table C2.2 that the flows at Kaliachari and at Parshuram represent at least two-thirds, and probably even more, of the contributing area. The water from 'hilly Selonia' is assumed to contribute completely: the local dams are probably provisional, and will be removed in the wet season to prevent flooding.
The corrected formula to calculate wet season reservoir inflow from the flow of the Muhuri at Parshuram can now be concluded from Table C2.3:

$$\begin{aligned} \text{inflow} &= (\text{contributing area/Parshuram area}) \times \text{Parshuram flow} \\ &= \frac{2566.5}{648} \times \text{Parshuram flow} \\ &\approx 3.96 \times \text{Parshuram flow} \end{aligned}$$

2.2.1.5.Corrected values

Now a corrected table of average inflow values can be presented: because a factor of 4.78 was used for the calculation of Table C2.1, the inflow values have to be reduced by a factor ($\frac{3.2}{4.78}$=) 0.669 for the dry season, and by ($\frac{3.96}{4.78}$=) 0.828 for the wet season.
The only matter that is left to be decided, is where to use which factor. According to [SRP'92/2], irrigation is required from the second half of December until the end of April. Using these dates for the change of 'seasons' in this calculation is appropriate, as irrigation plays an important part in the determination of the contribution factors. The following table results:

MONTH	AVERAGE INFLOW [m³/s]	FACTOR	CORRECTED INFLOW [m³/s]
January	22.9	0.669	15.3
February	20.3	0.669	13.6
March	19.3	0.669	12.9
April	31.0	0.669	20.7
May	72.2	0.828	59.8
June	241.1	0.828	199.6
July	285.9	0.828	236.7
August	234.2	0.828	193.9
September	145.4	0.828	120.4
October	118.3	0.828	98.0
November	66.3	0.828	54.9
December/1	31.0	0.828	25.7
December/2	31.0	0.669	20.7
Total average	107.3		87.4

Table C2.4: Average monthly inflow with corrected catchment

2.2.2. Water use and losses

2.2.2.1. Introduction

The amount of water that flows into the reservoir is not the same as the amount of water that flows out of it: apart from deformation of the inflow curve by the reservoir, there is also input from rainfall and output through evaporation and irrigation use.
There is also a certain groundwater flow, but because of the small gradients this is probably negligible.

2.2.2.2. Net precipitation

The data on rainfall and evaporation that will be used originate from the same table in [IECo'83] as the inflow data (Table A4.2). To assess the influence of the net rainfall on the reservoir as compared to the inflow from the rivers, the given values in millimetres have to be transformed to cubic meters per second. The area of the reservoir is 17 km^2.
The required expression is:

$$P_{net}[m^3/s] = \frac{P_{net}[mm] \times 10^{-3} \times 17[km^2] \times 10^6}{30.5[days] \times 24[h] \times 3600[s]}$$

or

$$P_{net}[m^3/s] = 6.45 \times 10^{-3} \times P_{net}[mm]$$

This results in the following table:

MONTH	RAINFALL [mm]	EVAPORATION [mm]	CONTRIBUTION [mm]	CONTRIBUTION [m^3/s]
January	10.2	58.4	-48.2	-0.3
February	25.4	66.0	-40.6	-0.2
March	63.5	94.0	-30.5	-0.2
April	177.8	119.4	58.4	0.4
May	302.3	132.1	170.2	1.1
June	566.4	106.7	459.7	3.0
July	630.0	83.8	546.2	3.5
August	630.0	73.7	556.3	3.6
September	411.5	94.0	317.5	2.0
October	193.0	88.9	104.1	0.7
November	35.6	94.0	-58.4	-0.4
December	12.7	58.4	-45.7	-0.3
Total	3058.4	1069.4	1989.0	

Table C2.5: Meteorological data

The rainfall values are monthly totals, based on 54 years of record; the evaporation data are monthly totals at Noakhali for 1979-1980 (see Figure A2.3)

2.2.2.3.Irrigation use

According to [SRP'92/2], there is an irrigation requirement from mid December until the end of April. The values of irrigation requirement that are calculated, are expressed in millimetres on cultivated land per half month. For comparison with the inflow into the reservoir, these values have had to be transformed to cubic meters per second as well.

According, again, to [SRP'92/2], the average achieved irrigated area from 1986 to 1992 was 17300 ha; this value is much higher than the calculated possible irrigable area (about 14000 ha). Still, because there is clearly a rising trend over the last years, a value of 20000 ha has been selected for the calculation of irrigation use. The required expression is:

$$I[m^3/s]=\frac{I[mm]\times 10^{-3}\times 20000[ha]\times 10^4}{15.25[days]\times 24[h]\times 3600[s]}$$

or

$$I[m^3/s]=0.15\times I[mm]$$

This formula was used to compute Table C2.6.

PERIOD	IRRIGATION REQUIREMENT [mm]	IRRIGATION REQUIREMENT [m^3/s]
December/2	46	7.1
January/1	111	17.1
January/2	149	23
February/1	155	23.9
February/2	134	20.7
March/1	163	25.2
March/2	153	23.6
April/1	97	15.0
April/2	36	5.6
Total	1043	

Table C2.6: Irrigation requirement

2.2.3.Corrected values

A combination of the results of Tables C2.4, C2.5 and C2.6 yields the average amount of surplus water that is available for hydro power production. The resulting available water values in the last column of Table C2.7 are the result of addition of the net rain values to and substraction of the irrigation requirements from the inflow values.

MONTH	CORRECTED INFLOW [m^3/s]	NET RAIN [m^3/s]	IRRIGATION REQUIREMENT [m^3/s]	AVAILABLE WATER [m^3/s]
January/1	15.3	-0.3	17.1	-2.1
January/2	15.3	-0.3	23.0	-8.0
February/1	13.6	-0.3	23.9	-10.6
February/2	13.6	-0.3	20.7	-7.4
March/1	12.9	-0.2	25.2	-12.5
March/2	12.9	-0.2	23.6	-10.9
April/1	20.7	0.4	15	6.1
April/2	20.7	0.4	5.6	15.5
May	59.8	1.1	0	60.9
June	199.6	3.0	0	202.6
July	236.7	3.5	0	240.2
August	193.9	3.6	0	197.5
September	120.4	2.0	0	122.4
October	98.0	0.7	0	98.7
November	54.9	-0.4	0	54.5
December/1	25.7	-0.3	0	25.4
December/2	20.7	-0.3	7.1	13.3
Total average	87.4	1.1	6.7	81.8

Table C2.7: Water availability

The total average values in this table are overall annual averages. The data from this table are presented graphically in the following figure, which is also included as Figure C2.1.

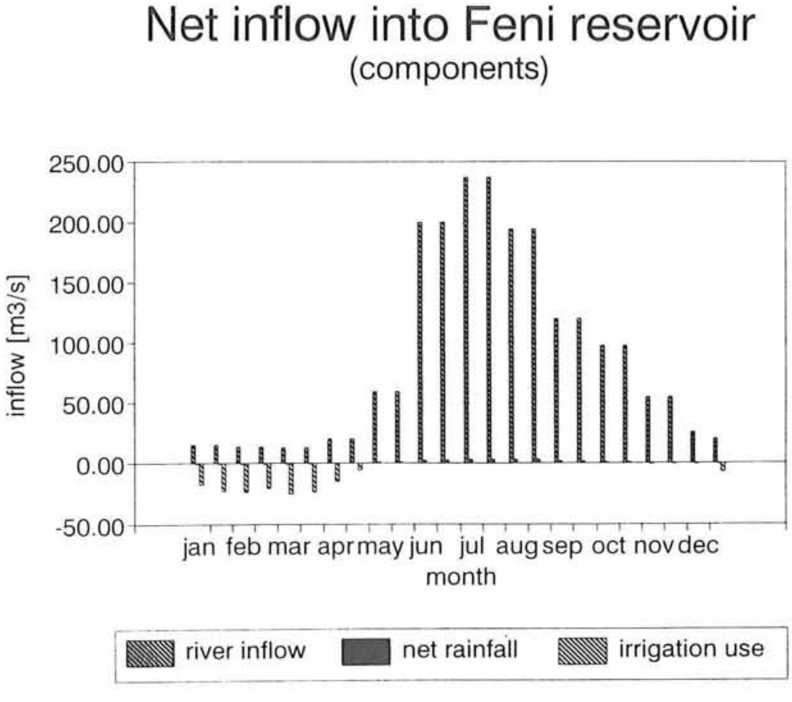


Table C2.7 and Figure C2.1 clearly demonstrate that energy production is only possible during a certain part of the year, during the wet season. The negative results during the dry season indicate that the reservoir water level gradually drops during an average dry season. The wet season values represent the amount of water that is spilled through the Regulator in an average year.

2.3. Availability distribution

2.3.1. Introduction

In order to decide upon the turbine capacity to be installed, a clear and detailed impression of the water availability will have to be attained. The design of a power plant is usually based on a turbine discharge with a certain probability of exceedance; the relation between discharge and probability of exceedance is derived from a duration curve. In this case, the design will be based on a duration curve of the reservoir inflow, as the data that are available are inflow data, and not discharge data. To determine this duration curve, the long-term average values in Table C2.7 cannot be used: more detailed data are required. Table C2.7 can be used later on to check the obtained values.

The distribution of the availability over a year cannot be called homogeneous, and only a certain part of the year matters for hydro power production, so a more or less homogeneous and relevant period of time will have to be selected; from this period a set of measured data will be analysed. Taking into account both Figure C2.1 and the actual datasets, the period from the beginning of May to the end of October is selected.

The datasets that are available are measured water levels of the Muhuri river at Parshuram and of the Feni river at Kaliachari from November 1990 to February 1994. Therefore, the wet season datasets for both gauging stations comprise three wet seasons.
These datasets consist of two-weekly measured water levels, so for each year there are thirteen measurements during the selected period. From these levels the inflow into the reservoir can be calculated as described in Section C2.2.1.

The average duration of a flood in the region is six to ten days [SRP'92/1], so the distinct measurements of each dataset are independent on that scale (whether they are fully independent is doubtful, as the weather is determined by cycles of high and low pressure areas). The datasets of Parshuram and of Kaliachari have to be treated separately: the Kaliachari measurements have all been taken two days after the Parshuram measurements, so these two sets are not independent.

2.3.2. Analysis

As already indicated in Section C2.2, the discharge of a river can be calculated from the water level using a rating formula of the type $Q=Q_0 \times (H-H_0)^m$; the coefficients of this formula are revised each year. [SRP'92/1] describes a study into dry season water availability for the MIP; they concluded that for the gauging station at Kaliachari, 1989's curve is the best one, and for Parshuram 1988's. From presented datasets in [SRP'92/1], the following formulas can be reconstructed:
Kaliachari:
$Q=4.57 \times (H-3)^{2.542}$

Parshuram :
$Q=3.86\times(H-9)^{3.256}$

with Q in [m^3/s] and H in [m+PWD] (PWD = Public Works Datum).
The reference level of PWD is equal to SOB -1.51 m.

The rating curves are presented in Figures C2.2a and C2.2b. These two formulas have been used to calculate the discharges of the rivers out of the datasets of measured water levels. This calculation results in two new datasets, containing two-weekly values of the wet season flows of Feni river and Muhuri river.

Out of these datasets, the total inflow into Feni reservoir can be calculated, both from Parshuram and from Kaliachari. It has to be stressed that it is the total inflow, and not just the inflow of either of these rivers: the river discharge is multiplied by a factor to make it represent the complete catchment area.

For both these gauging stations proportionality relations as explained in Section C2.2.1. can be used. For Parshuram, the same formula as in Section C2.2.1 can of course be used again:
total reservoir inflow $= 3.96 \times$ Parshuram flow

The appropriate factor to calculate the total inflow from the flow at Kaliachari, follows from Table C2.3 in the same way:

$$\text{total reservoir inflow} = \frac{2566.5}{1264}\times\text{Kaliachari flow}$$
$$\approx 2.03\times\text{Kaliachari flow}$$

The calculated inflow values then have to be corrected for the net rainfall onto the reservoir. As Figure C2.1 indicates, this results in minor changes.

The results of the calculations are presented graphically in Figure C2.3. For comparison's sake, the average availability values from Table C2.7 are included in this figure. This figure clearly shows how flashy, unpredictable and irregular the flow of the Feni and Muhuri river is.
Some aspects to be noted:

- the highest peak inflow, in August '91, of 1150 m^3/s, has a recurrence interval of about 5 years [IECo'83].
- quite often, peaks of inflow do not seem to coincide. It has to be noted however, that the measurements of both stations have been taken two days apart; so the flashy character of the rivers can explain this phenomenon.
- the wet season of 1992 was a very dry one.
- it has been assumed in this section that the backwater curve from the reservoir does not extend as far as Kaliachari or Parshuram (see Figure A2.3).

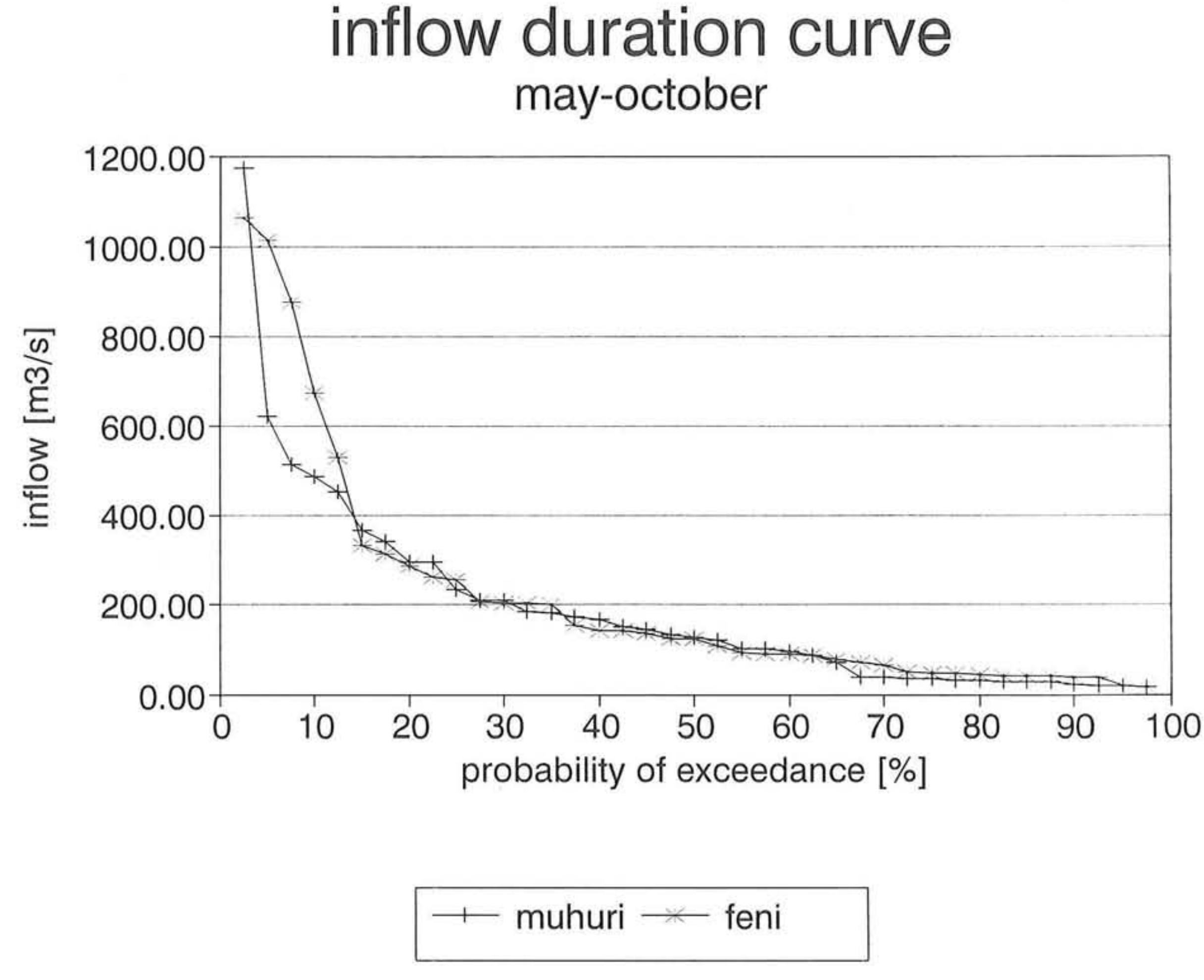
2.3.3.Inflow duration curve

For a better impression of the distribution of the inflow, it is useful to know what the probability of exceedance of each value of inflow is. The curve that represents this relationship is called an inflow duration curve. To construct this, the inflow values from the selected period are put into ascending order, after which the probability of exceedance of each value is calculated. The probability of exceedance of a value is the percentage of the measurements higher than this value; the formal definition is as follows:

$$p = 100 \times (1 - \frac{i}{n+1})$$

in which i = ordinal number of the measurement
n = total number of measurements

The ordered datasets are presented in Table C2.8; mean values and standard deviations are added. The inflow duration curves as computed from both rivers are presented in the following figure, and also at the end of the report as Figure C2.4.



Compared to discharge duration curves of 'normal' rivers, the profile of Figure C2.4 is flat in the lower reaches and steep in the higher reaches. This reflects the flashy character of the rivers. The catchment area is relatively small; also, the rivers are relatively short, so the time between rainfall and outflow is short. Furthermore, precipitation is the only source of the flow, so an irregular, flashy precipitation pattern results in an irregular, flashy discharge pattern.

ORDINAL	PROB. OF EXC.	MUHURI	FENI
1	97.5	17.9	18.4
2	95	20.3	18.8
3	92.5	21.3	38.3
4	90	22.9	38.7
5	87.5	28.2	41.5
6	85	29.0	41.5
7	82.5	29.7	42.4
8	80	31.2	45.2
9	77.5	32.8	46.7
10	75	34.3	47.6
11	72.5	35.0	49.5
12	70	37.7	67.2
13	67.5	38.6	73.4
14	65	72.1	77.0
15	62.5	88.5	88.5
16	60	97.2	90.3
17	57.5	102.6	91.3
18	55	103.5	92.6
19	52.5	121.3	110.1
20	50	127.7	122.8
21	47.5	132.4	123.8
22	45	144.7	137.8
23	42.5	151.9	142.3
24	40	168.2	142.9
25	37.5	174.7	155.0
26	35	181.4	199.6
27	32.5	186.2	203.2
28	30	209.1	203.3
29	27.5	210.0	208.1
30	25	235.3	255.9
31	22.5	296.1	262.9
32	20	297.0	287.7
33	17.5	342.2	314.4
34	15	367.8	333.1
35	12.5	451.4	528.1
36	10	487.4	674.8
37	7.5	515.2	876.0
38	5	622.9	1014.6
39	2.5	1173.3	1064.0
	mean	190.7	214.6
	st.dev.	223.5	264.2

Table C2.8: Probability of exceedance of reservoir inflow

Both curves seem to agree quite well, especially in the lower part of the graph. The average values of this three-year period of observation (see Table C2.8) are apparently higher than the long term average: from Table C2.7, a long-term average May-October inflow of about 155 m^3/s can be computed.

However, it has to be noted that the lowest inflow values in Table C2.8 are almost all values from the 1992 datasets (which was a very dry year, see Figure C2.3), while the rest of the list is nicely mixed. To assess the effect of this dry year on the duration curve, an identical curve has been constructed for the Muhuri river of 1991 and 1993 (Figure C2.5). It appears that the whole curve is situated about 50 m^3/s higher than in Figure C2.4. Still, Figure C2.4 will be used for further analysis in the design phase. The main reason for this is, that the data that are used only date back three years, which is very short. Because of that, a rather conservative approach is called for.

3.LEVELS

3.1.Introduction

In this chapter a description of the water levels inside and downstream from the reservoir will be given. Following the conclusions of the preceding chapter, attention will be focused on the wet season.

Some research has been done into the water levels at the site with regard to the structures of the MIP; this research has been focused either on the tidal curve in the dry season (as used for the closure of the Dam in 1985) or on the extreme water levels (as used for the design of the Dam and the Regulator). For this study though, it is the representative wet season hydrostatic head that matters.

From 1987 onwards, there has been a gauging station at the Regulator. In [Halcrow'91], wet season water levels were included for research into the stilling basin that was being attacked. The report contains data of the wet seasons of 1987, '88, '89 and '90. All recorded data are presented in Figures C3.1a and C3.1b. Some additional wet season data (mostly in Bengali) from the gauging station are available as well.

3.2.Reservoir levels

During the daily production period, the reservoir level will decrease from its maximum to its minimum, because the production period is also the time in which the reservoir has to be emptied for the next high tide. The only data that are available are daily maximum levels.

The curves of the maximum reservoir level in Figures C3.1a and C3.1b do not show any statistical pattern during the wet season: the water level inside the reservoir is determined by the inflow from the rivers and by the extent to which outflow is possible, depending on downstream levels. These are both more or less random processes. Therefore, the average level and the variance of the level of the reservoir are the only significant characteristics that can be derived.

The average of all measured data from the beginning of May to the end of October is SOB +2.83 m. The standard deviation is 0.7 m.

It is questionable whether the measured levels are relevant for the power plant. As stated, they are partly determined by the natural inflow, but also partly by the outflow. The outflow pattern is a product of the reservoir management: in the present situation, outflow starts as soon as the difference between reservoir level and downstream level is large enough to open the flapgates. The radial gates of the Regulator are usually kept open in the wet season, in order to release as much water as possible as quickly as possible, to prevent flooding. In the situation with a power plant, it might be possible to adapt the operation regulations, and to delay the start of the outflow as long as there is no risk of flooding, depending on flood forecasts. In that manner, the hydrostatic head for production will be larger. This will be elaborated upon in the design phase.

The minimum water level, which occurs at the end of the production period, is determined by the management of the reservoir as well. Any measured levels in the present situation are thus irrelevant.
It must not be forgotten, that the first priority of the MIP in the wet season still is the improvement of the drainage of the project (see Section A3.1). This means that the total outflow during low tide must create enough space in the reservoir for the amount of water that will flow into the reservoir during the following high tide. The design discharge and the production period are determined such, that this condition is fulfilled; the minimum water level then follows as well. The determination of these values will be performed in the design phase.

Other important levels with regard to the reservoir (from the reconnaissance phase) are:
- SOB +4.91 m:maximum reservoir level, accompanied by flooding of lateral embankments;
- SOB +3.81 m: top of radial gates, spillway level of the Regulator;
- SOB +0.00 m: sill level of the Regulator.

3.3.Downstream levels

3.3.1.Introduction

There is no official gauging station in the area, and therefore no tide forecasting tables: the nearest gauging station is at Sandwip Island (Figure A2.3). An indicative relation between Sandwip and Feni high tide levels is given in [Haskoning'83], but this is only valid in the dry season and is based on less than three weeks of data. Apart from that, the pattern of the tide must have been influenced by the construction of Feni Dam in 1985.
A similar curve for the wet season could be constructed, but this is not considered necessary: not a certain exact tidal curve is important (as it was in [Haskoning'83] for the closure of Feni estuary), but a representative schematisation of the curve that occurs at the site.

In this section, the high and low tide levels from Figures C3.1a and C3.1b, and the additional data will be analysed; however, it is not these daily extremes that are important: what matters is, how long the water level is below a certain point, and how much lower it is. Therefore, a representative wet season tidal curve will have to be constructed.

In the reconnaissance (in Section A4.3.3), some remarks were made concerning the interaction of tide and outflow that determines the measured downstream water level. These remarks will be reevaluated in this section.

3.3.2.Daily maximum level

Maximum downstream levels are available for 1987 and 1990 (see Figure C3.1a and C3.1b). The graphs clearly show that the maximum downstream level is (at least mainly) determined by the tide; a period of about 15 days (from springtide to neaptide and back) is easily distinguishable. Moreover, because in the wet season the high tide is usually higher than the reservoir level, the flapgates will be closed around the moment of high tide.

This means the maximum water level is only determined from the seaside, in other words by the astronomical tide combined with wind set-up (contrary to what was indicated in Section A4.3.3).

Following this, one way to determine the high tide level would be to use mathematical models for both the astronomical tide and for the wind set up at the site, and then compute the statistical distribution of the water level. This method would be very complicated: the cross section of the discharge channel (which is not constant), the distance that the tidal wave has to travel from the nearest gauging station (Sandwip Island), a statistical description of tidal levels at Sandwip and wind data would all interact in such a model. This is considered too complicated for this project.

Therefore, the datasets from [Halcrow'91] will be used again.
The following statistic characteristics can be computed:

- Average wet season springtide high water:
  SOB +4.55 m with a standard deviation of 0.36 m (based on 24 data);
- Average wet season neaptide high water:
  SOB +2.84 m with a standard deviation of 0.51 m (based on 24 data);
- Average of all wet season high waters:
  SOB +3.77 m with a standard deviation of 0.66 m (based on 350 data).

These are the values that will be used to construct the representative downstream level curve for the power plant.

3.3.3. Daily minimum level

Daily minima are available for 1988, '89 and 90 (see Figure C3.1a and C3.1b). The figures show that the influence of the astronomical tide is hardly discernible. Even the neaptide (when the tidal amplitude is at its minimum, so that the astronomical low tide is at its maximum) does not show in the datasets.
There does however seem to be a strong correlation with the water level inside the reservoir. This agrees with what was stated in Section A4.3.3: at the moment of the lowest tidal level, there is almost certainly an outflow through the Regulator, which will cause the measured water level to be higher than the astronomical tidal level. The discharge capacity of the channel is limited, so larger outflow leads to higher water levels.

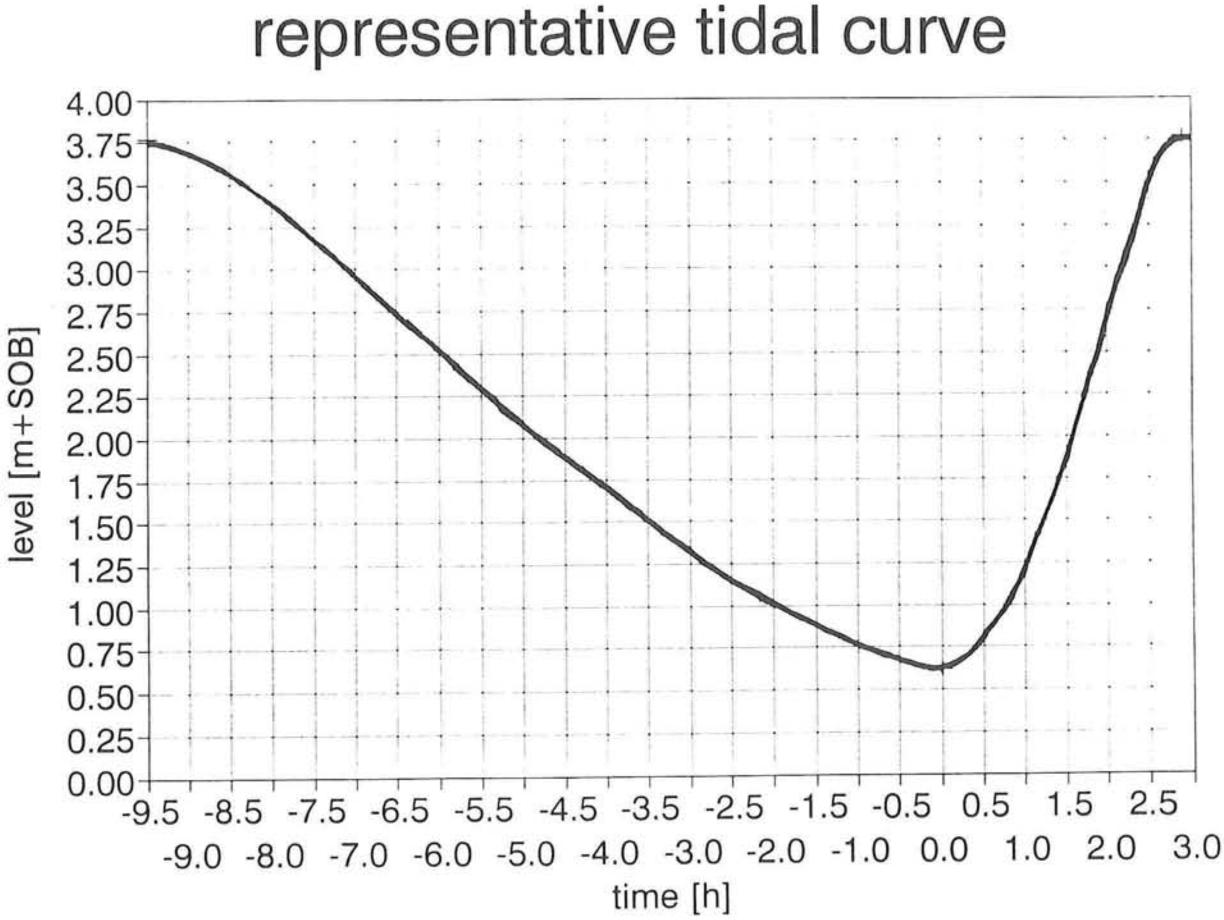
The average of the wet season minimum levels is SOB +0.62 m, with a standard deviation of 0.73 m (based on 436 data). Contrary to the high levels, there is no statistic pattern, so this level will be assumed to be the low tide level for all tides.

The question can be asked, whether the measured minimum downstream values are valid for the installation level of a power plant as well: as stated, the minimum levels, in most cases, are determined by the local discharge capacity downstream from the Regulator combined with the discharge. Both the discharge values and the discharge capacity in the situation with a power plant will differ from the present situation, depending on location and design of the structure: the maximum discharges will be smaller, as discharge peaks will partly be guided through the Regulator; on the other hand, the channel that will guide the water from the plant to the existing discharge channel may constitute an extra impediment for the flow, causing a stronger backwater effect.

It is difficult to make any assumptions about these two factors, but because they will counteract each other, it seems fair to assume pre-plant levels for the situation with a power plant.

3.3.4. Downstream curve

In Section A4.3.3.1 it is mentioned that the tidal curve at the site of Feni Dam is asymmetrical, with an ebb flow of 9½ hours and a flood flow of 3 hours. This feature was determined before the construction of the Dam; it was caused by the fact that the outflow was slowed down by the limited capacity of the ebb channels, and by the continuous river discharge. Therefore, while the sea level was already rising again, there was still an ebb flow going on. In the present situation, and at the Regulator site, the tidal curve does not have to be asymmetrical to the same extent. The Regulator is somewhat closer to the open sea than the Dam, but on the other hand, the capacity of the tidal channels has decreased due to siltation. Therefore, the same asymmetry will be assumed for the Regulator site in the present situation.
In the situation with a power plant this means that the tidal level rises during a period of about 3 hours and falls during a period of about 9½ hours each tidal cycle. This feature will be incorporated in the representative curve.
The representative curve is given in the following figure, and as Figure C3.2 at the end of the report.



It has been constructed by combining the given shape of the tidal curve in the dry season before construction of Feni Dam from [MIP'82] with the levels as determined in Sections C3.3.2 and C3.3.3. This curve still does not completely agree with the physical reality as far as the minimum level is concerned. It can be concluded from Section C3.3.3 that the water level should remain at or around the minimum level for some time, after which it should rise with the astronomical tide again. This feature has not been incorporated in the schematisation, as no data are available on the amount of time that the level remains around its low.
Even though it may not be correct, or even based on correct assumptions, this indicative curve will be used for further design, as at least it resembles the shape of a tidal curve. The design process can then be based on a realistic boundary condition.

Phase D: Design

1.INTRODUCTION

In this final phase, the results of the data analysis are used to design the power plant, according to the objectives set in Phase B.

In Chapter 2, the size and number of the turbines are determined, based on a selected probability of exceedance. The emphasis in this chapter is on the design path that is followed; the first priority of this study is educational, after all. The chapter concludes with indicative values of investments and proceeds of hydro-electric development of Feni reservoir.
In Chapter 3, the location of the power plant is determined, and the different aspects of the lay-out are discussed.
Technical drawings have been made of the selected solution. Chapter 4 deals with the assumptions and choices that have been made for that purpose.

The remarks in Chapter A6 about the position of the project with regard to the rest of the Muhuri Irrigation Project, the natural environment and the electricity grid Bangladesh are essential to this chapter. They are therefore recapitulated first.
Firstly, hydro power production is only the second priority of the MIP in the wet season: flood control comes first. However, if a professional and accurate management system could be implemented, the production could be optimised while maintaining sufficient safety against flooding.
Secondly, the character of the plant is a combination of run-of-river plant characteristics and tidal power plant characteristics. Long term storage is impossible, so a certain amount of water must be spilled, depending on the inflow. At the same time, the times and dimensions of production are determined by the astronomical tide.
Finally, it is possible to link the production of the plant to the national grid. In this way, the actual moment of production (whether in the middle of the night or at times of peak demand) does not influence the proceeds, because it can always replace firm power from the hydro power plant at Kaptai.

2.TURBINE DIMENSIONS

2.1.Introduction

In this chapter, the results of Phase C (i.c. Figures C2.4 and C3.2) will be used to determine the dimensions of the turbines of Feni power plant. These dimensions are a major factor in the design of a power plant, and determine for a large part the investment costs of the project.

In this chapter, the location of the power plant has not been considered, even though sometimes the exact location may influence the boundary conditions. As already stated in the introduction to Phase C: the determined water availability is valid for the reservoir; the determined water levels are valid for the Regulator site (see Figure A2.4).
Because of this, exact economic calculations cannot be performed. Therefore, the design selections in this chapter are often based on guidelines from literature. The economic considerations that should govern the design are mentioned in a qualitative sense.

The emphasis in this chapter lies on the path that has been followed to combine the requirements with the boundary conditions for this project. The exceptional character of the project (the fact that electricity production should be considered a smaller priority than flood control and irrigation, the extreme and extremely variable natural circumstances, the shortage of data, and especially the mixture of run-of-river and tidal plant characteristics) means, that no standard design process is available to determine turbine parameters from the natural input parameters. In other words, the material is too complicated for a complete optimisation procedure. Therefore, design selections are made by comparison of several alternatives; if necessary, reiteration to earlier selections takes place.

