BANGLADESH WATER DEVELOPMENT BOARD

FENI RIVER CLOSURE DAM
( Part of Muhuri Irrigation Project )

FINAL DESIGN REPORT

SEPTEMBER 1983

MIP
DESIGN CELL
DHAKA

HASKONING
Royal Dutch Consulting Engineers and Architects
Nymegen, Netherlands
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SUMMARY AND CONCLUSIONS

1. The final design of Feni River Closure Dam is predominantly governed by hydraulic and geo-technical considerations.

2. Hydraulic considerations concern the tidal difference expected, the tidal volumes entering and leaving the estuary during neap and spring tides, expected wave action and future reservoir water levels.

3. Geo-technical considerations concern the presence of silt, which forms the sub-soil of the closure dam and the majority of the fill material for the dam body, while it must be borne in mind that the area is sensitive for earthquakes.

4. Because of the considerations given the construction of Feni River Closure Dam has to be planned very carefully.

5. After studying various closure methods it was decided to stop the tidal flow through Feni River by constructing a low neap tide dam across the full width (1160 metres) of the river during one neap tide. It is estimated that 10,000 labourers will be required for this operation.

6. Preparatory measures to closure include the placing of a bed protection of the river over an area of 1160 x 150 metres. On this bed protection a sill is constructed in order to obtain the same base level for the neap tide dam over the full width of the tidal channel.

On the sill stockpiles of bags filled with clay will be made at regular distances.

7. After completion of the neap tide dam the most critical item of closure operations has to be carried out: Construction of the so-called winter spring tide dam (using clay) during five days, in order to reach the elevation required for coping with water levels expected during the next spring tide.

8. Cross-sections of neap and spring tide dam as well as their location within the final dam profile are essentially determined by geo-technical considerations.

9. Final dam body will mainly consist of silt but measures have been taken to minimize the damages caused by liquefaction processes induced by earthquakes.

10. Crest level (SOB + 10.50 m) and slopes of final dam are primarily determined by hydraulic (wave attack, wave run-up) considerations but also geo-technical aspects (slope stability, settlement) play a role.
11. While type and thickness of revetments is mainly determined by wave attack, the extent of revetments as well as the incorporation of clay layers as proposed, is also governed by the desire to limit the consequences of a possible future neglect of regular maintenance.

12. The programme for construction of the works is based on closure by February 1985. In order to achieve this the contract has to be awarded by January 1984.

13. The most critical operation (for programming purposes) of the works will be the transport of construction materials to site in view of huge quantities (700,000 tons) involved and poor state of access roads.

14. The present state of (sections of) the access roads to the site prohibits extensive use by lorries. It is therefore necessary to upgrade these roads under separate contracts (or direct labour) at the earliest possible date.

15. Cost of construction of Feni River Closure Dam including Plank embankments will be in the order of 60 Crore Taka which is equivalent to US $ 24 million.
CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND

The Fen River Closure Dam to be constructed near the mouth of the estuary is considered to be the most difficult and also the costliest of all the structures required for Muhuri Irrigation Project.

Without this structure the main prerequisites of the Project viz. prevention of saline intrusion and storage of sweet water of the river systems for irrigation cannot be achieved.

After a number of studies, carried out by others, *(1), *(2), *(3), it was finally decided that M/s. Haskoning, Royal Dutch Consulting Engineers and Architects of Nymegen, the Netherlands, would be asked to prepare plans, design, construction drawings, cost estimates and tender documents, to evaluate tenders and to supervise the construction all as related to Fen River Closure Dam.

The Consultancy Agreement was signed on 14th January 1983 and Haskoning (hereinafter referred to as the Consultant) started its work one week later.

1.2 SUMMARY OF DESIGN ACTIVITIES

In accordance with the Agreement the Consultant would review all available reports, drawings and documents, etc. within a period of nine weeks after the Agreement had become effective.

This planning did not materialize as in fact, various team members had to be stopped coming out or alternatively, had to shorten their visit. The reason for this revised planning was that the data upon which the design had to be based were either out dated or insufficient for a well - balanced design.

Note: *) When reference is made in this report to one of the documents listed in Annex - E this is done as follows: *(3), (3) being the reference No. of the document referred to.
Therefore, it was proposed by the Consultant, and accepted by BWIEB, to collect new data on bathymetry of the estuary, to carry out tidal discharge measurements and to execute a geo-technical boring and testing programme.

This data collection campaign was carried out during the months of March to June 1983 (incl.) and was supervised and coordinated by the Consultants Team Leader.

The first Team member arrived again in Bangladesh on July 3rd and gradually the design team was built up until it consisted of seven experts by mid-August 1983.

Optimization of the design took place during the second half of August by Haskonings Project Sponsor and its Team Leader. These gentlemen received valuable contributions from their Senior Advisor on closure works, Prof. J. F. Agema of Delft Technological University.

On the 25th August 1983 the design was discussed with the Chairman of BWIEB, his staff members and the staff of NIP.

1.3 COMPOSITION OF STUDY TEAM.

In accordance with the Consultancy Agreement the Consultant has provided the services of expatriate specialists experienced in dam design, closure techniques, tender procedures and having practical knowledge of the various closure techniques.

Composition of the Study Team was as follows:

- J. Van Duivendyk: Project Sponsor and Closure Design Expert.
- Prof. J. F. Agema: Senior Advisor to the Study Team;
- C. Riemond: Closure Design Expert, responsible for construction methods and costs;
- N. W. A. Broug: Geo-technical expert, responsible for geo-technical design aspects and construction methods;
- J. C. Van der Meulen: Closure Design Expert, responsible for hydraulic and morphological aspects;
- E. C. Smith: Closure Design Expert, responsible for evaluation of local construction methods and bottom protection works;
- G. Te Slaa: Senior Supervisor/Team Leader, responsible for tender documents/Tendering procedures;
1.4 ACKNOWLEDGEMENTS

During their stay in Bangladesh individual Team members had the pleasure of meeting many BWDB officers and to discuss interesting technical issues related to the Closure of the Peni estuary.

It is not possible to mention all persons but the following persons should be mentioned in particular:

Mr. G.H.A. Islam Jaigirdar who was initially Project Director & Chief Engineer, MIP and who was succeeded by Mr. Lutfur Rahman, Project Director & Chief Engineer, Muhuri Irrigation Project.

Mr. Sadhan Chandra Das: Superintending Engineer, Muhuri Irrigation Project Design Cell.

Mr. Tushar Kanti Ganguly: Deputy Director.

Mr. Md. Lutfur Rahman: Deputy Director.
CHAPTER TWO

PROJECT AREA

2.1 GENERAL

The Kuhuri Irrigation Project is an agricultural development comprising a gross area of about 100,000 acres located in the tidal zone of the Noakhali and Chittagong District around the confluence of the Kuhuri and Feni Rivers adjacent to the Bay of Bengal in South-East Bangladesh.

The project area is adversely affected by saline intrusion caused by tidal action during much of the year, loss of upland fresh water flow during the dry season and flooding of the area during the wet season. The basic improvement techniques required to overcome these effects include facilities which permit storing of fresh water discharging flood flows, improving drainage and preventing salinity intrusion.

The required major facilities comprise coastal embankments, access roads, internal irrigation and drainage facilities, the Feni Regulator and the Feni River Closure Dam.

It is this last facility, the Feni River Closure Dam, which is the subject of this Report.

The site for the Feni River Closure Dam has been selected in June 1982 as part of the redesign of the closure dam prepared by MIP Design Cell-II of BWIB. *(3) Consultants agree to this selection.*

The data required to design the dam comprise mainly geo-technical data on sub-soil and building materials, hydrographic data about the whole estuary, observations and measurements about tides.

The available information as well as those data especially collected, will be reviewed in the following sections of this chapter.
2.2 GEO-TECHNICAL DATA

2.2.1 Introduction.

Following two sections present a compilation of the design governing soil characteristics, details of which are given in Annex B to this Report.

- The first section will deal with the interpretation of the site investigation results and the results of subsequent laboratory testing.

- The second section covers seismicity and its influence on design parameters and/or dam detailing. The general design features resulting from the inter-active phenomena between geo-technical design parameters and those stemming from the preferred closure method and the integrity of the finished dam, will be detailed in Chapter 4.

2.2.2 Site investigation and Laboratory testing.

(a) General.

The site investigation, comprising the drilling of 19 borings in conjunction with logging, sampling and SPT testing, was carried out by the "Ground Water Circle" of the BWDB.

Laboratory tests, carried out on selected samples and comprising sieve analyses, triaxial testing, wet and dry density, critical density, Atterberg limits and compaction testing, were carried out by the River Research Institute of the BWDB.

The results of the site investigation and subsequent testing were reported by the River Research Institute of the BWDB in their reports number: Soils - 89 (83) and 94 (83) of June 1983.

(b) Site investigation results.

(i) Ground level and ground water table.

Ground levels are fairly constant varying between approximately SGB + 0.60 and SGB + 0.20 for most of the river cross section. At the location of the main gully ground level is approximately SGB - 0.50 m. The ground water table is less relevant as, depending on tidal conditions, soils will have to be regarded as saturated up to ground level.
(ii) **Soil Composition.**

The soil composition is very consistent all over the dam axis. It varies from silt, often with some fine sand, to very fine sand, with some silt. Relatively thin clay layers have been encountered, some at shallow depth. At one instance, boring H23, a 3m thick clay layer was encountered at greater depth.

(iii) **Density.**

The density of the earlier described silts and sands shows a relatively consistent stratification with every where an at least 0.6 m thick very loose top layer (N-Values from 2 to 12). Loose and very loose layers are encountered from ground level to greater depth in boring H3, H11 & H17, with an average depth 6.50 meters. Some loose layers, emedded in between medium dense layers, are also encountered in boring H9 and H19. The remainder of the silts and sands, up to the investigated depth of approximately 20 meters, is predominantly medium dense (N-Values from 10 to 30).

(c) **Laboratory test results.**

(1) **Sieve analyses.**

Sieve analyses have been carried out on approximately 70 samples. They practically all confirm the prevailing presence of fine sands and silts as indicated on the bore logs.

With sieve No. 200 (particle size < 74 mm) as criterion for the change from sand to silt it can be concluded that:

- 84% of all samples has 10% or more particles < 74 mm
- 47% " " " 30% " " " < "
- 15% " " " 80% " " " < "

$D_{50}$, taken from 27 tests in the top 3 meters, shows a prevailing particle size of 95 mm for 22% of the tests.

$D_{50} \leq 95$ mm was found for 78% of the tests.

$D_{50}$, taken from 48 tests in the top 6 meters, shows prevailing particle sizes of 95 and 125 mm for respectively 12.5 and 10% of the tests. A $D_{50} \leq 95$ and 125 mm was found for respectively 42% and 71% of the tests.

---

Note: D... indicates percentage passing through sieve size as shown.
River bed samples, taken at 16 locations, can be characterized by means of following average diameters and standard deviations:

<table>
<thead>
<tr>
<th>Average (mm)</th>
<th>Standard deviation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D$_{50}$</td>
<td>88</td>
</tr>
<tr>
<td>D$_{15}$</td>
<td>44</td>
</tr>
<tr>
<td>D$_{95}$</td>
<td>158</td>
</tr>
<tr>
<td>D$_{90}$</td>
<td>189</td>
</tr>
</tbody>
</table>

The above results are of particular importance for the assessment of the feasibility of compaction procedures, the choice of filter structures and material, the scour pattern, etc., etc..

(11) Triaxial Testing

Shear strength parameters ($\phi', c'$) were established by means of 8 consolidated undrained (CU) tests with pore pressure measurement (pp) and 6 consolidated drained (CD) tests.

Taking into account the developed pore pressure in the CU-test, results are presented as effective stress parameters, allowing comparison with the results of CD-tests.

It appears that the parameters $\phi'$ and $c'$ are consistently lower for the CU-test compared with the CD-test. This is not unexpected but confirms the metastable character of the particular layers. These soil properties may also result in liquefaction phenomena under influence of repetitive and alternating shear stresses. For the choice of the design parameters it seems prudent not to take an overall average but to choose a lower limit; the poorest zones under the dam will govern the overall integrity.

To allow for the loadings during construction and from earth-quakes (being dealt with later) the following design parameters have been introduced:

- The relatively loose top layer $\phi' = 20^\circ$ $c' = 5$ kN/m$^2$
- The medium dense to dense layers $\phi' = 20^\circ$ $c' = 25$ kN/m$^2$

(111) Critical Density

Tests carried out with triaxial test equipment at constant isotropic stress allow the establishment of a void ratio that does not change under influence of shear stresses. This is called the critical void ratio. This void ratio can be compared with the in-situ void ratio. When the latter exceeds the critical void ratio the soil can be regarded as susceptible to liquefaction.
Tests were carried out on 5 samples and were compared with in-situ densities. Three tests clearly indicated an in-situ void ratio exceeding the critical void ratio. These results also confirm the bore logs indicating the presence of loose and very loose layers.

(iv) Atterberg Limits.

With soil composition being very consistent in the area, only two samples were tested for Atterberg Limits. With the results of previous studies in the area they confirm that the prevailing sand and silt layers have to be regarded as non-plastic. This also implies that the material has to be regarded as very susceptible to piping (internally) and erosion (externally). Materials originating from excavation of the diversion channel and intended for use as fill in the closure dam clearly showed signs of erodibility. This property should be regarded as critical. The silty soil will also be poor for the development of vegetation.

(v) Compaction Testing.

Standard Compaction tests were carried out on two samples consisting of silt with some fine sand resulting in an OC of 19.1% and 18.7% for maximum dry densities of respectively 16.3 & 16.9 Kn/m³.

2.2.3 Seismicity.

(a) Data

Data on seismicity and its implications has been collected from:


- Discussions with the Seismicity Dept. of the Royal Dutch Meteorological Institute (see attached map with epicentre).

- Considerations in the earthquake - resistant design of earth and rockfill dams, H.B. Seed (1979), Geotechnique 29, No.3.

(b) Definition of problem.

To allow establishment of the earthquake loading one has to quantify a basic seismic coefficient after which various factors have to be introduced, allowing for the importance of the structure, type of structure, soil conditions, etc. Analyses based on a basic seismic coefficient method and 0.25 in the Response Spectrum Method result in a Design Seismic Coefficient of 0.04 - 0.09 g taking into account an importance factor of 1.5. \(\sqrt{0.05}\) in the Seismic Coefficient indicates that earthquake loading will have to be allowed for.

(c) Impact of seismicity on dam design.

(i) Previous studies on embankment design did already allow for Design seismic coefficients of 0.05 and 0.10 g. They do well cover the previous analyses and consequently that part of the slope stability analyses has been adopted without change.

(ii) The horizontal accelerations in conjunction with the zones of loose and very loose material result in subsoil conditions that have to be regarded as being susceptible to liquefaction.

Though compaction methods as vibroflotation, dynamic compaction, SPTC-method, etc., do exist, it has to be expected that soil composition, with a percentage of particle \(<74\) mm much higher than 10%, is prohibitive to an effective treatment. The conditions have to be taken for granted and consequently design should incorporate design feature to minimize the adverse effects of such phenomena.

(iii) Poor compaction properties of the silt in general, in the dam body as well, require that silt and fine sand is placed as densely as practically possible without fines being washed out. In this context hydraulic fill is not recommended.

As case histories clearly indicate that the presence of cohesion is of paramount importance to dam stability under earthquake loading, it is therefore decided to introduce a more stable clay core in the dam body to ensure that any damage will be limited to one outer silt shell, whereas the clay core will remain stable. The clay core will also serve as a vertical above ground seal to minimize the negative effects of piping.

(iv) To increase the stability of the dam on a potentially locally liquefied subsoil it was decided that the whole dam base will be placed on a woven fabric
layer on the under side of which fine mazed non-woven membrane will be attached. The latter will also serve as a filter during the stage that this composite layer will be used for the construction of the sill.
FENDI RIVER CLOSURE PROJECT.

Graph 1

D.50 of "n" sieve analysis

27 Test results in top 3 meters.

48 Test results in top 10 meters

Fig. 1
2.3 HYDROGRAPHY.

Hydrographic information, made available to the Consultants at the commencement of their services, consisted of:

- a contour line map based upon surveys in 1972;
- cross sections and a contour line map based upon surveys in January/February 1982.

The information made available led to the conclusion that:

- considerable siltation had taken place between 1972 and 1982.
- the most recent information was (from 1982) insufficient for a determination of the geometry of the Feni River estuary and its tidal capacity.

Therefore Consultants proposed, and BWMB accepted, to conduct cross sections measurements over the estuary between the "old closure site" and the highway bridges of the new Feni by-pass roads. Cross sections were to be measured at 600 m intervals; the whole survey campaign was to encompass:

- the Feni River (from "old closure site" to new highway bridge);
- the Muhuri River (from confluence with the Feni River to new highway bridge);
- the Kalidas Pahalia (from its mouth near the Feni river to new highway bridge).

The actual measurements were done by the Hydrology Department of BWMB between March and May 1983. In all the following cross sections were measured:

- 25 nos. across the Feni River;
- 6 nos. across the Muhuri River;
- 20 nos. across the Kalidas Pahalia.

No. measurements were made across the khals discharging into the estuary.