First, in Section D2.2, the probability of exceedance of production, or the percentage of time in the wet season when the inflow cannot be guided completely through the power plant, is selected. This results in a minimum volume of water that the power plant must be able to handle within one tidal cycle.
Section D2.3 concentrates on the proceedings within one tidal cycle, in the design situation. Consecutively, the production period, the hydrostatic head, the discharge capacity and the number and size of the turbines are determined for the design situation. Then, in Section D2.4, the selected solution is described, giving indicative values of dimensions and production. Finally, in Section D2.5, the selected probability of exceedance is reviewed, and solutions with a different probability of exceedance are compared to the selected one.

2.2.Probability of exceedance

2.2.1.Introduction

With regard to the capacity of the turbines, the project can be compared to a run-of-river plant, because unlike a tidal plant a certain uncontrollable amount of water has to be spilled. Normally, the basis of the selection of the discharge capacity is the outflow itself, obviously. However, in this case it is different: the data that are available concern the inflow into the reservoir, and not the outflow. The actual turbine discharge is then determined by this inflow and the selected production period per tidal cycle.

So, the first design selection that has to be made is the maximum inflow that must be guided completely through the power plant in the part of the tidal cycle when the tidal level does not prevent outflow. In other words, how often will the spillway (in this case the Regulator) have to be used to spill a sufficient total amount of water. Figure C2.4, the inflow duration curve, represents the boundary conditions with regard to the inflow into the reservoir. The following selection is based on costs and profits: a larger power plant capacity leads to larger proceeds and larger investment costs, so an optimum has to be found.

2.2.2.Selection

The selection of the optimum probability of exceedance for this project is slightly different from other projects, for two reasons: the fact that a part of the required infrastructure is already in place, and the fact that Feni plant can be connected to a grid.

The economic considerations for Feni plant are based on a power plant-spillway system, of which the spillway is already operational, with supposedly sufficient capacity to comply with flooding criteria (the Regulator is designed for a flood with a recurrence interval of 50 years, 2014 m^3/s). Therefore, the considerations that determine the capacity are slightly different from other projects; normally, an increase of the turbine capacity (resulting in an increase of the production of energy), would have to balance enlarged costs of construction of the power plant, but at the same time diminished costs of construction of the spillway. In this case, the 'positive' effect of decreasing spillway costs cannot be taken into account, and the rising investment costs have to be balanced solely by rising production of the plant.
As stated, these considerations cannot be quantified. Therefore, guidelines from practice have to form the basis of the design.

Practice has shown that, for solitary power stations of the run-of-river type, the discharge with a probability of exceedance of 75% is an appropriate design value; however, for stations that are part of a grid, and that can operate as a supplement to base load stations, the probability of exceedance of the design value can be as low as 15%. A value between these two extremes has to be selected.

In this case, it can be expected that the number and size of the turbines will be relatively large because of the combination of large discharge and small hydrostatic head. This means that the increase of the investment costs for each increment of the proceeds of the project will be relatively large; therefore, a too small probability of exceedance cannot be justified. The initial calculations will be performed for a design inflow with a probability of exceedance of 30%; the inflow with this probability of exceedance is 200 m^3/s (Figure C2.4 and Table C2.8). After the first iteration, the same calculation will be performed for smaller (15%) and larger (50%) probabilities of exceedance.

This value of 200 m^3/s is the average inflow during the complete tidal cycle: the design discharge of the power plant is then determined by the selected production period and hydrostatic head.

2.2.3.Remarks

It has to be noted, that the determined value of 200 m^3/s is the design value: in the daily reality, the total outflow will have to be balanced with the expected inflow, if a more or less stable production (on a larger scale) is required. In other words: if the amount of water that is used for production is larger than the inflow, the reservoir level falls more during production than it rises during the rest of the tidal cycle; this means that the hydrostatic head and the available amount of water in the next production period are smaller. The actual discharge capacity that will be used during a certain production period therefore depends on the reservoir level and the forecasts of inflow. If these forecasts are reliable, production could be optimised, as indicated in the introduction to this phase.

2.3.Turbine operation scheme

2.3.1.Introduction

With regard to the hydrostatic head over the plant, the situation is comparable to a tidal power plant of the simplest kind: a single basin system, in which flow in one direction (ebb flow) generates electricity. The difference is, that in this project the basin is filled from the upstream side instead of by the tide itself.
The advantage of this is, that the plant does not have to be designed for flow in two directions. On the other hand, pumping water from the sea into the basin to increase production is no option in this case, because the reservoir is also used for irrigation. The loss of irrigation benefits would certainly be larger than the gain of electricity benefits in such a situation.

In tidal power plants, it is common practice (as far as there is any practice in this field) to operate the plant with a constant hydrostatic head: the power plant discharge can be regulated to make the reservoir level fall at the same rate as the downstream tidal level. The advantage of this is, that the turbines do not have to be adapted to a changing head, and can function with a maximum efficiency. Clearly, the downstream level does not fall at a constant rate (Figure C3.2), so in order to keep the hydrostatic head about constant, the number of turbines in operation may have to be adapted to the actual falling rate[8].

In this section, first the type of turbine that is best suited for the circumstances will be selected. After that, a number of design characteristics, such as the production period, the hydrostatic head and the discharge capacity will be determined. Finally, the number of turbines in the power plant will be selected on the basis of, mainly qualitative, design criteria.

The input data for this section are the design average inflow of 200 m^3/s, and the representative tidal curve (Figure C3.2) for the downstream levels.

[8]The discharge through one turbine cannot be adapted very much without loss of efficiency: the optimum discharge is determined by the shape and dimensions of the turbine and by the hydrostatic head.

2.3.2.Type of turbine

2.3.2.1.Introduction

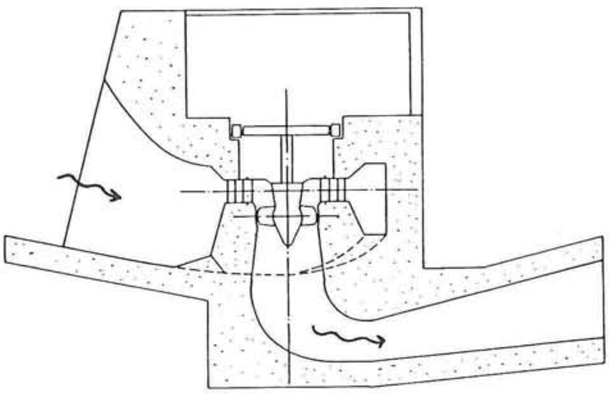
A wide range of types and sizes of turbines is used in hydro-electrical engineering. This aspect of the project actually lies in the field of mechanical engineering rather than civil engineering. Normally, type and size of the turbines are regarded as a 'black box' by civil engineers: the engineer provides the turbine manufacturers with the basic required data, and the turbine manufacturers then determine the turbines that will be used. Obviously, the shape and size of turbines and electrical equipment partly determine the dimensions of the power plant; therefore, a type and size of turbine will have to be selected for this project as well. This selection will be based on qualitative grounds and on empirical data from literature.

2.3.2.2.Options

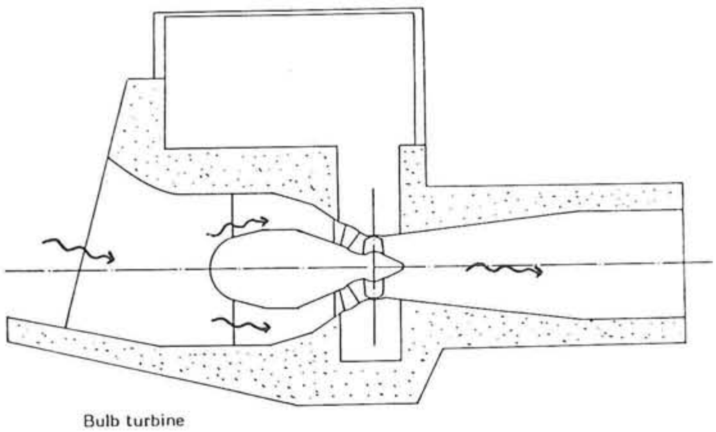
With a hydrostatic head around 2.5 m, the turbines will have to be of an axial type (this means that the flow enters the turbines parallel to the axis of the turbine) [Duivendijk'89]. There are three main axial turbine categories: Kaplan, Straflo and Propeller.

Obviously, the actual hydrostatic head will be far from constant over the year, depending on both reservoir level and tidal curve. Therefore, to reduce the loss of efficiency that results from this, the turbines of Feni power plant will have to be adjustable to different values of hydrostatic head. Contrary to Straflo turbines and Propellers, Kaplan turbines do have this feature: they can be throttled down, adapting the discharge to the actual hydrostatic head. Therefore, the obvious choice is a Kaplan turbine in this case.

There are two main types of Kaplan turbines: they can be installed vertically, which is the classic type, or horizontally, as practised in bulb turbines. These two options are presented in the following figures.



Kaplan turbine



Bulb turbine

This figure is included at the end of the report as Figure D2.1.

In a vertical Kaplan, the flow enters the plant horizontally, then makes a 90° angle to flow through the vertical turbine. The turbine is connected to the rotor, that rotates inside the stator to generate electricity. The stator is situated above the turbine. Rotor and stator constitute the generator.
The principal feature of the horizontal bulb turbine is, that the generator and all equipment are concentrated inside a watertight bulb, around which the water flows. The turbine is at the downstream end of the bulb.

2.3.2.3.Criteria

The criteria that govern the selection of the type of turbine (Kaplan or bulb) are the following:

- total length of the structure has to be small→
  - less turbines;
  - smaller centre to centre distance of the turbines;
- excavation depth has to be small;
- foundation pressure has to be low→
  - smaller turbines;
- operation and maintenance have to be simple;
- efficiency has to be optimum.

The first three points are purely economic, and their value is determined by the size and number of the turbines. The other two mainly depend on the structural characteristics of the two types.

2.3.2.4.Characteristics

The difference in structure of the two types of turbine induces principal differences in operation and size. These differences are presented in this section.

The main advantage of bulb turbines is, that the dimensions of the power plant structures are reduced as a result of the construction method, which is very compact compared to vertical Kaplans. Furthermore, the flow follows a straighter line (which increases the efficiency), and concrete structures are comparatively simple and cheap.

In other respects, bulb turbines are less suited than vertical Kaplans:

- for bulb turbines, there is a greater risk for cavitation[9] as a result of the horizontal position, which enlarges the pressure difference within the flow;
- the machine itself is more expensive than a classic Kaplan turbine;
- maintenance is more difficult as a result of the reduced accessibility;
- the construction method causes the plant to be less stable, and therefore more sensitive to oscillations;
- it is more difficult to cool down the generator during production;

[9]Cavitation: the rapid formation and collapse of vapour pockets in the flow in regions of very low pressure, a frequent cause of structural damage to turbines

- as a result of the smaller size and of the construction method, the diameter of the generator is small, and therefore the inertia of the generator is small as well; this means that the inflow has to be very stable in order to have stable production.[10]

The mentioned disadvantages pose limits to the maximum size of bulb turbines: the largest one in operation has an external diameter (D_R) of 7.7 m (Racine, USA).

The size of the turbines (of either the vertical or the bulb type) depends on the hydrostatic head and the turbine discharge. There are methods to get an indication of practical turbine dimensions under given circumstances, as is described in [WPDCsep'88] and [WPDCnov'92]. These methods are based on statistical analysis of actual projects, using regression analysis with dimensionless parameters, in order to be able to compare projects of a different size. [WPDCsep'88] describes this method for bulb turbines, [WPDCnov'92] for axial turbines in general. The method that is followed is described in Appendix D1.

The resulting formulas for the turbine diameter D_R depend on both turbine discharge and hydrostatic head. The hydrostatic head can be assumed to be a known constant in this design situation (2.5 m, for example), so D_R can be expressed as a function of the turbine discharge Q. This calculation is performed in Appendix D1, with the following results:

For bulb turbines:

$$D_R = 0.368 \times \sqrt{Q}$$

so

$$Q = 7.384 \times D_R^2$$

For Kaplan turbines:

$$D_R = 0.419 \times \sqrt{Q}$$

so

$$Q = 5.696 \times D_R^2$$

These relations are presented graphically in Figure D2.2. Apparently, the discharge through a bulb turbine of certain dimensions is about 30% higher than through a vertical turbine of the same dimensions. This is an important feature in the context of this project: it means that a choice for bulb turbines will reduce the number of turbines and/or the size of the turbines, compared to vertical Kaplans. This is illustrated by Figure D2.3, in which the approximate number of turbines is expressed as a function of the turbine diameter D_R. For this purpose, an indicative power plant discharge capacity of 1300 m^3/s (loosely based on a production period of 2h30) has been assumed. Figure D2.3 only serves to compare both types of turbines; the actual values are not important.

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[10] Modern bulb turbines are equipped with an 'epicyclic gearbox', which increases the speed, and therefore the inertia of the generator; this eliminates the problem.

2.3.2.5.Selection

Despite the disadvantages of bulb turbines, they have to be considered more suitable in this case than vertical Kaplans, especially because of the smaller number and size of the turbines; consequently, the dimensions of the power plant structure as a whole will be smaller, and foundation pressures will be lower. Furthermore, with the already low hydrostatic head, it is very important to reduce the hydraulic losses as much as possible; this requires horizontal turbines. Finally, the fact that nowadays, low head power plants are almost all equipped with bulb turbines, and not with vertical Kaplans, makes the choice for bulb turbines obvious.
Therefore, bulb turbines are selected for the design of Feni power plant.

2.3.3.Design characteristics

2.3.3.1.Introduction

Hydro turbines are designed to have maximum efficiency for one design set of Q and H (discharge and hydrostatic head). An indicative value for this maximum efficiency is 94%. This design set of Q and H is usually selected as the situation in which the annual average production occurs, or in other words, 'above and below which the average annual generation of power is approximately equal' [ICE'90]. The efficiency decreases as the conditions change in either direction. In this section, it is this design situation that is determined from the natural boundary conditions; with the acquired values, the number and size of the turbines will then be computed.

The basis of this section is, on the one hand, the selected probability of exceedance of 30%, or a design reservoir inflow of 200 m^3/s, and on the other hand the representative tidal curve of Figure C3.2.

2.3.3.2.Required discharge curve

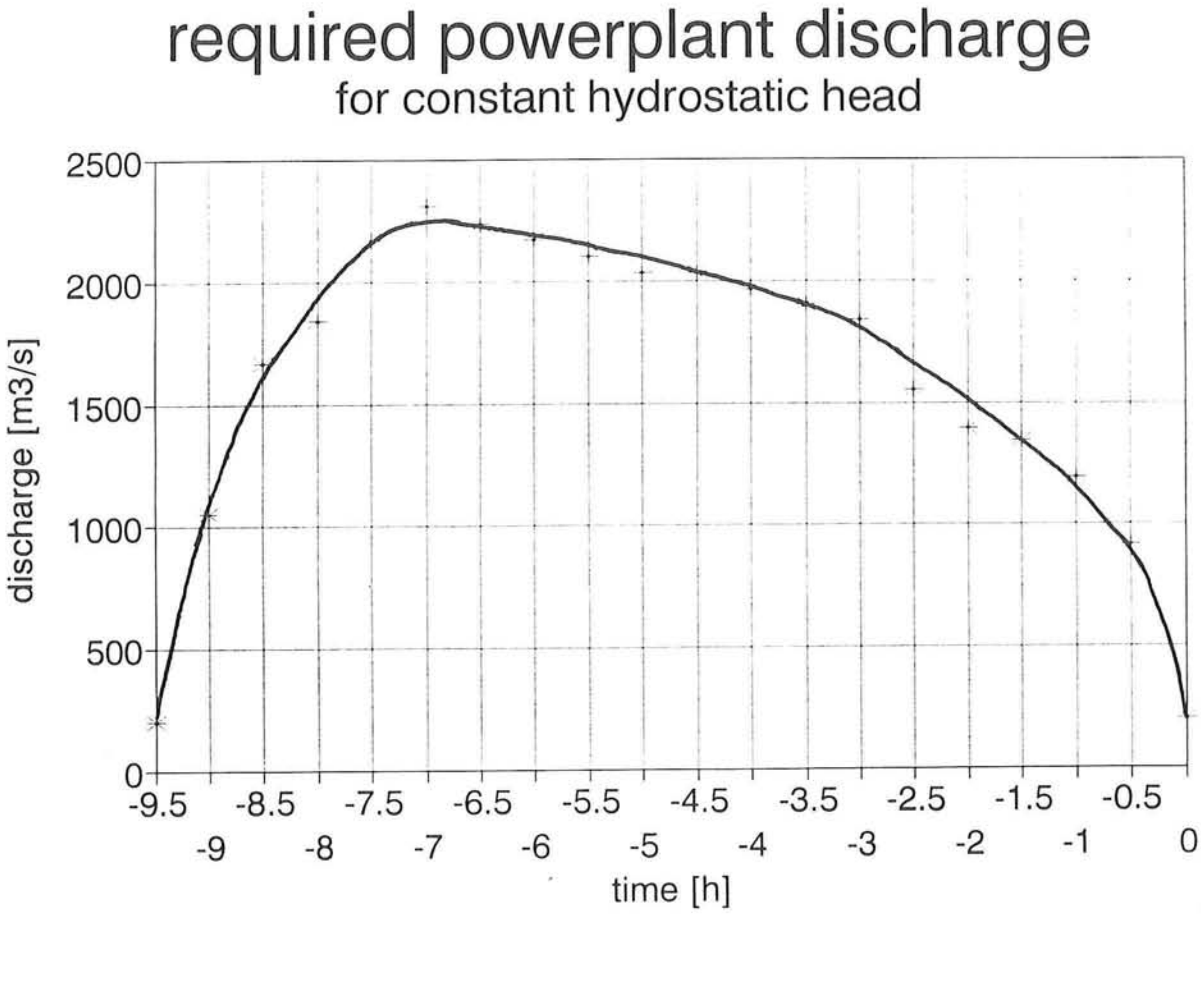
As stated, production is most efficient if the hydrostatic head remains relatively constant, or near the design value, during production. The hydrostatic head is equal to the reservoir level minus the downstream level. The downstream level is assumed to be a boundary condition: it cannot be influenced, and is represented by the curve of Figure C3.2. Therefore, in order to keep the hydrostatic head constant, the falling rate of the reservoir level has to be synchronised to the tidal falling rate.

A graph of the power plant discharge that is required to keep the hydrostatic head constant can be constructed. For this purpose, a graph of the downstream **falling rate** can be derived from Figure C3.2, by measuring the gradient dh/dt at various points during the tidal cycle. This yields a set of downstream falling rate values in m/h. The falling rate of the reservoir has to be about equal to the downstream falling rate thus determined.

The falling rate of the reservoir is directly related to the power plant discharge; the falling rate values in m/h can be transposed to **'net outflow'** values in m^3/s by multiplying the falling rates with the reservoir area of $17*10^6$ m^2, and dividing them by 3600 s (the multiplication factor then is: $17 \times 10^6 [m^2]/3600[s] = 4700[m^2/s]$). This calculation is based on two assumptions: in the first place, that the reservoir level falls at the same rate all over the reservoir at all times, so that the outflow can be divided by the whole reservoir area; secondly that the sides of the reservoir are vertical, so that the area is independent of the water level (17 km^2 at all levels). The validity of these assumptions will be discussed in Appendix D2.

The net outflow values that are obtained are not equal to the power plant discharge that is required to keep the hydrostatic head constant: in the design situation, there is a constant reservoir inflow from the rivers of 200 m^3/s, also during the production period. Using the assumption of the horizontal reservoir level again, an increase of the net outflow by 200 m^3/s at all times finally yields the **required discharge** values.

The resulting curve for the required discharge between t = -9½ h and t = 0 (falling tide) is presented in the following figure.



This figure will be referred to as Figure D2.4.

The three subsequent datasets (**falling rate, net outflow** and **required discharge**) are presented in Table D2.1.

TIME [hours before low tide]	FALLING RATE [m/h]	NET OUTFLOW [m^3/s]	REQUIRED DISCHARGE [m^3/s]
9.5	0	0	200
9	0.18	854	1054
8.5	0.31	1466	1666
8	0.35	1643	1843
7.5	0.42	1969	2169
7	0.45	2113	2313
6.5	0.43	2040	2240
6	0.42	1969	2169
5.5	0.40	1900	2100
5	0.39	1833	2033
4.5	0.39	1833	2033
4	0.37	1768	1968
3.5	0.36	1705	1905
3	0.35	1643	1843
2.5	0.29	1354	1554
2	0.25	1195	1395
1.5	0.24	1144	1344
1	0.21	996	1196
0.5	0.15	717	917
0	0	0	200

Table D2.1: Construction of required discharge curve, Figure D2.4

2.3.3.3.Production period

Figure D2.4 can replace Figure C3.2 as a boundary condition for the reservoir level from now on. The design conditions are now satisfied if the design inflow of 200 m^3/s during one tidal cycle can be guided through the turbines, while following the curve of Figure

D2.4. If the inflow is 200 m^3/s, the amount of water that flows into the reservoir during one tidal cycle of 12h24 is $8.928*10^6$ m^3. This is the amount of water that has to be spilled through the turbines during one production period.
The optimum production period is the time for which the reservoir level drops by the same amount as the downstream level, while spilling the design amount of water. This period follows directly from Figure D2.4: the area below the curve between t_0 and t_1 is the cumulative outflow between those t_0 and t_1. In formula:

$$V_d = \int_{t_0}^{t_1} Q_R(t)dt$$

in which
V_d = design volume, $8.928*10^6$ m^3
t_0 = beginning of production period
t_1 = end of production period
$Q_R(t)$ = required discharge curve

For t_1, the point when the downstream level is at its minimum will always be used in practice: the hydrostatic head is maximised, and the production period is longer as well (the downstream curve is less steep around low tide (Figure C3.2), so the falling rate is lower (Figure D2.4), so the required discharge is smaller, so the time it takes to spill the design amount of water is longer). This is useful, because if the same production can be spread out over a longer period, the installed discharge capacity can be smaller for the same production. Concluding: t_1 = 0 h.[11]

An expression for $Q_R(t)$ cannot be determined exactly: some approximation will have to be used for this. In order to minimise the error, the first approximation has to be as accurate as possible. For convenience's sake, all times will be rounded off to the nearest quarter of an hour in the following stage: the errors that are caused by this will appear to be minor.
The first assumption for $Q_R(t)$ (which will be called $Q_R^*(t)$) is a parabolic one. The production period is expected to be about 3 h, so $Q_R^*(t)$ has to be matched as much as possible with $Q_R(t)$ during the last few hours of the curve. The method of calculation of $Q_R^*(t)$ is described in Appendix D3, and the result (reflected in the $Q_R^*(t)$-axis) is:

$$Q_R^*(t) = -5.643\times10^6\times t^2 + 0.193\times t + 350$$

with

$$t \in [0;34.2\times10^3 s]$$

and with $Q_R^*(t)$ in m^3/s.
Figure D2.5 contains both the actual and the approximate curve.

[11]In reality, the production period could be extended into the rising tide: the required discharge is still positive for a while after t_1, because of the constant inflow of 200 m^3/s. However, the curve of Figure 4.1 is at its steepest around this time, so the required discharge will be negative again very soon. Therefore, this error is considered insignificant.

The time that it takes to spill $8.928*10^6$ m^3, exactly following the $Q_R^*(t)$-curve, follows from

$$8.928 \times 10^6 = \int_{t_0}^{t_1} Q_R^*(t)dt$$

The calculation of this integral is described in Appendix D4, the result is an optimum design production period of 2h23.

The resulting value is not too far from the value of 3h that was initially assumed for the construction of the parabolic approximation of $Q_R(t)$. A rough indication of the development of the hydrostatic head can now be determined to assess the correctness of $Q_R^*(t)$. It is repeated that the production period will be rounded off to the nearest quarter of an hour, because this facilitates later computations to a great extent.
If a production period from t = -2h30 to t = 0 is selected, the downstream level falls from SOB +1.13 m to SOB +0.62 m, or a total of 0.51 m, during the production period (Figure C3.2). The distance that the reservoir level falls in the same period can be calculated as well, following this formula:

$$\Delta h_r = \frac{V_d - Q_d \times PP}{A_r}$$

in which:
h_r = reservoir level
V_d = design outflow volume, = $8.928*10^6$ m^3
Q_d = design inflow discharge, = 200 m^3/s
PP = production period in seconds
A_r = area of the reservoir, = $17*10^6$ m^2

For a production period of 2h30, this results in a fall of the reservoir water level of 0.42 m. This means that the hydrostatic head is increased by 9 cm (0.51 m - 0.42 m) during the production period of 2h30 (for the actual value of the hydrostatic head, see Section D2.3.3.4). Apparently, the production period is selected somewhat too large, or in other words, $Q_R^*(t)$ is situated below $Q_R(t)$. No clear data are available on the amount of variation that can be allowed; in this case, it would vary by about 4% (9 cm out of approximately 2.5 m), which is acceptable.

Still, if the start of the production period can be delayed, proceeds would increase as the hydrostatic head would be larger. Therefore, the situation with a later start of production will be regarded as well. The same calculation can be carried out for a production period of 2h15. The downstream level now falls 0.45 m (Figure C3.2) while the reservoir level falls 0.43 m, so the cumulative change of hydrostatic head is only 2 cm now. This means that, if the power plant discharges can be adapted sufficiently accurately to the required discharge curve of Figure D2.4, the variation of the hydrostatic head during the production period will be well within limits.
Therefore, a production period from -2h15 to 0h is assumed for the design calculations.

It should be noted that the variation of hydrostatic head that is used as a criterium in this section, is the cumulative or resulting variation. This value is not influenced by the turbine operation scheme that is selected for the plant, as long as the cumulative outflow remains unchanged; if the actual plant discharges do not follow the curve of Figure D2.4 closely, the variations during the production period may be large, but the resulting change of hydrostatic head from t = -2h15 to t = 0h will be the same in all cases. The optimum turbine operation scheme will be determined in Section D2.3.3.5.

2.3.3.4. Hydrostatic head

The hydrostatic head during the production period is determined by the levels at the beginning of the production period. Figure C3.2 indicates, that the downstream level at t = -2h15 is SOB +1.07 m. If the initial reservoir level is assumed at SOB +2.83 m (as determined in Section C3.2), the constant head is 1.76 m. However, if an accurate management system would be available, as indicated in the conclusions of the reconnaissance (Section A6.2), the reservoir levels could be controlled much better than in the present situation, and it would be possible to work with higher reservoir levels (depending on inflow and tidal forecasts).

The level at which major flooding from the reservoir starts to occur is SOB +4.91 m, and the spillway level of the radial gates of the Regulator is SOB +3.81 m (Section C3.2). If reliable forecasts of inflow are available, it should be possible to maintain this spillway level as the initial production level in most cases. A rise of the reservoir level by 1.1 m (from spillway level to flooding level) during one 'high tide' of about seven hours would require an inflow of about 750 m^3/s. This occurs once or twice a year. A relatively extreme situation like that can be foreseen.

The (approximately) constant hydrostatic head during the production period in the design situation will therefore be assumed at 3.81 - 1.07 = 2.74 m.

2.3.3.5. Power plant discharge capacity

The next aspect to be determined is the discharge capacity of the power plant. As stated in the introduction to Section D2.3, the usual practice in low head hydropower plants is, to adapt the total discharge in such a way that the hydrostatic head remains as constant as possible, in order to keep the efficiency as high as possible. The question is, how closely the curve of Figure D2.4 has to be followed by the power plant discharge in order to keep the hydrostatic head between limits. If the curve is followed very closely, the hydrostatic head will vary only little, and increase almost linearly by 2 cm in the design situation (Section D2.3.3.3). If the curve is not followed closely, the cumulative change of hydrostatic head at the end of the production period will still be 2 cm, but the hydrostatic head will have varied more in the course of the production period, reducing the overall efficiency.
On the other hand, if the hydrostatic head can be allowed to vary more, it is possible to install a smaller discharge capacity with a longer average production time per turbine. In that case, each turbine could be utilised more effectively, and investment costs would be smaller for the same production.

To assess this, the development of the hydrostatic head during the design production period will be calculated for two situations: the situation that the total discharge is constant all through the production period (Situation A), and the situation that the total discharge follows Figure D2.4 as closely as possible with a power plant that consists of six turbines (Situation B). A number of six turbines is chosen because this seems a reasonable maximum for the number of turbines on the basis of qualitative economic considerations. Because of the indicative nature of the calculation, no more exact considerations are needed yet: Situation B just represents the situation in which the hydrostatic head has a minimal variation.

The average discharge during the production period in the design situation is known:

$$Q_d[\frac{m^3}{s}]=\frac{V_d[m^3]}{PP[s]}$$

with
Q_d = average discharge;
V_d = design outflow, $8.928*10^6$ m^3;
PP = design production period, 2h15 * 3600 = 8100 s.

This results in a value of 1100 m^3/s. That is the optimum value as far as efficient use of all the turbines is concerned. A smaller power plant discharge capacity would be out of range: it would then be impossible to spill the design amount of water within the design production period. Therefore, this is the value for the total turbine discharge capacity in Situation A.

The value for Situation B is the discharge for which the required discharge curve can be followed as good as possible. Figure D2.4 shows, that for six turbines this value is certainly less than 1600 m^3/s. The total turbine discharge capacity for Situation B will be assumed at 1500 m^3/s.

The development of the hydrostatic head during the design production period for both situations has been determined with a procedure that is described in Appendix D5. The operation schemes of both situations are included in the same appendix. The resulting hydrostatic head curves are presented graphically in Figure D2.6. The predicted difference between Situation A and Situation B (a smaller variation if the curve is followed more closely) is very clear.

The maximum variation in the curve of Situation A is about 2.6% (2.81 m / 2.74 m = 1.026). The significance of this error should be assessed by calculation of the turbine efficiency through the production period. Efficiency data have to be derived from mussel charts of turbines, but no exact data on this are available, and this aspect has to be considered beyond the scope of a civil engineer anyway. Comparison with mussel charts in WPDC and [Duivendijk'89] indicates that a deviation of 2.6% only has a minor influence on the efficiency.

Clearly, it can be concluded that a constant outflow of 1100 m^3/s in the design situation is the best solution, as the investment costs are much lower and the loss of efficiency is insignificant.

2.3.4.Number and size of the turbines

2.3.4.1.Introduction

The only matter that is now left to be determined, is the number of turbines in the power plant. In Section D2.3.2, bulb turbines have been selected as the best type for the plant, and in Section D2.3.3, the production period per tidal cycle, the hydrostatic head in the design situation and the total discharge capacity for the plant have been determined.

Some possibilities for the number of turbines will be compared in this section; first, the criteria that govern the selection will be presented; some indicative values of costs and profits and other characteristics of some plausible alternatives will be calculated. Finally, the best alternative will be selected.

2.3.4.2.Criteria

Some of the criteria for this selection are comparable to those in Section D2.3.2:
- low structure costs require a small number of turbines;
- low foundation pressure requires small turbines.

Other considerations are:
- transport of the turbines to the location is easier and cheaper for smaller turbines[12];
- larger turbines are more efficient;
- application of a large number of turbines makes a certain degree of standardisation in the production of both turbines and power plant structure possible, which again reduces investment costs;
- application of smaller turbines reduces the risk of cavitation, and therefore the required excavation depth;[13]
- a large number of small turbines increases efficiency in case of varying discharge.

Most of these conditions are 'soft': they pose no absolute limitations to the design, the optimum design point is the economic optimum. The available space for construction, for example, is limitless within a certain range: if the plant is located in Feni Dam or in a future embankment more downstream, the available length is more than 3 km. The low bearing capacity of the subsoil (Section A3.3) also has to be taken into account: again, the degree to which the foundation can be improved is only limited by economic considerations.
The only strict condition for the selection is the D_R, that will have to be within the limits of experience, and preferably even within a certain range of often used dimensions; furthermore, the transport capacity of the roads leading to the location poses (unknown) limits to the weight and size of the turbines. In theory these conditions are soft as well, but in this case they can be regarded as strict.

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[12]This factor is difficult to quantify, but may well be decisive.

[13]The problem of cavitation is usually solved by placing the turbines below a certain water depth, so that the pressure difference is relatively smaller.

The value of a solution is the extent to which it suits the mentioned considerations. This value is determined by the characteristic features of the solution; the distinguishing features are summarised in the following list.
With regard to construction:

1. *Excavation depth;*
2. *Length of structure;*
3. *Turbine and electrical equipment investments;*
4. *Transport;*
5. *Installation;*
6. *Required subsoil improvement.*

With regard to operation:

7. *Adaptability to varying discharge;*
8. *Loss of production in case of malfunctioning of one turbine;*
9. *Efficiency of production;*
10. *Maintenance characteristics.*

The characteristics in this list are the distinguishing factors between the alternative solutions; therefore, some important characteristics, such as the production in the design situation, have not been included: they are equal for all three alternatives. Data like this will be determined in the description of the selected solution, in Section D2.4

2.3.4.3.Alternative solutions

A number of four installed turbines is generally regarded as a minimum; on the other hand, a too large number of turbines will probably not be economically feasible, considering the limited period with production of power. The discharge capacity has to be 1100 m^3/s.

Therefore, the selected three alternatives are the following:

- A: 4 turbines of 275 m^3/s;
- B: 6 turbines of 183 m^3/s;
- C: 8 turbines of 138 m^3/s.

For these three solutions, values for the 10 characteristics of the preceding section will be determined; with those values a comparison can be made.

Some characteristic values that are necessary for the selection have to be calculated first. In the first place the external turbine diameter D_R, that can be computed for each alternative with the formulas in Appendix D1. The results are the following:
For Alternative A: D_R = 6.08 m;
For Alternative B: D_R = 4.96 m;
For Alternative C: D_R = 4.30 m.

From the design values of the turbines, the Thoma cavitation factor σ can be calculated; this factor is used for the determination of the minimum excavation depth that is required to prevent cavitation. The calculation is performed in Appendix D6; the result is a σ of 4.605.

2.3.4.4.Selection

The resulting marks are gathered in Table D2.2. Some of these characteristics can be quantified. Most however can only be assessed on a qualitative basis. This results in marks on a ±-scale, in which a + stands for positive or more feasible.

CHARAC-TERISTIC	ALTERNATIVE A (4*275)	ALTERNATIVE B (6*183)	ALTERNATIVE C (8*138)
1 [m-SOB]	15	12.7	11.4
2 [m]	65	87	109
3 [+/-]	+	±	-
4 [+/-]	-	±	+
5 [+/-]	-	±	+
6 [+/-]	-	±	+
7 [+/-]	-	±	+
8 [+/-]	-	±	+
9 [+/-]	+	±	-
10 [+/-]	+	±	-

Table D2.2: Criteria for selection of number and size of turbines

The values in Table D2.2 are based on the following considerations:

1.*Excavation depth*:

These values follow from formulas that have been determined in Appendix D6. The resulting values seem rather large; it has to be taken into account however, that they are based on a conservative approximation of the cavitation factor, and that in practice installation above the calculated level is no exception. Cavitation can also be reduced by injection of air into the water flow [WPDCaug'86].

Clearly, the excavation depth is an important indication of the costs of the civil works, partly because it influences the amount of soil that has to be replaced, partly because it determines the level to which the groundwater table has to be lowered for construction. In Appendix D6, values for the excavation depth have been calculated for vertical Kaplan turbines under the same design conditions, to re-evaluate the selection of bulb turbines in Section D2.3.2. As stated there, vertical Kaplans are less vulnerable to cavitation, so that the elevation of the top of the turbines can be higher. Still, bulb turbines compare favourably in this case: the larger dimensions of turbines and installations for Kaplans with the same capacity lead to larger excavation depths for vertical Kaplans.

2.*Length of structure*:
The length of the structure (perpendicular to the flow) is made up of a constant value times size times number of turbines plus a constant value, plus a constant value. Or:
$L_p[m]=N\times(C_1\times D_R+C_2)+C_3$

In other run-of-river or tidal projects this length is also determined by other considerations, such as a limited availability of space. No generally applicable formula is available; therefore, indicative values for each constant have to be assumed. They will be based mainly on [ICE'82], a study about tidal power in the Severn estuary in Great-Britain.
C_1 determines the size of one power plant section as far as it depends on the turbine size. This mainly concerns the bulb itself, which has an approximate diameter of $1.15*D_R$.
C_2 consists of the flow channel around the bulb, including concrete casing, and is possibly also determined by the required space for operation and maintenance. It will be assumed at 8 m.
Finally, C_3 is the length of the structure that is independent of either size or number of the turbines. This includes both the side walls, for instance. This will be assumed at 5 m. The following formula then results:
$L_p[m]=N\times(1.15\times D_R+8)+5$

For other bulb turbine projects, the following coefficients for this formula can be derived:
For the plant at Maurik, in the Netherlands (4 bulbs, D_R: 4 m):
C_1 = 1.15, C_2 = 2.4 m;
For the tidal plant at La Rance, France (24 bulbs, D_R: 5.35 m):
C_1 = 1.15, C_2 = 7.8 m.
The factor C_3 cannot be determined because of lack of data.
Apparently, the factor C_2 is not independent of D_R either: the size of the flow channel (that determines C_2) of course depends on the expected turbine discharge, which is correlated with D_R.
Clearly, all three constants may vary a lot per project. As the formula is only intended to serve as an indication, no further investigations need to be performed on this subject, at this stage of the study. It may be that the calculated numbers will have to be revised when the selected solution is designed in more detail later on.

3.*Turbine and electrical equipment investments*:
This value is usually expressed as an amount of money per installed kW. For a smaller number of larger turbines, this value is usually lower than for a larger number of smaller turbines.

4.*Transport*:
As explained in Section D2.3.4.2, the transport of the turbines and other equipment may cause problems, considering the state of the infrastructure in the region. Transport over water may be an option, but that would probably require investments for port facilities in the area (which might, on the other hand, have some positive side effects). As stated before, this aspect can hardly be assessed, let alone be quantified. Clearly though, smaller turbines require smaller transport facilities.

5.*Installation*:
For the installation of larger turbines, larger cranes and other equipment would be required.

6.*Required subsoil improvement*:
The distinct turbines constitute a large concentrated load on the subsoil. Improvement measures will probably be necessary, but for larger turbines these measures will have to be even more extensive. On the other hand, a longer structure length will increase investments as well. Still, it is expected that selection of larger turbines will lead to higher costs in this respect.

7.*Adaptability to varying discharge*:
It will not be necessary or possible to operate all turbines during each production period, because of the varying water availability. Any deviation from the design value leads to a lower efficiency, so the better the capacity can be adapted to the situation, the higher the efficiency will be. A larger number of smaller turbines is therefore better in this respect.

8.*Loss of production in case of malfunctioning of one turbine*:
This characteristic is comparable to the preceding one: a smaller capacity per turbine means that the influence of one single turbine is smaller.

9.*Efficiency of production*:
Larger turbines are more efficient than smaller turbines [Duivendijk'89].

10.*Maintenance characteristics*:
Maintenance has traditionally been a problem in bulb turbines. All equipment is concentrated inside the bulb, which decreases the accessibility, and which means clearances in between equipment are very small. A larger turbine has larger clearances and a better accessibility [WPDCoct'85].

In a proper multi-criteria analysis, all criteria should be supplied with a coefficient according to its importance for design. This coefficient should be multiplied with the mark that each alternative receives for each criterium, after which the alternative with the highest score would be the selected alternative. This procedure will not be followed in this study though: too many factors are unknown, and none of the criteria can be interpreted financially at this stage.
Alternative B will be selected for further study, because the quantified values seem reasonable and not extreme.

2.4.Selected solution

2.4.1.Introduction

The best solution with a probability of exceedance of 30% has now been determined. The main characteristics with regard to production, proceeds and investments will be summarised in this section. These data will be used in Section D2.5 for comparison with the best solutions with probabilities of exceedance of 15% and 50%.

2.4.2.Production

The power plant consists of **6 bulb turbines** with an external diameter D_R of **4.96 m**. The .0hydrostatic head for design is **2.74 m**, the discharge capacity is **183 m³/s** for each turbine, which makes a total of **1100 m³/s**. The axis of the turbines is at an elevation of SOB -5.3 m. The total excavation depth that is required for prevention of cavitation is **12.7 m**, if no remedial measures are taken. The length of the construction, perpendicular to the flow, is estimated at **87 m**. In Figure D2.6, the development of the hydrostatic head during production in the design situation is given (the upper curve).

An indication of the power that can be produced (the *capacity* of the plant) follows from the indicative formula $P[kW]=8 \times Q \times H$; this yields a value of **24.1 MW**, or **6.0 MW** per turbine. The total amount of energy that can be generated annually (the *output* of the plant) has to be calculated with the formula $E=P_{av} \times T$, or average power times production time.

The average power during production obviously is not equal to the maximum power of 24.1 MW: that value is based on the maximum discharge of 1100 m³/s, which is again based on the design inflow with a probability of exceedance of 30%, 200 m³/s. For the calculation of the annual output, the average power plant discharge is required.
It is assumed that when inflow is larger than the design value of 200 m³/s, a cumulative amount of water of $8.928*10^6$ (based on this 200 m³/s) is still guided through the plant in one production period, while the rest is spilled through the Regulator. The average reservoir inflow that is guided through the plant thus is the average of Table C2.8, with all values larger than 200 m³/s substituted by a value of 200. For both calculation methods (inflow calculated from the Muhuri and from the Feni, in other words column 3 and 4 of Table C2.8), this yields an inflow of 120 m³/s. An inflow of 120 m³/s during a tidal cycle of 12h25 leads to a required power plant discharge of **660 m³/s** during a production period of 2h15.
The hydrostatic head for the calculation of the output will be assumed at **2.74 m**. The downstream curve on which it is based is the average wet season tidal curve. The reservoir level of SOB +3.81 m is in reality no average value: it is the level that could be maintained in case of a proper management system (Section D2.3.3.4). Still, this level will be used, resulting in a hydrostatic head of 2.74 m. This means that the average capacity is **14.5 MW**.
The average output per tidal cycle of 2h15 then is 32.6 MWh, per day 63 MWh and per half year, from May to October, **11.5 GWh**. (For comparison's sake: in [Haskoning'85/1] an annual output of 15 GWh was calculated.)

The availability of a hydro turbine is usually expressed as the number of days per year that a turbine functions. In this case, it is more appropriate to work with the number of tidal cycles per year.

For this calculation a production period of 2h15 per tidal cycle is assumed once again. Moreover, it is assumed that a turbine can produce energy with a discharge as small as 20% of its capacity.[14] Finally, this calculation is based on Table C2.8, which is again

[14]The efficiency is lower for such low discharges, but not dramatically for bulb turbines: down to about 85% for a discharge of 30% of the capacity [WPDCnov'83].

based on inflows from May to October. Some production may also be possible before May and after October, despite irrigation requirements, but the size of this production is unclear.

NUMBER OF TURBINES	NUMBER OF TIDAL CYCLES PER YEAR	AVAILABILITY [%]
1	353	9.1
2	265	6.8
3	229	5.9
4	185	4.7
5	159	4.1
6	124	3.2
average	219	5.6

Table D2.3: Turbine availability

The resulting values for the annual number of tidal cycles that a certain number of turbines is operating, is given in Table D2.3. The total number of tidal cycles per year is 707, and from May to October 353. In the third column, the annual availability is expressed as a percentage. This is the percentage of time in a whole year that a certain number of turbines is operating, based on 2h15 production period per tidal cycle.

2.4.3.Proceeds

An indication of the annual proceeds can be calculated with help of Chapter A5.

According to Table A5.1, the availability factor of the hydro power plant at Kaptai is 100%, and installed power is 100 MW. This means that it should always be possible to replace 25 MW from Kaptai on the electricity grid with an equal amount of power from Feni. Therefore, all production from Feni plant can be sold at the normal electricity price, even when the production period happens to occur at times of low demand.

According to Section A5.3.4, the supply costs of energy in Bangladesh for 91/92 were Tk 2.47/kWh, or 6.18 cents (US)/kWh (Table A5.2). This means that the annual proceeds of Feni power plant can be estimated at **US$ 711,000.**

2.4.4.Investment costs

It is more difficult to estimate the investment costs. These are usually expressed as the costs per installed kW. Practical indications from literature cannot really be used in this case: on the one hand, a major part of the construction costs does not have to be included

(construction of the dam and the spillway, creation of the reservoir), on the other hand the low head can be expected to increase costs per kW.
Some indicative values:

- The Kaptai plant in Bangladesh was constructed at a cost of 478 US$/kW[15] (Table A5.1). This would result in an investment of **11.7 million US$** at Feni. The plant at Kaptai consists of two 50 MW Francis turbines, a major dam and a large reservoir.
- A bulb turbine project in the USA (Main Canal, Washington) was constructed at 760 US$/kW. With this ratio, the investment costs for Feni plant would be **18.3 million US$**. This project consists of one 27 MW turbine with a D_R of 5.4 m.

Apart from the already mentioned doubts, the applicability of these figures is uncertain because the figures date from around 1984 for both projects.

The other way to assess the investment costs of a project is, to calculate the cost of every aspect of the project separately. Very little data on this are available yet: at least more data on the price of turbines would be needed. This is considered beyond the scope of this study.

2.5.Different design values

2.5.1.Characteristic values

The basis of the design process in this chapter is the probability of exceedance of the inflow, for which a value of 30% has been selected in Section D2.2. The grounds on which this value was selected were not very firm: literature showed, that values somewhere between 15% and 75% were possible.

To evaluate the correctness of this selection, the same calculations have been carried out for probabilities of exceedance of 15% and 50%.
The same boundary conditions that are the basis of the solution in Section D2.4 (Figures C2.4, C3.2 and D2.4) have been used for this calculation. The calculation is not discussed again, as it is similar to the computation that has been carried out in this chapter.
The values that have been calculated for a probability of exceedance of 30% in Sections D2.3 and D2.4 are also included. A number of six turbines has been assumed for each solution, to enable fair comparison. The order of the characteristics in this table is the order in which they are computed in Sections D2.3 and D2.4. In Table D2.4, all characteristics are summarised. In the next section, the characteristics in this table that can serve as an indication for either costs or profits are discussed separately and in more detail.

[15]updated investment costs, 1984

CHARACTERISTIC	15%	30%	50%
design inflow [m^3/s]	350	200	125
design inflow [$*10^6$ m^3]	15.6	8.9	5.6
production period	3h15	2h15	1h30
hydrostatic head [m]	2.46	2.74	2.98
discharge capacity [m^3/s]	1335 (6*223)	1100 (6*183)	1033 (6*172)
specific speed	603	564	535
turbine diameter [m]	5.5	4.96	4.79
Thoma factor	5.12	4.61	4.23
excavation depth [m-SOB]	13.8	12.7	12.4
structure length [m]	91	87	86
installed capacity [MW]	26.3	24.1	24.6
average capacity [MW]	11.3	14.5	17.7
output [GWh/annum]	12.9	11.5	9.4
availability	13%	9%	6%
proceeds [US$/annum]	797,000	711,000	580,000

Table D2.4: Comparison of solutions for different design values

2.5.2.Comparison

An indication of the investment costs can be derived from the values for the turbine diameter, the excavation depth and the structure length. Additional costs and problems following from the construction process and operation and maintenance have to be derived from these factors as well, mainly from the turbine diameter (see Section D2.3.4.4). These three factors are presented separately in the next table, Table D2.5.

CHARACTERISTIC	15%/350 m^3/s	30%/200 m^3/s	50%/125 m^3/s
turbine diameter [m]	5.5	4.96	4.79
excavation depth [m]	13.8	12.7	12.4
structure length [m]	91	87	86

Table D2.5: Indications of investment costs

From this restricted dataset, a certain pattern can be derived. Apparently, investment costs would rise more than linearly with a sharper probability of exceedance, and also (to a smaller extent) with an increasing design inflow.

Indications of production and proceeds are given by the factors capacity, output, availability and proceeds. They are presented separately in Table D2.6.

CHARACTERISTIC	15%/350 m^3/s	30%/200 m^3/s	50%/125 m^3/s
installed capacity [MW]	26.3	24.1	24.6
average capacity [MW]	11.3	14.5	17.7
output [GWh/annum]	12.9	11.5	9.4
availability	13%	9%	6%
proceeds [US$]	797,000	711,000	580,000

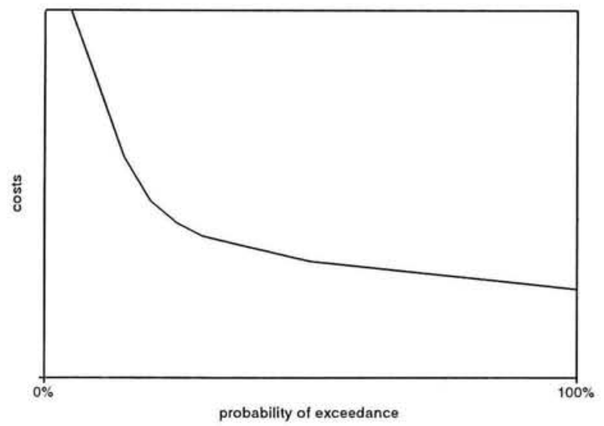
Table D2.6: Indications of production and proceeds

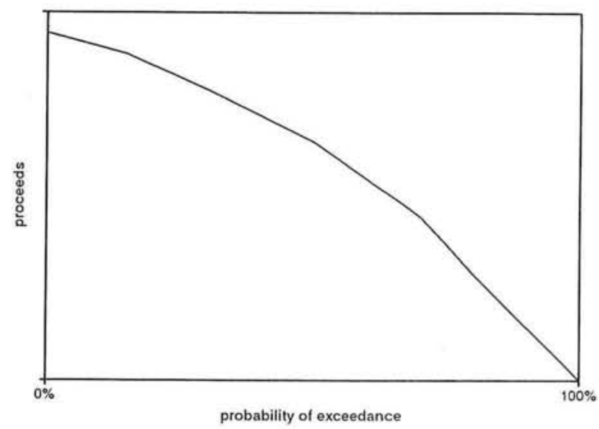
In a way, the value of the proceeds in the last row summarises this table: average capacity, output and availability are all included.

The curve of the installed capacity as a function of the design inflow seems peculiar. This is caused by the shape of the curves of Figures C3.2 and D2.4. For a higher probability of exceedance (50% instead of 30%), the required outflow volume is smaller. This means that the production period required to spill this volume is smaller, so the production period starts later, when the downstream level is lower, and the hydrostatic head higher. The values for the design head are given in Table D2.4.
At the same time, the maximum discharge can be smaller because the tidal curve is less steep, so that the required discharge curve is lower (see Figure D2.4). The determined values for the maximum discharge are also given in Table D2.4. The same reasoning is valid for a lower probability of exceedance, mutatis mutandis.

The capacity of the plant simply follows from $P = 8 \times Q \times H$, and the resulting influence of these two factors appears to cause a non-linear relation between capacity and probability of exceedance.
It has to be noted, that the value of the installed capacity has no influence on the proceeds: it only indicates what the highest peak in production over the year can be, and a larger installed capacity directly leads to a smaller availability or higher investment costs. It is expected that all power will be used as a substitute for production at the Kaptai hydro power plant anyway, so there are no peak demands that have to be met by the Feni power plant itself.

An indication of the relations of costs and proceeds with the design probability of exceedance is given in the following two tables.



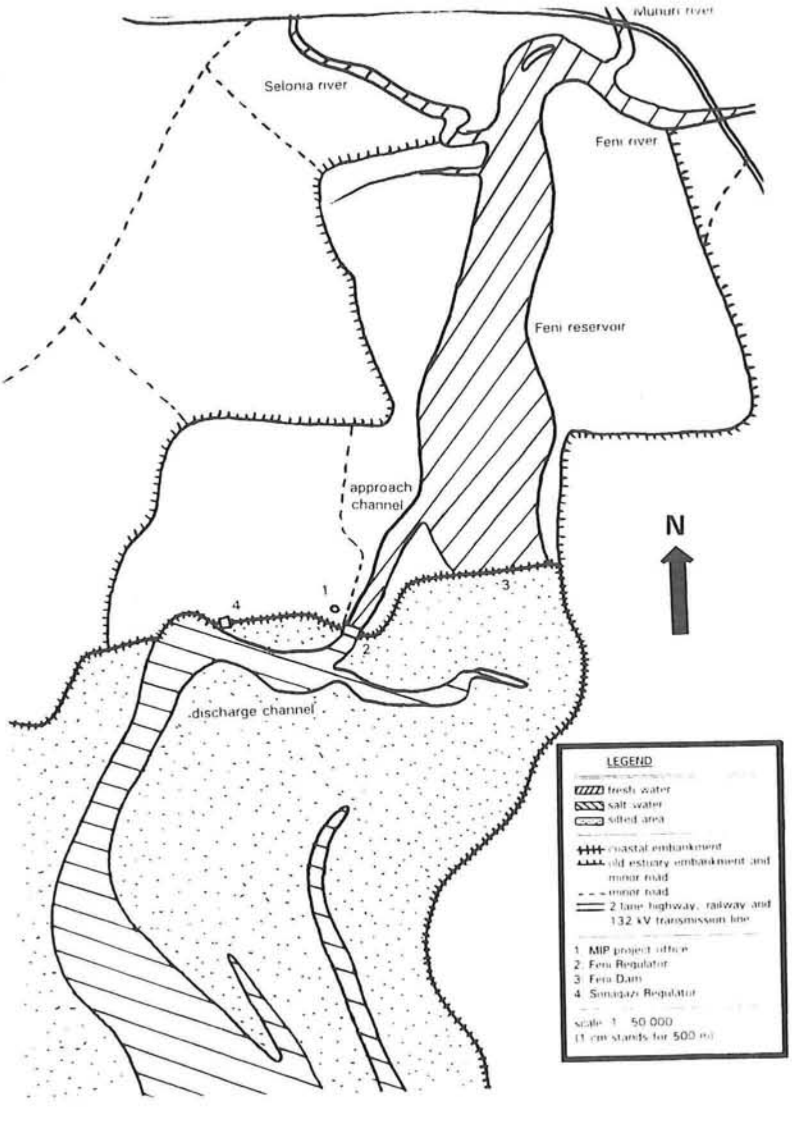


The curve of costs is based on the turbine diameter, the curve of proceeds on the value of proceeds in Table D2.6. As no exact curves can be determined, the evaluation has to be based on qualitative grounds.
It appears that a value of 30% was a good selection: the curve of costs starts its ascend toward infinity just left from 30%, while the direction coefficient of the curve of proceeds decreases as the probability of exceedance decreases. Therefore, it can be concluded that the calculation process has been based on a correct first assumption; the solution as presented in Section D2.4 will be adopted for further design in the following sections of this study.

3.LAY-OUT

3.1.Introduction

In the final part of this study into multi-purpose operation of Feni Reservoir, the location and lay-out of the power plant will be selected. The phases that precede this one, the reconnaissance, the data analysis and the design of the turbine dimensions, serve as input for this chapter; the power plant of which the location is selected in this report is described in Section D2.4.



For the purpose of selection, a schematic map of the project area has been constructed, using literature and aerial photographs (see Appendix A2). The constructed map is also included as Figure D3.1, and will be referred to as such.

As in the preceding chapter, the considerations in this chapter are mainly based on qualitative grounds; normally, economics would be the overriding criterium for selection, but a complete and reliable calculation of the costs of the alternatives is impossible, due to shortage of data and complexity of the situation.

In Section D3.2, Figure D3.1 is elaborated upon, while features of the area that are relevant to the subject are discussed. In Section D3.3, the boundary conditions for the lay-out are presented, in Section D3.4 some alternative solutions for the lay-out are discussed; the best solution is selected in Section D3.5.

3.2. Relevant features of Figure D3.1

3.2.1. Introduction

Some features of Figure D3.1 that are relevant to the selection of the location of the plant are discussed in this section. For some part, this is a recapitulation of information from earlier phases. Two categories of features will be discussed: Land and water, and Transport and structures.

3.2.2. Land and water

The features in this section will be treated from upstream to downstream, or from top to bottom of Figure D3.1.

- There is river inflow from four sources: Feni, Muhuri, Selonia and local inflow. The share of each source in the wet season inflow is as follows:
  Feni river : 53%
  Muhuri river : 37%
  Selonia river : 8%
  local inflow : 2%
  These figures can be derived from Table C2.3.

- The location of the old estuary embankments gives an indication of the historic development of the river bed: it is clear that heavy meandering has occurred in the past. Since the construction of the Dam and the Regulator, the river bed is more fixed, as measures have been taken to guide the outflow towards the Regulator.
  The area between the present reservoir banks and the old estuary embankments is much less densely populated, as aerial photographs indicate (see photos 1 and 2). It is used for agriculture, however.

- The approach channel towards the Regulator was designed by IECo for non-scouring conditions in case of the mean annual flood discharge. This discharge amounts to a flow of 840 m^3/s, according to [IECo'83]. A combination of inspection and remedial measures in case of failure were considered more economic than stricter design conditions.

- The area downstream from the Regulator is very active with regard to morphology. The stilling basin of the Regulator is silted up to SOB +1 m in the dry season, and consequently scoured down to about SOB -5 m (which is 1.5 m above the bed protection), in the wet season, as a result of the difference in the discharge pattern between the seasons. Silt is carried in with the high tide, and is either washed away during the wet season, or allowed to settle due to lack of discharge in the dry season.
  The same applies to the discharge channel further downstream. The discharge channel meanders heavily, and the large and varying discharges in the wet season have even caused a breach in the coastal embankment, just west from Sonagazi Regulator (Figure A4.3). Various measures to redirect the channel to its left (southward) have been considered; for example the digging of a pilot channel, protection of the embankment with bamboo-brick structures (porcupines), and lately the construction of groynes at

both ends of the embankment breach. This last measure was in fact being implemented in February 1994 (see photo 8).

- Some flood channels that do not also function as discharge channels, can still be seen in the figure. Comparison with older maps (from 1984 onward) indicates, that these channels follow a natural process of retreat, as a result of the siltation that cannot be washed away.

- Large areas outside the coastal embankments have gained so much elevation as a result of the siltation, that they are only flooded in case of extreme springtides and cyclones. Because of the severe shortage of land in Bangladesh, reclamation of these areas is a desirable option. For the area south of the Dam and to the east of the Regulator, some superficial planning has been done; these plans range from the construction of a new coastal embankment to the planting of mangrove trees in order to protect the area against cyclones.
  There are several reasons why these plans have not become more definite yet: obviously, the salinity of the land is still quite high, so that agricultural possibilities are initially limited to the grazing of cattle. Other reasons are the high investment costs for such a project and the rather weak organisational structure of the authorities. Anyway, whether there is cyclone protection or not, people are already starting to live in the unprotected area (see photo 7).

3.2.3. Transport and structures

- As a result of the fact that Bangladesh is only some fifteen km wide at this point, three major transport grids pass near the project. The highway at the top of the figure is the main highway between the two largest cities of Bangladesh, Dhaka and Chittagong. It has two lanes, and should be able to cope with any kind of major transport.
  The railway line is close to the highway. There is a major railway station in the town of Feni, which is some ten km to the north-west of the edge of the figure.
  Finally, the electrical grid of Chittagong division is linked to the grid of Dhaka division via a transmission line that runs along the highway. This line has a capacity of 132 kV.

- The minor roads in the figure have a limited transport capacity. They are paved with asphalt, but of uncertain quality; two cars can hardly pass each other and the roads are very curved, and often closely lined with trees. The road on top of Feni Dam is probably sufficiently wide, but the road on top of the Regulator is rather narrow and lined with a concrete wall (See Appendix A2).

- From the figure, the shortest distance between the highway and the project can be measured to be about 7.5 km, to the eastern end of the Dam.

- Feni Regulator and Feni Dam are described in more detail in Section A3.3.
  The MIP project office near the Regulator houses some personnel for the operation of the Regulator gates and the measuring of water levels. There is also a 200 kW emergency generator, to operate the gates in case of power failures.
  Sonagazi Regulator drains the area between the old embankment and the minor road that leads to the project office. Its presence is not essential because of the small size

and population density of its drainage area, and the fact that this area could also be drained onto Feni Regulator, if some adjustments were made.

3.3.Boundary conditions for selection of location

3.3.1.Introduction

In this section, the boundary conditions for the location of the plant are presented. Some of these conditions are valid because the design of this particular power plant has been based on them, others are valid for hydro power plants in general and still others for construction projects in general.

Generally, these conditions do not pose absolute limitations to the location of the plant: most of the conditions can be satisfied with the help of remedial measures, for any location. Obviously, for one location these remedial measures would cost more than for another location; these are the costs that determine the selection. It should be noted, that these costs can relate to the construction, but also to operation and maintenance.

The conditions for selection are divided into three categories:

A. **Upstream conditions**
B. **Downstream conditions**
C. **General conditions**

The conditions will be summed up in Section D3.3.2, explained and interpreted in Section D3.3.3 and conclusions will be drawn in Section D3.3.4.

3.3.2.Conditions

A. **Upstream conditions:**
1- The plant must be connected to the reservoir;
2- Bed level must be sufficiently low;
3- Bed level must be sufficiently constant;
4- Inflow must be regular and undisturbed (in the horizontal and the vertical plane);
5- Capacity of the approach channel must be sufficient (based on maximum power plant discharge of 1100 m^3/s).

B. **Downstream conditions:**
1- Bed level must be sufficiently low from the plant down to the Bay of Bengal;
2- Bed level must be sufficiently constant;
3- Capacity of the outlet channel must be sufficient (based on maximum power plant discharge of 1100 m^3/s);
4- The power plant structure must be protected from extreme springtide levels and cyclones.

C. **General conditions:**
1- Access to the site must be adequate, both for construction and for operation and maintenance;

2- Total discharge capacity of the project (Regulator + power plant) must not be reduced as a result of the location of the power plant.

3.3.3.Interpretation

In this section the boundary conditions from Section D3.3.2 will be explained and interpreted into Figure D3.1.

A1.The plant must be connected to the reservoir.
This condition is based on the design of the power plant; the design upstream water level for the plant is SOB +3.81 m, and the reservoir boundaries (Dam and Regulator) can provide this level. This condition limits the location area in such a sense, that the area downstream from the Regulator is no option. Apart from that, any location not directly at the edge of the reservoir in the present situation could be made possible by the excavation of canals.

A2.Bed level must be sufficiently low.
The flow channel of the turbines is connected to the intake basin by inlet tubes, of which the length and orientation can vary, according to the circumstances. Obviously, longer and more curved tubes are more expensive and reduce the efficiency.
The efficiency of the flow (i.e. the ratio of effective and actual hydrostatic head) is determined by the shape of the streamlines. If the inlet tubes can be placed horizontally, the streamlines will be straighter; this decreases the loss of energy in the flow. Therefore, a lower intake depth increases the efficiency of the plant. Obviously, a lower intake depth requires a lower bed level. In the selected plant, the axis of the turbine will be at SOB -5.3 m (Section D2.4). Depending on the diameter of the tubes (which is about $2 \times D_R$), the lowest point of inflow (the invert level) will have to be about SOB -10 m or deeper.

There are two aspects to the bed level at the intake:
- The first one is, that the bed level must simply be lower than the minimum water level that occurs during production, in order to have an inflow.
- Secondly, there has to be some additional depth between the bed of the basin and the minimum water level: a discharge of 1100 m^3/s may cause erosion and carry a sediment load into the turbines; the extent to which this occurs is determined by the velocity of the water and by the grain size. Therefore, depending on the other parameters of the approach channel (width, sensitivity to turbulence), a certain minimum depth will be required to keep the speed, and thus the erosion, within limits. This factor will pose the strictest limits to the required depth.

It has to be noted, that an excavation down to SOB -13 m will be needed for the construction of the power plant anyway, so that the nature and elevation of the upstream bed pose no strict limitations to the selection of location; a higher elevation and a more difficult terrain will increase investment costs. If horizontal inlet tubes were selected, the main concern would be the protection of the bed of the intake basin against sedimentation and erosion.

A3.Bed level must be sufficiently constant.
This point has already partly been mentioned in the interpretation of A2: excessive erosion may undermine the structure, and an excessive sediment load may damage the turbines, or cause problems downstream from the plant. In the first place this determines the required excavation of the inlet basin. Apart from that, it limits the selection of the location to the downstream end of the reservoir, because in such a situation the river sediments would be allowed to settle inside the reservoir[16]. In Section A4.4.3, it was mentioned that siltation of the reservoir has not caused any problems yet, even though a certain amount of reservoir siltation can be expected in this situation. Still, if excavation equipment is present in the area anyway (for the construction of Feni power plant), it may be useful to study the need for dredging inside the reservoir.

A4.Inflow must be regular and undisturbed (in both the horizontal and the vertical plane).
Irregular inflow into the power plant can reduce the efficiency and can cause oscillations of the turbines [Duivendijk'89]. As stated in Section D2.3.2.4, bulb turbines are rather sensitive to oscillations, so that this aspect is quite important.
The approach channel to the plant has to be straight and sufficiently wide (at least 87 m), and symmetrical placing of the turbines is preferable.

A5.Capacity of the approach channel must be sufficient (based on maximum power plant discharge of 1100 m³/s).
In a situation of constant flow through a channel, there is an equilibrium of forces: the force of gravity, caused by the gradient of the flow, is balanced by the shear force from the bed, which is a function of grain size, Reynolds number and hydraulic radius.[17] The gradient can only be changed by a lowering of the water level at the downstream end of the channel.
If the shear force from the bed that counteracts the flow is too large, the gradient for a discharge of 1100 m^3/s has to be large, so that the water level at the intake of the power plant has to be low. This leads to a smaller hydrostatic head, which would result in a loss of production.

B1.Bed level must be sufficiently low from the plant down to the Bay of Bengal.
The considerations with regard to the elevation of the downstream bed level are comparable to those for the upstream bed level (see A2). The downstream level, therefore, must be about SOB -10 m. Because of the excavation for construction, this does not really limit the selection.
The discharge out of the outlet tubes can be considered as a jet without sediment load. The combination of large discharges with the small grain size of the bed inevitably leads to scour. To prevent damage to the structure, some sort of bed protection is required; furthermore, there will have to be a connection between the low bed level of the outlet basin and the higher bed level of the outflow channel toward the Bay of Bengal. These aspects will be treated later on, after the selection of the location.

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[16]Sediment traps, that could solve the sedimentation problem at any location, are a theoretical possibility. However, structures of this kind don't function very well in practice.

[17]This is somewhat simplified: factors like wind shear stress and local acceleration aren't taken into account.

The additional requirement for the bed level down to the Bay of Bengal is important. In the wet season (which is the relevant period), the Regulator discharge channel is assumed as a boundary condition, and is also assumed to meet all requirements with regard to level and capacity. Therefore, connection of the power plant outlet channel to the Regulator discharge channel is considered sufficient.
From the plant down to that point though, it is considered better (and possible) to guide the water through a fixed outlet channel (the maximum discharge is known: 1100 m^3/s). The dimensions of this channel are important factors in the investment costs; therefore, the distance between power plant and discharge channel should be minimised.

B2.Bed level must be sufficiently constant.
This issue is more important for the downstream side than for the upstream side: if the downstream side is within the tidal influence, a situation similar to that of the Regulator might develop (excessive siltation in the dry season, excessive erosion in the wet season). Operation of a plant in such a situation is impossible: if the outlet basin is silted up each dry season, the structure could be harmed and expensive dredging works would be required each year.
It seems rather difficult to counteract this phenomenon, as the bed level of the power plant outlet channel will have to be lower than SOB 0, at least, so that each high tide would normally reach the plant. From this point of view, some sort of protective structure inside the channel, to block the entry of the tide but pass the discharge, should be considered.