The cross section measurements enabled the Consultant to determine the basic parameters needed for the mathematical model of the Feni estuary, as well as the (approximate) volume of the tidal prism.
2.4 TIDES AND DISCHARGE MEASUREMENTS.

2.4.1 Tides.

Reliable observations on the (tidal) water level fluctuations near the closure site were available for the period from 19th February till 8th March 1981. No official tide forecasting tables exist for the Peni River estuary. Such tables, however, do exist for Sandwip (see Bangladesh Tide Tables published by the Department of Hydrography of BITWA, Dhaka). Comparison of simultaneously measured tides at the closure site and Sandwip indicate a clear relation for high water, while the low waters at the (old) closure site are governed by the sea bed configuration and reach almost the same level of SOB = 1.0 m (whether at neap tide or spring tide).

The highwater relation was concluded to be:

\[
\text{HW Closure Site (ref: to SOB)} = \text{HW Sandwip (ref: to chart datum)} - 1.30 \text{ m}
\]

High water reaches the closure site 15 to 45 minutes later than Sandwip.

Tide predictions for the closure site, which are extremely important for the actual closure operations, could be made by adjusting the BITWA tide predictions for Sandwip as indicated above. (For other periods of the year, with important river discharges, the relation between high water levels has yet to be established more accurately. It is therefore recommended to install a permanent automatic tide gauge at the earliest opportunity).

2.4.2 Tidal Discharge Measurements.

In the winter and spring of 1982 tidal discharge measurements have been done near the closure site during tidal cycles of 12½ hours. The discharge measurements comprised simultaneous measurements of velocities and water levels.

The main results have been collected in the following table.
TABLE 1: Discharge measurements near closure site in 1982.

<table>
<thead>
<tr>
<th>Date</th>
<th>HW (m + SCB)</th>
<th>LW (m + SCB)</th>
<th>Q max flood (m³/s)</th>
<th>Q max ebb (m³/s)</th>
<th>Flood Vol (10⁶ m³)</th>
<th>Ebb Vol (10⁶ m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 Feb.82</td>
<td>2.88</td>
<td>0.40</td>
<td>949</td>
<td>877</td>
<td>5.7</td>
<td>8.8</td>
</tr>
<tr>
<td>10 Mar.82</td>
<td>3.05</td>
<td>0.43</td>
<td>1034</td>
<td>1053</td>
<td>6.2</td>
<td>13.7</td>
</tr>
<tr>
<td>19 Mar.82</td>
<td>1.19</td>
<td>0.24</td>
<td>163</td>
<td>102</td>
<td>1.2</td>
<td>1.4</td>
</tr>
<tr>
<td>8 Apr.82</td>
<td>3.37</td>
<td>0.33</td>
<td>1212</td>
<td>1878</td>
<td>8.7</td>
<td>21.3</td>
</tr>
<tr>
<td>22 Apr.82</td>
<td>3.66</td>
<td>0.34</td>
<td>1299</td>
<td>1664</td>
<td>8.8</td>
<td>19.4</td>
</tr>
</tbody>
</table>

The velocity measurements have been carried out in one vertical only. Current velocities in other verticals of the cross section have been determined proportionately to the squarefoot of the depth. This method can introduce in-accuracies in tidal areas with different ebb and flood channel and where the flow can cross-over shoals.

For the 1982 discharge measurements, gauge and velocity readings were only done at hourly intervals. This period is too long for a proper description of the tide curve, especially in the flood stage, which lasts only 3 to 3½ hours. Difference between flood and ebb discharges are too high (taking in-to account a likely combined river inflow of 25 m³/s) and must probably be attributed to the "single vertical method" and the long measuring intervals.

To get a better insight in the flow and discharge patterns of the estuary, the Consultants have proposed, and the BWIB has agreed to carry out simultaneous gauge readings and current velocity measurements at strategic locations in the estuary.

These locations were:

- At the old closure site:
  - gauge readings at both banks;

- At the new closure site:
  - gauge readings at both banks;
  - current velocity measurements in three verticals (left channel, shoal and right channel).

- At the mouth of the Kumira Khal ("half way" the estuary):
  - gauge readings at both banks.
At the new highway bridges across the Feni and Kuhuri Rivers and the Kalidas Tahalia:

- gauge readings (one bank only)
- current velocity measurements (one vertical only).

Actual measurements were done by staff of the Hydrology Department of BWDB.

Two measurement campaigns took place:

- around 23rd March 1983, i.e. during neap tide;
- around 29th March 1983, i.e. during spring tide;

(An earlier campaign around 16th March had to be stopped prematurely because of adverse weather conditions).

The results of the two measurement campaigns have provided the data required for the mathematical model of the Feni estuary.
2.5 **TOPOGRAPHY**

Topographical maps of the project area had been produced by others at an earlier stage of the project.

Together with the new river cross sections (see section 2.3) these maps formed an adequate basis for the design of the Fenl River Closure Dam.

From the maps it has become clear that shifting of channels, erosion and accretion take place continuously.

It is therefore necessary to conduct regular surveys at the closure site, so that adjustments to the design, if any, can be made without delay.
CHAPTER THREE

THE PROJECT

For the purpose of this Final Design Report the project concerns the design of the Feni River Closure Dam located near the mouth of the estuary.

At the selected site the tidal river has a width of approximately 1160 metres while the level of the river bed varies between SCB - 0.30 m and SCB + 0.50 m.

The main dam through the estuary will have a crest level of SCB + 10.00 m (after initial settlement).

This main dam will have to be connected:

- On the left bank to the existing coastal embankment by means of a flank embankment having a length of 383 m and an average height of 4.5 m.

- On the right bank to the Feni Regulator, (which structure is in an advanced stage of completion) by means of a flank embankment having a length of 1622 m and an average height of 5 m.

Prior to construction of the earth works and revetments of main dam and flank embankments the tidal channel has to be closed by means of a low bund in order to stop the tidal flow.

In view of the prevailing circumstances in the Feni estuary this closure is a difficult feature which requires careful planning and design followed by construction through methods and time schedules which have to be followed closely to prevent major mishaps.

Design aspects are covered in Chapter Four of this Report while the construction, works planning and costs are dealt with in Chapters 5, 6 and 7 respectively.
CHAPTER FOUR

DESIGN OF FENI RIVER CLOSURE DAM

4.1 SCOPE OF PROBLEM:

The problem to be studied can be summarised by recalling the two different issues involved:

- Closure of the tidal channel of the Feni River;
- Construction of a permanent dam across the Feni River which can act as a barrier between a fresh water lake and a tidal estuary and also as a defense against high tides and wave attack.

The two issues are interrelated and cannot be studied separately. The main aspects which play a role within the context of the studies and which each, to a certain extent, determine the final design of closure works and final dam profile, are the following:

(a) The tidal movement in the Feni River estuary is considerable. At the site of closure tidal high water levels normally vary between SCB + 1.3 metres (neap tide) and SCB + 4.0 m (spring tide) during the dry season from November to March (inclusive) while the spring tide high water level can reach SCB + 6.5 metres outside the dry season.

(b) During spring tide the incoming tide is characterized by a tidal bore.

(c) The subsoil in the Feni estuary consists predominantly of loosely packed layers of silt and fine sand.

(d) Data on wind and waves are not available for the neighbourhood of the closure site. Wave observations in as far as available for the Bay of Bengal have to be correlated to Feni site circumstances.

(e) At the closure site shoals and gullies are at a level of SCB + 0.40 and SCB - 0.20/-0.50 m respectively.
(f) Building materials like suitable clay, boulders, reed and brushwood are not available near the closure site and may have to be transported over considerable distances.

(g) It is desirable to construct the works as much as possible by manual labour which is, however, not available in unlimited number outside the dry season.

(h) Closure can best effected during yearly lowest tides without presence of waves of importance viz. in the month of February but this implies that final dam profile must be completed during the monsoon period by mid June.
4.2 GEO-TECHNICAL DESIGN ASPECTS

4.2.1 Introduction.

Both design and construction, intimately connected, are strongly influenced by the characteristics of the non-plastic sand and silt layer present under the dam base and to be used for the dam body.

As indicated elsewhere in the report the main geo-technical concept of the dam involves the reduction of the adverse effects of sub-soil liquefaction by means of the introduction of woven fabrics, having a certain tensile strength, in conjunction with a stable clay core. Further allowance has been made for the forces from horizontal acceleration in the dam body itself by introduction of appropriate slope angles.

The following sub-sections will deal, successively, with the various details in order to support the overall dam design and recommended closure operation.

4.2.2 Below existing ground level.

4.2.2.1 Susceptibility to liquefaction.

The presented analysis in Annex - B clearly indicates that liquefaction of the subsoil constitutes a potential hazard. However, soil composition, showing in general a very high silt content, renders it practically unsuitable for compaction. It is known from literature that a percentage > 10-15% particles < 74 μm (silt and clay) will render compaction methods ineffective. Any suggestion to do so should be preceded by in-situ tests in similar conditions.

Soil improvement by means of excavation and replacement by coarser material is not regarded feasible. The extremely fine material will result in very flat underwater slopes and even in liquefaction and consequent loosening of major areas.

4.2.2.2 Loose top layer/membranes.

The whole area is covered with a very loose top layer that may induce horizontal sliding and, or mud waves when structures are placed upon it without special measures.

The behaviour of the top layer has been improved by means of the introduction of a protective layer able to take tensile forces. This layer, under influence of the ballast placed upon it will confine the loose top layer. It will also allow gradual consolidation as the layer will be drained under influence of the dam weight. An analysis, carried out to establish the likelihood of high stresses in the membrane under earthquake loading indicated that no special requirements emerge. Of course, construction procedures will result in certain strength requirements to ensure proper behaviour under handling and ballasting.
Another design condition, as anywhere else in the dam, stems from the requirement that no fines from the dam base shall be allowed to migrate through the membrane. This requirement results in the need to apply a non-woven sand-tight but permeable membrane as well. For this purpose the sieve analyses present data to allow the choice of a matching fabric in accordance with laboratory test results. The latter are often presented by the manufacturers of the membranes.

4.2.2.3 Scour of top layer - Minimal required length of bed protection.

The development of any concentrated flow on exposed river bed material will result in scour and may contribute to regressive erosion generated by "static" liquefaction phenomena. An approximation of the required length of the bed protection with respect to a structure can be calculated under the assumption that the angle of the unstable slope on the downstream side will develop also on the upstream side when a flow due to liquefaction occurs. Further it stands to reason that eroded material will partly fill up the hole and consequently the overall volume remains the same. Based on the previous approach we have established that with regard to a predicted scouring depth "d" a minimum distance of 10 d should be adhered to.

4.2.2.4 Settlements of the dam foundation.

Previous studies have already shown that the amount of settlement as a result of compression of the under ground will be of the order of 0.3 to 0.5 m. A major part of this settlement will go unnoticed as the sub-soil layers will exhibit a relatively short consolidation period. This means that most of these settlements will take place during construction.

4.2.2.5 Monitoring of settlements.

Settlements of the dam shall be monitored during various stages of construction. For this purpose at least 5 settlement beacons shall be installed in the clay bag depots. The beacons shall be placed in the center of a depot and shall be made from hollow steel pipes with a welded-on steel plate to be placed on the ballast layer. Similar beacons shall also be installed in other dam sections.

4.2.3 Above existing ground level.

4.2.3.1 Measures against piping.

The possibility of liquefied zones due to earth quakes, the presence of the loose top layer and the proposed vertical closure are the main reasons to introduce a composite membrane, consisting of woven and non-woven fabric, on top of the existing ground. The membrane will be ballasted to avoid erosion and, depending on ground level more successive ballasted layers may be used to reach uniform level to allow the construction of the neap tide dam (see sketch).
By their nature these layers will be extremely permeable in horizontal direction when clay and silt for the dam body is placed upon it. Without special measures water pressures under the dam body may become unacceptably high and piping will be promoted. Further it has to be expected that silt and clay placed on the ballast layer will not be able to penetrate sufficiently into the pores between the ballast. When this happens after actual construction it may result in loosening of the dam body which is unacceptable for the already sensitive silt layers. Besides, this phenomenon will contribute to surface settlement and damage to the slope protection.

To cater for the above conditions following measures have been introduced into the design:

- Use of a provisional grouted seal-I (cement/sand) prior to construction of the neap tide dam in those areas where more than one ballast layer has been placed.

- Thoroughly sanding of the top ballast layer at the location of the neap tide dam on the reservoir side:

- Placing of non-woven fabric over the neap tide dam and the adjoining dam base section of the spring tide dam. The non-woven fabric should match the particular characteristics of the clay so that clay cannot migrate through this membrane. The membrane shall have sufficient length to provide the vertical sealing of the ballast layers as well;

- The use of a provisional seal II in conjunction with a clay dam placed on the first composite layer. In the zone between the neap tide dam and the grouted seal I II on the reservoir side ballast layer shall be removed leaving only the first composite layer, in contact with the river bed, with some sparse loading with ballast but without any of the originally fixed-on bamboo. Leakage on the reservoir side to be suppressed by means of clay to be placed outside dam too.
A clay seal shall be placed in the contact zone with the permeable ballasted zone and on top of the bent down upper composite layer (see above) to ensure that no piping can occur at this location at any time.

The placing of silt/sand on the central part of the dam body between the clay embankments in such a way that the original density of the borrow material will be maintained or improved as much as possible. In this situation placing by hand, chunk-wise, is to be preferred without too much levelling and scraping by bulldozers.

4.2.3.2 Slope stability (Macro) of the Closure Dam

Slope stability analyses in previous studies, allowing for angles of internal friction of 27° and 30°, have already been undertaken. They considered various loading conditions for a very similar dam profile as shown in the previous paragraph allowing for slope as well as foundation failures. The analyses clearly confirm that horizontal loading due to earthquakes will govern design.

To allow for this situation the proposed dam has been checked against earth-quake loading taking into account a relatively low friction angle $\phi \leq 10^\circ$ and cohesion $c' = 5 \text{kN/m}^2$. The worst loading condition can then develop with a filled reservoir (500 + 5 m) and low tide in conjunction with a horizontal acceleration of 0.1 g.

The reviewed loading condition on the sea side can be compared with a situation on the reservoir side after relatively quick emptying of the reservoir as a result of a calamity involving damage of the spring tide dam on the sea side. As such a safety factor slightly $\leq 1$ becomes acceptable.

Following conclusions can be drawn from the above design approach:

- The average slope for $h \leq 5.5 \text{m}$ (height of spring tide dam) can be $\leq 3:2$, design allowed for a slope angle of 1 in 4 to allow for construction and maintenance.

- For $5.5 \text{m} < h < 10 \text{m}$ the minimum slope angle is 1 in 5 but wave height attenuation in this zone gives preference to a 1 in 6 slope.

Taking into account the result of previous analyses and a number of case histories of damaged silt dams due to shaking we have given preference to the introduction of a clay core in the dam body. Such an element, consisting of clay and exhibiting sufficient cohesion, can be regarded as a stable body that can be used to minimize and/or localize potential damage.

The location of the clay core on the sea side has been chosen to ensure that a potential sliding will not affect that part of the dam which has to provide protection against the sea.
In this context it has to be kept in mind that the type of soil has shown that a failure will normally not happen during but some time after the shaking. This happens especially when the density of the dam body has become insufficient. The shaking will then introduce zones in which an on-set of liquefaction will build up and extend in the dam body till some time after the shaking shear stresses cannot be taken up any more.

The design has now concentrated the weaker element, the silt embankment, on the reservoir side to create more favourable conditions to slide into one direction only. For this reason we have also considered the average reservoir side slope of 1 in 4.6 as sufficiently safe compared with a slope angle 1 in 5 for a safety factor 1. This design approach may contribute to the overall integrity as well, because sliding preference is on the reservoir side where repair can take place, under protection of the winter spring tide dam.

4.2.3.3 Slope stability of the winter spring tide dam.

The stability of the winter spring tide dam is critical on the down stream side only. Important in the analyses is the fact that the ballast layers underneath the dam base will be permeable and water pressures may develop here relatively easily (see sketch). It is quite clear that the up-stream provisional grouting and sanding— in are of extreme importance.

Besides, one has to take care that the ballast layers are closed at the very end in such a way that free entry of water can never occur. Seepage water shall only be allowed to pass through the non-woven fabric. For the clay properties adhered to the relatively conservative values of $\phi' = 20^\circ$, $c' = 5\, \text{kN/m}^2$. Wet density of $16\, \text{MN/m}^3$.

With a base length of the core of 50 meters the hydraulic gradient becomes $\frac{5\phi_0}{c} = \frac{5}{5}$. No earth quake loading during construction has been allowed for.

Based on the previous parameters calculations were carried out indicating that the down stream slope section was stable under the given slope (1:4), even if cohesion was neglected.

Note: The terms "meap tide dam" and "winter spring tide dam" are introduced and explained in more detail in Section 4.3.
4.2.3.4 Slope stability of the neap tide dam.

The slope stability of the neap tide dam has to be looked at in the light of the fact that it consists of clay bags. As e.g. polythene bags do have low friction characteristic they will govern the dam stability on the downstream side. For this purpose \( \phi' = 15^\circ \) has been introduced however without making any allowance for cohesion. The density of the material is taken at 16 KN/m\(^2\) and for traffic load on top of the dam a surcharge of 10 KN/m\(^2\) has been taken into account. For the downstream dam section a loading condition similar to the winter spring tide dam will be allowed for. The hydraulic gradient is chosen as approximate 1 in 6 = 0.16.