B3.Capacity of the outlet channel must be sufficient (based on maximum power plant discharge of 1100 m^3/s).
As for the upstream side, the dimensions of the outlet channel should be based on two considerations: the behaviour of the flow and the behaviour of the bed of the channel.
The behaviour of the flow is determined by the dimensions in such a way, that insufficient capacity will cause higher water levels just downstream from the plant, and will therefore result in a loss of production. With regard to the behaviour of the bed, erosion, siltation and also meandering will have to be limited.

B4.The power plant structure must be protected from extreme springtide levels and cyclones.
For the protection against extreme springtide levels and cyclones, two options exist: one is, to take such extreme conditions into account for the design of the structure; the other is, to take measures to protect the structure from those conditions. As stated in B2, a structural protection against siltation is worth considering. Protection against cyclones could be added to the functional requirements of such a structure.

C1.Access to the site must be adequate, both for construction and for operation and maintenance.
The accessibility will have to be improved for construction anyway, to enable transport of the turbines to the site. Clearly, the accessibility influences the costs of construction to a great extent. Safe and easy access, even in extreme natural circumstances, is important for operation and maintenance. Obviously, a location near existing roads is preferable.

C2.The total discharge capacity of the project (Regulator + power plant) must not be reduced as a result of the location of the power plant.
Only this last condition is of a different type than the other conditions: if this one would not be followed, major changes to the existing structures would be necessary.

This condition is based on the assumption, that hydro power production does not have the highest priority for the MIP, and in fact only comes third, after flood control and irrigation. However, this condition does not have to mean that any meddling with the Regulator and its canals is impossible: it could be an option to replace a part of the Regulator discharge capacity with the power plant capacity in some way.
According to [IECo'83], the discharge capacity of the Regulator suffices for a 50-year flood during an extreme springtide situation. The maximum Regulator discharge in this situation would be about 2700 m^3/s, as determined with flood routing studies.

3.3.4.Conclusion

The conditions for the selection of the location of the power plant can be summarised by the following two points:

- Upstream: Inflow basin must be wide and the inflow undisturbed; location at the existing reservoir rim, or connected to the reservoir by a wide channel.
- Downstream: Downstream basin must not be located within the tidal influence, but it has to be connected to the discharge channel; location behind a protective structure, possibly with an outlet channel.

The complete lay-out would then consist of the following components:

- (possibly) intake channel from the reservoir;
- (possibly) intake basin;
- power plant structure;
- outlet basin;
- outlet channel towards coastal embankment or Dam;
- protective structure;
- (possibly) connection to discharge channel.

3.4.Options

3.4.1.Introduction

In this chapter, the possible locations and lay-outs, following from Section D3.3, will be determined and presented. A choice has to be made between two possibilities: either to combine the power plant lay-out with the Regulator or its approach channel in some way, or to avoid any interference with the present structural elements of the MIP.

First, the extent to which the power plant lay-out can be combined with elements of the present situation will be determined in Section D3.4.2. Then two possibilities that do not involve a combination will be treated in Section D3.4.3.

3.4.2.Combination with present discharge facility

3.4.2.1.Introduction

Clearly, to use the hydrostatic head between reservoir and discharge channel , these two have to be connected in some way. In the present situation, they are only connected by the approach channel and the Regulator. In this section, the options to combine elements of the power plant project with these two elements of the present situation will be explored.

3.4.2.2.Regulator

The protective structure that will be required is comparable to the existing Regulator: it has to prevent siltation at the foot of the plant, it has to protect the structure against extreme natural circumstances and its discharge capacity has to be in the same range as that of the Regulator. Therefore, it seems obvious to look for a solution in which the Regulator could act as the required protective structure. An impression of such a solution could be, to place a cross-dam in the present inlet of the approach channel, and install the plant and a spillway facility (to reach the required 50-year-flood discharge capacity) in this dam. A solution like that would fulfil all requirements; furthermore, no protective structure would be required, and no special power plant channels would have to be excavated.

There are two problems though. In the first place, the drainage of the project area would be obstructed during the construction. This could be solved by careful planning of the construction schedule, or by the construction of a temporary diversion channel. The other problem has a more fundamental nature. The sill level of the Regulator is at an elevation of SOB 0. In order to spill the power plant discharge through the Regulator gates, the water level in the channel between plant and Regulator will have to be some distance above this sill level.
An indication of this distance can be derived from Table A3.2: for a Regulator discharge of 1100 m^3/s, an upstream water level of about SOB +2.75 m would be required, even when the tidal level is at its minimum. Checking this value with the theory of broad crested weirs (as performed in Appendix D7) shows that this value is based on a situation with permanently raised flapgates and with a downstream water level that is low enough not to influence the flow. The first condition is possible (it seems that originally facilities were provided for this in the design of the Regulator); the second condition however is only valid during a limited period of time, as the representative tidal curve of Figure C3.2 shows.

Clearly, with such high downstream levels, no hydro-power production on a scale as described in Section D2.4 is possible. This problem could be solved by an adaptation of the Regulator structure; the sill level could be lowered, or discharge capacity could be added on the sides of the Regulator. This has to be considered beyond the scope of this study, and a combination of the plant with the Regulator will not be considered anymore.

3.4.2.3.Approach channel

The approach channel could be used in a different way as well, though. A possible solution is, to use the approach channel down to a certain point, excavate a diversion channel from that point towards the coastal embankment, and include the power plant and the protective structure in that diversion.
As stated in Section D3.2.2, the approach channel was designed to handle a flow of 840 m^3/s without scour. In its short history, the channel has often had to handle larger flows than that. In fact, the daily maximum Regulator discharges in the wet season seldom fall below 840 m^3/s, and can even amount to 2000 m^3/s [Halcrow'91]. Still, no problems connected to erosion or siltation have been reported for the approach channel or the upstream side of the Regulator, so the present approach channel probably fulfils condition A3 of Section D3.3.3. If it would appear that the capacity of the channel is too small (condition A5), or if siltation or erosion problems arise after all, minor adjustments to the present channel would suffice.
If the approach channel is utilised, there are two options for the lay-out: on the right bank and on the left bank of the approach channel (that is, west and east from the Regulator).

The point where the deflection starts is determined by the length that is required for all the components. The route of the diversion towards the coastal embankment has to be determined by the value of the land that would be used and by the accessibility of both power plant and Regulator in the various options.

3.4.3.No combination with present discharge facility

3.4.3.1.Introduction

If the present discharge facilities are not utilised, the reservoir and the discharge channel have to be connected by an excavated channel. The advantage of this option is, that the intake basin can be very large (because the reservoir is directly at the upstream side), which increases the regularity of the inflow and decreases the risk of siltation in the intake basin. The main disadvantage is the need for excavation of a long channel, and the increased use of land.

Roughly two options can again be distinguished: to the west and to the east of the Regulator. A short description of these two options, and of the possibilities within these options, will be given in this section.

3.4.3.2.West of the Regulator

The western solution would have to be connected to the reservoir just upstream from the inlet of the approach channel. This solution can be qualified as a variation of the western solution as described in Section D3.4.2.3. Apart from the already mentioned advantage with regard to the intake basin, there is more freedom in the selection of the route. This implies, for instance, that the MIP project office near the Regulator could be saved, or that the old drainage channel of Sonagazi Regulator could be used, or that the protective structure could be combined with a permanent solution to the erosion problem west of Sonagazi Regulator.

3.4.3.3.East of the Regulator

Any solution east of the Regulator would have to pass through Feni Dam in some way, and would involve the excavation of an outlet channel through the silted area in front of the Dam. There could be a conflict of interests between this option and the (sort of-) planned land reclamation in front of the Dam. On the other hand, a future coastal embankment could be combined with the required protective structure.
In any case, the connection to the reservoir should be made as much to the west as possible, to minimise the length of the channel.
The small piece of land that is presently enclosed by the approach channel, the reservoir and the Dam could be used as well; its present value and use are probably low anyway.

3.5.Selection

From the four options that were mentioned in Section 3.4, the solution to the east of the Regulator, passing through Feni Dam, is selected. This selection is based on the following considerations:

- There is sufficient space, both upstream and downstream from the plant, for the intake and outlet works;
- The disturbance of other facilities in the area will be reduced to a minimum (e.g. transport, Regulator operation, agriculture);
- The influence of the tide is minimal, so conditions for design are less strict and construction will be easier.

The main disadvantage is, that the required protection from siltation will have to be quite elaborate, as a result of the fact that the outlet basin is placed outside the coastal protection. 'Just' a protective structure inside the outlet channel does not suffice: the banks of the channel have to be protected as well. The best solution for this is probably the construction of the new coastal embankment that is planned in the framework of land reclamation, as mentioned in Section A4.4.7 and D3.2.2.

Due to lack of time, there will not be a complete design of the selected lay-out in this study. Some aspects of implementation into the environment, such as the intake basin and the outlet basin and channel will be determined and some aspects of the dimensions of the power plant will be revised. The results will be presented in a set of technical drawings as well. The protective structure that is required against siltation will not be considered in more detail; it is an essential aspect of the project, but more detailed design would almost require a complete study just for this aspect.

4.TECHNICAL DRAWINGS

4.1.Introduction

The final part of this phase of the study is the visualisation of the design with technical drawings. The drawings include:
- a lateral section;
- perpendicular to the flow: upstream and downstream views and a transverse section at the turbine axis;
- a view from above.

The drawing of the lateral section is based on examples in [CFGD'85] and [ÖNT'91]. Some of the dimensions that were determined in the preceding chapters have been adapted slightly, to facilitate drawing. For example, instead of a turbine diameter of 4.96 m, a value of 5 m is taken. All elevations are relative to SOB.
The bed level of the inlet basin is just assumed at the inlet level that follows from the shape of the inlet tube, that is again based on examples from literature. No detailed calculations have been performed in this respect.

In this chapter, first some general assumptions that were needed to make the drawing, mainly concerning the dimensions of the plant are treated. Then two elements concerning the implantation of the structure into the environment are discussed separately: whether piping will occur below the structure, and the dimensions of the outlet channel.

4.2.Power plant

The general build-up of the lateral section (perpendicular to the flow) is based on examples in [CFGD'85] and [ÖNT'91].

The exact location of the plant relative to the Dam is determined by the location of the road on top of the plant; this road is placed in one line with the road on top of the Dam. Furthermore, location of the main body of the plant near the core of the Dam reduces the excavation volume, as the width of the excavation is minimal there.
On the other hand, if this location in the core of the Dam is selected, problems might arise concerning the functioning of the Dam itself: if watertightness has to be guaranteed, the method of construction may become very complicated. In that case, it might be better to place the power plant structure on the sea side, and guide the water through the Dam via longer intake tubes.

To make sure that the implantation of the plant does not harm the protection against cyclones, the top of the inclined concrete wall on the downstream side is given the same elevation as the Dam itself (SOB +10.5 m).[18] The concrete wall is connected to the top of the Dam.

[18]The actual safety against flooding isn't equal, because the elevation of the Dam is partly determined by wave run-up, and the slope angles aren't equal.

The tubes that guide the water through the power plant change their shape from rectangular to circular to rectangular again. This is common practice in power plants. The reason for that is, that it is much easier to work with rectangular intake and outlet gates. The dimensions of the inlet opening are 10 by 10.5 m; the gates are provided with trash racks. At the downstream end, the dimensions are 12 m width by 8.5 m height. The area of the outlet opening is 102 m^2, so the average velocity of the water at that point is about 1.8 m/s.

4.3. Dimensioning of the outlet channel

4.3.1. Introduction

The dimensions of the channel depend on the discharge and the bed properties. The particle size of the bed material leads to a critical velocity above which scour occurs. The friction angle of the bed material leads to a maximum slope for the banks of the channel. With these two characteristics and the given design discharge, the dimensions of the channel can be determined. The transition between the outlet basin and the channel, and the extent to which bed protection is required in this transition zone will be treated as well.

4.3.2. Stability

4.3.2.1. Critical velocity

The non-scouring velocity can be determined in various ways. In [Grishin'82], the empirical formula of A.M.Latyshenkov and B.I.Studenichnikov is given as

$$u_{ns}=5\times d_{av}^{0.3}\times h^{0.2}$$

for d_{av} < 1-10 mm.
(d_{av} : average grain diameter, h : waterdepth)
With $d_{av} = 1\times10^{-4}$ m and h = 5 m, this yields a value of 0.435 m/s.
This value has to be corrected for the porosity of the bed material (leading to a reduction of 15%), and for the uneven horizontal velocity distribution (another 15% reduction); the non-scouring velocity then becomes 0.31 m/s.
More widely known methods are described in [Schiereck'92], mainly the method of Izbash and of Shields. The Izbash formula yields a critical velocity of 0.07 m/s. This formula is not very accurate though: the position of this critical velocity above the bed is not defined, and the influence of the waterdepth is not taken into account.

The Shields formula does take the water depth into account. Shields found values for the relation f in the following equation:

$$\Psi=\frac{u_*^2}{\Delta gd}=f(\frac{u_*d}{\nu})$$

with
u_* = shear velocity;
Δgd = effective weight of a grain of bed material;
ν = kinematic viscosity.

Values for Ψ as a function of f are presented in the Shields-diagram. As u_* appears on both sides of the equation, an iterative process is required to solve it. For $d = 1 \times 10^{-4}$ m, $\Delta = 1.65$, $g = 9.8$ m/s^2 and $\nu = 1 \times 10^{-6}$ m^2/s, u_* appears to be about 0.013 m/s and Ψ about 0.1. In a uniform flow, the shear velocity is related to the depth averaged velocity, and the following formula for the depth-averaged velocity u_c results:

$$\frac{u_c}{\sqrt{g\Delta d}}=\frac{C\sqrt{\Psi}}{\sqrt{g}}$$

For a Chézy-coefficient C of 70 m$^{1/2}$/s [Haskoning'83], this leads to a critical depth averaged velocity of 0.28 m/s.

The Russian formula and Shields appear to agree quite well. Therefore, a critical depth averaged velocity of 0.3 m/s will be assumed for design.

4.3.2.2.Embankment slope angle

According to [Haskoning'83], the friction angle of the bed material is about 20^0. This is equal to a slope of 1 : 2.75. If this material is used for channel embankments, the slope has to be appreciably smaller for stability. There are two reasons for this: groundwater flow out of the embankment in case of a low water level inside the channel, and the flow of the channel perpendicular to the slope. For the first factor, the general rule exists that the slope of the embankment has to be about half the internal friction angle for stability. This leads to a slope of 1 : 5.5. For the situation of the channel flow perpendicular to the slope, the computed critical velocity for a flat bed has to be reduced, according to [Schiereck'92]. The appropriate factor follows from the formula:

$$K(\beta)=\sqrt{1-\frac{\sin^2\beta}{\sin^2\phi}}$$

in which
β = slope angle
ϕ = friction angle.

This factor K applies to forces; for velocities the square root has to be used. The resulting reduction factor for the critical velocity of 0.3 m/s then is 0.93, which leads to a critical velocity of 0.28 m/s. As the initial calculation was already rounded off and based on assumptions, the same critical value of 0.3 m/s will be applied for further analysis anyway.

4.3.3.Loads

4.3.3.1.Introduction

If the behaviour of the plant outflow could be schematised as a flow from one outlet, the behaviour of the water velocity would be comparable to a 3-dimensional wall jet. 3-Dimensional wall jets are described in [Schiereck'92]. The velocity profile at a certain distance from the origin (x) in such a jet can be modelled as Gaussian, both in the vertical (z) and the horizontal (y) plane. An indicative formula for the maximum speed at a distance x, from [Schiereck'92], is the following:

$$u_{m0}=\frac{8u_0}{x/\sqrt{A}}$$

According to this formula, the maximum velocity would decrease below 0.3 m/s in about 480 m. This value only serves as an indication: this maximum velocity is not relevant for bed scour: as stated, the average velocity is needed for that.

In case of the design discharge of 1100 m^3/s though, there will be full flow out of all six gates. The six jets will overlap in that case, which means that the resulting velocity profile is only Gaussian at the fringes (near the embankments), and about constant in the middle. In other words, the energy of the jet is distributed more equally.
For the remainder of this analysis, the horizontal distribution of the flow at a certain x will therefore be assumed to be constant in the horizontal plane.

4.3.3.2.Channel dimensions

The downstream end of the outlet channel is its connection with the Regulator discharge channel. For this downstream end, a bed level of SOB -5 m is assumed. Furthermore, the inclination of the channel bed (i) is selected at 1×10^{-4}, the slope of the embankments at 1 : 5.5. If the channel has to be designed for non-scouring conditions with a flow of 1100 m^3/s, a channel cross-section of about 3700 m^2 would be required. A trapezoidal profile is selected, and if the water level is assumed at SOB 0, the resulting bed width is about 700 m.
If the flow diverges with a slope of 1 : 6 (valid for jets, [Schiereck'92]), the horizontal transition zone has to be almost 2 km long. This is approximately the distance between the power plant and the Regulator discharge channel. If no scour is allowed, the bed would have to be protected all the way down to the Regulator discharge channel. This has to be considered impractical: in practice, bed protection will be applied to such a point, that the dimensions of the downstream scouring hole will be within limits. This point will be determined at the end of this section.

In the transition zone between the basin and the channel, the velocity profile is still developing. Due to complicating factors such as the changing bed level, calculations for this section are difficult. However, an upstream boundary condition is required for the channel, so indicative dimensions have to be given.

The average velocity of the water at the point of outflow is 1.8 m/s. For the downstream end of the transition zone, the cross-section will be selected such that the average velocity is 1.5 m/s[19]. The dimensions of this downstream end then have to be as follows:
The bed level is about SOB -4.8 m (following from the assumed bed level inclination of 1×10^{-4} and the downstream bed level of the channel of SOB -5 m), the maximum value of the water level during production is assumed at SOB 0 for this calculation. Therefore, the wet cross-section has to be 733 m^2, below the line of SOB 0. If the channel banks are assumed to have a slope of 1 : 5.5 (not really necessary yet, because the bed is still protected at this point), and with a trapezoidal profile, the bed width at this point has to be 125 m. If the flow diverges along this same 1 : 6 slope[20], the distance between the outlet basin and the start of the channel has to be about 115 m. The outlet basin is assumed with a constant bed and with a length of 10 m.

The final longitudinal profile of the channel then consists of three sections:

■ Outlet basin :	bed width	: 87 m;
	length	: 10 m;
	inclination	: 0;
	bank slope	: from 1 : 1 to 1 : 2;
■ Transition :	bed width	: from 87 m to 125 m;
	length	: 115 m;
	inclination	: + 1 : 22 (upward);
	bank slope	: from 1 : 2 to 1 : 5.5;
■ Outlet channel:	bed width	: from 125 m to 700 m;
	length	: 1725 m;
	inclination	: -1 : 10 000 (downward);
	bank slope	: 1 : 5.5.

4.3.3.3.Bed protection

As stated, it is not practical to apply bed protection down to the point where the velocity decreases below the critical velocity. At the downstream end of the protection, a scouring hole will develop. As long as this scouring hole is controllable, this does not have to be a problem.

According to [Schiereck'92], the development of the depth of the scouring hole is an asymptotic process. The asymptotic depth for a plane jet is given by the following formula:

$$\frac{h_{sm\infty}}{2B_u} = 0.008\left(\frac{u_0}{u_{*c}}\right)^2$$

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[19] It is doubtful whether the assumption of a horizontally constant velocity is already valid at this point.

[20] The diverging of the flow near the banks is neglected: only the bed width is taken into account.

In shallow water, as is the case in this situation, the depth of the scouring hole is about half, which yields

$$\frac{h_{sm\infty}}{2B_u}=0.004\left(\frac{u_0}{u_{*c}}\right)^2$$

The flow in the channel is schematised as a plane jet. In that case, B_u is the width of the flow as it has diverged from the outlet gates. U_0 is the average velocity of the flow and u_{*c} is the critical velocity of 0.3 m/s. The resulting scouring hole will be calculated for some values of u_0; u_0 leads to a required cross-sectional area; that leads to a bed width (B_u), from which both h_{sm} and the distance from the outlet can be calculated. The following combinations of scouring hole depths and bed protection lengths result:
For u_0 = 0.4 m/s:
h_{sm} : 7.5 m; length : 1200 m.
For u_0 = 0.5 m/s:
h_{sm} : 8.8 m; length : 825 m.
For u_0 = 0.6 m/s:
h_{sm} : 10.9 m; length : 645 m.

For this situation, the solution with a velocity of 0.5 m/s will be selected. From [Schiereck'92], the length of the scouring hole can be expected to be about 12 times the depth of the hole, which leads to about 100 m.

The elevation of the banks of the outflow channel is put at SOB +5 m. Obviously, these banks do not have to suffice for the same conditions as the Dam. In the drawings, the soil level downstream from the Dam is estimated at SOB +3.5 m.

4.4. Piping

4.4.1. Introduction

The second matter of concern is the foundation of the structure. A complete analysis will not be given: the focus in this section will be on the transitional zone between structure and subsoil.
A substructure will probably not be needed, so the structure can be placed directly on top of the soil. As a result of the head over the structure, a groundwater flow will occur through the soil. In such a situation, there is a risk that the groundwater flow will concentrate just below the structure, where the resistance to groundwater flow is minimal. Because of this smaller resistance, the velocity of the flow is higher, and may even become high enough to transport grains. In this way, piping might develop.

The extent to which piping occurs depends on the grain size and on the gradient of the flow. The gradient of the flow is equal to the hydrostatic head divided by the length of the seepage path. For this analysis, the hydrostatic head has to be an extreme design condition. A reservoir level of SOB +4 m and a downstream level of SOB -0.5 m are used for this, leading to a hydrostatic head of 4.5 m. Various empirical methods have been developed to calculate the relation between these factors. Two of these methods (Bligh's and Lane's) will be used for this analysis.

4.4.2.Seepage path

According to Bligh, the length of the seepage path (L) has to fulfill the following requirement:
$L \geq 1.5 \times C_B \times h$

in which
C_B = soil constant, = 15 for fine sand (according to Bligh);
h = hydrostatic head, = 4.5 m.

This formula yields a value of 100 m.
The length of the seepage path, according to Bligh, is the total length (horizontal and vertical) of the transition between structure and subsoil. In the determined situation, the horizontal distance is about 48 m, and the vertical distance is 3.85 m (upstream side) plus 5 m (downstream side), or a total of about 57 m.

According to Lane, the same formula is valid, but the implementation is different, and so are the soil constants: C_B = 7 for fine sand. This yields a value of 58 m.
The definition of the length of the seepage path differs from Bligh's: according to Lane, the horizontal sections of the seepage path only count for one third of their actual length. Therefore, the appropriate value of the length of the seepage path is 16 + 3.85 + 5, which is about 25 m.

For this analysis, Lane's method is selected. Usually, this is the method that leads to smaller improvement measures; at least in the Dutch circumstances, structures that are designed according to this method appear to be quite safe against piping.

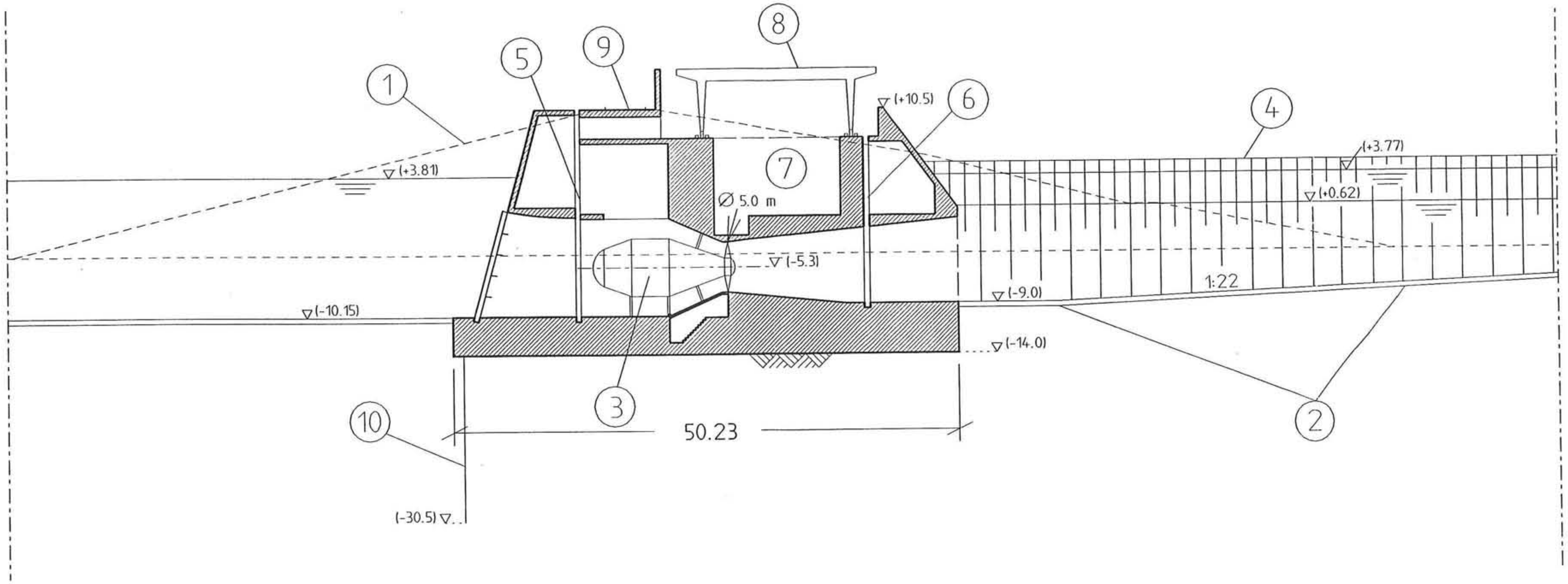
4.4.3.Improvement measures

Apparently, measures are required to counteract piping underneath the power plant structure. Principally, two solutions exist: decreasing of the soil constant C_B, or increasing of the length of the seepage path.

The first solution would imply injection of the subsoil. Even in developed countries, this method is considered too expensive to be practical, so it will not be considered for this case.
The second solution offers several possibilities. One solution would be to project a watertight bed protection, preferably at the upstream side. A solution that is used more often is the installation of sheet piling. This is selected for this case as well.

According to Lane, there is deficiency of seepage path length of about 33 m. This means that 16.5 m of sheet piling is required for safety against piping. The sheet piling will be placed at the upstream side of the structure, as in that way the upward pressure of the groundwater against the structure is reduced as well.

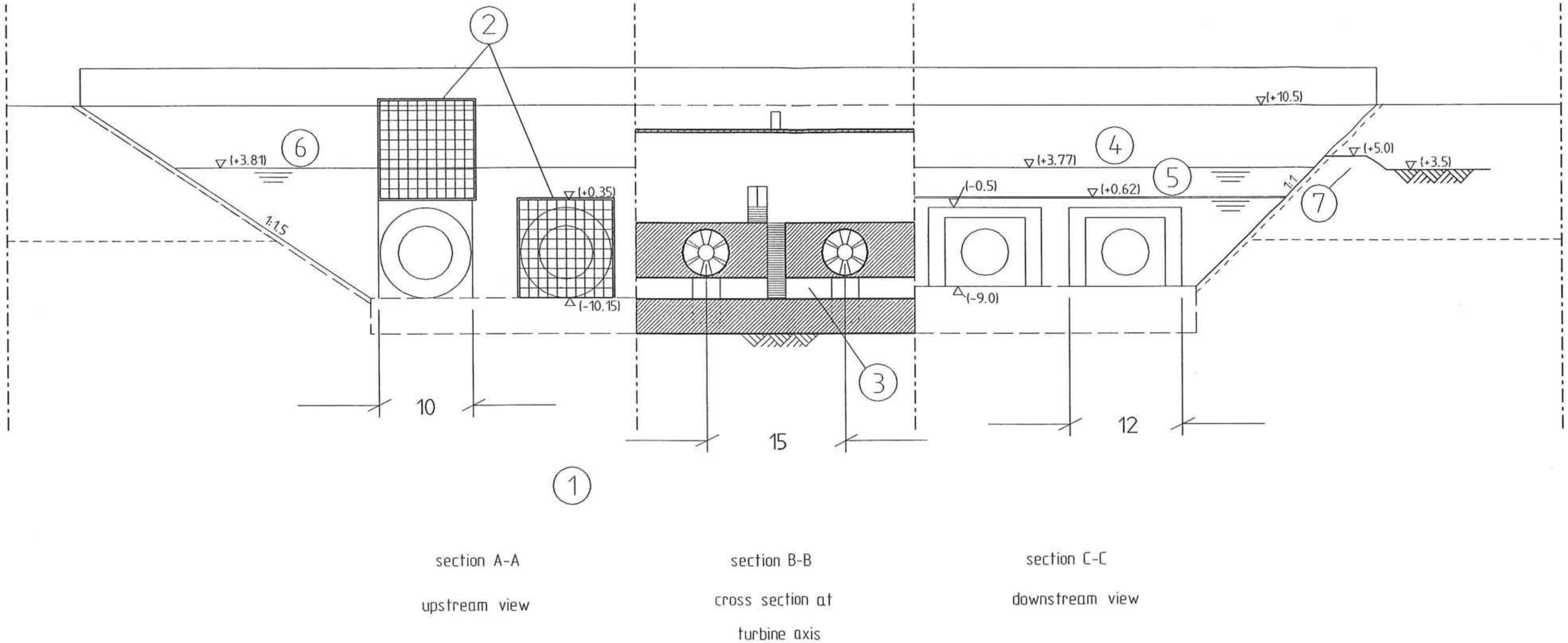
If a sub-structure appears to be required for the power plant after all, more sheet piling would be needed: in that case, settlement of the soil is not followed by the structure itself, which means that the resistance against groundwater flow is reduced even more near the bottom of the structure.



Lateral section

scale 1:500

1. feni dam outline
2. bed protection
3. bulb unit
4. outlet channel embankment
5. intake gate
6. outlet gate
7. control floor
8. gantry crane
9. road
10. sheet piling



scale 1:500

1.fine sand (100 mu)
2.screen
3.inspection gallery
4.normal high water
5.normal low water
6.normal high reservoir level
7.outlet channel embankment

Transverse views

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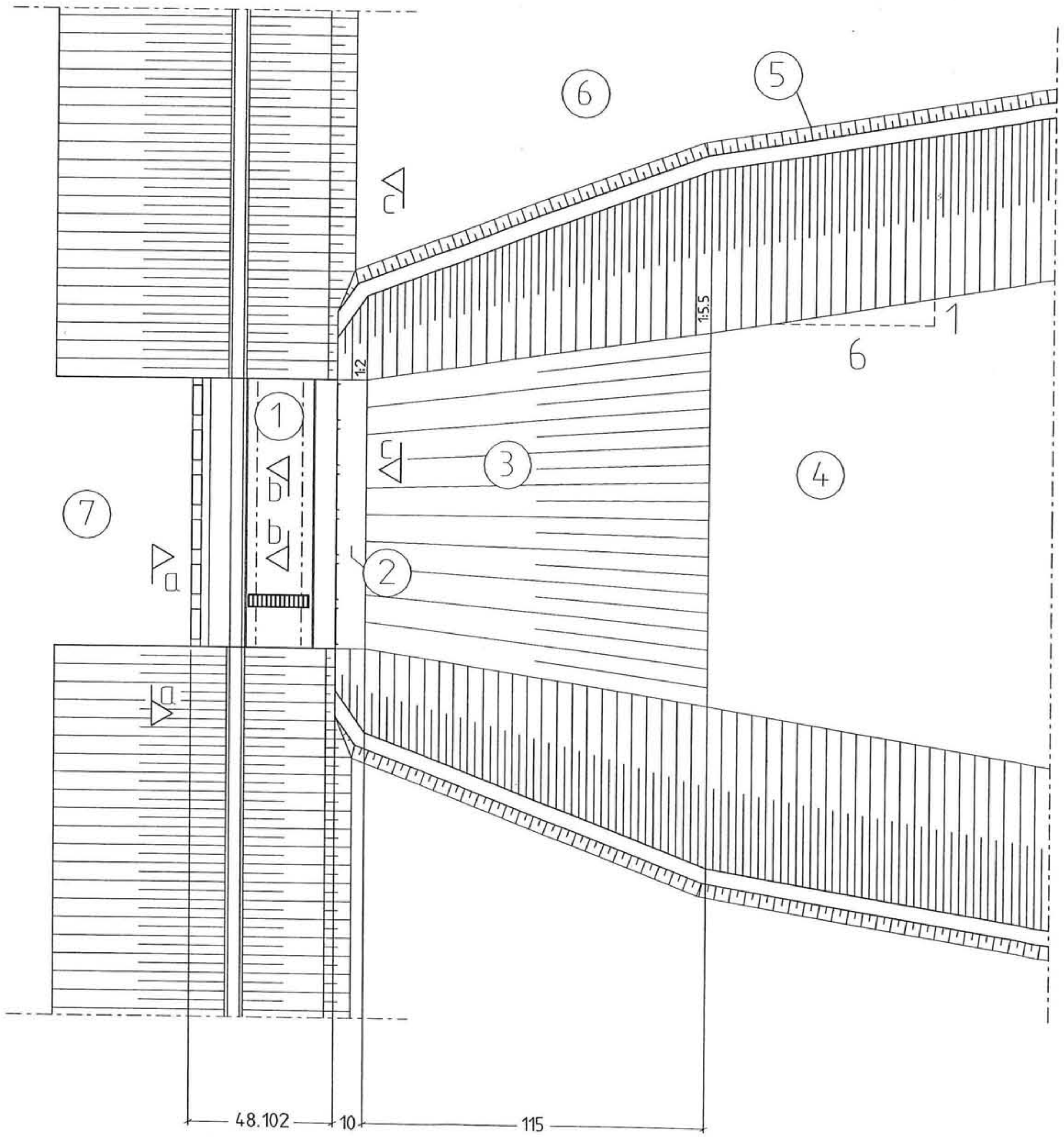

Top view

scale 1:1800

1. power plant structure
2. outlet basin
3. transition basin-channel
4. outlet channel
5. channel embankment
6. silted area
7. intake basin
8. reservoir

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Phase E: Evaluation

1.INTRODUCTION

At the end of a study such as this, the preceding work has to be evaluated. Firstly, in Chapter 2, the feasibility of the selected solution is assessed. Then, in Chapter 3, a reiteration to the problem definition is performed.