Based on the earlier parameters it becomes evident that without special measures no stability can be obtained on the downstream side. For this purpose both sides will have to be connected with each other. The use of bags or any other measures inducing a proven friction angle of 30\(^\circ\) will result in a downstream slope of approximately 1:3.

The total stability of the dam (with \( \phi' = 15^\circ \)), acting as a monolithic structure, will then be sufficient to withstand forces from the water pressure only.

4.2.3.5 Stability clay bag depots.

The closure operation involves the erection of 10 clay bag depots to cater for the construction of the neap tide dam. The height of these depots will be some 6 meters and its minimum cross section 60 m\(^2\). This means that without special measures the foundation pressure, acting on and via the ballasted layers will be of the order of 20 KN/m\(^2\) which is quite considerable. Though deformation of the sub-soil is allowed to happen it should at no time lead to uncontrollable movements of the sides and consequently failing of the subsoil. The only way to overcome such events is to place the depots on bamboo mattresses extending at least 4 meters outside the depot base.

Note: During execution the load combination of high sea water level and truck load has to be (and can be) avoided.
The slopes of the depot should not be chosen oversteep to avoid toppling. Assuming the slopes of 1:1 we will find a foundation base of approx. 16 m and a total width, incl. mattresses 24 meters assuming an approximately parabolic foundation pressure, we will find then a foundation pressure of $50 \times 1.5 = 75 \text{ KN/m}^2$, which can be accepted.

Taking into account the properties of the very soft layer it is recommended that stockpiling will start in the center of the depots to ensure that settlements in this area will be maximum and along the side less.

Quantitatively it seems prudent to allow for 0.2 - 0.3 m of settlement in the centre and 0.10-0.15 m along the perimeter.

**4.2.4 Settlement of the dam body.**

It has already been mentioned in paragraph 4.2.2.4 that underground settlement, 0.3 m - 0.5 m, will partly go unnoticed during construction. However these settlements will effect quantities to be allowed for and heights to be set when comparing with the theoretical cross section.

Further allowance has to be made for movements in the dam body itself which could be as much as 0.5 meters at the centre and 0.3 meters of the location of the spring tide dam. These movements have to be taken into account for the calculation of quantities.

To ensure that a proper crest level will be maintained a camber has to be introduced. Monitoring of settlements, with beacons at various levels in the dam body, has to be carried out during and after construction to verify and/or adjust the previous figures and to monitor over-all dam behaviour.

**4.2.5 Compaction.**

The prevailing silty layers to be used to build-up the dam body will be difficult to compact when placed too wet. Compaction will then result in a transformation into a fluid state. Consequently borrow material should be taken from a dewatered area with the optimum moisture content slightly on the dry side. Measures to improve the final density are:

- No fill to be placed as hydraulic fill;
- No fill to be placed under water. This may result in placement of the fill in the lower dam sections at low tide;
- The chunky character of the fill to be maintained as much as possible with minimum levelling by means of dozers;
- To use vibrating rollers with a layering adjusted to the type of equipment. Vibratory rollers with 100 - 150 KN
drum weight can be used for lifts of 0.7 to 1 meter. Lighter vibratory rollers 30 to 60 kN in weight, can be used for lifts of 0.3 to 0.45 meters.

- Depending on the type of roller the number of passes shall be 4 to 8.

- The minimum degree of compaction obtained shall be 95 percent Standard Proctor.

The availability and type of clay for the various embankment sections consisting of clay has not yet been determined from its borrow area. Following criteria shall be met:

- The clay to be used of medium plasticity with a liquid limit not exceeding 50%, No high plasticity and / or highly compressible clays shall be used.

- The clay shall be of low to medium strength with an unconfined compression strength of 50 - 200 kN/m².

- Compaction can be carried out with sheepfoot rollers of a capacity and number of passes adjusted to clay type and shift thickness (layer thickness appr. 0.3 to 0.4 meters).

- The O.C shall be adjusted to the clay type.

- The minimum degree of compaction obtained shall be 95 to 100% per cent Standard Proctor.

- The degree of compaction shall be checked in the field.

4.2.6 Micro-stability.

The non-plastic characteristics of most of the fill material render it very susceptible to erosion. This erosion due to leakage through slope protecting layers could have serious consequences for the structure as a whole. All detailing and execution of works shall therefore allow that filter layers do have sufficient overlap, are not damaged, do devide coarse and fine material by means of matching filters etc, etc.
4.3 PRINCIPAL HYDRAULIC ASPECTS

4.3.1 GENERAL

In this section the most important hydraulic aspects related to the closure works of the Feni River estuary are reviewed. The review will be based on hydraulic calculations carried out and hydraulic considerations formulated by means of the basic information available (Section 2.3, 2.4). These calculations and considerations are presented in a more detailed form in Annex A to this Report.

The basis of all tidal calculations made is the mathematical model which was specially developed for the Feni estuary and closure works projected. It is now operational in Bangladesh and will be transferred to the Design Cell of MIN after completion of the work. The mathematical model was in first instance calibrated by means of the hydrographic and hydrometric measurements carried out during the months of February and March, 1993. After calibration various "design tides" were introduced as well as different closure methods. The results of these calculations will be discussed in Sections 4.3.3 to 4.3.10 inclusive, and conclusions will be drawn.

4.3.2 Initial approach to closure

In line with earlier studies (ref. *(1), *(2) and *(3)) Consultant has, as a first approach, studied so-called horizontal - vertical closure. This means that the natural tidal channel at the future closure site is narrowed from both ends "as much as possible" in order to limit the work needed for closing the last remaining tidal gap by vertical method.

To determine how far one could narrow the channel beforehand, it will be necessary to calculate which flow conditions will exist in the closure gap. It should be realised that narrowing the channel will inevitably lead to higher current velocities in the closure gap. These higher velocities would result in a higher sediment transport capacity in the gap. As the sediment could only be "delivered" by the channel bed at the gap, degradation of the bottom would occur, Roughly speaking degradation would continue till the original cross section area would again become available for the tidal flow.

To counteract this degradation, it would be necessary to apply a protection to the channel bed, which should:

(a) Consist of elements heavy enough not to be transported by the increased current velocities.

(b) Leave the original bed material in place.

The protection work should also cover an area upstream and downstream of the closure gap (to prevent scour near the centre line of the future closure dam), as well as an area at both sides of the gap (to withstand higher velocities during the initial narrowing operations).

Actual closure would take place during a neap tide, and the closure gap should be wide enough to allow the preceding spring tide to pass, without causing damage to the earlier applied bed protection works.
In drawing up an implementation scheme for closure the following aspects must be considered and a certain balance between them reached:

(a) If the gap is relatively wide, a lighter bed protection is required of increased length in the direction of the axis of the dam, but of decreased length perpendicular to the dam (less contraction, lower velocities).

(b) If the gap is relatively wide, the work capacity required to close during neap tide is large; this applies also for the work required to withstand a spring tide following closure during neap tide.

(c) A low sill in the closure gap limits the current velocities during the spring tide precedent to the closure operation.

(d) A low sill causes high velocities in the closure gap during the last stages of the (neap tide) closure.

(e) A low sill has an effect on the work load to be carried out as noted in (b).

The Consultants approach has initially been:

(a) To determine a closure gap wide enough to limit current velocities during a design (winter) spring tide (4.50 m ~ SOB) to a maximum of 3 m/s. This criterion has been set having regard to materials available for bed protection and the possibility of using (manual) labour.

(b) To determine to which extent further horizontal narrowing of the gap would be possible between spring tide and the subsequent neap tide at which actual closure would be effected.

(c) To determine the weight of any units able to withstand velocities in the final closure gap during neap tide.

4.3.3 Discussion of results of computer runs.

To investigate the influence of the dimensions of the closure gap (width and sill level) on the maximum current velocity 39 computer runs have been made. The most important results of these runs, viz. the maximum (flood and ebb) current velocities and the tidal volume have been presented in Annex A (Table A - 2). In order to better illustrate the effect of variation of the three parameters "width of closure gap", "highest tide level" (representing in each case a typical design tide) and "sill level" on the maximum current velocity encountered in the closure gap graphs have been drawn up (Figures 2 and 3).

From these graphs the following can be concluded:

- Narrowing of the closure gap (while not elevating the sill) leads to higher maximum current velocities in all cases.

- A raising of the sill (while leaving the closure gap width constant) leads to higher maximum current velocities for all tides higher than approximately SOB + 2.70 m.
Maximum velocities. Flood.

(Graphs connect points with some highest tide level and sill elevation as indicated)

Fig. 2
Maximum velocities.

Ebb.

(Graphs connect points with same highest tide level and sill elevation as indicated)

Width of closure gap
(effective) (m)

(Graphs connect points with same highest tide level and width of closure gap as indicated)

Sill level.
(m + 5.08)

Fig. 3
Having regard to the natural closure site configuration and a gradual horizontal closure (to the point where velocities in the gap do not exceed the adopted limit of 3 m/s), as a first approach the following closure method would seem to be conceivable:

(a) Apply bed protection "in and around" the final closure gap.

(b) Narrow the natural width at the closure site so that a current velocity of 3 m/s is not exceeded during a severe winter spring tide of S0B + 4.60 m. The effective width of the closure gap then would have to be approximately 425 m.

(c) If tide and weather predictions are favourable, the gap could be narrowed so that the spring tide preceding the closure operation would not result in damage to the bed protection. Typical design figures for this stage are:
   - Spring tide: S0B + 3.60 m
   - Effective width: 360 m.

(d) Following the aforementioned spring tide further narrowing could be realised until neap tide occurs. Typical design figures:
   - Neap tide: S0B + 2.40 m
   - Effective width: 175 m

(e) During neap tide the final closure gap would have to be closed. This final closure would have to be achieved by means of a gradual raising of the sill. Further narrowing of the gap (horizontal closure) would not be possible as the current velocities would become too large.

4.3.4 Effect of friction and obstructions on closure width:

Apart from the direct effect on the closure gap width imposed by decrease of the actual area by narrowing operations, there are indirect effects which tend to decrease the width as well. These are:

- Friction losses due to the introduction of a bed protection having a roughness which is significantly higher than the roughness of the natural river bed.

- Contraction of flow due to the presence of a vertical (sill) and a horizontal (built-out embankments) decrease of original river profile available for discharge purposes.

- Obstruction of gap by scaffolding

(For instance a jetty required for closure operations)

These indirect effects can, together, be expressed in one single discharge coefficient. The "effective" widths as used in the computer calculations and presented in the graphs of Figures 2 and 3, must be divided by this discharge coefficient to arrive at the "real" width of the closure gap. Table 2 shows the result of this exercise. (+)

*) See Section 4.3.5 of this Chapter.

(+) The effective widths as presented in the Table are the ones found for the maximum allowable current velocity of 3 m/s (sill level 0.25 m > S0B)
TABLE 2 Effect of discharge coefficient on width of closure gap.

<table>
<thead>
<tr>
<th>Typical design tide (as characterized by highest tide level in SOB +)</th>
<th>4.60</th>
<th>3.60</th>
<th>2.40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective width of gap (m)</td>
<td>425</td>
<td>360</td>
<td>175</td>
</tr>
<tr>
<td>Sill level (SOB +)</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>Discharge coeff. (no jetty)</td>
<td>0.77</td>
<td>0.77</td>
<td>0.72</td>
</tr>
<tr>
<td>Discharge coeff. (with jetty)</td>
<td>0.55</td>
<td>0.55</td>
<td>0.51</td>
</tr>
<tr>
<td>Real width (no jetty) (m)</td>
<td>550</td>
<td>470</td>
<td>245</td>
</tr>
<tr>
<td>Real width (with jetty) (m)</td>
<td>770</td>
<td>655</td>
<td>340</td>
</tr>
</tbody>
</table>

When studying the figures of Table 2 it must be realized that the gullies are not wide enough to accomodate a closure gap of 550 metres at a sill elevation of 0.25 m + SOB. The closure gap would have to extent on the showal where higher sill levels would prevail. A higher average sill level implies higher current velocities than acceptable. To reduce these current velocities to the allowable maximum of 3 m/s the gap would have to be widened (say to 650 - 700 m in the case without a jetty).

4.3.5 Weight of closure elements in case of horizontal closure.

The bed protection will be made when the existing natural conditions (width, current velocities) at the site of closure have not changed. Consequently, relatively small and light individual elements can be used for ballasting the bed protection. When the tidal channel is narrowed current velocities will increase and shear forces induced by the current will try to move the ballast elements laying on the bed protection. As will be discussed in Section 4.4, this can be avoided by securing the ballast elements to the bed protection by means of wire mesh. In this manner relatively light ballasting of the bed protection is sufficient to withstand maximum current velocities of 3 m/s.

Horizontal closure implies the pushing out (earth) or dumping (rock) from both ends of the embankments built-out from both river banks. Narrowing of the gap results in higher current velocities and it has been demonstrated in practice and during model tests that heavy, large elements are required if dumped during current velocities of 2 to 3 m/s. In fact, these elements cannot be fixed to the bed protection with wire mesh and before reaching the bottom many will be carried away by the current and deposited outside the actual axis of closure.

Depending on the stage of closure the individual elements should have an average diameter of initially, 0.35 m increasing to 0.60 m in final stages. Moreover, these dimensions are only valid for rock elements having a specific weight of 2.65. For elements having a lower specific weight larger diameters are required.

4.3.6 Main reasons for rejection of closure method developed during initial approach.

It is recalled that the initial approach was based on the horizontal-vertical method; i.e. try to narrow the natural channel as much as
possible until further narrowing is not any longer possible because of exceedance of the maximum allowable current velocity. Subsequently, the remaining gap (called the closure gap) is then closed in the shortest possible time by raising the sill simultaneously over its full width.

The hydraulic calculations by means of the mathematical model as well as other hydraulic and construction considerations have led to the conclusion that this method of closure is not feasible in the prevailing circumstances.

Main reasons for arriving at this conclusion are the following:

(a) Mainly because of the natural high elevation of the river bed at the site of closure it is not possible to narrow the natural river channel substantially (say to more than half of its original width) prior to closing the final gap.

(b) Narrowing as far as possible, say from 1160 to 700 m would necessitate the introduction of expensive flow guiding devices to prevent so-called vortex streets causing heavy damage to bed protection and/or deep scour holes. Moreover, even the gradual concentration of flow followed by spreading after passing the narrowest point would lead to a heavy attack of the bed protection possibly resulting in scour holes of considerable depth which in turn would adversely affect the risk of closure (+) also bearing in mind the geo-technical condition of the sub-soil.

(c) To close the final gap, of say 700 m, heavy elements with individual weights of approximately 450 kg would be required. These cannot be placed by manual labour.

Therefore costly infrastructural works or placing methods would be required (*) (for instance the construction of a cable way car or placing by helicopters), to bring the elements in the final closure gap. Moreover, the voids within the elements would require difficult sealing measures, for which only a very limited amount of time would be available.

(d) The limited working space and long haulage routes for materials would make the whole closure operation a very hazardous one.

(*) Application of a jetty (walk-way own coffer dam as used for the much smaller Chakamaya and Antali closures) is not feasible because of the consequential increase in width of final closure gap and the heavy structure required for trucking operations.

(+) For a discussion on scour see also Annex A.
4.3.7 Second approach to closure.

As discussed in Section 4.2.6, narrowing of the closure gap prior to final closure leads to unfavourable and undesirable hydraulic and to some extent, geotechnical, conditions. A new approach has subsequently been made whereby no initial narrowing of the closure section is envisaged.

In this approach the closure has to be effected over the full width in a single operation during a very short time i.e., in a six hour operation during neap tide.

From tidal records it can be concluded that in February neap tides occur with high waters not exceeding SGB + 1.30 to + 1.40. Such neap tides however are rare, and it would not be prudent to assume such a tide for design purposes. Tides with high waters below SGB + 1.30 occur more frequently and this tide will be used for design purposes.

With the level of the shoal (including bed protection) at SGB + 0.70, the largest water depth above the shoal is only 1.10 m. Within a few hours after high water this height will reduce to approx. 0.30 to 0.40 m, which would render it possible for labourers to walk over the bed protection and to deposit clay filled bags in the closure gap. The bags should form a small neap tide dam with an elevation of approx. SGB + 2.50 m, which should be able to withstand the next high water (neap tide).

As the gullies are deeper than the shoal, the water depth would be larger. This would result in difficult access for labourers (water depth to approximate 0.50 m), and higher current velocities up to 2.5 m/s during closure.

To avoid these conditions it would be necessary to raise the level in the gullies to match the level of the shoal (including bed protection), so that uniform flow conditions will exist over the whole width of the closure gap. This is achieved by dumping after placing the initial bed protection, boulders (gabions) in the gullies in order to create a sill of sufficient width and elevation to build a neap tide dam.

Closing in a six hour period requires short walking distances and the possibility to employ a large number of labourers. Closing from the two river banks would be too restrictive: There would not be enough working space and the walking distances would become too long. To overcome this problem a number of stockpiles have to be created at regular intervals in the closure gap. These stockpiles should not form a major obstruction to the currents.

To stockpile the required amount of clay filled bags nine stockpiles, in the form of bridge piers (length 60 m, width 10 m) are constructed parallel to the current.

A potential problem forms the construction of the winter spring tide dam following the construction of the neap tide dam. Huge amounts of clay have to be transported and deposited in place within only four days time, so that the following spring tide can be successfully "held at bay". In Chapter 5, it has been indicated how this filling operation could be completed successfully with lorries and labourers.
4.3.8 Results of computer runs

As the dimensions of the closure gap are more or less fixed, the number of runs required to determine the maximum velocities in the closure gap is limited. The following table gives the most important results of the various runs.

Table 3  Current velocities and volume of tidal prism for typical tides (effective width 963 m, sill at SCB + 0.70 m)

<table>
<thead>
<tr>
<th>Tide (ref., SCB + m)</th>
<th>Run No.</th>
<th>Vmax flood (m/s)</th>
<th>Vmax ebb (m/s)</th>
<th>Volume of tidal prism ($10^6$ m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>+ 4.60</td>
<td>59</td>
<td>2.08</td>
<td>1.74</td>
<td>46.0</td>
</tr>
<tr>
<td>+ 3.20</td>
<td>50</td>
<td>1.65</td>
<td>1.28</td>
<td>17.6</td>
</tr>
<tr>
<td>+ 2.40</td>
<td>53</td>
<td>1.00</td>
<td>0.85</td>
<td>7.1</td>
</tr>
<tr>
<td>+ 1.80</td>
<td>52</td>
<td>0.78</td>
<td>0.55</td>
<td>3.9</td>
</tr>
<tr>
<td>+ 3.20*</td>
<td>51</td>
<td>2.05</td>
<td>2.17</td>
<td>12.2</td>
</tr>
</tbody>
</table>

In all computer runs the oed protection on the shoals was supposed to be at a level of SCB + 0.70 m while the sill in the gullies also has been assumed to be at a level of SCB + 0.70 m over a length (perpendicular to axis of dam) of 50 metres.

Effective width of the closure gap was calculated at 963 m(+) allowing for an overall width of the estuary of 1160 m and "obstruction" by nine stockpiles having a width of 10 m each.

4.3.9 Use of regulator during closure operations

As stated in Section 4.3.8 actual closure would take place during a neap tide. As it is essential to have a maximum outflow to the sea to reduce the water level on the sill / matresses the regulator must be utilized. It is therefore proposed to open the regulator immediately after high water. For calculation purposes this high water level is assumed to be SCB + 1.80 m (sill level is at SCB + 0.70 m). To increase the discharge already at an early stage the heavy flaps gates should be opened, for instance, by using hand winches.

Details of the hydraulic calculations are presented in Annex A, Section 6. The results of these calculations have been summarized in Figure 4. It can be seen that without the use of the regulator the water level on the sill, after an initial lowering, would go up to SCB 1.30 m (which is waterdepth 0.60 m) while thanks to discharge through the regulator the water levels would godown to the level of the sill and even lower.

* For calculation of scour (See section 7 of Annex A, this run has been made for a narrowed closure gap.

(+) Of this 963m width 333 m is located in the gullies and 630 m on the shoals.
Discharge through Regulator and over sill during construction of Neap tide dam.

Opening of regulator

Neaptide dam closed.

Water level above sill without use of regulator.

Water level above sill with use of regulator.

Water depth above sill

Level of sill in River

Water level at sea

Level of sill in Regulator

508

hours after high water

300

Total discharge

dam closed.

discharge through regulator

regulator open

(m3/s)

0 1 2 3 4 5 6 7

hours after high water

Figure - 4
4.3.10 Main reasons for accepting method developed during second approach to closure.

The following are the main reasons for further developing the closure method discussed in Sections 4.3.7 to 4.3.9 (incl.)

(a) The fact that narrowing prior to closure will not be done, results in relatively low current velocities in the closure gap, for which a relatively light bed protection can be used, while also its length perpendicular to dam axis can be smaller.

(b) As a mat under the dam is also required for geotechnical reasons, the bed protection mattress over the full width of the river would serve for this purpose as well.

(c) Only moderate scour would result from the closure works. Apart from decreased risks during closure, this is also desirable from a geotechnical point of view.

(d) No materials or (temporary) structures are used which do not have a function in the final dam profile.

(e) No heavy elements are required to stop the flow in the final closure gap.
4.4 SEA BED PROTECTION AND SILL

4.4.1 General

In Section 4.3.2 it has been explained why the bed of the shoals and gullies in the estuary at, and in the direct neighbourhood of, the axis of closure has to be protected against scour. If no protection of the tidal channel would be made it would not be possible to construct a dam across the Feni River.

In fact the bed protection can only prevent scour there where it is actually made. It is known that scour holes will form immediately outside the protected zone. Though the number of available data is very limited, the possible depth scour holes may reach has, tentatively, been calculated. On the basis of this calculation and bearing in mind the liquefaction potential of the fine, loosely packed sands of the channel bed, it was decided to protect the channel bed (shoals and gullies) over a width of 150 m (perpendicular to axis of closure) over the full length (1160 m) of the closure works across the tidal channel.

In this manner scour holes cannot endanger the area where actual closure works (sill, stockpiles, neap tide dam, spring tide dam) will take place.

The bed protection will, apart from prevention of scour during construction, also have a task in the functioning of the completed dam. As has been explained in Section 4.2 the bed protection can serve as tension string underneath the dam in order to decrease the risk of loss of stability due to liquefaction due to earthquakes.

The required tensile strength is 40 kN/m in both directions.

4.4.2 Bed protection

To prevent erosion (scour) of the underlying loosely packed soils (silt and fine sand) the bed protection must function as a filter layer. This can be achieved by introducing a fabric (a so-called geo-textile) having a maximum pore size equal to the D90 of the bed material. This relationship between pore of a filter fabric and grainsize distribution of the bed material to be prevented from washing out was established after extensive model testing in the Delft Hydraulics laboratory. In Table B-3 (Annex B) the grainsize distribution of samples of bed material collected during March/April 1983 is summarized.

By taking the lowest D90 found the maximum pore size of the filter fabric is defined at 0.09 mm. This requirement can only be met by a synthetic non-woven membrane. This membrane has to be combined with a woven filter - fabric in view of the low strength and vulnerability for damages of the former. The filter fabric must be strong enough to render the tensile strength mentioned in Section 4.4.1.
The geo-textile combination of non-woven and woven filter fabric must be fixed to the sea bed by means of ballast. The weight of this ballast is related to current velocities expected and turbulence. Maximum current velocities for uniform flow over the bed protection were calculated on the basis of the boundary limits (sea side and reservoir side of protection) computed by means of the mathematical model. Details of these calculations are presented in Annex C of this Report. It was found that the maximum current velocity to be expected is 2.1 m/s in the gullies at locations near the sill (see Section 4.4.3) and 1.5 m/s on the shoals. Consequently, it was decided to utilize river boulders having a specific density of 2.65 (2650 kg/m³).

**On the shoals:**

These boulders should have a D50 = 0.10 m (size varying between 3 and 12 cm) and the average ballast weight should be 250 kg/m². These round and relatively small boulders will only stay in place when covered by wire mesh which in turn is tied to the filter fabric by means of nylon ropes. In this manner a stable protective layer can be established. As the shoals become dry and accessible during considerable periods the wire mesh can be laid out and fixed to the filter fabric without any difficulty.

The mesh size of the wire mesh must be smaller than the smallest size boulder to be used. Wire mesh must be of 3 mm diameter, galvanized mild steel.

**In the gullies:**

Current velocities will be higher than on the shoals and, consequently, boulders having a diameter of D50 = 0.20 m are required while size may vary between 10 and 30 cm. Because of the impossibility to apply wire mesh here the average ballast weight is increased to 400 kg/m² in order to allow for losses and a possible uneven distribution. Moreover, reed rolls of 0.30 m diameter will be provided on the filter fabric in lines parallel to the dam axis at 1.50 m centre to centre to prevent movement of boulders by the current. These reed rolls must be connected to the filter layer in such manner that a tensile force of 2000 N per m length can be transferred.

The bed protection must extend on to the banks of the tidal river. Here the length is increased by 50 m in u/s and d/s direction (overall length 250 m). The composition of the bed protection will be the same but additional reed rolls are introduced (perpendicular to dam axis) in order to prevent rolling down of boulders. Wire mesh will be placed from a level of 50 m + 0.0 m upwards. This provision is made to avoid damage to the protection works in working areas. Prior to placing of bed protection the banks must be levelled to slopes not steeper than 1 in 3.
4.4.3 Design and construction of sill:

It is recalled that bed protection is foreseen over the full river width in a zone of 150m.

In Section 4.3.7 a sill was introduced having a width of 60 m and an elevation of SOB + 0.70 m as part of the adopted closure method.

On the shoals the top of the bed protection will generally reach a level of SOB + 0.70 m, but may locally be situated 0.10 to 0.40 m lower. These deeper spots should be raised to match the general level of SOB + 0.70 m. Filling out should be by (hand) placing of boulders or bricks. If the boulders are not heavy enough to withstand the current forces, they should be covered with wire mesh which should be connected to the bed protection mattress. Transition slopes should not exceed 1:3.

In the gullies the top of the bed protection in general is at SOB + 0.0 m while the lowest point will be at SOB - 0.30 m. To construct a sill over a width of 80 m and a level of SOB + 0.70 m it will be necessary to dump boulders to the desired sill level.

The maximum current velocity on top of the sill in the gullies will not exceed 2.7 m/s (see Annex C). However, this current velocity will only occur during a very short period. Current velocities of about 2 m/s are expected more frequently.

It was therefore decided to use boulders having a size D_{50} = 0.20 (sizes 10 to 30 m). If required clusters of boulders can be made by using nylon rope nets (in earlier studies called "gabions")

4.4.4 Sealing works.

Sealing works of bed and sill will be required to prevent piping underneath the completed nesptide dam, spring tide dam and main dam. It will be dealt with in other sections concerning the items mentioned.
4.5 NEAP TIDE DAM (DESIGN)

The function of the neap tide dam is to stop the tidal flow when tidal prism is at its lowest i.e., during a neap tide.

Prior to start of construction of the dam, grout seal I has to be made. This implies that over the full length of the sill the boulder layer is grouted by means of a sand - cement grout in order to prevent piping after completion of the neap tide dam (see also Section 4.2.3.1).

At the start of building the dam, no time will be available to remove the ballast, etc., on the sill. Instead thereof sanding of the ballast layer at the reservoir side of the dam location is foreseen.

The neap tide dam is located at 40 m distance of edge of bed protection in view of possible settlement of the bed protection due to scour holes (liquefaction).

The dam must be built up in such manner that slopes of 1 in 1 (sea side) and 1 in 3 (reservoir side) are constructed while the sill is planned at elevation SEC + 2.50 m (width 3.0 m); the elevation allows for some settlement to occur, (0.15 m has been calculated).

A point of concern are still the bags to be used: Only jute bags (gunny bags) will provide the friction required to accept a gradient of 1 in 3 for the reservoir side slope. (Section 4.2.3.4).

In case polyethylene bags have to be used either a flatter slope is required at the reservoir side or driving on the crest by lorries has to be postponed until the crest has been widened or to be limited to periods with small differences in water level between both sides of the dam, or finally, the dam has to be "prestressed" by means of bamboo frames and nylon ropes.

It is essential that the neap tide dam is constructed by using good quality clay.
SPRING TIDE DAM. (DESIGN)

The winter spring tide dam (hereinafter called WSTD) has two functions:

(a) To prevent spring tides from entering the Peni River during the construction of the main dam profile.

(b) To retain a water retaining structure after completion in case of an earthquake followed by liquefaction of the main part of the dam body consisting of silt.

This double function determines the location, the slopes and the construction material of the WSTD.

By locating the WSTD as much as possible at the sea side, it is possible to place the slope protection on the WSTD slope and to start doing this immediately after its completion and prior to significant deposition of loosely packed layers of sediments.

The slopes of the WSTD are determined bearing in mind:

- slopes of sea side main dam (1 in 4)
- high water level spring tide (SCB + 5.0 m)
- low water level reservoir side (SCB + 0.0)
- traffic by lorries on crest of WSTD.

This resulted in a reservoir side slope of 1 in 2½ (Section 4.2.3.3).

Essential for the stability of WSTD are the careful execution of grout seal I and the fabric to be placed over the neap tide dam body (Sections 4.2.3.1 and 5.4).

Settlements have been calculated at 0.30 m which leads to a crest level at time of construction of SCB + 5.50 m.

On the sea side a layer of clay bags is placed to prevent washing out of freshly placed clay layers.

The WSTD will be completely built up of clay. The specification was already presented in Section 4.2.5.
4.7 KAINI DAM THROUGH ESTUARY.

4.7.1 General.

When the actual closure works as described in Sections 4.2 to 4.6 (inclusive) have been carried out the more permanent dam structure has to be constructed. Final profile of this dam is governed by various reasons. The main points are the following:

- Expected (high) water levels at sea and in the Poni Reservoir.
- Wave attack and wave run-up.
- Geotechnical considerations as presented in Sections 2.2 and 4.2 which are based on availability and characteristics of building materials, sub-soil characteristics, earthquake statistics in the Region, water levels and waves.
- Required transport facilities along the crest of the dam.
- Maintenance requirements for completed works.

In the following sections the design of various parts of the dam will be discussed bearing in mind the main points mentioned. Basic data on water levels, wind and waves as well as subsequent calculations based on these data are presented in Annex D to this Report.

4.7.2 General cross-section of dam.

Hydraulic and geo-technical considerations determine the overall profile of the dam. The application of woven and non-woven fabric under the base of the dam allows slopes of 1 in 3½ on both sides.

For practical reasons it was decided to apply slopes of 1 in 4. Only the upper part of the sea side slope (above SCB + 6.00 m) will have a flatter slope in order to reduce wave run-up and, consequently, the elevation of the crest of the dam.

As far as the lower part of the sea side slope is concerned one should bear in mind that accretion will take place after completion of closure works.

It is in fact expected that in the near future a major part of this slope will be covered by sediments.

At the reservoir side a berm having a width of 4 metres is foreseen as a transfer zone between two different types of slope protection (rip rap and revetment).
By taking into account construction, hydraulic and geo-technical aspects it has been assured that stability and redundancy of the dam will be as good as possible under the circumstances. In order to achieve this the cross-section has been designed as follows:

(a) The (clay) spring tide dam is located at the sea side of the main dam.

(b) A small clay dam is constructed after closure against the lower slope of the reservoir side of the main dam.

(c) In between both dams the main dam body of silty sand and silt will be built up in a careful manner (see Section 4.2.5).

The road on the crest of the dam has a width which is in line with earlier designs and the width of the road from Feni to the regulator. A surfaced width is provided of 4 metres. This road can be used also for inspection and maintenance. The gradient of the slopes and types of revetment allow the access of lorries on to the slopes for maintenance purposes.

4.7.3 Crest of dam.

(a) Level.

It was considered by the Consultant that the Regulator, flank embankments and the main dam across the estuary all act as one high water protecting system. Consequently, the elevation of the dam crest should be at least the same as that of the regulator crest (i.e. S0B + 9.00 m).

The crest elevation of the main dam is determined by the design water level, say S0B + 8.1 m (which level has a return period of 50 years), and a design wave height of 1.7 m as calculated by IECO *(1).*

Wave run-up calculations (Annex D) result in a crest elevation of S0B + 10.00 m (after settlements of subsoil and dam body).

For lower extreme water levels also the wave run-up will be reduced. This phenomenon gives the main dam an extra redundancy which is a positive feature for this relatively exposed part of the afore mentioned high water protection system.

It is pointed out that the value of the design water level (S0B + 8.1 m) probably is too high by 0.7 m because of possible inaccuracies in high water level measurements. However, this range (0.7 m) can be used for local wind set-up which, according to calculations will be in the order of 0.2 to 0.3 m.

Finally, it should be mentioned that Consultant has studied the configuration of the estuary and adjacent tidal coastal
waters and based on this data does not expect higher water levels in the mouth of the estuary as a result of reduction of storage due to closure of the Fen River.

(b) **Width**

Crest width is determined by the surfaced width of the road (4 m) and its verges (each 1 m). So overall width is 6 m.

Road surfacing consists of a layer of bricks (0.10 m) a layer of chipped bricks (0.15 m) and crushed boulders penetrated by tar (0.07 m). On the verges concrete blocks are laid as applied in the revetment of the seaward slope (upper part).

4.7.4 **Slope protection.**

(a) **Sea side**

Along the slope wave loads and their duration varies. In a range around the higher water levels, say from SGB + 5 m to SGB + 9 m wave attack is relatively high. Only concrete blocks containing as aggregate crushed boulders and sand (400 mm) do have the required specific density ($\rho_b = 2250 \text{ kg/m}^3$). It is recalled that specific density for concrete blocks, made by using chipped bricks as aggregate, amounts to $\rho_b = 1850 \text{ kg/m}^3$ only. For so-called brick blocks the specific density is even lower ($1600 \text{ kg/m}^3$).

Consequently, concrete blocks (2250 kg/m$^3$) are selected, size will be 0.50 x 0.50 x 0.25.

For relevant calculations reference is made to Annex D.