2.FEASIBILITY

In the present situation, it has to be considered very doubtful whether a power plant as designed in this study is feasible. Some reasons for this have already been mentioned earlier, in the study itself. They are repeated in this chapter.

The first reason concerns the fact that a large part of the water for production originates in India (see Section A4.4). Because of this, uncertainty exists about the future availability of this water.
The second reason concerns the operation of the project if a power plant is added to the MIP. This has been mentioned in Section A6.2. A professional system, using accurate forecasts of inflow and tides, is probably impossible in Bangladesh at the moment; apart from that, implementation of such a system will probably not be considered worth the trouble.
Another problem has to do with the influence of the natural circumstances on the power plant. This is treated in Section D3.3. If no measures are taken to counteract siltation of the outlet basin by the high tide, the basin will be filled up in a short time; dredging may be a possibility, but it will have to be done very frequently. This factor at least increases the costs.
Finally, it is doubtful whether the required investment costs are not too high for the relatively small proceeds that can be expected. This factor is hard to assess: it depends on the actual costs and proceeds, but also on such unpredictable features as the price of oil.

3.DESIGN PROCESS

According to the problem definition, the optimum application of hydro power had to be found in this study. It is questionable and difficult to assess whether this result has been achieved. Anyway, the cost/profit ratio probably renders the project unfeasible, so it is imaginable that better solutions exist. For instance, maybe the scale of production should have been chosen somewhat smaller. The scale of the production was determined in Chapter D2; this chapter will be evaluated to find alternatives.

The first selection, and the basis of the rest of the chapter, concerned the probability of exceedance. Table D2.4 and Section D.5.2 indicate that a value of 30% was the right selection. This is clearly correct if the design process of Section D2.3 is followed for each alternative. The question remains: is this design process the optimum one?
An important basis of the design process is the requirement of an approximately constant hydrostatic head during production, in order to have an optimum efficiency of production. This requirement more or less determines the discharge capacity of the plant. As stated in Section D2.3.3, no clear data are available on the exact relation between design hydrostatic head, actual hydrostatic head and the efficiency of production. If a smaller

efficiency would be allowed, a larger variation of hydrostatic head could be allowed. In that case, it might be possible to install a significantly smaller discharge capacity, that could spill the same design water volume in a longer production period. The annual output would be smaller, because of a lower average hydrostatic head and a smaller efficiency, but a solution along these lines would also be quite a lot cheaper.
On the other hand, major parts of the project, such as the concrete structures and the outflow channel would still be required, though their size would be somewhat smaller. The difference therefore, would not be a principal one.
It is difficult to assess both the change of production and the change of costs; therefore, such a solution is not necessarily better.

A principally different solution would be to work with units of a very small scale. In that case, no large structural investments would be needed. However, the one feature that could make the site feasible for hydro power is the magnitude of the wet season discharges; this compensates the extremely low hydrostatic head.
Anyway, such an application would have been less interesting from an educational point of view, and one of the assumptions in the program of requirements (Phase B) was the prevalence of the educational value above the realism of a solution.

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Appendices

Appendix 1: List of Tables

Appendix 2: List of Figures

In Phase A:

In Phase C:

In Phase D:

Appendix 3: List of abbreviations

The abbreviations are listed in alphabetical order.

- BIWTA : Bangladesh Inland Water Transport Authority
  Agency of the Government of Bangladesh that controls river transport.
- BUET : Bangladesh University of Engineering and Technology
  Polytechnical section of the University of Dhaka.
- BWDB : Bangladesh Water Development Board
  Agency of the Government of Bangladesh that controls services and projects that are concerned with water.
- CIDA : Canadian International Development Agency
  Governmental agency for international development.
- EEC : European Economic Community
- EPWAPDA : East Pakistan Water and Power Development Authority
  Equivalent of BWDB before independence of Bangladesh (March 1971).
- e.r.r. : Economic rate of return
- HYV : High Yielding Varieties
  Varieties of crops that involve a more intensive use of land.
- IDA : International Development Association
  Organisation within World Bank.
- IECo : International Engineering Company
  American consulting engineers' company; designed Feni Regulator and initial plan for Feni Dam.
- MIP : Muhuri Irrigation Project
  Irrigation project in the south-east of Bangladesh, near the town of Feni, of which Feni Dam, Regulator and reservoir are components.
- SOB : Survey of Bangladesh
  Reference level; mean sea level.
- SRP : Systems Rehabilitation Project
  Consortium of Dutch companies, that consults BWDB with regard to irrigation projects.
- Tk : Taka
  Currency unit of Bangladesh; Tk 40 equals about US$ 1.
- WPDC : (International) Water Power and Dam Construction
  Monthly magazine.

Appendix A1: Formal description

Multi-purpose operation of Feni Reservoir, Bangladesh

The Feni estuary was closed in 1985. The freshwater reservoir behind the dam is providing water for irrigation in the Muhuri Irrigation Project.
Surplus water (i.e. water which cannot be stored in the reservoir because it is full and which originated from the river Feni and local rain on the reservoir) is spilled through the Feni Regulator into the Bay of Bengal.
Spilling is at present done on a more or less continuous basis during the wet season in order to counteract siltation of the channel between Regulator and Bay of Bengal.
It has been proposed to construct a small hydropower plant which would generate energy by guiding the surplus inflow through the turbine(s).

Aspects to be studied:

- inflow into reservoir (average, minimum, peak) as it varies during a season and also during the past 8 years;
- outflow as spilled through the reservoir and used for irrigation;
- optimum water use as follows from agricultural practices and related costs and benefits of irrigation water supply and hydropower energy;
- design of hydropower plant.

Appendix A2: Photographs

In this appendix, some photographs that were taken in Bangladesh and that could serve to clarify aspects of this study are presented. These photos are the following:

- Photo 1: Aerial photograph
- Photo 2: Aerial photograph
- Photo 3: Panorama downstream from the Regulator
- Photo 4: Radial gate
- Photo 5: View of the Regulator
- Photo 6: MIP at a glance
- Photo 7: Panorama downstream from Feni Dam
- Photo 8: Groyne building near Sonagazi Regulator

Photo 1: A complete view of the project area. From left top corner to right bottom corner: the tributaries and their catchment areas, the reservoir, Feni Dam with silted downstream area, approach channel and Regulator, the (silted) meandering bed of the discharge channel and the breach of the coastal embankment near Sonagazi Regulator.

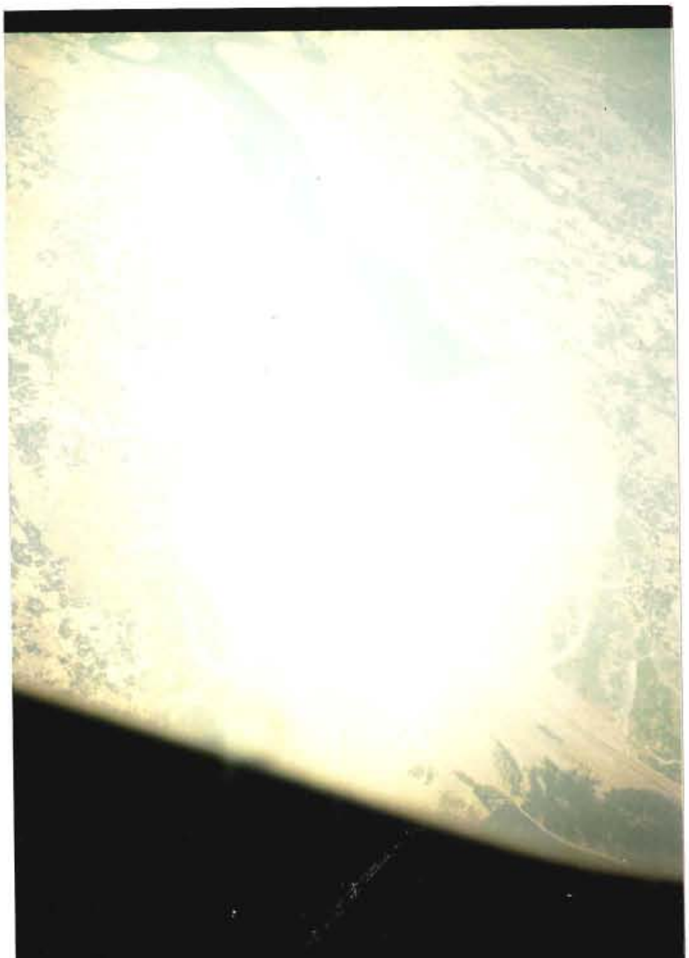


Photo 2:
A closer view of the silted downstream area and the breach of the coastal embankment. The location of the old estuary embankments can be distinguished, some distance from the reservoir banks.

Photo 3: A panoramic view from the Regulator toward the downstream stilling basin, taken on February 2, 1994. This is the dry season, and the photo was taken at low tide, so the view mainly consists of silt. Small settlements can be seen on the banks of the channel.

Photo 4: One of the 40 radial gates, in a closed position because it is the dry season, and irrigation is still required. Apparently, there is a surplus of inflow: there is a small flow over the top of the gate.

tream stilling basin,
e photo was taken at
ments can be seen on

position because it is the dry season,
, there is a surplus of inflow: there is

Photo 5: A complete view of the Regulator from the side of the approach channel. The radial gates are on this side, the flap gates on the other. One function of the Regulator is coastal defence: it is connected to the coastal embankment on both sides. Feni Dam starts on the left edge of the photo.

Photo 6:
The MIP at a glance. These data represent the MIP as it was intended at the start. Some aspects, like the internal rate of return and the minimum reservoir level, have been adapted since.

Photo 7: A panoramic view from Feni Dam toward the silted area downstream. The land has already been divided, and cattle is grazing, even though a small tidal gully still extends to the foot of the Dam.

Photo 8: A groyne being built the Bengali way. This photo was taken at the eastern edge of the coastal embankment breach, near Sonagazi Regulator (see photo 1).

stream. The land
a small tidal gully

hoto was taken at the eastern edge
onagazi Regulator (see photo 1).

Appendix D1: Calculation of turbine parameters

This appendix describes the results of a statistical analysis of the relation between turbine parameters and hydraulic conditions in actual projects. An analysis of this kind was performed for bulb turbines in [WPDCsep'88] and for vertical axial turbines in [WPDCnov'-92].

The basis of the calculation is the general dimensioned specific speed formula

$$N_q = n \times Q^{\frac{1}{2}} \times H^{-\frac{3}{4}}$$

in which
N_q = specific speed
n = rotational speed (rev/min)

With regression analysis, the following relations between N_q and H are then found:
$H = 69180 \times N_q^{-1.6}$ (1976 to 1985, [WPDCsep'88])
$H = 920650 \times N_q^{-2.058}$ ($P > 2$ MW, [WPDCnov'92])

Then a peripheral velocity coefficient is introduced (K_u), that is defined as follows:

$$K_u = \frac{\pi \times D_R \times n}{60 \times \sqrt{2 \times g \times H}}$$

Again with regression analysis, the following relations between N_q and K_u can be found:
$K_u = 1 + 0.0038 \times N_q$ [WPDCsep'88]
$K_u = 0.8434 + 0.00456 \times N_q$ ($P > 2$ MW, [WPDCnov'92])

Now the external diameter of the turbine runner (D_R) follows from the definition of K_u:

$$D_R = \frac{60 \times K_u \times \sqrt{2 \times g \times H}}{\pi \times n}$$

For these initial calculations, a hydrostatic head of 2.5 m is assumed. When this value is incorporated in the given formulas, the relation between discharge and dimensions can be established.

For that situation, the following expressions are valid:
For bulb turbines:
$N_q = 1059.2 \times H^{-0.625} = 597.4 \rightarrow$
$n = 1059.2 \times H^{0.125} \times Q^{-0.5} = 1187.7 \times Q^{-0.5} \rightarrow$
$K_u = 1 + 4.025 \times H^{-0.625} = 3.27 \rightarrow$
$D_R = 0.368 \times Q^{0.5}$

For Kaplan turbines:
$N_q = 790.7 \times H^{-0.486} = 506.5 \rightarrow$
$n = 790.7 \times H^{0.264} \times Q^{-0.5} = 1007.1 \times Q^{-0.5} \rightarrow$
$K_u = 0.8434 + 3.606 \times H^{-0.486} = 3.15 \rightarrow$
$D_R = 0.419 \times Q^{0.5}$

In [WPDCmay'88], some other comparable statistical analyses have been performed for axial turbines, that resulted in the following relations between D_R and Q:

$D_R = 0.434 \times Q^{0.5}$
$D_R = 0.368 \times Q^{0.5}$
$D_R = 0.437 \times Q^{0.4696}$

These relations will not be used for design: the first one because it has been calculated with a procedure that is comparable to the one in [WPDCnov'92], but with less recent data, the other two because they have been determined with regression analysis of the direct relation between D_R and Q, without taking the hydrostatic head into account. Therefore, these two relations are probably not valid for the Feni plant, as the hydrostatic head of 2.5 m is exceptionally low, compared to other projects. In fact, the lowest hydrostatic head that was incorporated in the analysis for [WPDCnov'88] was 3.58 m.

Appendix D2: Validity of assumptions

First some background information:
The actual location of the plant has quite a large influence on the validity of the assumptions; this location has not been selected yet, so it is assumed that the approach toward the power plant is sufficiently wide and does not obstruct the flow too much. This assumption will be one of the criteria for selection of the location.

In the design situation, there is a flow of 200 m^3/s into the reservoir at the upstream end, near the confluence of the Muhuri and the Feni, and there is an outflow of 1100 m^3/s at the downstream end, through the power plant.
The cross sectional area of the reservoir is far from constant from the confluence down to the Dam; originally the reservoir area was part of the estuary, so the shape of the bed has been formed by ebb flows and flood flows. It is therefore made up of gullies and shoals. Generally speaking though, the cross sectional area increases, and the elevation of the bed decreases from the confluence toward the Dam. In [Haskoning'83], a computer model is discussed that served to predict the tidal flow during the closure of Feni Dam in February 1985. The data that have been used for this have to be treated carefully: obviously, the situation has changed quite a lot since then. They can serve as an indication though.
A cross-sectional area of about 4000 m^2 is given for a water level of SOB +4 m near the site of the Dam. At this same water level, the cross-sectional area at the confluence is 150 m^2. Obviously, if the water is flowing the water table at the confluence has to be higher than at the Dam, so these numbers cannot really be compared. Anyway, it is clear that the velocity of the water during production will probably not be excessive: between 0.25 and 1 m/s. This conclusion was to be expected of course: in the present situation, the discharges are often much larger than 1100 m^3/s, and no problems of erosion inside the reservoir have been reported.

The method of construction of the required discharge curve of Figure D2.4 is based on two assumptions:
- that the reservoir level falls at the same rate all over the reservoir at all times, so that the falling rate is equal to the discharge divided by the reservoir area;
- that the reservoir area is independent of the reservoir water level.

As far as the first assumption is concerned:
There are two different situations: the stationary one, in which the flow has been constant for some time, and the non-stationary one, for example at the start of production.

In the stationary situation the assumption is quite correct, because of the sufficient cross-sectional area of the reservoir. At the start of production though, it takes some time for the whole area to be 'activated'. The celerity of the 'shock-wave' that is created by the opening of the turbines can be calculated with the formula

$$c \approx \sqrt{g \times a}$$

in which g is the acceleration of gravity and a is the water depth. The value of a is, as stated, very variable. In Section A4.4.3, an average value of 1.7 m is mentioned. However, near the Dam and in the middle of the reservoir and in the tidal gullies a value of 3 m is more appropriate. If this is correct, the celerity of the wave is about 5.5 m/s. If the length of the reservoir is assumed at 7 km, it would take 1200 s, or 20 minutes, for the shock-wave to reach the other end of the reservoir.

This means that the falling rate of the upstream water level during the first minutes of the production period will be larger than calculated in the main report, so that the hydrostatic head will not be constant. This could cause problems with the operation of the turbines; these problems will have to be solved in practice by taking a sufficient amount of time for the starting up of production.

The second assumption is correct: according to [IECo'83], the capacity-elevation curve of the reservoir is a straight line above an elevation of SOB +2.6 m, which means that the reservoir area is approximately constant above this level.

Appendix D3: Determination of approximation of parabolic falling rate curve

A parabolic curve is used for the first approximation of Figure D2.4. The second degree function $Q_R^*(t)$ that is selected has a value of 350 m^3/s at t=0 and t=9h30, where Figure D2.4 is at 200 m^3/s, and a value of 2000 m^3/s as its maximum, at t=4h45. $Q_R^*(t)$ is only valid during the receding tide, which is between t = -9h30 and t = 0h.
Time has to be expressed in seconds to enable computation. To facilitate the computation, the curve is mirrored in the $Q_R^*(t)$-axis. The interval of validity thus becomes [0, 34200 s].

The second degree function can be expressed as $Q_R^*(t) = a \times t^2 + b \times t + c$; three datapoints are required for the determination of the constants a, b and c. Those three points are available:

$t = 0$ s	: $Q_R^*(t) = 350$ m^3/s;	(1)
$t = 17.1 \times 10^3$ s	: $Q_R^*(t) = 2000$ m^3/s (at 4.75 h);	(2)
$t = 34.2 \times 10^3$ s	: $Q_R^*(t) = 350$ m^3/s (at 9.5 h).	(3)

From (1), parameter c follows directly: c = 350 m^3/s.
The parameters a and b are then the only unknowns in equations (2) and (3):
$2.92 \times 10^8[s^2] \times a + 17.1 \times 10^3[s] \times b = 1650\ m^3/s$
$1.17 \times 10^9[s^2] \times a + 34.2 \times 10^3[s] \times b = 0$

The resulting values are:
$a = -5.643 \times 10^6\ [m^3/s^4]$
$b = 0.193\ [m^3/s^3]$,

so the following function results:
$Q_R^*(t) = -5.643 \times 10^6 \times t^2 + 0.193 \times t + 350$

with
$t \in [0; 34.2 \times 10^3 s]$

and with $Q_R^*(t)$ in m^3/s.

Curves $Q_R^*(t)$ and $Q_R(t)$ are presented graphically in Figure D2.5.

Appendix D4: Integration calculation

The following calculation has to be executed:

$$8.928 \times 10^{6} = \int_{t_0}^{t_1} Q_R(t)\,dt$$

or

$$8.928 \times 10^{6} = \int_{t_0}^{t_1} (-5.643 \times 10^{-6} \times t^{2} + 0.193 \times t + 350)\,dt$$

The primitive function of $Q_R^{*}(t)$, $F_R^{*}(t)$, is:

$$F_R^{*}(t) = -1.881 \times 10^{-6} \times t^{3} + 0.0965 \times t^{2} + 350 \times t$$

As t_0 is 0, the solutions of the problem are the values of t_1 for which:

$$-1.881 \times 10^{-6} \times t_1^{3} + 0.0965 \times t_1^{2} + 350 \times t_1 - 8.928 \times 10^{6} = 0$$

This third grade equation can be solved with a spreadsheet program, such as QUATTRO PRO, and the resulting values are

t_1 = -10407 s = - 2h53
t_1 = 8586 s = 2h23
t_1 = 53123 s = 14h45

As $Q_R^{*}(t)$ is only defined between t = 0 s and 34.2×10^{3} s, the second value is the only physically relevant solution.

Appendix D5: Determination of turbine operation scheme

The design turbine operation scheme for Situation B has been determined with Figure D2.4. The total outflow is known (8.928×10^6 m^3) and the production period is known, so the total amount of 'turbine hours' can be determined with the following formula:

$$T_t[h] = \frac{V_d[m^3]}{Q_t[\frac{m^3}{s}] \times 3600}$$

The resulting value is rounded off to the next quarter of an hour; then an operation scheme with the calculated number of 'turbine hours' that follows the required dišcharge curve as good as possible, is determined by trial and error. The resulting turbine operation schemes are presented in the two figures that follow this appendix.

Now all required information is available to calculate the development of the hydrostatic head. The most comfortable way of doing this is with a spreadsheet program. This results in Tables 1 and 2; they can be found on the following page. The significance of the columns in these tables is the following:

*The values in column two have been determined from Figure C3.2, and are the same for both situations.

*The third column follows directly from the figures of the operation scheme. The value in a certain row is the turbine discharge in the fifteen minutes that precede the point in time in the first column of the same row.

*The fourth column can be calculated from the third column by dividing the turbine discharge minus the design inflow of 200 m^3/s by the area of the reservoir. This area is assumed constant, at 17 km^2. So:

$$v_r = \frac{Q_t - Q_d}{A_r}$$

Like the turbine discharge, the value of the reservoir rate says something about the preceding fifteen minutes.

*The fifth column again follows from the fourth: each value is made up of the value of fifteen minutes earlier minus the fall of the reservoir level in the time in between. So:

$$z_1 = z_0 - \frac{v_r}{4}$$

The value of SOB +3.81 m in the first row is the design value for the reservoir level at the start of the production period.

*Finally, the hydrostatic head at each moment follows from substraction of the tidal level in column two from the reservoir level in column 5.

Average values and variation of the values is given as well. The variation is defined as the maximum value divided by the minimum value, expressed as a percentage.
The resulting curves are presented in Figure D2.6, and the significance of the values is discussed in the main document.

1100 m3/s

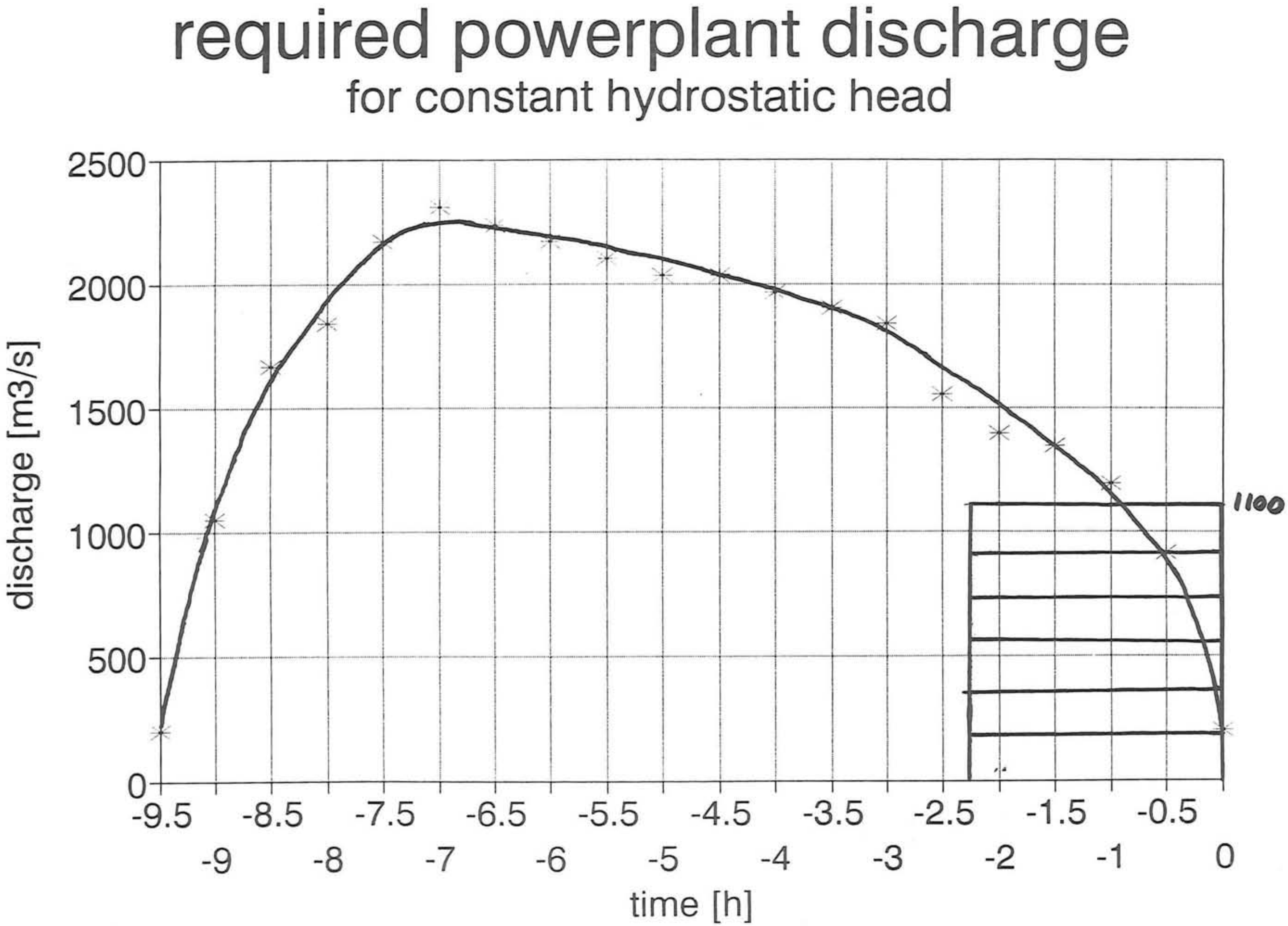
time [h]	tide [m+SOB]	tdis [m3/s]	resrate [m/h]	reslevel [m+SOB]	hh [m]
-2h15	1.07			3.81	2.74
-2h	1.01	1100	0.19	3.76	2.75
-1h45	0.94	1100	0.19	3.71	2.77
-1h30	0.88	1100	0.19	3.67	2.79
-1h15	0.83	1100	0.19	3.62	2.79
-1h	0.76	1100	0.19	3.57	2.81
-0h45	0.71	1100	0.19	3.52	2.81
-0h30	0.67	1100	0.19	3.48	2.81
-0h15	0.63	1100	0.19	3.43	2.80
0h	0.62	1100	0.19	3.38	2.76
				average	2.78
				max.var.%	2.70

Operation scheme for Situation A

1500 m3/s

time [h]	tide [m+SOB]	tdis [m3/s]	resrate [m/h]	reslevel [m+SOB]	hh [m]
-2h15	1.07			3.81	2.74
-2h	1.01	1500	0.28	3.74	2.73
-1h45	0.94	1500	0.28	3.67	2.73
-1h30	0.88	1250	0.22	3.62	2.74
-1h15	0.83	1250	0.22	3.56	2.73
-1h	0.76	1250	0.22	3.51	2.75
-0h45	0.71	1000	0.17	3.46	2.75
-0h30	0.67	1000	0.17	3.42	2.75
-0h15	0.63	750	0.12	3.39	2.76
0h	0.62	500	0.06	3.38	2.76
				average	2.74
				max.var.%	1.12

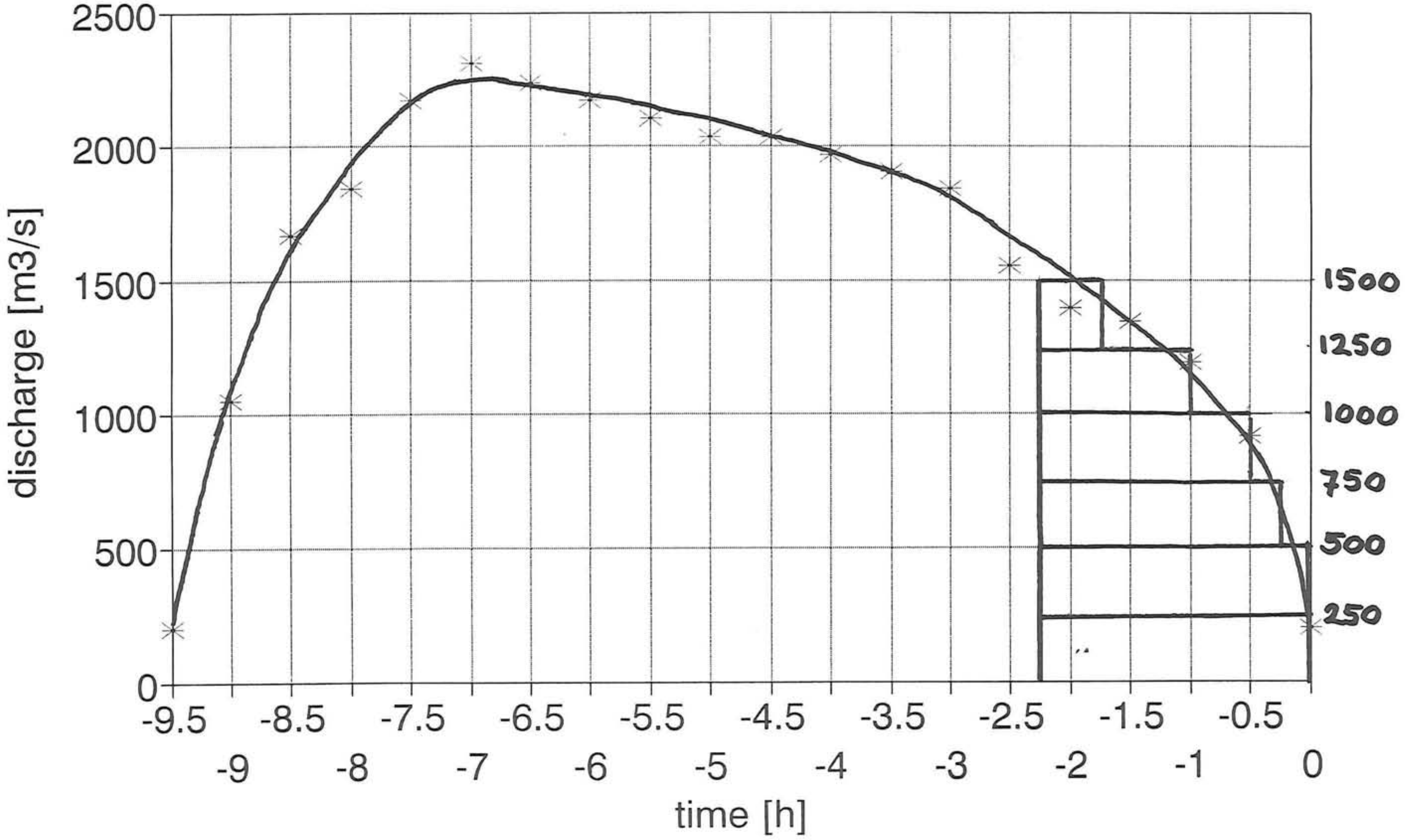
Operation scheme for Situation B


required powerplant discharge
for constant hydrostatic head
discharge [m3/s]
2500
2000
1500
1000
500
0
-9.5
-9
-8.5
-8
-7.5
-7
-6.5
-6
-5.5
-5
-4.5
-4
-3.5
-3
-2.5
-2
-1.5
-1
-0.5
0
time [h]
1100

Situation A

required powerplant discharge

for constant hydrostatic head



Situation B

Appendix D6: Calculation of Thoma cavitation factor σ

The cavitation factor of a turbine is normally determined from model tests by the turbine manufacturer. However, according to [WPDCoct'85], a 'conservative preliminary value' for σ can be calculated with the following formula:

$$\sigma = \frac{N_s^{1.6}}{28940}$$

in which
σ = Thoma cavitation factor;
N_s = specific speed on basis of power and head.

The definition of N_s is given in [Duivendijk'89]:

$$N_s = n \times P^{\frac{1}{2}} \times H^{-\frac{5}{4}}$$

in which
n = rotational speed [rev/min];
P = design power [kW]
H = design hydrostatic head [m].

In this study, instead of N_s, the specific speed factor N_q has been used. This factor is defined as (see Appendix D1):

$$N_q = n \times Q^{\frac{1}{2}} \times H^{-\frac{3}{4}}$$

in which
n = rotational speed [rev/min];
Q = design turbine discharge [m3/s];
H = design hydrostatic head [m].

The power quantity P is usually approximated as $8 \times Q \times H$, so N_s can be expressed in N_q, resulting in

$$N_s = \sqrt{8} \times N_q$$

With this expression, the Thoma cavitation factor can be expressed in N_q as well:

$$\sigma = \frac{N_q^{1.6}}{5483}$$

The value of N_q under the design hydrostatic head of 2.74 m can be calculated from the statistical expression in Appendix D1, and is 564. The cavitation formula then yields a value of σ = 4.605. This outcome is valid for all three alternatives, as N_q only depends on the hydrostatic head, according to Appendix D1.

The following formula now determines the elevation of the top of the turbine above the downstream water level:

$$H_s \leq H_b - \sigma \times H$$

in which
H_s = elevation of highest point of the turbine, relative to minimum downstream water level
H_b = atmospheric pressure [m];
H = maximum operating net hydrostatic head.

For turbines with a high specific speed, this formula usually yields negative values, which means that the turbines have to be installed below the downstream water level to prevent cavitation. With H_b = 9.8 m, σ = 4.605 and H = 2.74, the result is an installation level H_s of -2.8 m relative to the downstream water level. For design, a low downstream water level of SOB 0 will be assumed (see Figures C3.1a and C3.1b). Again, this outcome is valid for all three alternatives.

This is minimal depth of the top of the turbine; the centreline then is at a level of
$z_c = 2.8 + 0.5 \times D_R$ [m - SOB]

Again according to [WPDCoct'85], a representative value of the lowest point for excavation in case of bulb turbines is 1.5 times D_R from the centreline of the turbine; this yields a maximum excavation depth of:
$z_x = 2.8 + 2 \times D_R$ [m - SOB].

For comparison's sake, the same procedure has been followed for a vertical Kaplan turbine with the same hydrostatic head. The experimental relation between σ and N_q is derived from [WPDCmay'88], the relation between excavation depth and D_R from [WPDCoct'85]. The result is:
$\sigma = 3.711 \rightarrow$
$z_c = 0.35 + 0.5 \times D_R$ [m - SOB] $\rightarrow$
$z_x = 0.35 + 2.5 \times D_R$ [m - SOB].