For the revetment on the slope below SGB + 5.00 m down to SGB + 2.00 m concrete blocks ($\rho_b = 1850 \text{ kg/m}^3$) were selected having a size of 0.50 x 0.50 x 0.20.

This type of revetment is also applied between SGB + 9.00 m and the crest as well as for the earlier mentioned road berms.

Because of early sedimentation expected in front of the dam below SGB + 2.00 m brick blocks (Size 0.50 x 0.50 x 0.25), will be placed down to the toe beam.

Revetment blocks will be placed on a layer of crushed brick thick 0.15 m which in turn is placed on a filter fabric having a pore size of 0.09 mm. Between filter fabric and main dam body a 0.60 m thick layer of clay is foreseen.
(b) **Reservoir side.**

In Annex D it has been demonstrated that wave attack is concentrated between SCC + 1.50 m and SCC + 5.1 m while wave run-up is limited to levels between SCC + 3.5 and SCC + 6.1 m. As the lower part of the slope (below SCC + 3.0 m) has to be constructed under water it is proposed to apply mattresses of a design similar to the ones described in Section 4.7.4. These mattresses will be ballasted using rip rap, \( D_{50} = 0.30 \) m. Layer thickness will be 0.55 m.

From a transmission berm (width 4 m, level SCC + 3.00 - SCC + 3.50 m) up to a level of SCC + 6 m a revetment will be placed as described for the seaward slope. Concrete blocks (\( \rho_b = 1850 \) kg/m³) have been selected.

Between SCC + 6.0 m and the dam crest only erosion by rain and (seldom) overtopping by waves is expected. Consequently, a light protection of bricks (thick 0.10 m) will do. Clay layer under this reservoir side slopes has a thickness of 0.40 m.

4.7.5 **Toes.**

(a) **Sea Side.**

In order to ensure the stability of the block revetment and to establish a flexible transmission to the bed protection an adequate toe structure has to be introduced.

A continuous concrete beam (aggregates: chipped bricks, sand) having vertical joints at every metre has been selected.

A horizontal layer of boulders \( D_{50} = 0.20 \) m is placed adjacent to the beam over a width of 5 m.

The beams can best be cast in situ on a solid mass of boulders grouted with sand - cement mortar.

Grouting should be applied over a width of 1 m along the beam.

(b) **Reservoir side**

The mattress mentioned in Section 4.7.4 under (b) serves also as toe at the berm level of SCC + 3.50 m. The mattress slopes down on to the bed protection where it ends in a horizontal layer of 5 m length.
4.8 FLANK EMBANKMENTS

4.8.1 General

The principal function of the flank embankments is to connect the main dam across the Peri River with the Regulator structure (Right Bank) and the coastal embankment (Left Bank).

Moreover, the flank embankments carry a roadway which crosses regulator and main dam.

Crest elevation of flank embankments will be at SOD + 9.00 m which means that there will be a transition zone between crest of flank embankments and crest of main dam (SOD + 10.00 m).

4.8.2 General cross-section of embankments.

As the flank embankments are located on higher ground (SOD + 3 m to SOD + 4.5 m) the height is much lower than the height of the main dam.

Crest width is the same as for the main dam, (6 m) while the gradient of the slopes was taken as 1 in 4 for the reservoir side and 1 in 6 for the sea side. However, the parts of the embankments lying in one line with the main dam axis have the same slopes (1 in 6, 1 in 4, transition point at SOD + 6 m) as the main dam.

No berms are foreseen.

4.8.3 Flume protection.

For calculation of size of revetment blocks reference is made to Annex D.

At the sea side the revetment in general is similar to the one designed for the main dam with the exception of the embankment section parallel to the Regulator Inlet canal.

The latter section is not exposed to heavy wave attack. It was decided to apply here a "local" construction consisting of fine and coarse filter layers and brick blocks 0.50 x 0.50 x 0.25. No fabric will be applied underneath.

At the reservoir side a revetment of bricks placed on edge (thickness 0.10 m) is foreseen throughout.

As the dam body will consist completely of silt and silty sand Consultant considered it prudent to place a clay layer thick 0.6 m (sea side) and 0.4 m (reservoir side) between dam body and revetments.
Except for the sea side part of the embankment stretch, parallel to the Regulator inlet canal, fabric is placed throughout between clay and revetments.

4.3.4 **Toes.**

At the toe of all slopes wooden poles (Sundary wood) will be placed (diameter 0.10 m, length 1.50 m). This will prevent the revetment from sliding off by supporting the first row of concrete blocks/brick blocks/bricks.

At the toe (sea side) of all slopes mattresses will be placed (width 5 m), ballasted with boulders having a D<sub>50</sub> = 0.20 m.

A special situation exists near the Regulator. Here, wave attack is heavy and already now part of the fore shore has disappeared. It is the intention to place mattresses covered by boulders (D<sub>50</sub> = 0.25 m see Annex D) on the foreshore in the bend of the embankment up to the connection with the wing wall of the Regulator.
CHAPTER FIVE

CONSTRUCTION OF PENI RIVER CLOSURE DAM

5.1 INTRODUCTION

The information contained in this chapter is presented first of all to give interested parties an idea of how closure works can be carried out to achieve the desired result. A second aim is to point out critical issues which are typical for Peni River like tidal differences, subsoil conditions and which will have to be looked into very seriously for any closure method proposed.

Closure method and programme proposed can be adapted:

(a) If contractors equipment, availability of materials or envisaged construction procedures render this possible, always provided the basic principles adopted for closure are not touched and Client and Consultant do agree with the modification proposed.

(b) When the development of scour holes during the construction period require this.
5.2 **BED PROTECTION AND SILL.**

5.2.1 **General.**

Composition of mattresses developed for placing as bed protection or as sill together with tidal water levels and ground level determine the construction procedures to be followed.

It is recalled that composition of mattresses was determined by:

- sand tightness requirement;
- function as tensile string underneath the dam;
- need to retain the ballast on the mattress;

At present it is envisaged that the bank/bed/sill protection is placed in the following order:

1. Bed protection on the banks;
2. bed protection on the shoals starting with centre and going in two directions;
3. bed protection in the gullies starting with all mattresses in one gully on the sea side;
4. dumping of boulders in the gullies;
5. sill works on the shoals including wire mesh.

5.2.2 **Bed protection on shoals.**

Elevation of shoals differs between SCB + 0.20 and + 0.50 m. This implies that the tide leaves the shoals dry for several hours during each day. Therefore it is possible to construct the bed protection in the dry. Main problem will be the transport of materials. These could best be transported around high water slack.

A point of interest is the transfer of the tensile strength of 40 kN in the joints. A minimum number of joints is appreciated. This applies to the joints in both directions. Consequently, it has been decided to accept only mattresses having minimum dimensions of 24 x 50 m² (*). (excluding extra lengths for jointing).

Larger mattresses would be possible if it can be demonstrated that handling problem can be solved. Most probably mattresses can easiest be transported by floating them into place followed by waiting for the moment that they are stranded. Floating mattresses require a certain floating capacity and stiffness. This can be created by means of reed rolls and bamboo which are connected to the fabric by means of loops.

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**Note:** *) The synthetic fabric can usually be supplied in a width of five metres and in any length. Joints between five metre wide breaths can be made using special stitching machines.
Need rolls also create a certain minimum thickness of the stone covered mattress and provide a place to fix the wire-mesh cover.

It is also possible to start ballasting the mattress during high water slack and to continue till the whole mattress has been ballasted.

The mattress must have strengthened edges to allow for anchor ropes, etc.

Prefabrication of the mattress can be on a platform situated above high water level from which it can be pulled into the water.

Transport of mattress and ballast material must be by barges and tugs.

Because of the required silt/sand tightness of the joints overlaps between mattresses of:

- two metres for joints perpendicular to dam axis;
- four metres for joints parallel to dam axis; are foreseen.

An overlapping part of an already placed mattress must be cleared of stones and bamboos, leaving only the fabric before the next mattress is sunk.

The wire mesh must be placed as quickly as possible after placing the ballast.

Scour holes which develop at the edges of a placed mattress must be filled before placing the next mattress in such way that the overlapping part is as much level as possible and slopes of scour holes are not steeper than 1 in 3.

5.2.3 Bed protection in the sullies.

There will always remain water in the sullies so the bed protection cannot be constructed in the dry with the exception of the higher part of the bank protection. Consequently, ballasting and jointing has to be done under water. Tensile stresses at joints can be avoided by not having joints in the areas where a tensile string is required. By splitting up the 150 m length of protection into three equal parts this is feasible. Also therefore mattress size adopted is 24 x 50 m^2.

Silt - Sand tightness in the joints is essential. Because of the impossibility to work in a dry area this "Jointing" will require much attention.

Execution of protection works must start from the sea side row of mattresses onwards. Ballasting by the required weight of 400 kg/m^2 can be done, if desired, in two stages, provided it is done within one to two days.
Reed rolls high 0.30 m, placed in transverse direction, are required to increase stability of the ballast.

As for the bed protection on the shoals scour holes must be identified and treated in order to minimize abrupt changes in the protected estuary bed.

5.2.4 Bank Protection

The bank protection works will consist of small mattresses having a nett width of 25 m and a length of 23 m. The upper part of the mattress must give protection up to 4.00 m + S0B. The composition of the mattress is in principle the same as for the bed protection mattresses in the gully. The longest side is however placed parallel to the centre line of the dam axis. Reed rolls having a diameter of 0.15 m will be placed in both directions.

The top of the mattress can be made on the spot during low water after sinking of the mattress. The filter fabric of that part can be kept in rolled position during transport to the spot.

The joints perpendicular to the bank must be made by means of an overlap of 1 m. The overlap of the proceeding mattress should not be ballasted with stone and should have no bamboo. Placing mattresses should start from the sea side. Scour holes at the toe must be filled in such a way that a slope of maximum 1:3 is available at start of placing the mattress. The bed protection mattresses in the gully are sunk on top of the bank mattress by overlapping with 4 m.

Wire mesh cover is applied to bank mattresses above a level of S0B 0.00.

5.2.5 Sill

The sill in the gullies is constructed by dumping boulders till the required elevation (S0B + 0.70 m) is reached. In view of current velocities and expected difficulties in obtaining large boulders it is envisaged to apply nylon nets filled with boulders. Nets can be made to match the required size by using nylon ropes.

As an alternative to boulders sand-cement blocks can be considered.

Last but not least, the sill on the shoals must be constructed. Gabion type boxes can be placed where required and filled up with bricks, boulders etc. in order to obtain the correct height of S0B + 0.70 m. The wire mesh must be connected correctly to the under lying bed protection.

Construction of the sill on the shoal must be done in shallow water depths to obtain the desired accuracy of levels.
5.3 **NEAP TIDE DAM (CONSTRUCTION)**

5.3.1 **Stockpiles.**

Materials for the neap tide dam have to be stockpiled on nine or more stockpiles in the alignment of the closure dam.

Number, dimensions and shape of stockpiles will have to be agreed between contractor and consultant prior to start of stockpiling.

Stockpiles will be placed on bamboo rafts. Protection of materials stockpiled might be required and could also consist of bamboo placed in screens around the stockpiles.

5.3.2 **Fill material.**

Fill material for the Neap Tide Dam should be clay, with at least 15% clay minerals. The clay should be filled in bags which are sufficiently resistant against decay during filling operations and storage (lasting together approximately three months). As gummy bags have insufficient resistance against decay, plastic bags are proposed at present which allow air and water to pass in and out of the bags. Woven polyethylene bags having a capacity of approximately 50 kg would serve this purpose.

It is recognized that friction resistance of polyethylene bags is low. Consultant, therefore, intends to check whether it is possible to apply a conservation process to gummy bags in order to prevent or delay decay.

5.3.3 **Closure operations.**

As the time for closure is very limited (approximately 6 to 7 hours) it is essential that sufficient working space is created, so that labourers do not hinder one another.

One way to achieve this is to work simultaneously on a wide front. Groups of labourers would have to be instructed to follow specific paths over the bed protection. In view of the regular pattern of fascines and "side walk" blocks, they would have to follow certain track rather than taking the most direct route. The contractor should however be free to propose his own plan for building up the neap tide dam.

Approximately 10,000 labourers will be required for the closure operations.

It is essential that the regulator flap gates are opened as soon as possible after high water slack, in order to reduce the water height above the sill as much as possible. As the water level differences are very small, the flap gates will not open automatically, but have to be opened manually, for instance by using hand winches.
5.4 WINTER SPRING TIDE DAM (CONSTRUCTION)

Prior to start of filling operations and immediately after completion of neap tide dam to design profile the following operations must take place:

- make part of sill to be covered by winter spring tide dam body free from all stones, bamboo and reed rolls;

- place fabric mentioned in Section 4.2.3.1 from grout seal I over neap tide dam body (after sanding in) onto the cleared fabric of the bed protection;

- place steel plates on top of fabric (after sanding) in order to spread axle loads of vehicles.

The double function of WSTD implies that in total 120000 m$^3$ clay has to be placed. However, during the short period lapsing between neap tide and spring tide only 48 000 m$^3$ of this quantity has to be placed, i.e. 44000 m$^3$ in bulk and 4000 in clay bags. This quantity of 48000 will still require work during 3 shifts as only 4 days and 5 nights are available.

The exact construction procedures to be followed will have to be agreed by Contractor and Consultant as geo-technical considerations (loading of the sub-soil) as well as hydraulic considerations play a role.

Approximately 175 lorries will be required at this stage, as well as three shifts of 2500 labourers each. Construction of the winter spring tide dam requires a perfect organisation of equipment and labourers and is the most critical part of the whole closure dam works. During building up of the spring tide dam it will be necessary to (manually) open the flap gates of the regulator in order to release the inflow into the estuary (through Feni and Muhuri rivers), so that the sill will remain dry at the reservoir side of the neap tide dam.
5.5 MAIN DAM (CONSTRUCTION)

Construction of the main dam starts immediately after completion of the winter spring tide dam (SS23).

The various design features have been discussed in detail in Section 4.7 and will not be repeated here.

Main features of the construction are:

- the time element: only four months are available for filling operations (450,000 m³) and placing of slope protection (87,000 m²) *;
- the quality of fill placing required in the tidal zone and especially at base level;
- the placing of slope protection in the tidal zone (sea side).

In order to facilitate construction of the lower part of the dam body it is essential that reservoir level is kept as low as possible during construction.

For construction as well as for geo-technical reasons a clay dam has been planned at the reservoir side.

The sequence of construction proposed is now as follows (after completion of the SS23):

- construct grout seal No 2 (to stop piping under clay dam mentioned above);
- remove all boulders bamboo, etc., from bed protection below future clay dam;
- construct clay dam;
- remove (if vertical piping permits) all or nearly all boulders etc., from bed protection between the SS23 and the clay dam;
- start fill placing.

In case of piping under one of the clay dams upstream, dumping of fines normally stops seepage rapidly.

For requirements to be imposed on filling operations, compaction, type of equipment to be used, etc. reference is made to Section 4.2.3.1, 4.2.4 and 4.2.5.

---

Note: *) These quantities are for main dam only. Blank embankments will require 290,000 m³ of fill and 116,000 m² of slope protection works.
5.6 MATERIALS TO BE INCORPORATED IN THE WORKS.

Most of the materials to be incorporated in the works have to be provided and transported by the Contractor who will be awarded the contract for the construction of Fenil River Closure Dam.

These materials have been listed in Table 4. The quantities are for information only.

It is noted that the contractor will be entitled to duty free importation in line with World Bank Guide lines. Consequently, EWDE will have to make arrangements in time with Customs or alternatively, have the required funds available in time to avoid any delay to arrival of materials from outside Bangladesh on the Site of the works.