For Kaplan turbines with the same hydrostatic head and discharge as the three alternatives of Section D2.4, the following diameters are found (with the values for bulb turbines for the same situation in brackets):
Q = 275 m^3/s : D_R = 6.95 m (6.07) $\rightarrow$ z_x = 17.7 m (15);
Q = 183.3 m^3/s : D_R = 5.67 m (4.96) $\rightarrow$ z_x = 14.5 m (12.7);
Q = 137.5 m^3/s : D_R = 4.91 m (4.30) $\rightarrow$ z_x = 12.6 m (11.4).

Appendix D7: Calculation of water level for 1100 m^3/s Regulator discharge

As stated in Section D3.4.2.2, according to [IECo'83] a water level of SOB +2.75 m would be needed in the approach channel of the Regulator for an outflow of 1100 m^3/s.

In [Henderson'66], the following formula is given for a weir that has a crest broad enough to maintain hydrostatic pressure distribution in the flow across it. This is the formula for critical flow:

$$q=\frac{2}{3}H\sqrt{\frac{2}{3}gh}$$

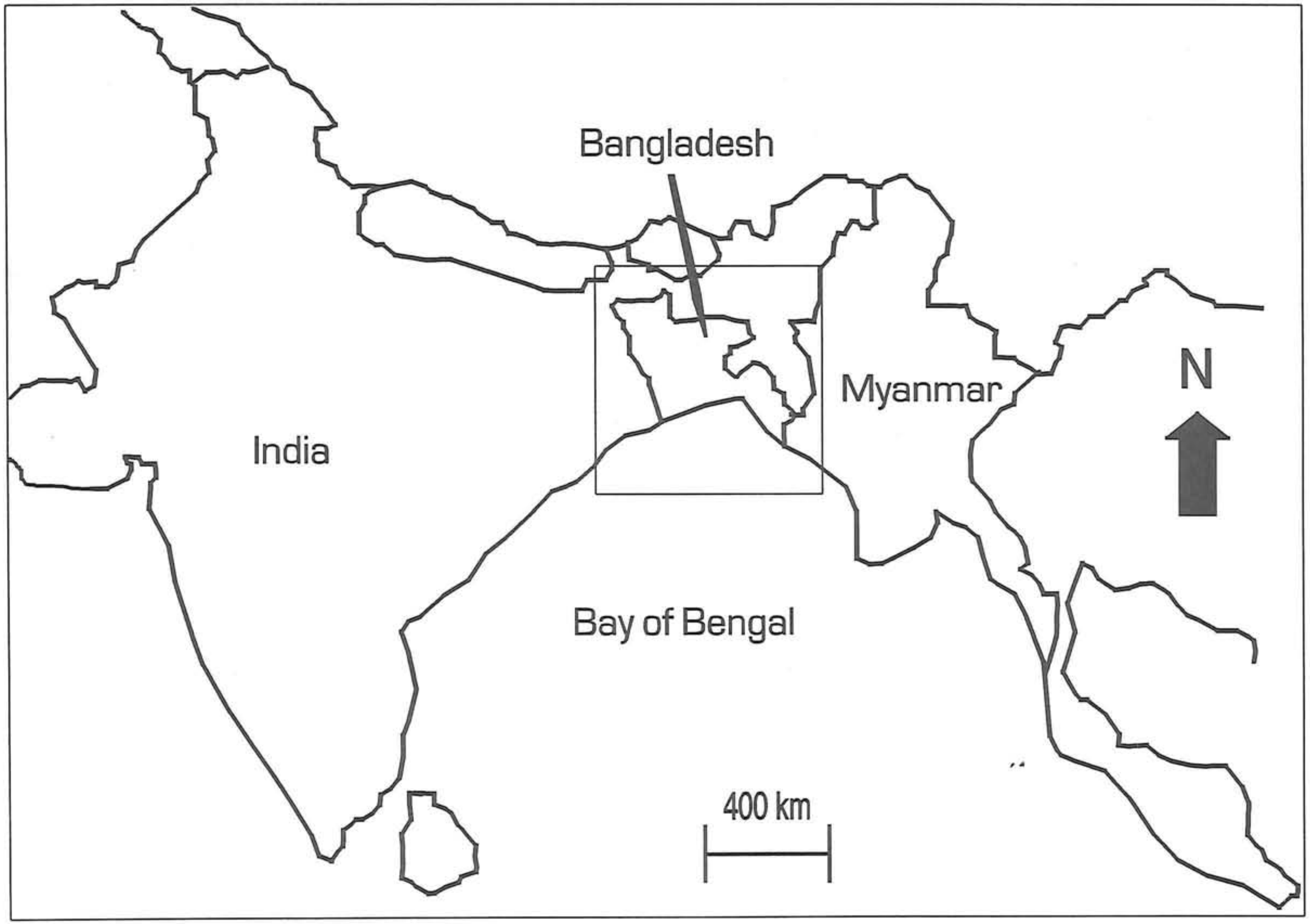
in which q is the discharge per unit of width and H is the water level above the sill of the weir, and therefore in this case the elevation above SOB.

For the determination of H, a representative value of q must be found. If the total discharge through the Regulator is 1100 m^3/s, the discharge through one of the 40 gates is 27.5 m^3/s. The width of the gates is 3.66 m (12'), so q would then be about 7.5 m^2/s. If this is correct, H can be calculated to be SOB +2.7 m.

It is very questionable if the given formula can be used for this situation: the Regulator structure (40 gates separated by 0.91 m wide piers) is certain to obstruct the flow to some extent. This would mean that an even higher level than SOB +2.7 m would be needed for a discharge of 1100 m^3/s.

More detailed calculations will not be performed in this respect, as the conclusion of the calculation is clear enough.

Figures



scale: 1 : 20 000 000
(1 cm stands for 200 km)

Figure A2.1: Topographical map of the Indian subcontinent

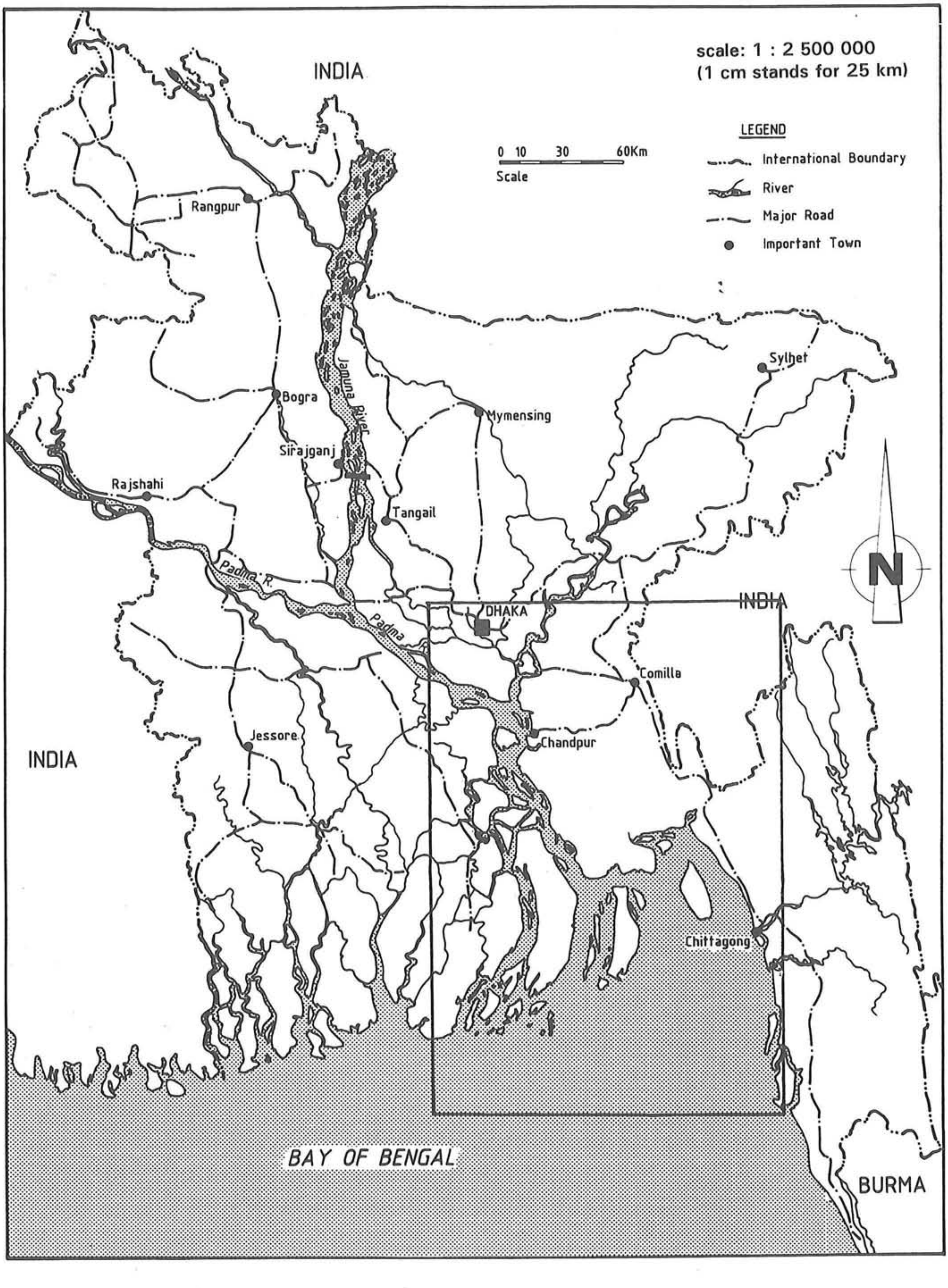
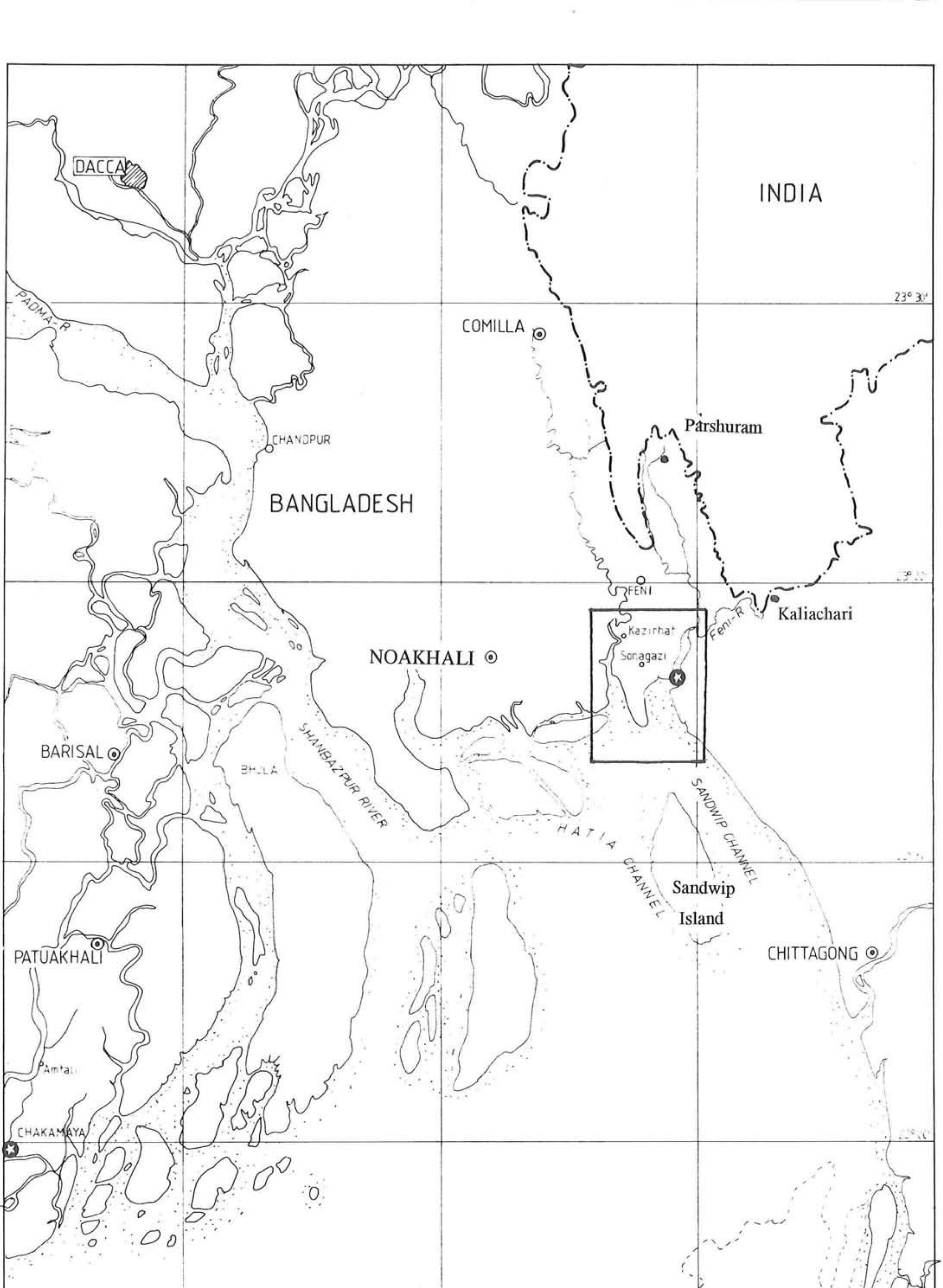
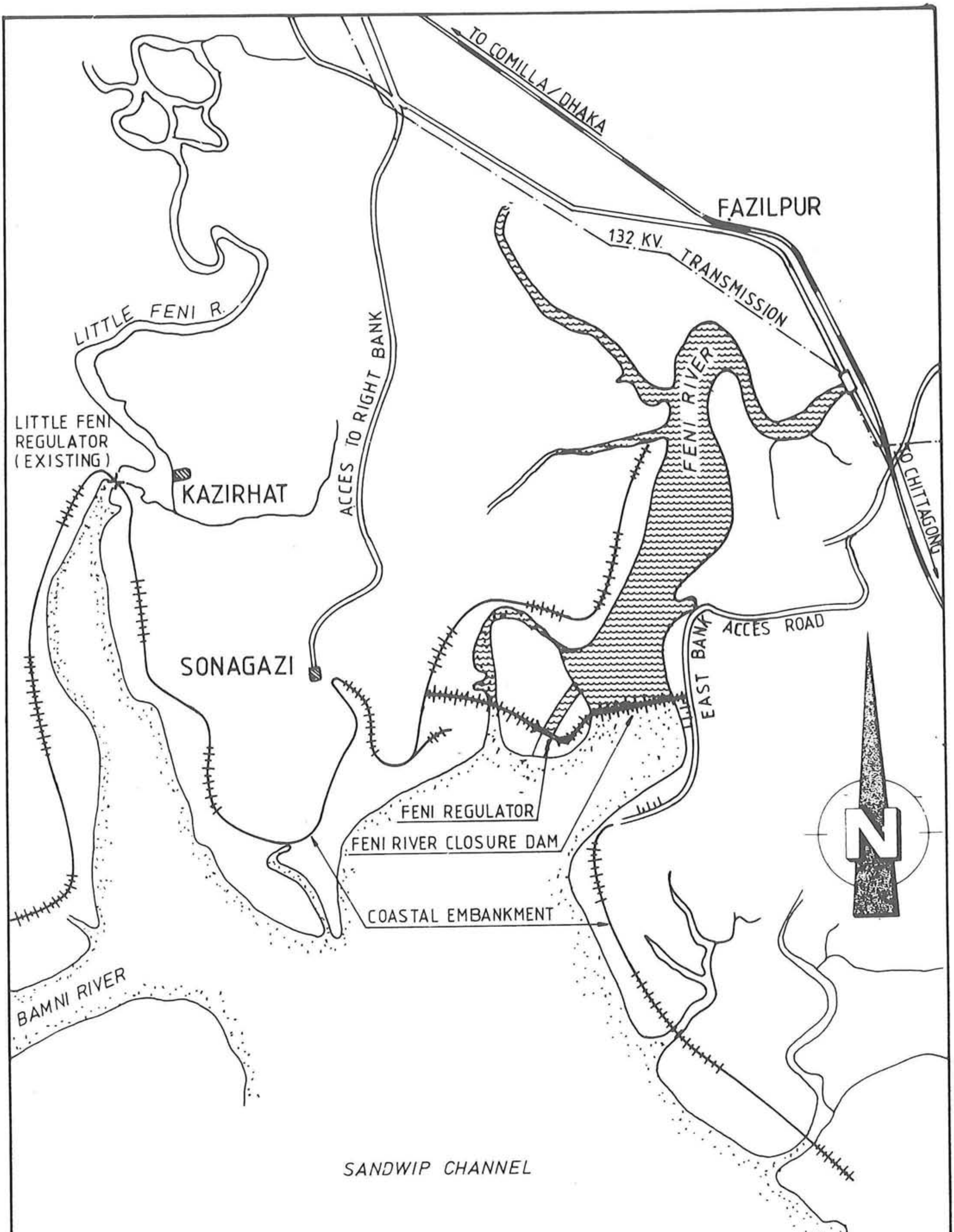


Figure A2.2: Topographical map of Bangladesh



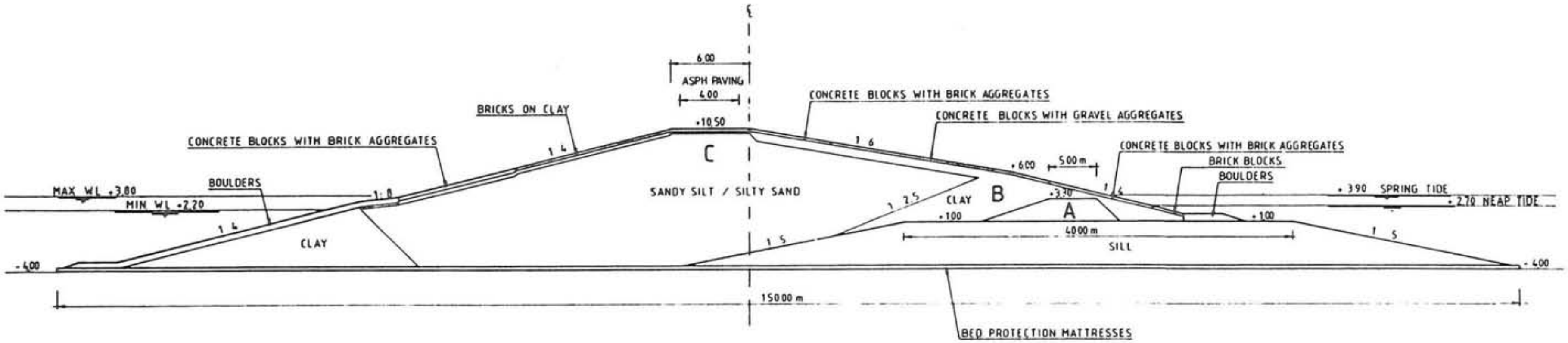
scale: 1 : 1 000 000
(1 cm stands for 10 km)

Figure A2.3: Topographical map of south-east Bangladesh



scale: 1 : 90 000
(1 cm stands for 900 m)

Figure A2.4: Topographical map of the project area



Cross section Feni River Closure Dam

A : Neap Tide Dam (NTD)
B : Winter Spring Tide Dam
C : Final Profile

Cross section of the main dam, levels above SOB

Figure A3.1

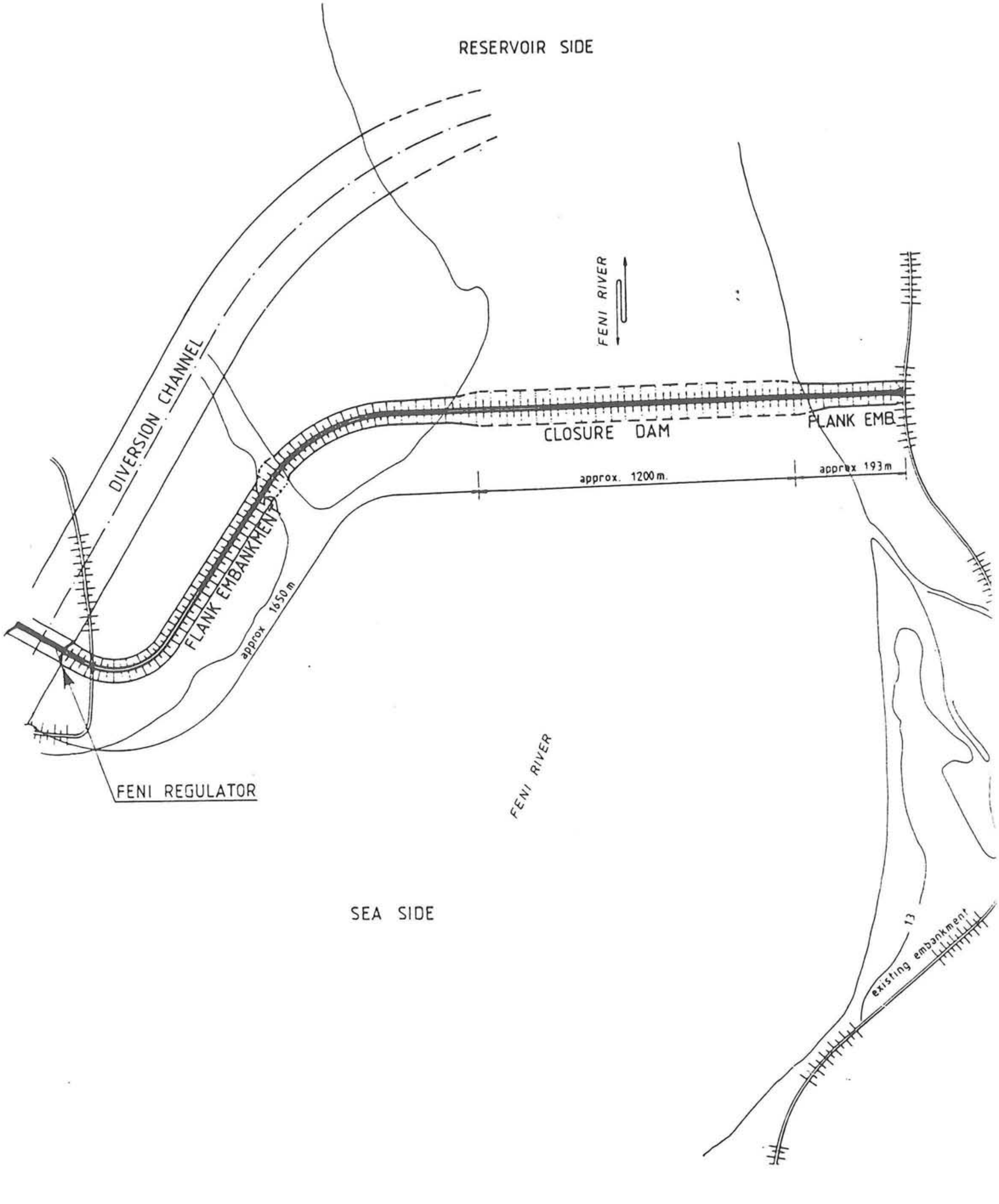


Figure A3.2: Plan of Feni River Closure Dam

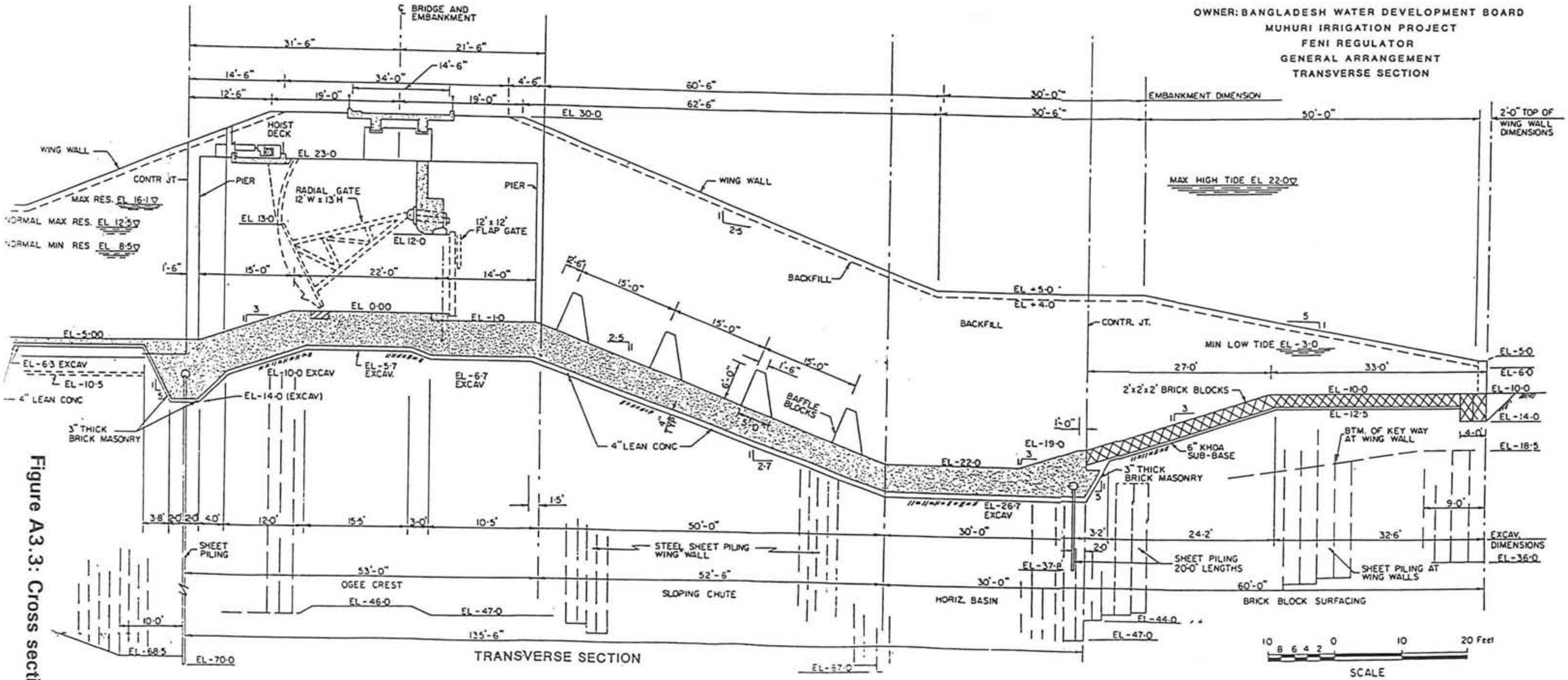
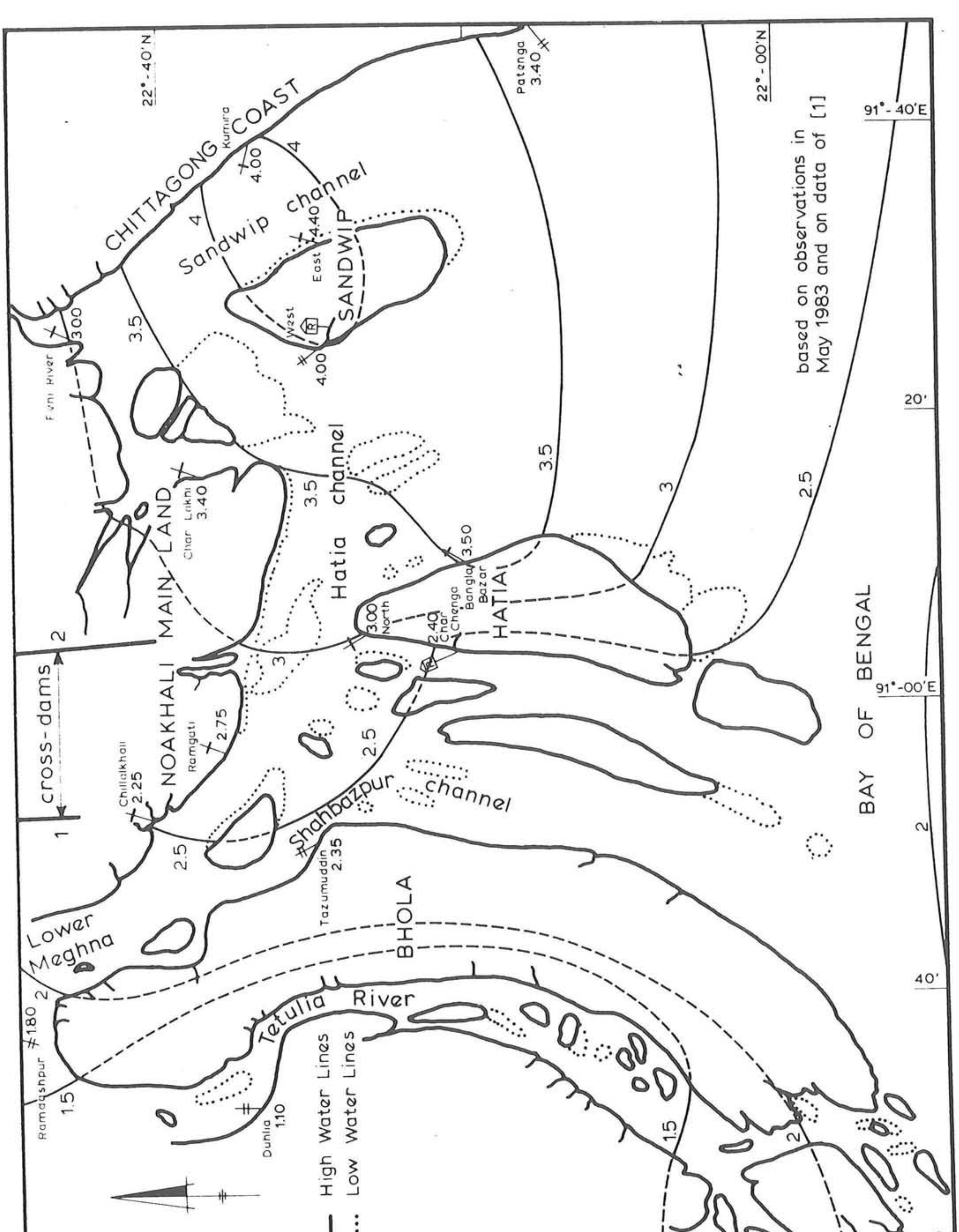


Figure A3.3: Cross section of Feni Regulator



TIDAL PROPAGATION APPROXIMATE MEAN TIDAL RANGE PATTERN	SCALE 1 : 625,000
LAND RECLAMATION PROJECT	Figure A4.1

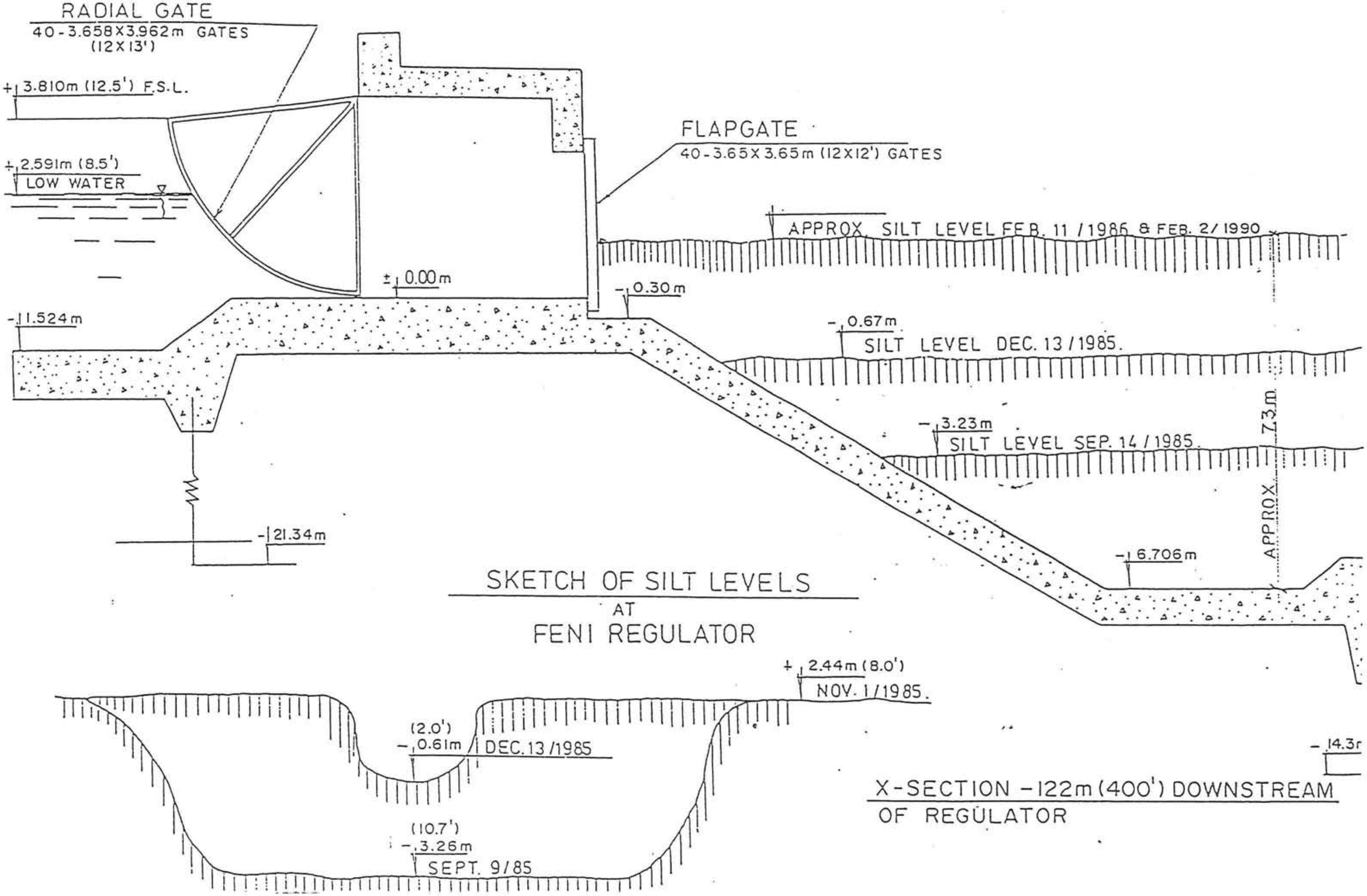


Figure A4.2: Siltation downstream from Feni Regulator

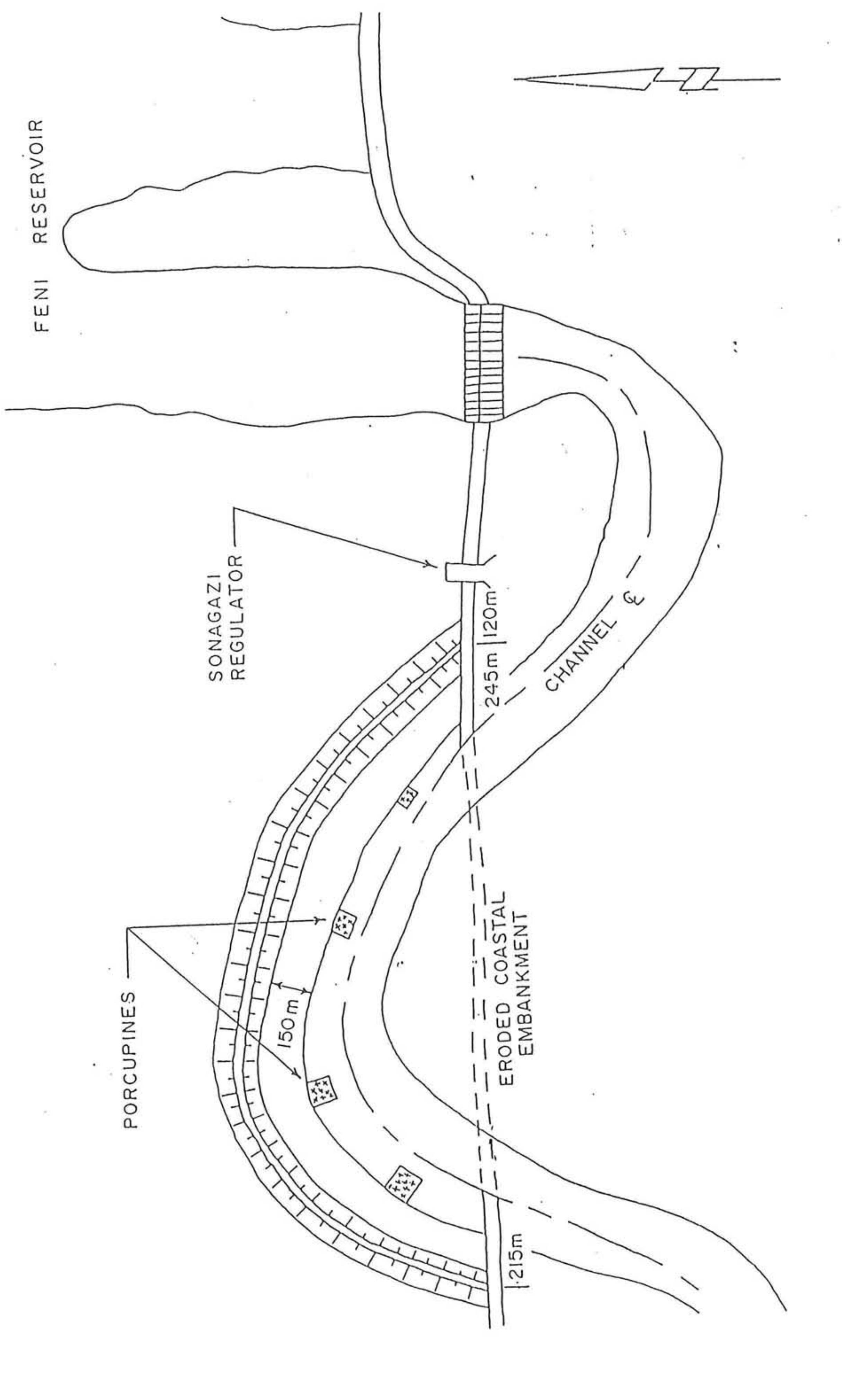


Figure A4.3: Meandering of Regulator discharge channel

Net inflow into Feni reservoir
(components)