<table>
<thead>
<tr>
<th>Ref No.</th>
<th>Type of material</th>
<th>Quantity</th>
<th>Weight in tons.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Boulder Ø 30 cm</td>
<td>7,200 t</td>
<td>7,200</td>
</tr>
<tr>
<td>2</td>
<td>Boulder Ø 20 and 25</td>
<td>85,500 t</td>
<td>85,500</td>
</tr>
<tr>
<td>3</td>
<td>Boulder Ø 10 cm</td>
<td>47,000 t</td>
<td>47,000</td>
</tr>
<tr>
<td>4</td>
<td>Gravel (crushed boulders)</td>
<td>22 x 10^6 nos.</td>
<td>72,000</td>
</tr>
<tr>
<td>5</td>
<td>Sand 400 mm</td>
<td>22,300 m^3</td>
<td>35,700</td>
</tr>
<tr>
<td>6</td>
<td>Clay</td>
<td>245,000 m^3</td>
<td>443,000</td>
</tr>
<tr>
<td>7</td>
<td>Cement</td>
<td>13,500 m^3</td>
<td>13,500</td>
</tr>
<tr>
<td>8</td>
<td>Bitumen</td>
<td>500 t</td>
<td>500</td>
</tr>
<tr>
<td>9</td>
<td>Reed bundles</td>
<td>150,000 Nos.</td>
<td>500</td>
</tr>
<tr>
<td>10</td>
<td>Sandary wood Ø 45 cm</td>
<td>41,000 Nos.</td>
<td>400</td>
</tr>
<tr>
<td>11</td>
<td>Full bamboo Ø 6.5 cm</td>
<td>900,000 Nos.</td>
<td>900</td>
</tr>
<tr>
<td>12</td>
<td>Half Dawn bamboo Ø 6.5</td>
<td>750,000 Nos.</td>
<td>750</td>
</tr>
<tr>
<td>13</td>
<td>Filter fabric</td>
<td>486,000 m^2</td>
<td>120</td>
</tr>
<tr>
<td>14</td>
<td>Bags for clay</td>
<td>960,000 Nos.</td>
<td>60</td>
</tr>
<tr>
<td>15</td>
<td>Wire mesh or equivalent</td>
<td>140,000 m^2</td>
<td>520</td>
</tr>
<tr>
<td>16</td>
<td>Nylon rope 1000 N</td>
<td>600,000 m</td>
<td>3</td>
</tr>
<tr>
<td>17</td>
<td>Nylon rope 2000 N</td>
<td>700,000 m</td>
<td>6</td>
</tr>
</tbody>
</table>

Apart from the materials listed in Table 4, silt and silty sand will be required for incorporation in the works. Part of the required quantities are already stockpiled on site (420,000 m^3 at Right Bank). The remainder will have to come from borrow areas...
on Right and Left Bank. It is understood that BWDB will take
care of land acquisition and right of way. Quantities of silt/
silty sand required *) are listed for information only:

Main dam 390,000 m$^3$
Flank embankment Right bank 208,000 m$^3$
Flank embankment Left bank 41,000 m$^3$

Note: *) including quantities already stockpiled.
CHAPTER SIX

PROGRAMME OF WORKS

6.1 GENERAL

When drawing up a construction programme for the Peni River Closure Dam it must be realised that mainly so-called hydraulic engineering works are concerned which have to be carried out in the tidal zone, in a relatively short period and by means of manual labour. In fact the elements construction method, building materials, equipment, labour force and time planning are so closely related to each other that minor deviations from, for instance "time planning" or "availability of equipment" can jeopardise the whole works programme and result in loss of tens of acres of Takas and postponement of completion by a full year.

Bearing this in mind it must be clear that certain operations of the works can only be started by the contractor when their timely completion is guaranteed through the elements: suitable working season, labour, equipment and materials while the working method must have been studied thoroughly before hand taking into account possible mishaps which always can occur during the implementation of this type of works.

6.2 KEYDATES

In Figure 5 the works programme is shown. It has been assumed that date of award can be as early as mid-January 1984 and that overall completion is by mid-May 1985. The latter date can be slightly adjusted (say to mid-June 1985) but in that case firm dates must be fixed for intermediate completion of certain levels of main dam body and revetments.

During the 16 months period available for construction key dates are:

- The completion of the sill by mid-November 1984;
- The completion of all preparatory works for closure by mid-January 1985;
- Construction of neap tide dam and winter spring tide dam during first half of February 1985;
6.3 MAIN ITEMS ON PROGRAMME.

Main items on the programme are:

No. 1

Haulage to site of 710,000 tons of materials.

The major part of these materials should reach the site of the works during the period February 1984 - October 1984 (9 months) prior to the start on the permanent works by the Contractor. Because of the hauling distances and limited capacity of roads near the works site this haulage will ask for a major effort.

Nos. 4-6

Slope, bed and sill protection works.

As not less than 310,000 m² is involved this work will require a major effort. It is estimated it will cost 9½ months to carry out these construction works and the work has to continue right through the rainy season in the understanding that closure has to be effected during February 1985.

Nos. 7-10

Stackpiles; Filling and stocking.

Also these operations concern items which have to be completed prior to closure and they are time-consuming (5½ months in view the large number of clay bags to be filled and loaded (960,000 Nos.).

Nos. 9-11

Closure works; Preparation, Heap tide dam and Winter running tide dam.

Though quantities involved are not huge (compare items 10, 11, 13 and 16), the time element is very important here. In fact the winter running tide dam is the most critical item of the closure operations. Only 5 days are available to complete this last item for 30% (44000 m³ of clay).

Nos. 13-16

For these items huge quantities are involved which will require a major organizational effort (transport and placing of 430,000 m³ earth fill within 3 months). Also for the revetments to be placed only 3 months are available which means that approximately 1000 m² has to be laid each day of which a part is located in the tidal zone.
Flank embankments.

Construction of these items is less critical but has to tie in with the earthworks for the main dam. Overlap of operations must be avoided as much as possible.

6.4 LABOUR REQUIREMENTS

Labour requirements have been calculated and plotted in a graph presented in Figure 6. It is noted that 10,000 men will be required during the peak (first week of February 1985) of the work. The Programme of works has been planned in such way that labour requirements are practically constant (2000) during the period May – October 1984 while a much larger number of labourers is required during the typical dry season months November 1984 – mid May 1985.

It is envisaged that build-up of labour force can be gradually from 2000 to 10000 during three months.

6.5 EQUIPMENT (TRUCKS)

In view of the large quantities to be transported on site during a limited period from stockpile or borrow area to place of incorporation into the works, the truck requirements have been calculated. It was considered that the diversity of building materials (boulders, bricks, chipped bricks, earth, clay, concrete blocks) to be handled asks for a universal transport unit like a 5 tons truck. The latter is most probably available in certain numbers in Bangladesh and does not require wide and special haulage roads as scrapers and large earth/rock moving dump trucks do. It was calculated that 200 trucks (inclusive allowance for repair, etc.) would be required during the peak period (Figure 7).
<table>
<thead>
<tr>
<th>WORK ITEM</th>
<th>QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>MUIRUA RIQUITATION PROJECT</td>
<td>7,000,000 TON</td>
</tr>
</tbody>
</table>
Fig. 7.
CHAPTER SEVEN

COST ESTIMATE

A detailed cost estimate of the Works has been made on the basis of current prices of materials on the World markets, cost of labour in Bangladesh and prices for materials listed in the "Schedule of Rates" of MIF (Pages 13-14).

The detailed cost estimate has been transferred into short Bills of Quantities. These Bills are for reference and discussion purposes only (Tables 6 and 7).

Costs for construction of actual closure works/main dam have been kept separately from the costs for construction of flank embankments.

Though quantities and prices in the opinion of the Consultant do take into account possible variations and minor items not estimated separately, it has been considered prudent to make an allowance in the overall budget for contingencies. Allowance has also been made for re-sectioning of existing access roads. The overall cost of construction then would be as follows:

TABLE 5: Overall estimate of construction cost of Feni River Closure Dam.

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Amount in CRCRES</th>
<th>Equivalent in mill. of Takas</th>
<th>U.S. $</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Feni River Closure and main Dam</td>
<td>50.3</td>
<td>20.1</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Flank Embankments</td>
<td>9.3</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Re-sectioning existing access roads</td>
<td>0.6</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>Contingencies</td>
<td>3.0</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Overall total:</td>
<td>63.2</td>
<td>25.3</td>
<td></td>
</tr>
</tbody>
</table>
TABLE 6 : Estimate of construction cost of Feni River tidal closure works and main dam.

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Description of item.</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit rate (Tk.)</th>
<th>Amount in Tk.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization and demobilization.</td>
<td>LS</td>
<td>-</td>
<td>-</td>
<td>1662.4</td>
</tr>
<tr>
<td>2</td>
<td>Bed protection on shoals and in gullies.</td>
<td>m²</td>
<td>190,000</td>
<td>600</td>
<td>1140.0</td>
</tr>
<tr>
<td>3</td>
<td>Sill</td>
<td>m³</td>
<td>93,000</td>
<td>400</td>
<td>372.0</td>
</tr>
<tr>
<td>4</td>
<td>Neap tide dam.</td>
<td>m³</td>
<td>19,700</td>
<td>800</td>
<td>157.6</td>
</tr>
<tr>
<td>5</td>
<td>Winter spring tide dam.</td>
<td>m³</td>
<td>114,000</td>
<td>475</td>
<td>541.5</td>
</tr>
<tr>
<td>6</td>
<td>Final profile main dam (earth works)</td>
<td>m³</td>
<td>420,000</td>
<td>100</td>
<td>420.0</td>
</tr>
<tr>
<td>7</td>
<td>Revetments main dam.</td>
<td>m²</td>
<td>87,000</td>
<td>800</td>
<td>696.0</td>
</tr>
<tr>
<td>8</td>
<td>Road on main dam.</td>
<td>m²</td>
<td>4,500</td>
<td>1000</td>
<td>45.0</td>
</tr>
</tbody>
</table>

Total construction cost = 5034.5 Tk.

TABLE 7 : Estimate of construction cost of flank embankments to Feni River Closure Dam.

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Description of item.</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit rate (Tk.)</th>
<th>Amount in Tk.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Earth works.</td>
<td>m³</td>
<td>250,000</td>
<td>75</td>
<td>187.5</td>
</tr>
<tr>
<td>2</td>
<td>Revetments and fore-shore protection.</td>
<td>m²</td>
<td>116,000</td>
<td>570</td>
<td>658.0</td>
</tr>
<tr>
<td>3</td>
<td>Road on crest of dam.</td>
<td>m²</td>
<td>8,000</td>
<td>1000</td>
<td>80.0</td>
</tr>
</tbody>
</table>

Total construction cost = 925.5 Tk.
ANNEX - A

PRINCIPAL HYDRAULIC ASPECTS.

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A - 1 - MATHEMATICAL MODEL

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   1.1.2 Mathematical description of tidal wave.
   1.1.3 Simulation of closure.

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   1.2.1 Varying cross sectional area.
   1.2.2 Varying storage area.
   1.2.3 Varying roughness.
   1.2.4 Tenti and Kaburi Rivers and Kalidas Pahalia.
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7.1 General.
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7.3 Effect of sediment transport.
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1.1 Type of model.

1.1.1 Tidal wave length versus basin length.

If the tidal wave length \( \lambda \) is long in comparison with the length of the estuary \( L \), the water surface in the estuary will always be more or less horizontal. The estuary can in that case be considered as a storage compartment, in which the whole area rises or falls simultaneously, depending on inflow and out-flow conditions. A criterion frequently used to determine whether this is allowed is:

Estimate of the tidal wave length.

Tidal wave length: \( \lambda = C \times T \)

\( C = \text{celerity, } T = \text{wave period} \)

Celerity without bottom friction: \( C_0 = \sqrt{gh} \text{ (m/s)} \)

For Peni estuary:

\( h \text{ (average)} = 2.5 \text{ m} \).

It follows that \( C_0 = 5 \text{ m/s} \)

\( T = 44,700 \text{ (Lg. - tide)} \)

Tidal wave length is thus: \( 5 \times 44,700 = 223,000 \text{ m or 223 km} \)

Celerity with linearized bottom friction:

\[ C = C_0 \frac{\sqrt{2}}{\sqrt{1 + \sqrt{1 + \left( \frac{gh}{C_0 h} \right)} \frac{1}{\omega}}} \]

in which \( u = 1.2 \text{ m/s} \), \( g = 10 \text{ m/s}^2 \), \( C = 70 \text{ m}^{1/2}/\text{s} \), \( h = 2.5 \text{ m} \), \( \omega = \frac{2\pi}{44,700} = 1.4 \times 10^{-4} \text{ rad/s} \).

It follows that \( C = 2.26 \text{ m/s} \)

The tidal wave length with bottom friction is then: \( \lambda = 100 \text{ km} \).
Estimate of estuary length.

For calculation purposes, the end of the estuary can be deemed to be the location where the volume of the tidal prism becomes insignificant compared to the tidal volume at the entrance. For the Peni estuary the new highway bridges across the Peni and Tahuri rivers and the Kalida Tahalia can be treated as "end of the estuary" (However river inflow at those locations in still possible).

The estuary length thus determined is:

\[ L = 15,000 \text{ m}. \]

The ratio of tidal wave length and estuary length is:

\[ \lambda / L = 6.5 \quad \text{or} \quad \lambda = 6.5 L \]

This does not meet the earlier set criterion of \( \lambda > 15 \text{ L} \), and therefore the estuary may not be treated as a storage compartment.

With a lower mid tide level, the celerity decreases and the friction increases, which lead to an even smaller tidal wave length and smaller ratio between \( \lambda \) and \( L \). This is further aggravated by the short duration of the flood. If the flood period (3 hrs.) would be representative for half the tidal wave length, the whole wave length would reduce to approximate 50 Km. the ratio \( \lambda / L \) becomes even smaller.

1.1.2 Mathematical description of tidal wave.

A tidal wave in an estuary can mathematically be described by the following equations:

\[
\frac{\partial C Q}{\partial t} + g A \frac{\partial h}{\partial x} + \frac{g (Q)^2}{C^2 A R} = 0
\]

in which

- \( C Q \) = discharge through a cross section (m\(^3\)/s)
- \( g \) = gravitational acceleration (m/s\(^2\))
- \( A \) = cross sectional area (m\(^2\))
- \( h \) = Water level
- \( C \) = Chezy roughness coefficient
- \( R \) = hydraulic radius

and
Equation of continuity

\[ \frac{\partial h}{\partial t} + \frac{1}{b} \frac{\partial Q}{\partial x} = 0 \]

in which,

\[ b = \text{width of river section}. \]

The above equations form a set of two simultaneous partial differential equations of the first order. Such a set of differential equations can be numerically solved by replacing the differential quotients by difference quotients. By using a scheme known as explicit difference scheme, it is possible to divide the estuary in a number of sections. When the water levels at the end of each section are known, and if the discharge was known at an earlier point of time, the new discharge at the middle of the section can be calculated with the equation of motion (I), though slightly re-written as a result of the substitution of the differential quotients by the difference quotients.

The newly calculated discharges are then used to calculate new water levels at the end of the sections, by making use of formula (II).

The whole procedure is repeated time and again, so that ultimately at every point of time during a tidal cycle the water levels and discharges (and indirectly the current velocities) will be known.

Discharges follow from:

\[ Q(t + \Delta t) = \frac{\frac{Q(t)}{\Delta t} + g A \cdot \frac{h_1 - h_2}{\Delta x}}{\left\{ \frac{1}{\Delta t} + \frac{g L Q(t)}{c^2 A R} \right\}} \]

in which

\( Q(t + \Delta t) \) = newly calculated discharge

\( Q(t) \) = earlier calculated discharge

\( \Delta t \) = time step

\( h_1, h_2 \) = water level in point 1, 2

\( \Delta x \) = length increment (along axis of estuary)

Levels follow from:

\[ \Delta h = \frac{(Q_{\text{out}} - Q_{\text{in}})}{b \cdot \Delta x} \]

in which

\( Q_{\text{out}} \) = out going discharge from a "Point"

\( Q_{\text{in}} \) = in going discharge to a "Point"
\[ \mathbf{b} = \text{width of estuary} \]
\[ \Delta x = \text{length increment} \]
\[ \Delta A x = \text{storage area of a "Point"} \]

The following sketch may elucidate the scheme of sections and points.

\[ \text{\ldots} = \text{storage area corresponding to "Point" 2.} \]

```
Point 1  2  3  4  5  6  7
Section  1  2  3  4  5  6
\]
```

\[ \Delta x = \text{length of a section (may vary per section)} \]

For any tidal wave problem to be solved it is necessary to know:

- boundary conditions at both ends of the river/estuary section to be studied.
- the initial conditions, i.e., discharges in every section and water levels in every point.

The boundary conditions may be:

- constant or varying water levels in the end points.
- constant or varying discharges in the end sections.

One set of boundary conditions at each end will suffice, for instance:

- varying water level in point 1.
- constant discharge in section 7.

( these conditions have been used for the Veni estuary ).
The initial conditions are generally not known. For tidal problems this is no problem, since calculations could start by assuming all discharges and water levels to be zero. Then two tidal cycles are calculated. When the discharges and water levels at the end of both cycles are similar, it is sufficient to calculate one full tidal cycle to find the initial conditions. (Friction is a very important aspect in generating a proper initial condition).

A model was developed by Haskoning to allow the above computations to be made for the Peni River estuary. More details are given in the operation manual of the model. Under the terms of the Consultancy agreement and subsequent correspondence, the software, operation manual and the minicomputer and accessories will be handed to the Bangladesh Water Development Board once all calculations have been performed.

1.1.3 Simulation of closure.

Once the model has been calibrated (for which simultaneous water levels and discharges during a whole tidal cycle should be measured), it will be necessary to simulate partial or complete closure of the estuary near its mouth. The explicit difference scheme is particularly useful, since the equation of motion for Section No. 1 can easily be replaced by a weir formula (or any other formula). However, attention should be paid to the area of the opening (for instance of a closure gap) compared to the original cross sectional area. When the new opening is too big, it can introduce instabilities in the calculation process. This can generally be overcome by reducing the time increment.

1.2 Creation of model.

1.2.1 Varying cross sectional area.

The Peni estuary can be characterised by a number of distinct gullies and shoals, which will be dry during low water. It is obvious that the flow characteristics will be different during high and low water. The cross section may therefore not be represented by a rectangular section, but should have a varying width depending on the water level.

In the mathematical model this has been realised by defining width and hydraulic radius at three different levels; between those levels an interpolation will have to be made.

1.2.2 Varying storage area.

The storage area will also vary, depending on the water level. Apart from storage in the estuary itself, which can be determined from the cross sections measured at regular intervals, storage will also take place in numerous khals. The storage in the khals will influence the rate of rise and fall of water levels at different locations of the estuary. For that reason the effect of storage should be accounted for separately from the estuary.
cross section (which determines, through the equation of motion, the discharge). In the model, storage has been affected by introducing storage areas of "Points" at three different levels, in between which interpolation will take place.

1.2.3 Varying roughness:

The Chezy roughness coefficient is not constant, but varies with the water depth. This variation has been introduced in the model by multiplying the Chezy roughness at a hydraulic radius of 1 m, with \( R^{1/6} \) (this is in fact the roughness according to Strickler).

The roughness coefficient itself is not known beforehand, but must be determined from several calibration runs of the computer program containing the model.

1.2.4 Feni and Kuhuri Rivers and Kalidas Paharia:

From the new regulator site to the confluence of the Feni and Kuhuri rivers, the estuary can be treated as a single channel. From the confluence up to the new highway bridges, the Feni and Kuhuri river should be taken as separate river branches having their own characteristics and boundary conditions. As these sections are of minor importance (compared to the section between new regulator site and confluence), they have been incorporated in the model as one single (hypothetical) river, with characteristics reflecting the joint characteristics of the Feni and Kuhuri River.

The (winter) flow through the Kalidas Paharia is very small, and the storage capacity, compared to Feni and Kuhuri rivers is limited. No inflow has therefore been taken into account through the Kalidas Paharia, while the storage capacity has been added to one of the model "Points".

1.2.5 Boundary conditions:

Boundary conditions should, per definition, not be influenced by any phenomena inside the boundaries. Whilst this criterion will theoretically almost never be met, it will generally suffice to define boundaries in which the influence of the river sections to be enclosed, is neglectable (always having regard to the purpose of the calculations).

The following boundaries have been used:

- A water level boundary at the sea side (varying with time)

and

- A constant inflow though the Feni and Kuhuri rivers at the new highway bridges.
1.3 Calibration of model.

During March 1978 simultaneous measurements of the flow conditions in the estuary were done, once during neap tide and once during spring tide.

The spring tide measurements have been used to calibrate the model. As boundary conditions were used:

(a) At the sea side: The average of four simultaneous water level measurements, viz. two at each side of the river, namely at the old and new closure site.

(b) The sum of the average inflow through the Peni and Muhuri rivers, being approximately 26 m³/s.

The measured discharges in left channel, shoal and right channel at the new closure site and the measured fluctuation in water level in the Peni and Muhuri rivers (near the new highway bridges), were used to calibrate the model.

After approximately ten trial runs the best possible compromise was used to determine the

- Apparent roughness coefficient.
- Apparent storage in khals.
- Apparent length of each river section.

A scheme of the several river sections and the values of the characteristics used in the model are given in figure A1 and table A2, respectively.

The measured and calculated water level fluctuations in point 10 (i.e., near the Peni and Muhuri highway bridges) are given in figure A2.

The measured volume of the tidal prism is 26.2 x 10⁶ m³, whereas the calculated volume is 25 x 10⁶ m³.

The small discrepancies between measured and calculated volumes is within the expected range and indicates that the model will forecast water levels and discharges with sufficient accuracy. The input data of the run representing most accurately the real situation (run No.7) have been used for all subsequent calculations (with or without simulation of closure).

In total more than 60 runs were made to investigate any conceivable situation before, during and after closure.
### TABLE A - 1. Characteristics of Sections and points.

#### Sections.

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#### Points.

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Note: Levels E1, H2 and E3 are relative to S03 + 2m.
DISCHARGE OVER BROAD CRESTED WEIRS.

Often the discharge in a closure gap can be determined by using the discharge formulas for broad crested weirs. Depending on the magnitude of the upstream total head (water level plus energy head) and the down-stream water level, both relative to the sill level, there are two types of flow with different discharge characteristics.

(I) Subcritical flow

\[ q = m \cdot h_1 \cdot \sqrt{2g} \left( H_0 - h_1 \right) \quad \left( \text{if } h_1 \geq \frac{2}{3} H_0 \right) \]

and

(II) Critical flow

\[ q = m \cdot 1.3 \cdot H_0 \cdot \sqrt{2g} \cdot H_0^{\frac{1}{3}} = 1.73 \text{ m} \cdot H_0^{\frac{3}{4}} \quad \left( \text{if } h_1 < \frac{2}{3} H_0 \right) \]

in which:

- \( q \) = specific discharge (m³ per m width of weir)
- \( m \) = discharge coefficient
- \( h_1 \) = down-stream water height above top of sill
- \( H_0 \) = upstream total head relative to top of sill
- \( g \) = gravitational acceleration.
## Table A-2

<table>
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<th>Tide (ref:SCB)</th>
<th>Run No.</th>
<th>Width (m)</th>
<th>Sill L. (ref:SCB)</th>
<th>V max flood (m/s)</th>
<th>V max ebb (m/s)</th>
<th>Vol. tidal prism (10^6 m^3)</th>
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EFFECT OF FRICTION AND OBSTRUCTIONS IN THE CLOSURE GAP.

4.1 Influence of friction.

Energy is lost in overcoming the friction of the bed protection. As the level of the bed protection in the closure gap is not significantly higher than the surrounding sea/river bed, it is not permitted to neglect this friction. The friction loss can be expressed as a portion of the energy head: \( \alpha \frac{V^2}{2g} \quad (0 < \alpha < 1) \)

A discharge coefficient can be derived for

(I) subcritical flow: \( m = \sqrt{\frac{1}{1 + \alpha}} \)

and

(II) critical flow: \( m = \left(1 - \frac{\alpha}{6 + \alpha} \right) \cdot \sqrt{\frac{1}{C (1 + \alpha/6)}} \)

For spring tide conditions, with mainly subcritical flow, the discharge coefficient \( m \) equals 0.84.

For average and neap tide conditions, with both critical and subcritical flow, the discharge coefficient \( m \) equals 0.78.

4.2 Obstructions in the closure gap.

4.2.1 Horizontal and vertical obstruction.

Horizontal obstructions in the flow cross section cause a contraction of the flow in and downstream of the closure gap. Such obstructions lead to smaller discharge coefficients. Vertical obstructions lead in general to larger discharge coefficients. In the following table, based on experiments in the Delft Hydraulics Laboratory (M 1731 - XIV), discharge coefficients have been summarised for several closure gap situations.
Closure gap width

**Total width.**

<table>
<thead>
<tr>
<th>Sill height Depth</th>
<th>0.75</th>
<th>0.75</th>
<th>0.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.02</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td>0.50</td>
<td>1.17</td>
<td>1.03</td>
<td>0.92</td>
</tr>
</tbody>
</table>

**TABLE A-3 Influence of horizontal and vertical contraction on discharge coefficients.**

4.2.2 **Influence of a Jetty.**

A jetty in the closure gap (to be used for final closure) means an obstruction for the flow through the closure gap. This obstruction has two aspects: energy loss and decrease of net width. The energy loss restricts the head difference that can be used for the acceleration of the flow in the gap. This results in larger differences between water levels at sea and in the reservoir, higher current velocities and lower depth on the sill. Therefore the width of the gap should be larger to meet a maximum velocity criterion. (The reciprocal value of the discharge coefficient is a measure for the increase of the width).

The layout of the jetty substructure could be as shown below.
Calculation of the discharge coefficient will then be as follows.

1) Net width obstruction \[3.5 \div 4 = 0.875\]

2) Energy loss

According to Ven to Chow Open channel Hydraulics pp 506 and 507

\[
\Delta H = C \frac{V^2}{2g} \quad V = \text{undisturbed current velocity} = 0.875 \times \text{dist.}
\]

\[C = \beta \left(\frac{S}{b}\right)^{4/3} \sin \delta\]

\[\beta = 1.8 \text{ for round pile}\]
\[S = \text{Width of the structure perpendicular to the flow.}\]
\[b = \text{Distance centre to centre of row.}\]
\[\delta = \text{Vertical pile} \quad \delta = 90^\circ\]

\[
\Delta H = 1.8 \left(\frac{0.5}{4}\right)^{4/3} \times \left(\frac{0.875 \times V}{2g}\right)^2 = 0.034 \frac{V^2}{2g}
\]

The six piles in one row have a total energy loss of 0.5 \(\frac{V^2}{2g}\)

The discharge coefficient due to this energy loss can be determined as:

\[
m_{cc} = \sqrt{\frac{1}{1 + 0.5}} = 0.81
\]

\[
m_{wo} = 0.375
\]

\[
m_{jetty} = 0.71
\]

So the width of the closure gap must be 1.4 larger to get the same situation as without jetty.

Remark

The given solution is an approximation only. If the method with jetty is selected, more accurate calculations are necessary with more particulars of the shape of the closure gap.
FORMULATION OF WEIR AND REPRESENTATION IN THE MODEL.

5.1 Formulation of weir.

The broad crested weir formulas for critical and sub-critical flow, as mentioned in section 2 of this Annex, cannot be used for the calculation of the flow over the bed protection and sill mattresses, since the friction will have a dominant role in the forces determining the flow pattern through the closure "gap".

Therefore new formulas have been developed which describe the flow over the mattresses, which are only dependent upon the water levels upstream and downstream of these mattresses.

\[ \text{energy head} = h + \frac{\partial h}{\partial x} \frac{dx}{x} \]

A drop in water level over a distance \( dx \) is caused by friction and an increase of the energy head (due to increase of the current velocity) (see also above sketch).

The mathematical description of the situation is as follows:

\[ q = \text{specific discharge} \quad (\text{m}^3/\text{s, m}) \]

assumed to be constant during a short time interval and only dependent on \( h_b \) and \( h_e \)

in which:

\[ h_b = \text{water level immediately above beginning (m)} \]

\[ h_e = \text{water level above end of mattress, (m)} \]

\[ V = \text{current velocity (m/s)} \]

\[ g = \text{acceleration of gravity (m/s}^2 \)]

\[ V = \frac{q}{h} \rightarrow \frac{\partial V}{\partial x} = -q \cdot h^{-2} \cdot \frac{\partial h}{\partial x} \]
\[ f(z) = \int_{\gamma} \frac{e^{-z}}{z} \, dz \]

Where \( f(z) = e^{-z} \) and \( \gamma \) is a path in the complex plane.
Now it should be realised that \( h_b \) is the water level immediately above the beginning of the mattress. This water level is lower than the water level in front of the mattress (for instance above a scourd hole). If the water velocity above the scourd area is assumed to be zero (which is a reasonable assumption), \( h_b \) can also be expressed in terms of the upstream water level and the velocity immediately above the beginning of the mattress:

\[
h_b = H - \frac{v^2}{2g} = H - 0.65 \left( \frac{q}{h_b} \right)^2 \quad (eq. \, no \, 2)
\]

(1) and (2) form a set of two algebraic equations with unknowns \( q \) and \( h_b \) and known values for \( h_c, L \) and \( H \).

Elimination of \( q^2 \) from these equations gives an algebraic expression with \( h_b \) as a single unknown factor:

\[
6.6 \, h_b^4 - 5.6 \, H \, h_b^2 \, \omega + (\omega - 5.6) \, h_b^2 \omega^2 + \left( 5.6 \, H^2 - 5.6 \, \omega \right) \, h_b^2 - \alpha = 0 \quad (eq. \, no \, 3)
\]

in which \( \alpha, \beta \) and \( \omega \) are constants, depending only upon other constants \( H, h_b \) and \( L \).

Once \( h_b \) has been solved from (3), it can be substituted in either (1) or (2) to obtain \( q \).

Expression (1) is in principle not valid if \( h_c \) is lower than the critical depth which is governed by \( q \) as follows:

\[
h_{cd} = \sqrt[3]{\frac{3}{2} \frac{q^2}{g^2}}
\]

If \( h_c \) is less than \( h_{cd} \), it would theoretically be necessary to adopt an iteration procedure to determine which length of mattress would be sufficient for the water level to reach the critical depth. Then determine \( h_b \) and \( q \) as described before.

Trial computations have indicated that a rigorous use of \( h_c \) (\( > 0 \)) produces only insignificant errors. Therefore no such iteration procedure has been applied.
5.2 Representation of amended weir formula in mathematical
Model.

If the water levels upstream and downstream of the mattress
are known, the discharge can be calculated as indicated in
section 5.1. Upstream and downstream water levels are
calculated for points \( H_0 \) and \( H_t \) (or vice versa during
high tide) when these water levels are related to water
depths above the mattress level, formula (3) of section
5.1 can be solved for the two separate sill configura-
tions:

A width of 80 m in the gullies and
A width of 200 m on the shoals. *)

The specific discharges \( Q_0 \) and \( Q_{200} \)
are subsequently multiplied with the effective widths of
the respective sills to arrive at a total discharge through
the closure gap.

In the computer program equation (3) is numerically solved by
using the method of Newton – Raphson, whereby \( H \) is used as
a starting value.

The effective widths are:

```
In the gullies:
\[ W_{200} = (250 + 150) - 3 \times 10 \text{ m} \times 333 \text{ m} \]
\[ h = \frac{1}{c_s} \text{ (see also last part of this para)} \]
```

On the shoals:
\[ W_{200} = (760 - 6 \times 10) = 630 \text{ m} \]

One point of attention is the influence of the new weir
formula on the stability of the calculation process.

As no momentum effects have been taken into account
in the derivation of the new weir formula, the calculated
flow is only the result of water level difference and
not of earlier calculated velocities.

Several trial runs have learnt that a decrease of the time
step in the calculation process produces adequate results.

The following is more elaborate discussion on the discharge
coefficient in case caissons are placed in the closure gap,
respectively the gap were to be narrowed horizontally prior
to closing.

Note: *) Later it has been decided to adopt a width of 150 m.
      This change has not been effected in the calculations.
5.3 Discharge coefficient for closure gap with caissons.

Situation:

Reference: Ven te Chow, Open Channel Hydraulics, page 503, Bridge piers.

For flow approaching at $10^\circ$, $m = 0.93$
For irregularities in placing $m = 0.90$

Total gross width $B = 1160$ m

Gross width with bed protection having length of 200 m: 760 m
Gross width with bed protection having length of 80 m: 400 m

Net width ($L = 200$):

$(760 - 6 \times 10) \times 0.9 = 630$ m

Net width ($L = 80$):

$(400 - 3 \times 10) \times 0.9 = 333$ m
5.4 Discharge coefficient for closure gap with horizontal obstruction and caissons.

**Situation:**

\[ \text{Net width: } 720 - 5 \times 10 = 670 \text{ m} \]

\[ \text{Width for horizontal obstruction from the side } \frac{720}{1160} = 0.62 \]

Discharge coefficient 0.79

Discharge coefficient caissons 0.83

Total discharge coefficient 0.7

Net width \( L = 200 \text{ m} \): \( 0.7 \times 670 = 470 \text{ m} \)

The values are estimations only.
6.1 General

Closure is envisaged to be achieved during a neap tide period. It is essential that the discharge to the sea is maximal in order to reduce the water level above the mattress / sill as much as possible during the shortest possible time. To this end the regulator should be utilised to increase the flow immediately after high water. For calculation purposes this high water has been assumed at S.O.B. + 1.80 m (this means that at high water the water depth above the mattress / sill is approximately 1.10 m). The flapgates of the regulator should be opened to increase the discharge (the small water level differences are insufficient to open the heavy flapgates and to produce a flow of significant magnitude. The flapgates could be opened for instance by using hand winches).

The flow through the regulator depends on the discharge characteristics of the regulator openings as well as on the hydraulic resistance of the diversion channel.

6.2 Discharge through diversion channel.

The latest information received indicate that the channel will be dredged to a level of S.O.B. - 1.50 m, with a bottom width of 150 m and side slopes of 1:2.

As the water level will never drop below S.O.B. - 0.00 m (this is the level of the regulator floor), for calculation purposes only the net area above S.O.B. 0.00 is subject to variation.

\[ A = 2D^2 + 156P + 229 \]

Wet circumference:

\[ o = 154.5P + 4.5P \]

Hydraulic radius:

\[ R = A/o \]
Longitudinal section along channel and regulator:

\[ P = \left( \frac{H_A + H_B}{2} \right) \]

Critical or subcritical depending on ratio \( H_C / H_D \)

Hydraulic resistance channel:

\[ \frac{V^2}{2g} + \frac{V^2 L}{C^2 R} = H_A - H_B \]

in which

\[ C = \frac{R}{0.032} \Rightarrow C^2 = 97.6 R^{1/3} \]

thus

\[ 0.05 \frac{V^2}{2g} + \frac{2000}{97.6 R^{1.33}} \frac{V^2}{2g} = H_A - H_B \]

or

\[ V = \sqrt{\frac{H_A - H_B}{0.05 + \frac{2.05}{R^{1.33}}}} \]

\[ C_Q = \frac{V}{A} \]

\[ C_Q^2 = \left\{ \frac{H_A - H_B}{0.05 + \frac{2.05}{R^{1.33}}} \right\} \cdot A^2 \]

It follows that \( V \) and \( Q \) can easily be determined when \( H_A \) (the water level at the entrance of the diversion channel) and \( H_B \) (the water level at the entrance of the regulator) are known.

\( R^{1.33} \) and \( A^2 \) are functions of \( P \) and therefore of \( H_A \) and \( H_B \).