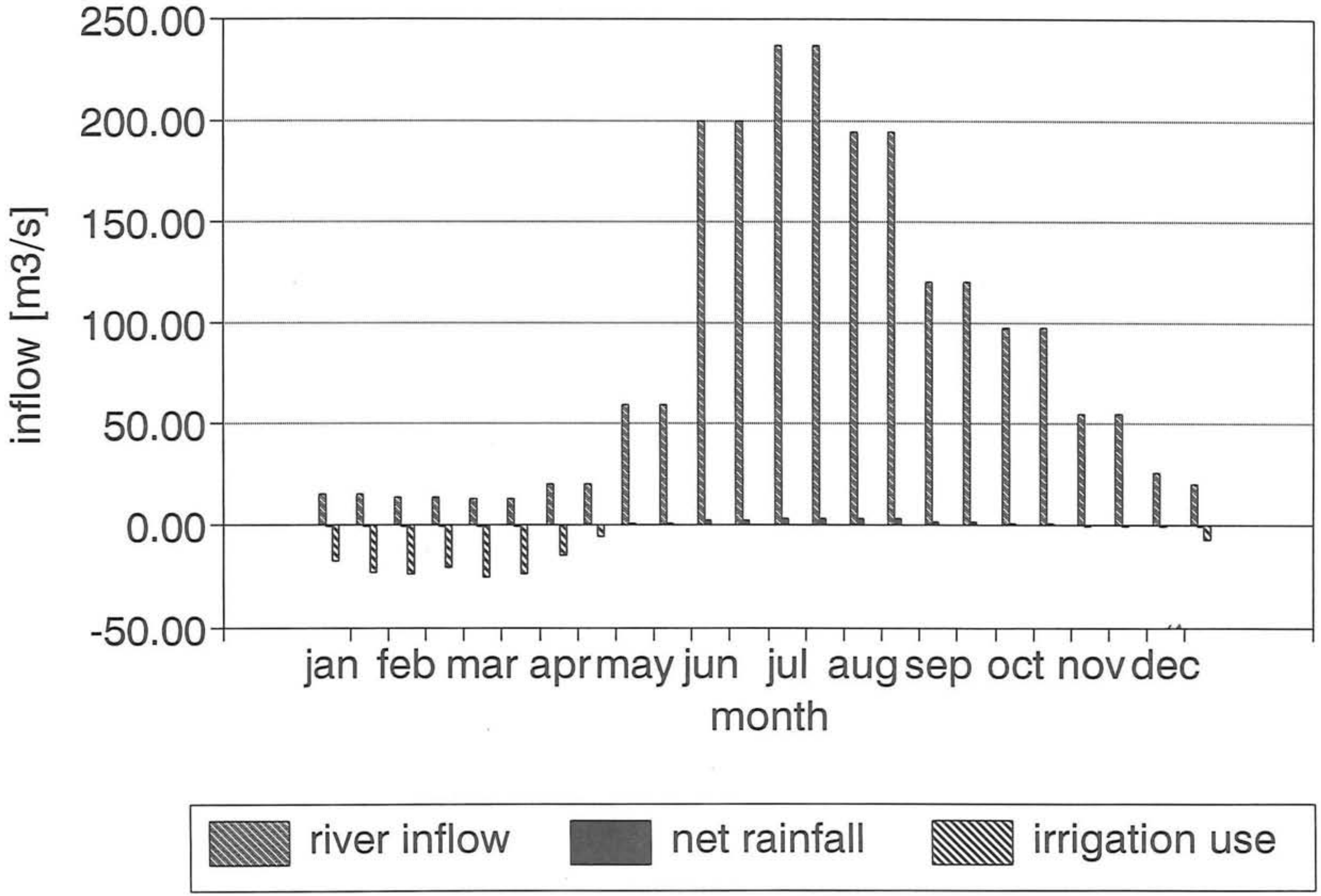


Figure C2.1

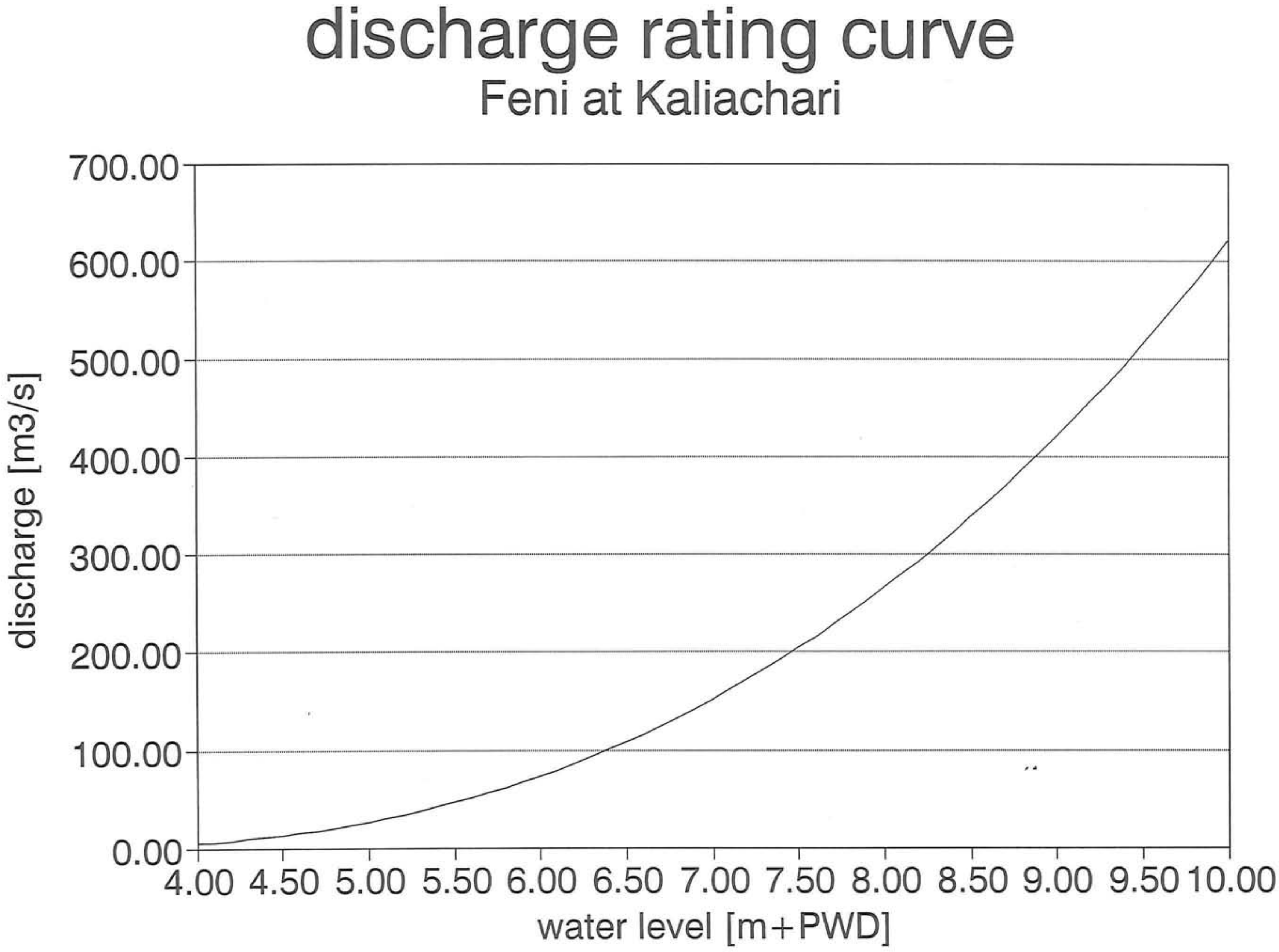

discharge rating curve
Feni at Kaliachari
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
discharge [m3/s]
4.00 4.50 5.00 5.50 6.00 6.50 7.00 7.50 8.00 8.50 9.00 9.50 10.00
water level [m+PWD]

Figure C2.2a

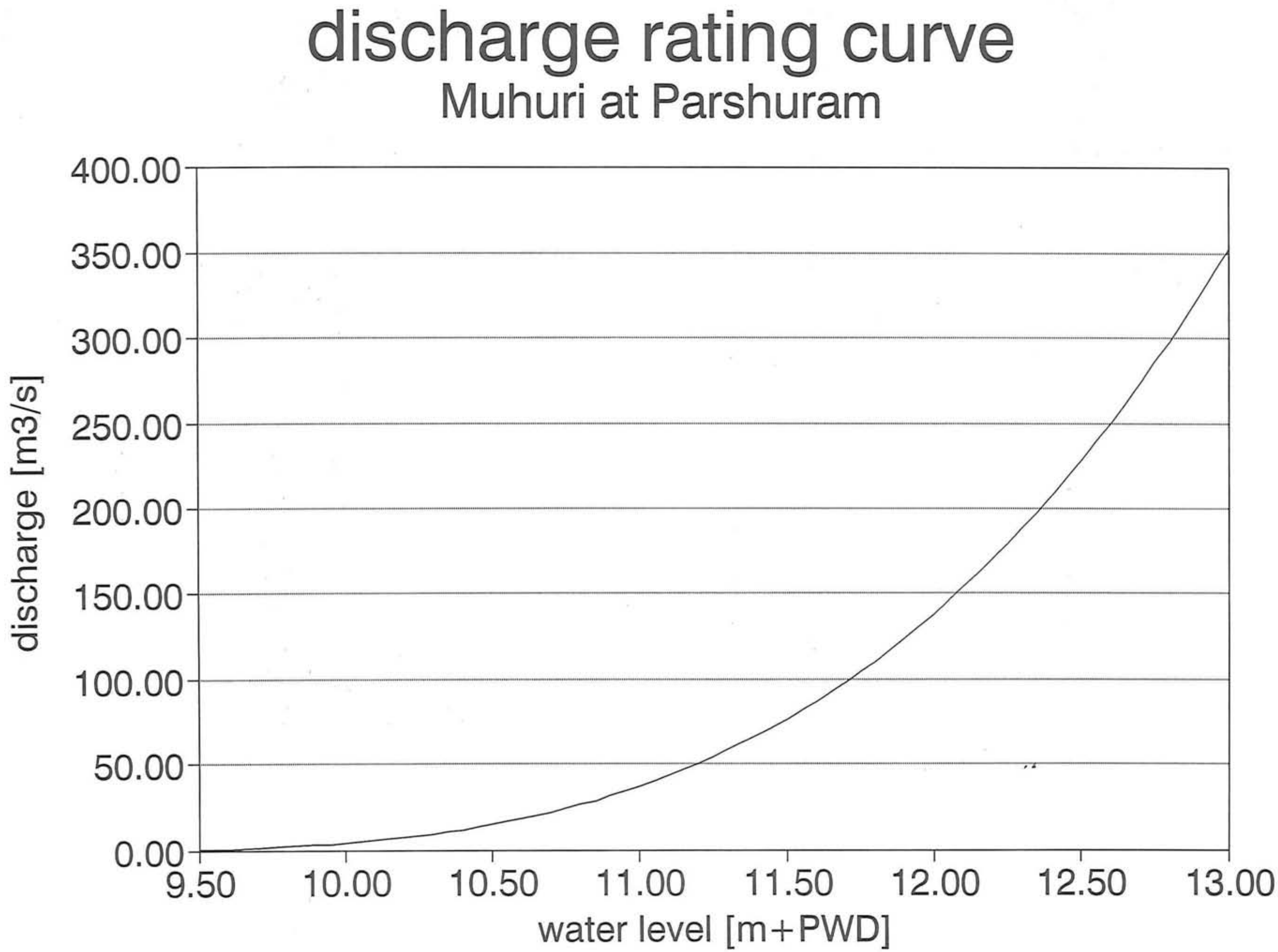

discharge rating curve
Muhuri at Parshuram
discharge [m3/s]
400.00
350.00
300.00
250.00
200.00
150.00
100.00
50.00
0.00
9.50
10.00
10.50
11.00
11.50
12.00
12.50
13.00
water level [m+PWD]

Figure C2.2b

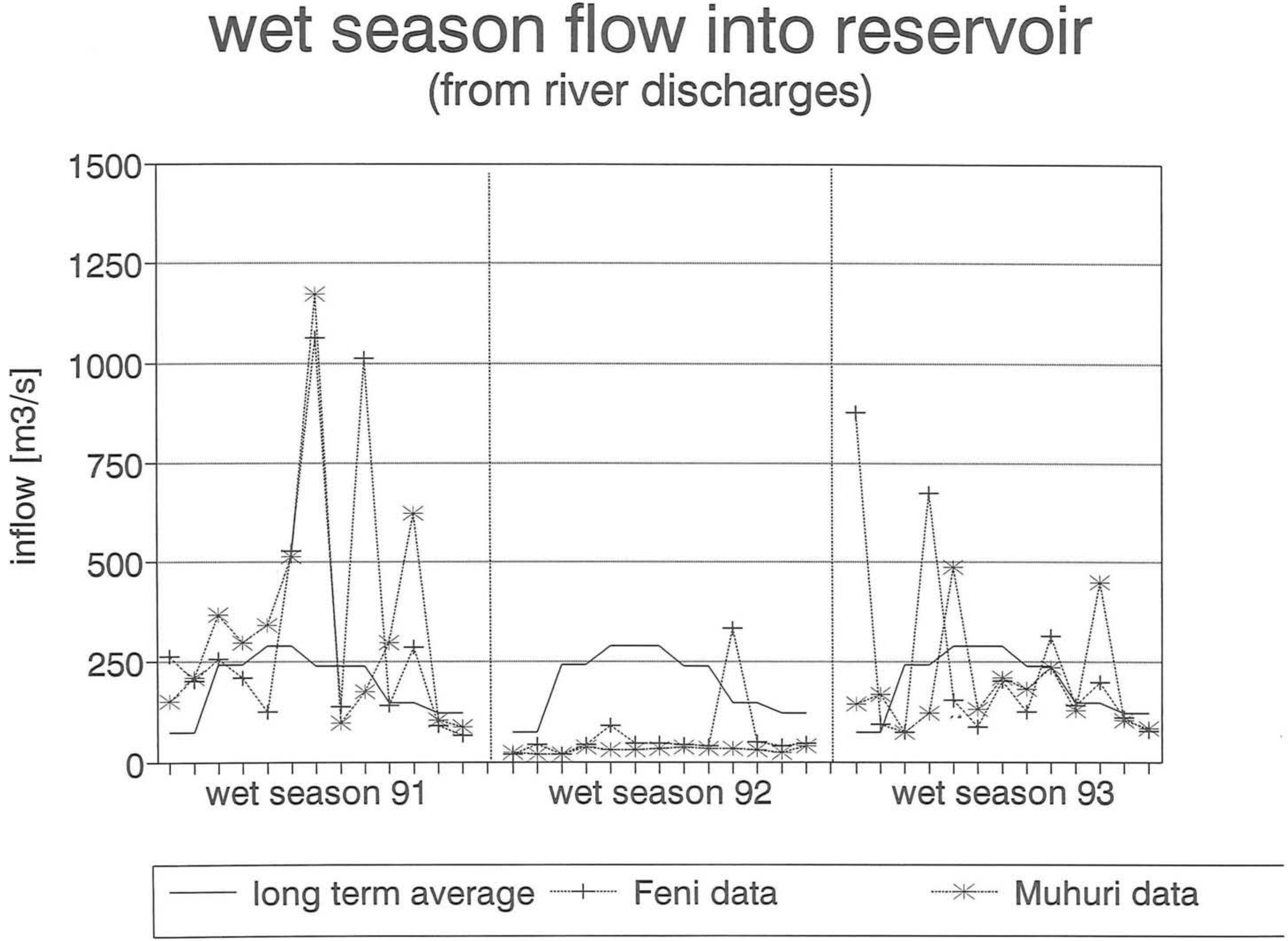

wet season flow into reservoir
(from river discharges)
inflow [m3/s]
1500
1250
1000
750
500
250
0
wet season 91
wet season 92
wet season 93
long term average
Feni data
Muhuri data

Figure C2.3

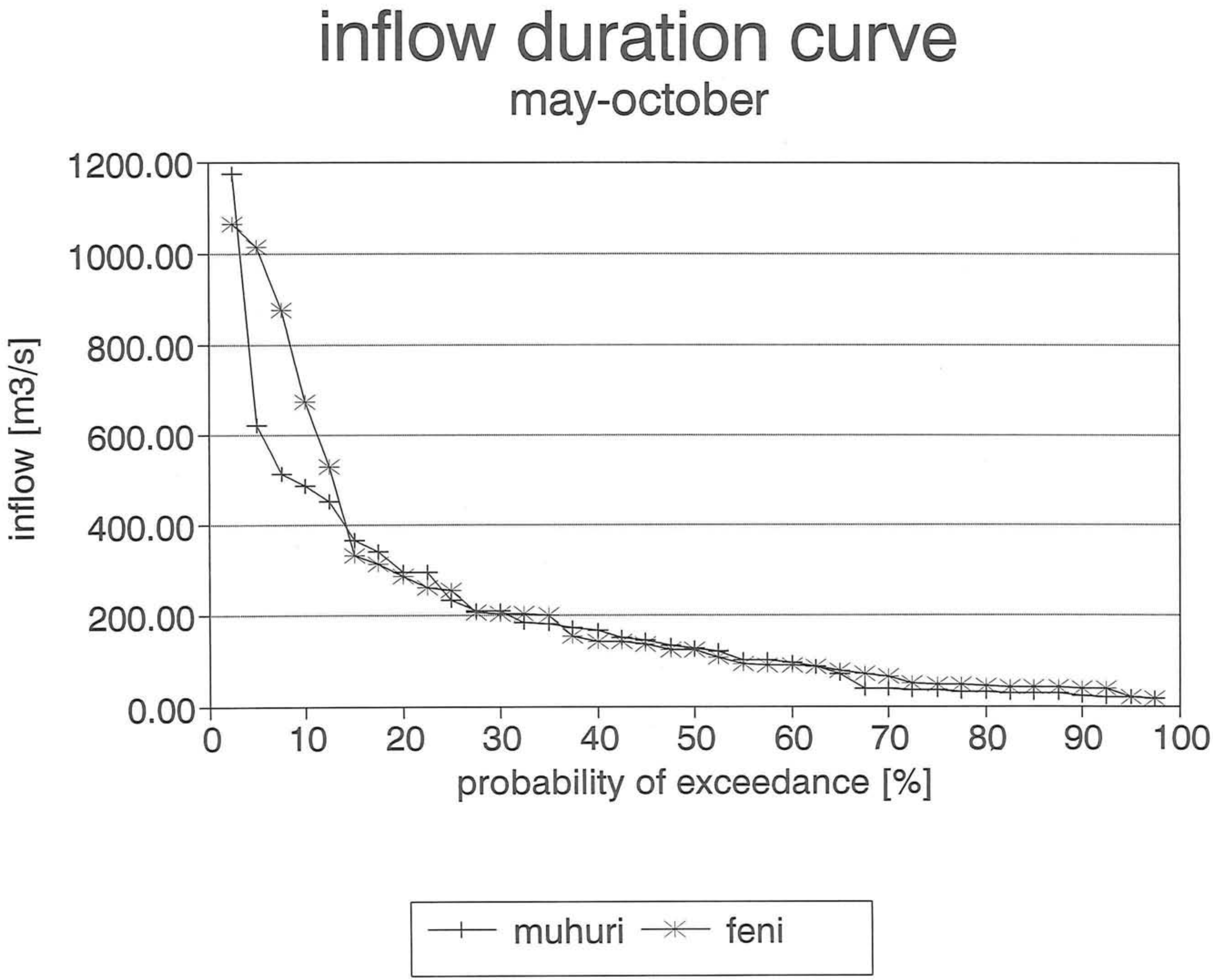

inflow duration curve
may-october
inflow [m3/s]
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
0
10
20
30
40
50
60
70
80
90
100
probability of exceedance [%]
muhuri
feni

Figure C2.4

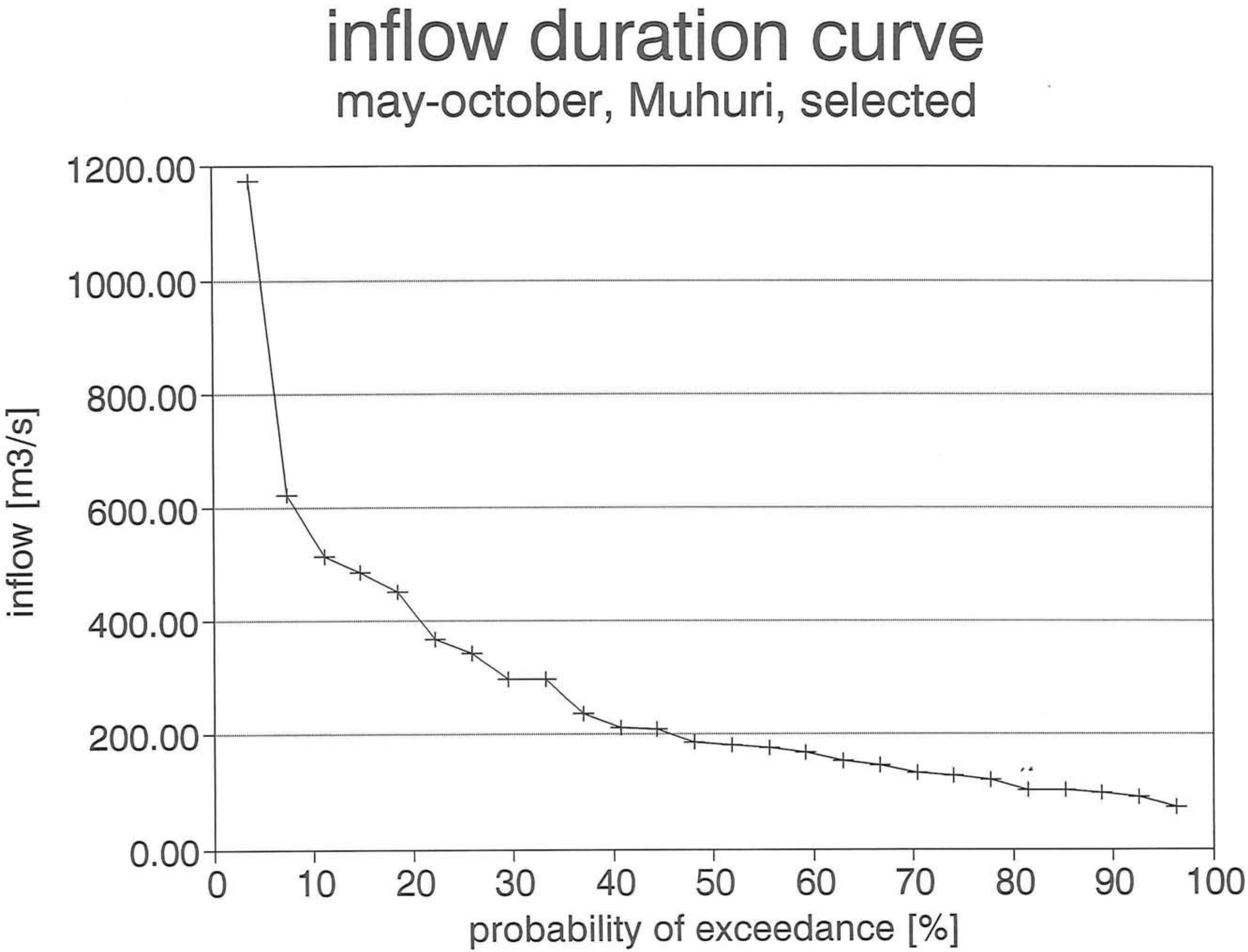
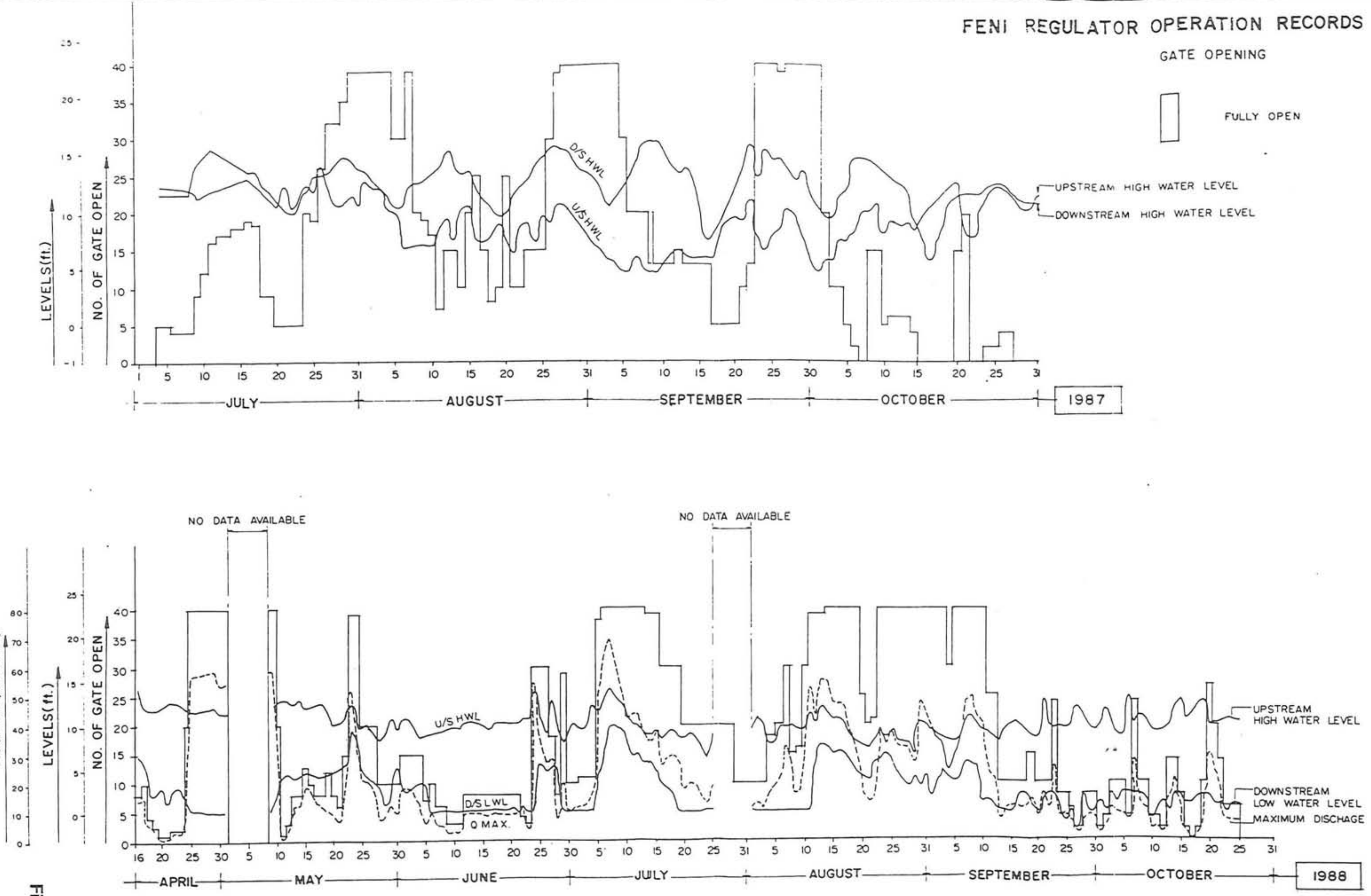
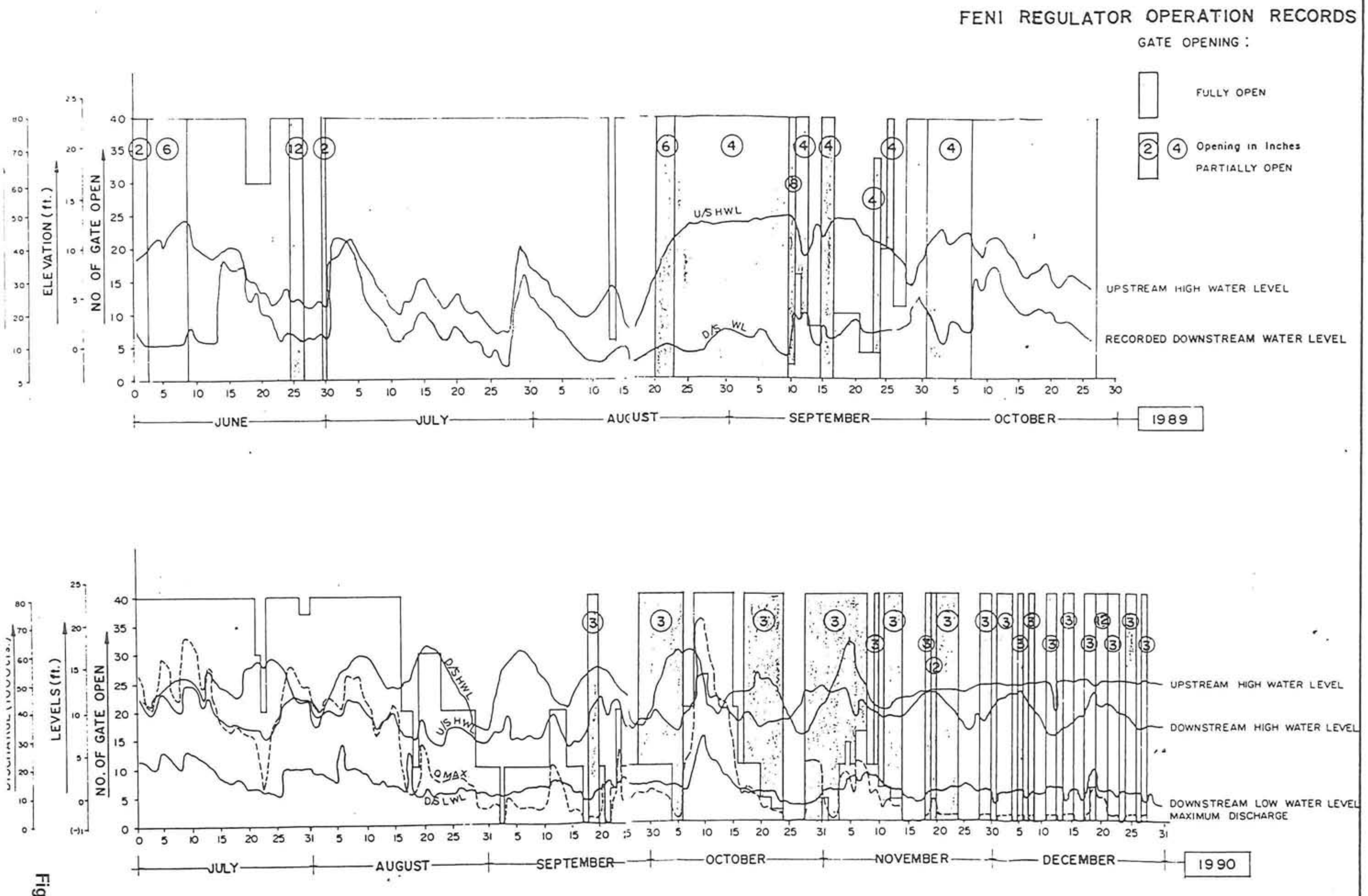

inflow duration curve
may-october, Muhuri, selected
inflow [m3/s]
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
0
10
20
30
40
50
60
70
80
90
100
probability of exceedance [%]

Figure C2.5



Water levels at Feni Regulator, '87-'88

Figure C3.1a



Water levels at Feni Regulator, '89-'90

Figure C3.1b

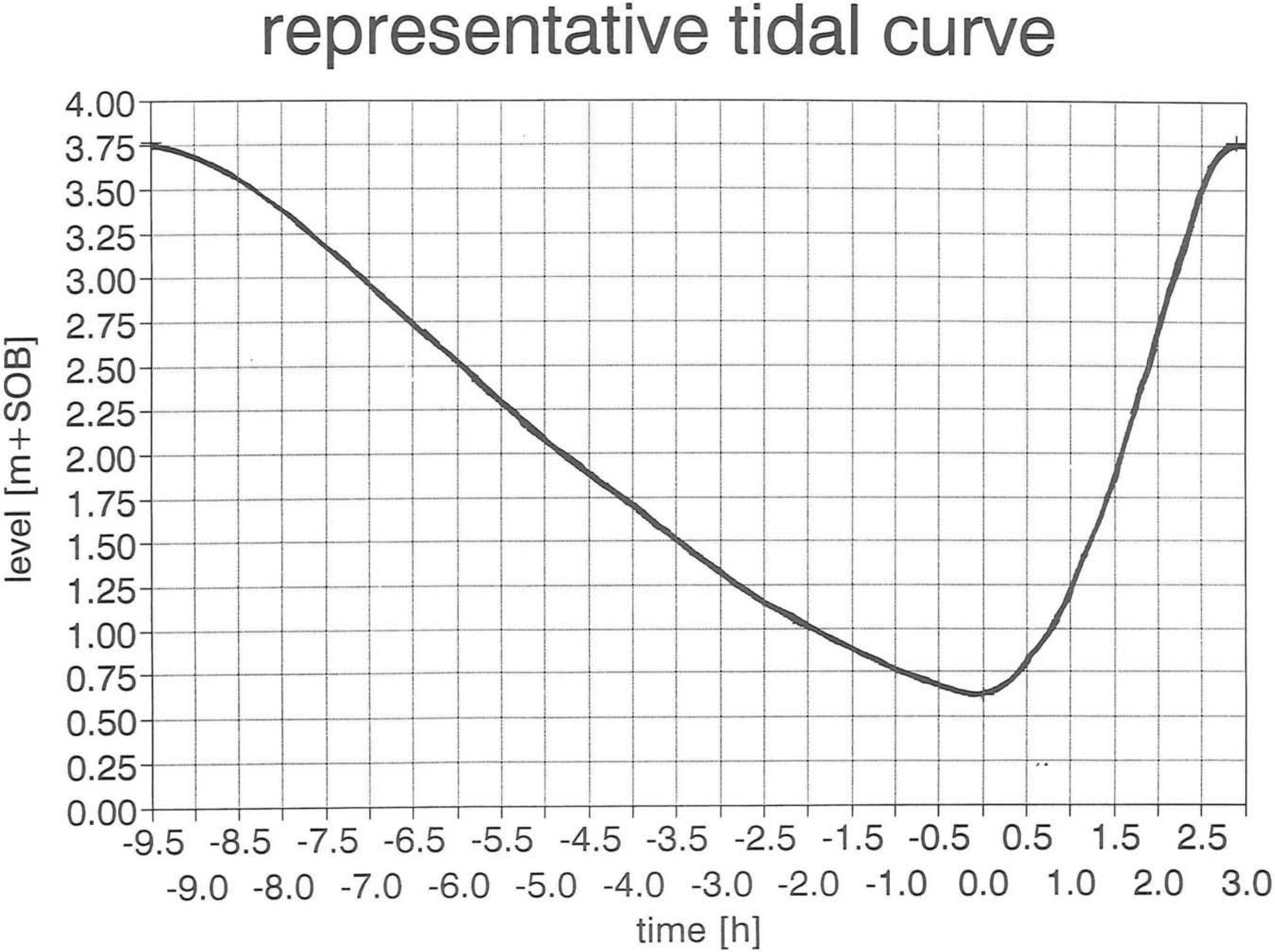
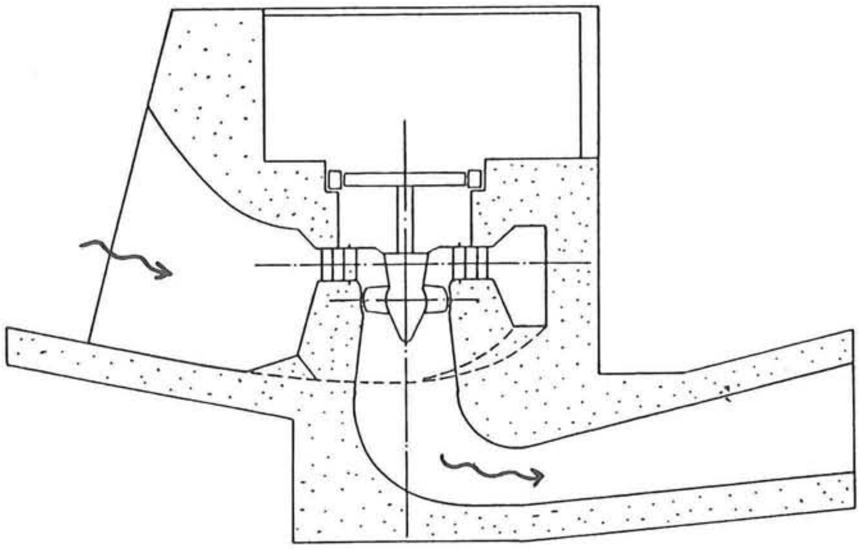
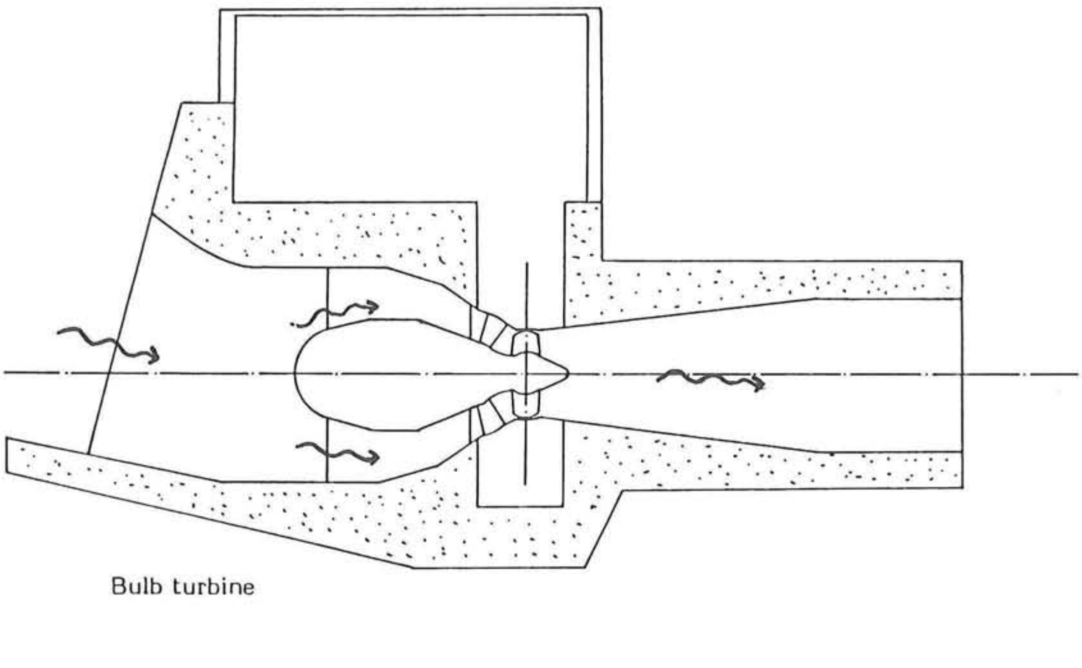

representative tidal curve
level [m+SOB]
4.00
3.75
3.50
3.25
3.00
2.75
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
-9.5 -8.5 -7.5 -6.5 -5.5 -4.5 -3.5 -2.5 -1.5 -0.5 0.5 1.5 2.5
-9.0 -8.0 -7.0 -6.0 -5.0 -4.0 -3.0 -2.0 -1.0 0.0 1.0 2.0 3.0
time [h]

Figure C3.2



Kaplan turbine



Bulb turbine

Figure D2.1: Axial turbine types

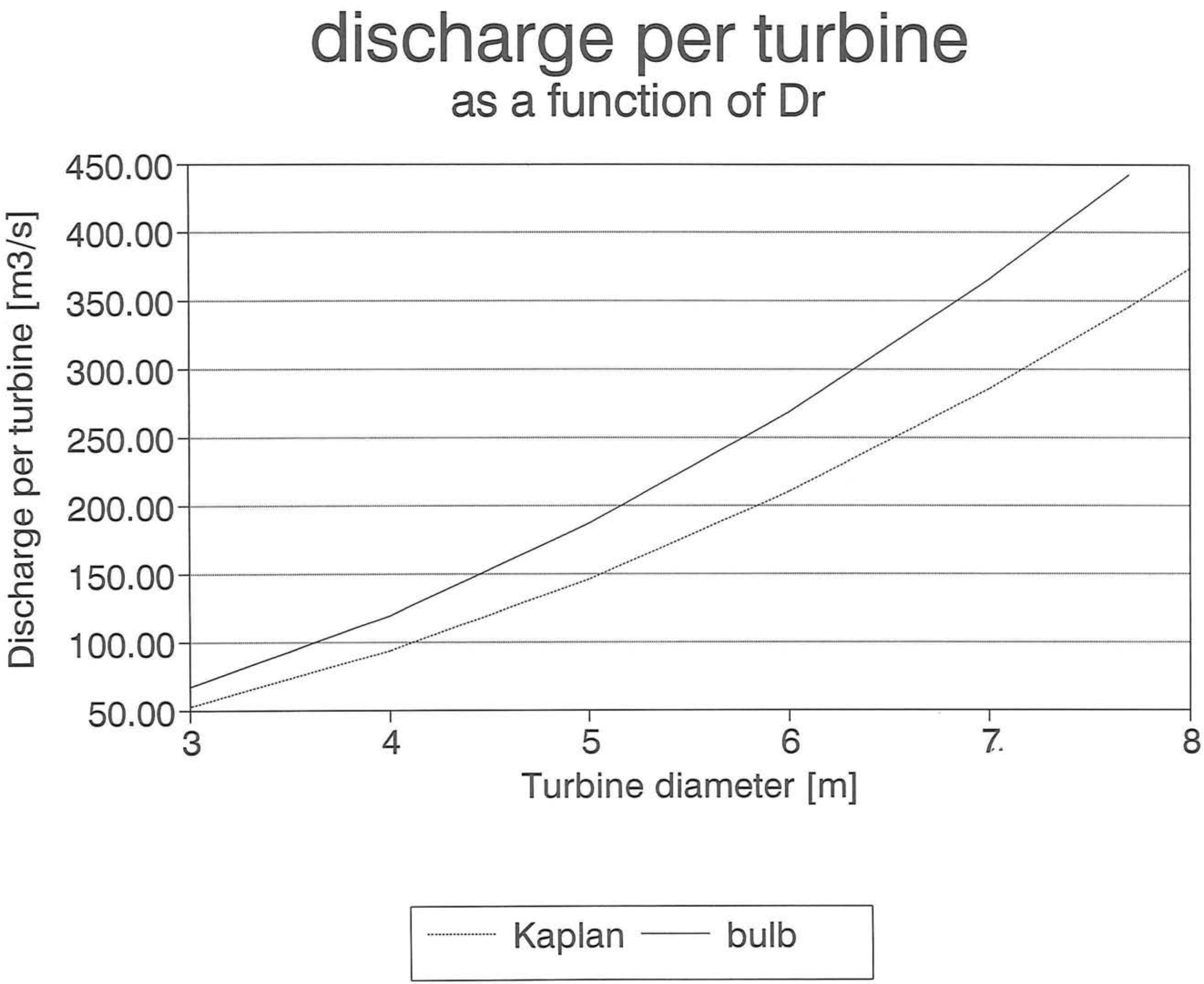

discharge per turbine
as a function of Dr
Discharge per turbine [m3/s]
450.00
400.00
350.00
300.00
250.00
200.00
150.00
100.00
50.00
3
4
5
6
7
8
Turbine diameter [m]
Kaplan
bulb

Figure D2.2

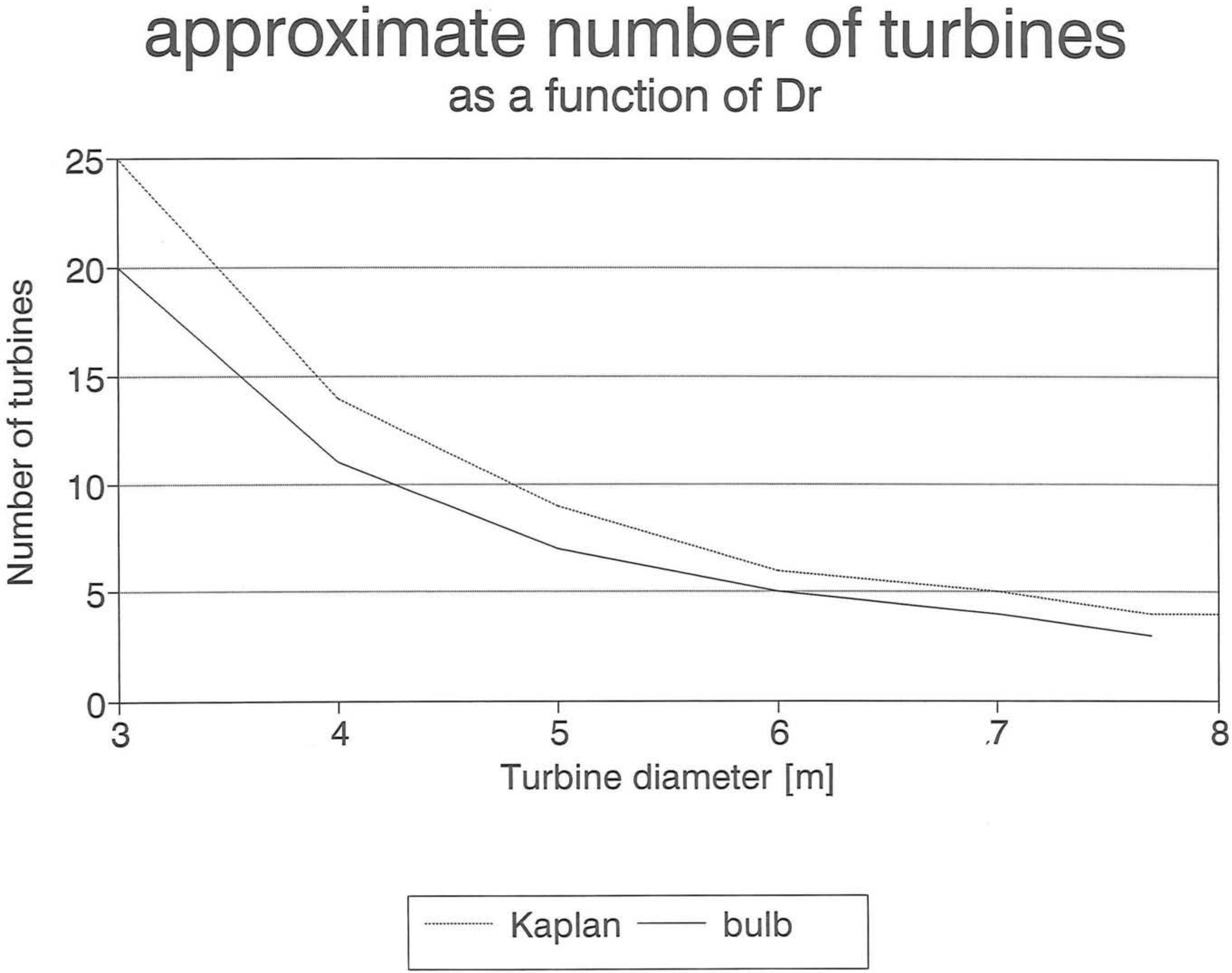

approximate number of turbines
as a function of Dr
Number of turbines
25
20
15
10
5
0
3
4
5
6
.7
8
Turbine diameter [m]
Kaplan
bulb

Figure D2.3

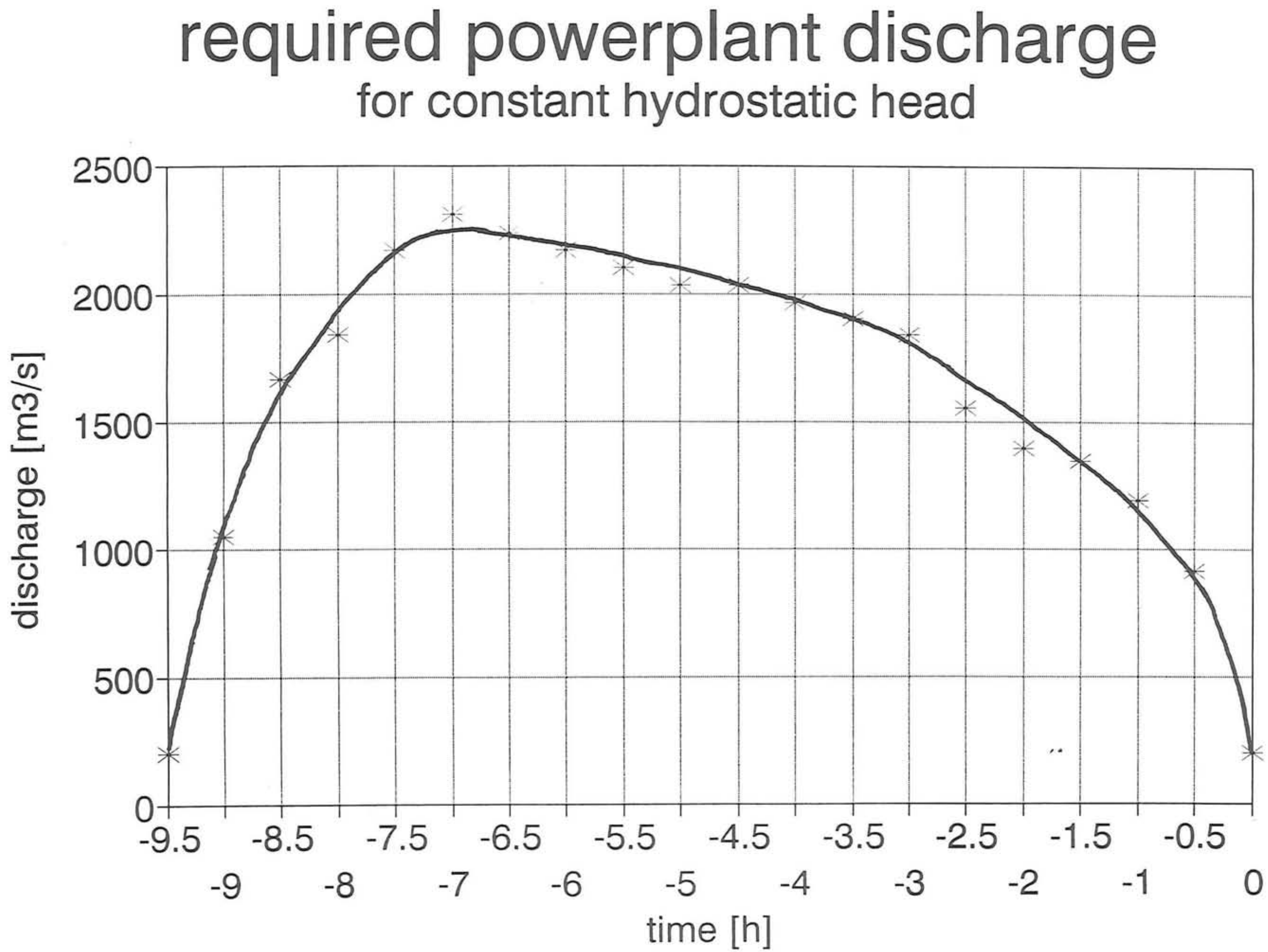

required powerplant discharge
for constant hydrostatic head
discharge [m3/s]
2500
2000
1500
1000
500
0
-9.5
-9
-8.5
-8
-7.5
-7
-6.5
-6
-5.5
-5
-4.5
-4
-3.5
-3
-2.5
-2
-1.5
-1
-0.5
0
time [h]

Figure D2.4

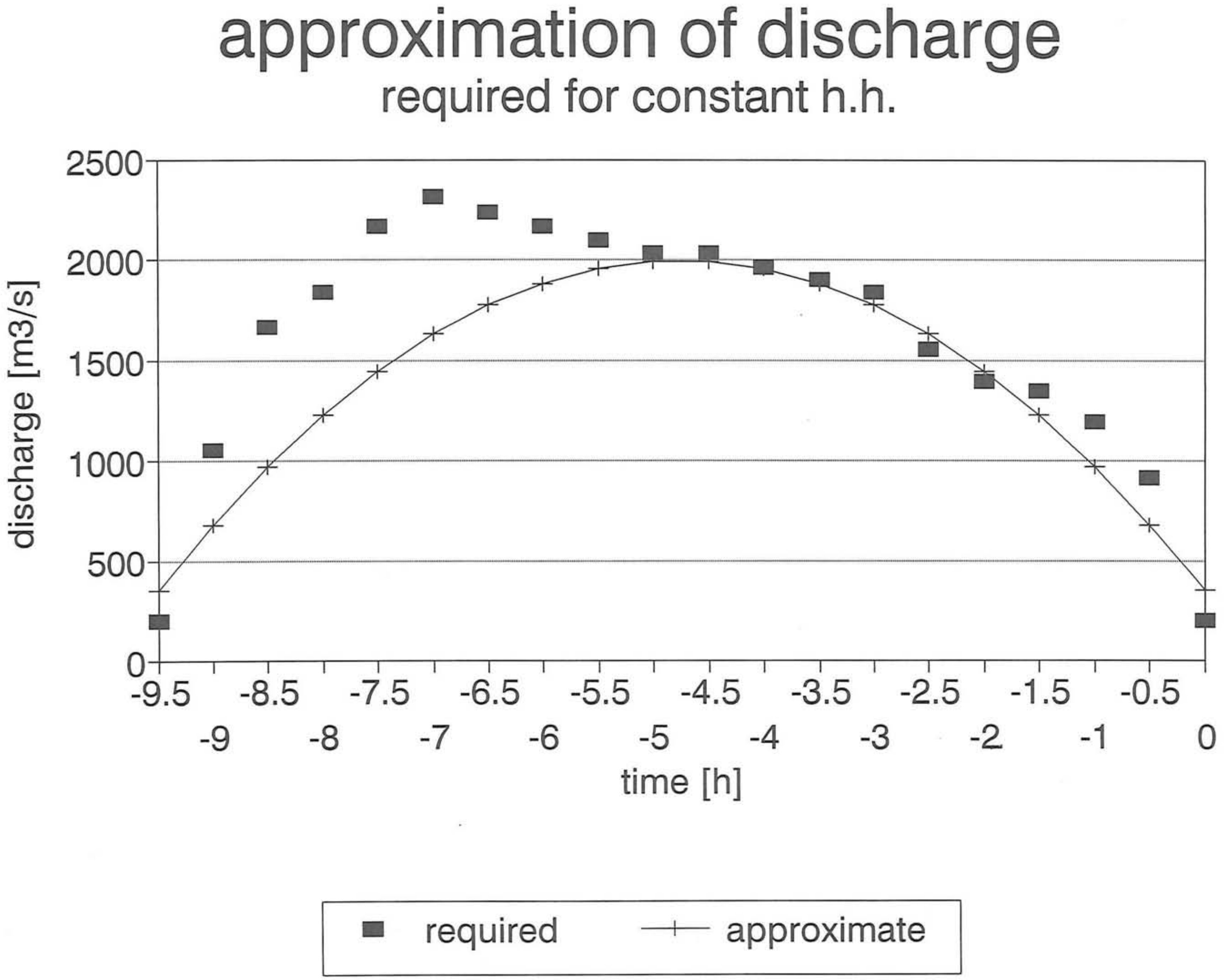

approximation of discharge
required for constant h.h.
2500
2000
1500
1000
500
0
discharge [m3/s]
-9.5
-9
-8.5
-8
-7.5
-7
-6.5
-6
-5.5
-5
-4.5
-4
-3.5
-3
-2.5
-2
-1.5
-1
-0.5
0
time [h]
required
approximate

Figure D2.5

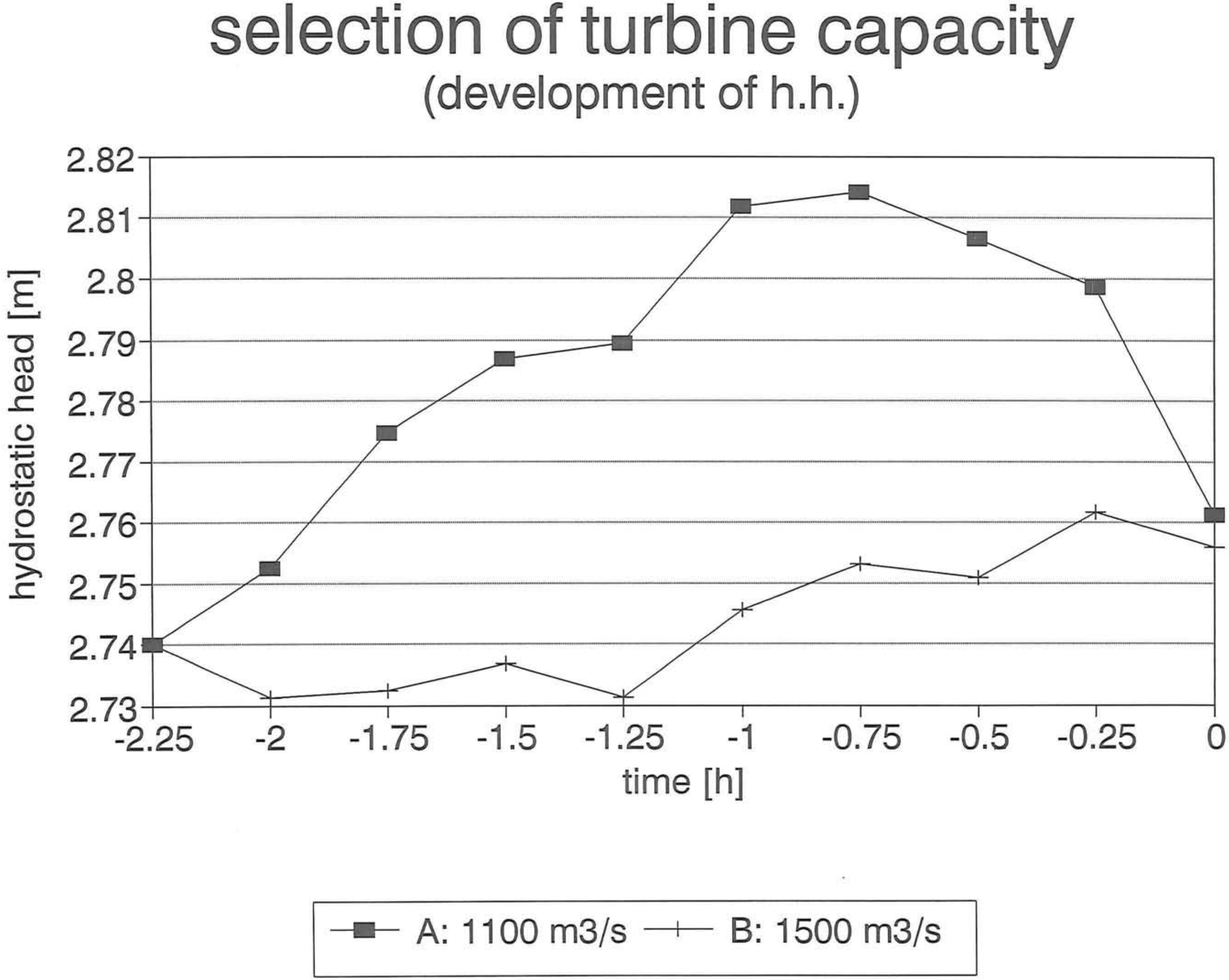
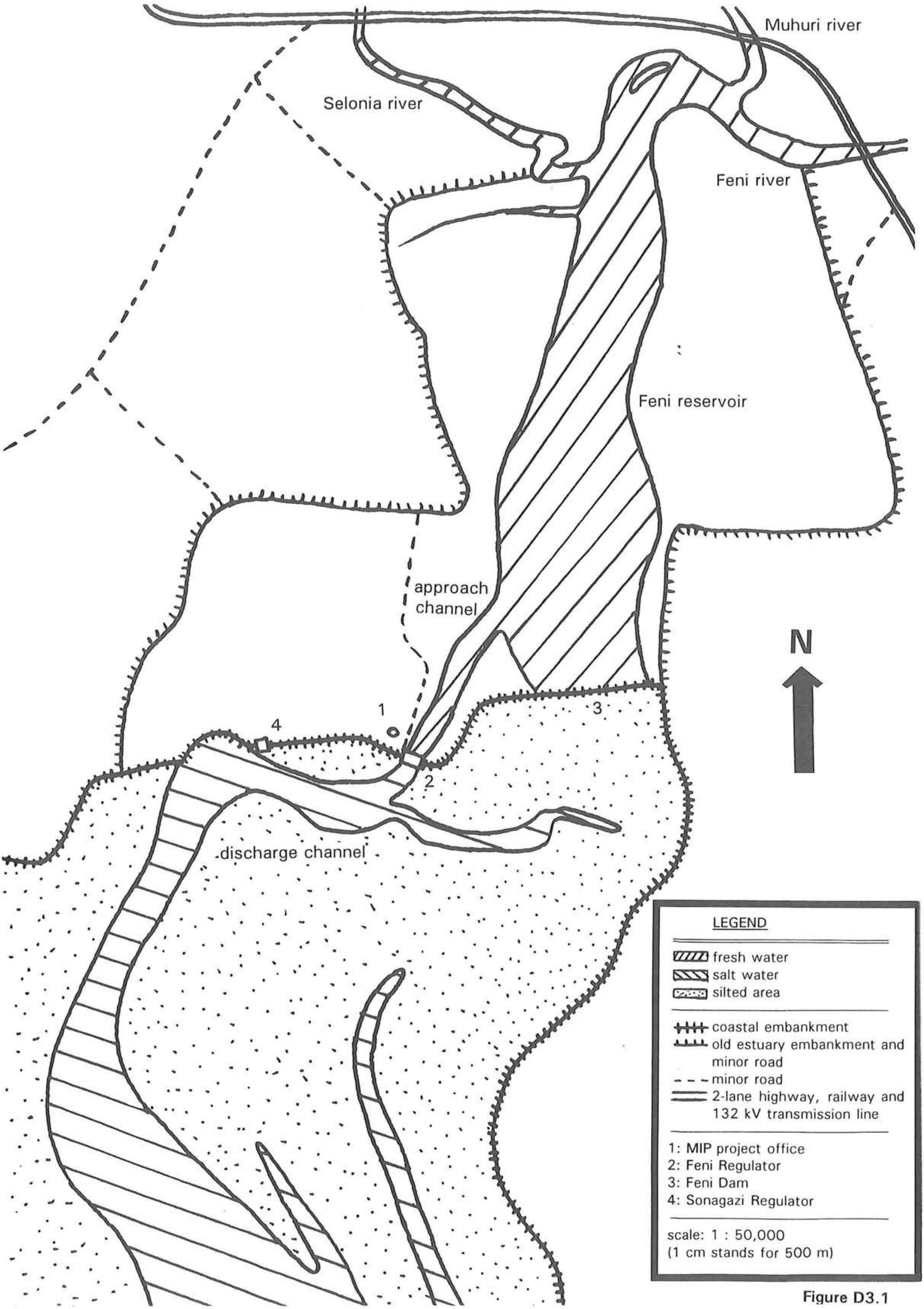

selection of turbine capacity
(development of h.h.)
2.82
2.81
2.8
2.79
2.78
2.77
2.76
2.75
2.74
2.73
hydrostatic head [m]
-2.25
-2
-1.75
-1.5
-1.25
-1
-0.75
-0.5
-0.25
0
time [h]
A: 1100 m3/s
B: 1500 m3/s

Figure D2.6



Muhuri river
Selonia river
Feni river
Feni reservoir
approach channel
discharge channel
N
1
2
3
4
LEGEND
fresh water
salt water
silted area
coastal embankment
old estuary embankment and minor road
minor road
2-lane highway, railway and 132 kV transmission line
1: MIP project office
2: Feni Regulator
3: Feni Dam
4: Sonagazi Regulator
scale: 1 : 50,000
(1 cm stands for 500 m)

Figure D3.1