In the region of interest (\( \frac{H_A + H_B}{2} \) is between 0.9 and 1.4)

term \( 0.05 + \frac{2.05}{R^{1.33}} \) can be approximated by:

\[ 0.80 + 0.356 \left( \frac{H_A + H_B}{2} \right) \]
Likewise can the term \( A^2 \) be approximated by:

\[ 73,000 \ (HA + HB) \]  
(3)

Substitution of (2) and (3) in (1) gives the following expression:

\[ Q^2 = HB^3 (-26,000) + HB^2 (-58,400 - 26,000 HA) + 
HB (26,000 HA^2) + (58,400 + HA^2 + 26,000 HA^3) \]  
(4)

### 6.3 Discharge through regulator.

The discharge coefficient for the regulator (for the water levels considered) has been determined at \( m = Q/3 \). The width is 40 bays \( 3.60 \ m = 144 \ m \). The effective width is thus \( 0.8 \times 144 = 115 \)

If \( HB < 1.5 HC \) the flow through the regulator is subcritical.

The discharge is then:

\[ Q = 115 \ HA \sqrt{2g (HB - HC)} = 515 \ HC \sqrt{(HB - HC)} \]  
(5a)

or

\[ Q^2 = 515^2 \ HC^2 (HB - HC) \]  
(5a)

If \( HB > 1.5 HC \) the flow through the regulator is critical.

The discharge will be:

\[ Q = 115 \ X \ \frac{2}{3} \ HB \sqrt{2g \ (HB - \frac{2}{3} HB)} \]  
(5b)

or

\[ Q^2 = 115^2 \ HC^3 \]  
(5b)

\( Q \) can be calculated when \( HB \) (in both cases) and \( HC \) (in case of subcritical flow) are known.

### Discharge through system of channel and regulator.

(4) and (5a) (subcritical flow) or  
(4) and (5b) (critical flow) form a set of two equations with basically two unknown values: \( Q \) and \( HB \) (HA and HC are the water levels, reduced to SAC, just upstream and downstream of the regulator).

When \( Q \) is eliminated from both equations, one equation with unknown \( HB \) remains (a cubic parabola), which can easily be solved by using the method of Newton–Raphson.
The equation in HB to be solved is:

$$HB^3 + \alpha_2 HB^2 + \alpha_1 HB + \alpha_0 = 0$$  \hspace{1cm} (6)

For subcritical flow:

$$\alpha_2 = HA + 2.25$$
$$\alpha_1 = - HA^2 + 10.2 HC^2$$
$$\alpha_0 = - 2.25 HA^2 - 10.2 HC^3 - HA^3$$

For critical flow:

$$\alpha_2 = \left(26 HA + 58.4\right) / 65.2$$
$$\alpha_1 = - 26 HA^2 / 65.2$$
$$\alpha_0 = - \left(58.4 HA^2 + 26 HA^3\right) / 65.2$$

In the program, it is firstly assumed that the flow is subcritical and (6) is solved accordingly. Later the assumption is checked and if found incorrect it is again solved for critical flow.

6.4 Computer run.

A computer run (No. 60) has been made in which the following was simulated:

- Tide: HW = SCB + 1.8 m (neap tide)
- Opening of regulator: One hour after high water.
- Closure of neap tide dam: Three and a half hours after high water.

Fig. 4 in Section 4.3.9 of the main report displays the most important aspects of the computer run.

At the moment of the critical closure a water depth of 0.18 prevails above the sill, which enables labourers to walk through the water and to deposit clay filled bags in the (ten) closure gaps. Should the regulator not be opened the water depth would be larger making it impossible to close the gap by manual labour. In addition a rise in water level would be experienced as a result of blockage of the flow by the dam.
The influence of the discharge through the regulator on the total discharge can also be clearly seen.

At the time of closing the neap tide dam, the regulator takes care of approximately 85% of the total discharge.
7.1 **General.**

For the considerations on the scour downstream of the bed protection time-scour relations have been used which are based on systematic investigations carried out by the Delft Hydraulics Laboratory.

The mentioned relation is:

\[ h_{\text{max}} = \frac{(\alpha \overline{u} - \overline{u}_{cr})^{1.7} h_0^{0.2}}{10 \Delta^{0.7}} \cdot t^{0.4} = k \cdot t^{0.4} \]

in which

- \( h_{\text{max}} \) = largest depth of the scour hole on time \( t \) (m)
- \( h_0 \) = original depth (m)
- \( t \) = time (hours)
- \( \overline{u} \) = average current velocity (m/s)
- \( \overline{u}_{cr} \) = critical current velocity for the bed material with water depth (m/s)
- \( \alpha \) = scour factor
- \( \Delta \) = density of bed material relative to water

From this relation it can be seen that scour is determined by the scour factor and magnitude of the current velocity relative to a critical velocity.

7.2 **Scour at closure site.**

In the case of the Feni estuary a favourable condition is the rather small water depth. A scour hole of several meters depth gives a substantial reduction of the current velocities in the scour hole. For the solution of the scour problem the factor \( \alpha \) must be known and model investigations are therefore indispensable.
However to get an idea of the scour to be expected, use can be made of earlier gained experience. For the calculations on scour, a tide with a high water level of $3.20^\text{th} + 3\text{CB}$ has been used, which is almost 20\% higher than the average high water level in the month of January, February and March.

The situation in the closure gap is as follows:

- bed protection over 200 m on the shoals at $3\text{CB} + 0.70\text{ m}$ in the gullies over 80 m at the same level.
- bed protection over 200 m in the gullies at the original level of $3\text{CB} + 0\text{ m}$.
- all caissons have been placed. *)

Runoff of the mathematical model gives the expected current velocities. Because of the larger current velocities and larger water depths the flood period will be decisive for the scour.

For calculation purposes the varying water depth and velocities have been converted into a single (constant) water depth and velocity with the same effect as the accumulated varying depths and velocities.

Four locations in the cross section have been considered (see figure A-3). The scour factor $\alpha$ has been derived from figure A-4. The locations are:

On the shoals:

I between the caissons

$\alpha = 2.2; \quad h_\varphi = 1.87\text{ m}; \quad V = 1.20\text{ m/s}$

II downstream of the caissons (a moderate vortex street will be possible)

$\alpha = 2.5; \quad h_\varphi = 1.87\text{ m}; \quad V = 1.20\text{ m/s}$

In the gullies:

III between the caissons, where flow can be regarded as two-dimensional with a vertical obstruction.

$\alpha$ varies from 2.1 to 3.2;

$h_\varphi = 1.37\text{ m}; \quad V = 0.87\text{ m/s}$
IV downstream of the caissons; three-dimensional effects are present due to combination of horizontal and vertical obstructions.

$v$ varies from 2.4 to 6;

$h_c = 1.97 \text{ m}, \quad V = 0.87 \text{ m/s}.$

Note: *) Calculations refer to preliminary design. In final design the length of 200 m became 150 m while caissons were replaced by stockpiles.
Case I: On shoal between caissons.
Case II: On shoal downstream of caissons.
Case III: In gullies between caissons.
Case IV: In gullies downstream of caissons.

Fig. A - 3.
VERTICAL NARROWING (%)

COINCIDING VORTEX STREETS
NORMAL 2-DIMENSIONAL SITUATIONS.

RELATION BETWEEN ALPHA AND VERTICAL NARROWING.
COURTESY: DELFT HYDRAULICS LABORATORY.

Fig. A-4.
Effect of sediment transport.

The effect of natural sand transport is not included in the scour expression of $\theta_{\text{max}}$. The available natural sand transport will reduce the scouring depth.

This effect can be taken into account as follows:

The volume of the scour hole is per $m^2$ width,

$$V = a \times h_{\text{max}}^2 = a \times k^2 \times t^{0.8}$$

in which $a$ is the shape factor of the scour hole.

If the sand transport is added, the volume of the scour hole will be reduced depending on the magnitude of the sand transport $T \left( m^2/s \right)$ and the time period considered.

$$V_{\text{red}} = a \times k^2 \times t^{0.8} - T \times t$$

For the Puri estuary with the very fine sand in the bed the sand transport will definitely influence the scour process.

The sand transport is determined with the following formula:

$$\frac{a_{S}}{\Delta \times d} = 0.012 \left[ \frac{\bar{u} - \bar{u}_{cr}}{(\Delta \cdot g \cdot D_{50})^{0.5}} \right]^{0.4} \left[ \frac{D_{50}}{d} \right] + D_{k}^{-0.6}$$

in which:

- $a_{S}$ = volume of sand $= T \left( m^2/s \right)$
- $\bar{u}$ = average velocity $\left( m/s \right)$
- $d$ = water depth $\left( m \right)$
- $\bar{u}_{cr}$ = critical current velocity of bed material

- $D_{50}$ = dimension of sieve through which 50% (by weight) of the grains of a sample will pass $\left( m \right)$

- $D_{k} = D_{50} \cdot \left[ \frac{\Delta \cdot g}{\nu} \right]^{1/3}$
- $\Delta$ = relative density of bed material ($\approx 1.65$)
- $g$ = acceleration of gravity $\left( m/s^2 \right)$
- $\nu$ = kinematic viscosity $\left( m^2/s \right)$
Calculations based on the current velocities from the mathematical model in the undisturbed situation of the estuary with a tide of SOL +3.20 m (run 48), give a total amount per flood period of 3.2 m³. For the reduction of the scour holes only 50% of this sand transport has been taken into account.

The results of the calculations are represented in the following Tables A-4 and A-5.

On the shoals and in the gullies the natural sand transport reduces the scour substantially and an equilibrium scour depth can be reached, which is restricted to a few meters in the worst case.

This limitation is very important in relation to the soil conditions on the closure site.

Large scour depths increase the possibility of local deformation of the soil on the upstream slope of the scour hole, which can lead to liquefaction of larger soil bodies. Notwithstanding the favourable results of the calculations, all closure operations should be checked upon their effects on scour.

Apart from the sand transport, the shape of the closure gap is a very important factor determining the rather shallow depths of the scour hole. To make clear what happens if large obstructions are placed in the closure gap, calculations have been made for a situation with 220 m obstructions on both sides of the gap. Only the two dimensional case between the caisson has been regarded. (Same as I)

The current velocities have been derived from Run No. 51 of the mathematical model.

The results are also represented in Tables A-4 and A-5 (cases V and VI). It is obvious that in this case the natural sand transport cannot restrict the scour so much.

The time necessary for the closure operations now determines the scour, which can reach larger depths.

7.4 Conclusions:

(a) The scour to be expected without initial narrowing of the closure gap (but with caissons in place) will be within acceptable limits. The bottom protection should however be extended sufficiently to prevent that scour holes will effect the stability of the dam.

(b) Initial narrowing of the closure gap will lead to decidedly deeper scour depths, which should be avoided in view of sub-soil conditions.
### Table A-4: Without adjustment for sediment transport.

<table>
<thead>
<tr>
<th>Time elapsed</th>
<th>Case I</th>
<th>Case II</th>
<th>Case III</th>
<th>Case-IV</th>
<th>Case-V</th>
<th>Case-VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 tide</td>
<td>0.40</td>
<td>0.50</td>
<td>0.25</td>
<td>0.54</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td>10 tides</td>
<td>1.00</td>
<td>1.25</td>
<td>0.62</td>
<td>1.35</td>
<td>2.30</td>
<td>2.80</td>
</tr>
<tr>
<td>100 tides</td>
<td>2.45</td>
<td>3.10</td>
<td>1.60</td>
<td>3.30</td>
<td>5.80</td>
<td>6.80</td>
</tr>
<tr>
<td>1 year</td>
<td>5.30</td>
<td>7.20</td>
<td>2.50</td>
<td>7.60</td>
<td>&gt;10</td>
<td>&gt;10</td>
</tr>
<tr>
<td>1000 tides</td>
<td>6.00</td>
<td>8.10</td>
<td>4.00</td>
<td>3.50</td>
<td>&gt;10</td>
<td>&gt;10</td>
</tr>
<tr>
<td>2 years</td>
<td>7.10</td>
<td>9.60</td>
<td>4.70</td>
<td>10.00</td>
<td>&gt;10</td>
<td>&gt;10</td>
</tr>
</tbody>
</table>

### Table A-5: With adjustment for sediment transport.

<table>
<thead>
<tr>
<th>Time elapsed</th>
<th>Case-I</th>
<th>Case-II</th>
<th>Case-III</th>
<th>Case-IV</th>
<th>Case-V</th>
<th>Case-VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 tide</td>
<td>0.30</td>
<td>0.40</td>
<td>MIL</td>
<td>0.45</td>
<td>0.85</td>
<td>1.05</td>
</tr>
<tr>
<td>10 tides</td>
<td>0.52</td>
<td>0.90</td>
<td>&quot;</td>
<td>1.20</td>
<td>2.10</td>
<td>2.50</td>
</tr>
<tr>
<td>100 tides</td>
<td>&quot;</td>
<td>1.50</td>
<td>&quot;</td>
<td>2.00</td>
<td>4.50</td>
<td>6.20</td>
</tr>
<tr>
<td>1 year</td>
<td>&quot;</td>
<td>1.55</td>
<td>&quot;</td>
<td>&quot;</td>
<td>3.05</td>
<td>&gt;10</td>
</tr>
<tr>
<td>1000 tides</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
<td>9.10</td>
<td>&gt;10</td>
</tr>
<tr>
<td>2 years</td>
<td>0.52</td>
<td>1.55</td>
<td>MIL</td>
<td>2.00</td>
<td>10.30</td>
<td>&gt;10</td>
</tr>
</tbody>
</table>
ANNEX - B

GEO - TECHNICAL INVESTIGATIONS AND CALCULATIONS

(Following presents a compilation of design relevant data a summary of which has already been given in section 2.2)

1. SITE INVESTIGATIONS.

1.1 Ground level.

Ground levels from SOB + 0.25 to + 0.60 m have been measured between boreholes H2 and H15 and at the location of boreholes H21, 22 & 23.

At the location of the gully, with boreholes H17, 19 & 24, ground level is at approximately SOB - 0.5 m. Borings H6, 10, 12, 16 & 18 have not been carried out. On-land borehole H24 showed ground level at approximately SOB + 3.40.

1.2 Ground water.

Ground water level practically equals ground level. Design has to allow for fully saturated subsoil conditions.

1.3 Soil composition.

Predominantly silt and/or silt fine sand layers have been encountered, some times with some mica or organic matter.

Relatively thin clay layers; 0.4 m to 0.75 m in thickness, have been encountered in borings H7, 9, 11 & 13 at depths of SOB - 10 m to - 20 m and in boring H19 (gully) between SOB - 2 m and - 4.50 m.

Relatively thick clay layers, 3 - 3.5 m in thickness, were encountered in boring H23 between SOB - 15 m and - 18 m and in the on-land boring H24 from ground level, at approximately SOB + 3.20 m.

The latter clay layer could be indicative for the presence of more clay possibly feasible for the core and seals in the dam body.

In-situ density indications stem from the Standard Penetration Test measuring the number of blows for 300 mm penetration. For this test standardized equipment is used. The number of blows is designated as N-value. To allow comparison with cone penetration test results the N-values should be multiplied by 3-4 to obtain a core resistance q., in lbf/cm². However it is stressed that for low N-values, 5-7, the test is less feasible and results should only be used qualitatively.

The top layer, from grade to a depth of 0.6 m, proves to be loose to very loose with N-values from 2 to 12. Similar loose and very loose layers to greater depth have been encountered at following locations (Table B - 1).
TABLE B - 1  Location and depth of very loose and loose layers.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (m S0B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H3</td>
<td>~ Surface to - 9.90</td>
</tr>
<tr>
<td>H9</td>
<td>~ 3.50 to - 6.30</td>
</tr>
<tr>
<td>H11</td>
<td>~ Surface to - 6.30</td>
</tr>
<tr>
<td>H17</td>
<td>~ Surface to - 4.50</td>
</tr>
<tr>
<td>H19</td>
<td>~ - 8.10 to - 11.00</td>
</tr>
<tr>
<td>H19</td>
<td>~ - 12.60 to - 16.50</td>
</tr>
</tbody>
</table>

The remaining layers are practically all medium dense with N-values between 15 and 30. Some dense layers with N-values $\geq$ 30 have been encountered.
2. LABORATORY TESTING

2.1 Sieve analyses.

The results of some 70 sieve analyses have been reviewed to suit the requirements for various design criteria.

To establish the compactibility the percentage particles \(< 74 \text{ mm} \) (sieve No 200 ) was determined resulting in the following table.

<table>
<thead>
<tr>
<th>Percentage of (\leq 74 \text{ mm})</th>
<th>Percentage of all samples</th>
<th>Cumulative percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td>16</td>
<td>100</td>
</tr>
<tr>
<td>10-20</td>
<td>21</td>
<td>84</td>
</tr>
<tr>
<td>20-30</td>
<td>16</td>
<td>63</td>
</tr>
<tr>
<td>30-40</td>
<td>20</td>
<td>47</td>
</tr>
<tr>
<td>40-50</td>
<td>4</td>
<td>27</td>
</tr>
<tr>
<td>50-60</td>
<td>4</td>
<td>23</td>
</tr>
<tr>
<td>60-70</td>
<td>1</td>
<td>19</td>
</tr>
<tr>
<td>70-80</td>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td>80-90</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>90-100</td>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

To obtain an impression of the prevailing particle sizes, expressed as \(D_{50}\), an analysis has been made of 27 curves from the top 3 meters and 48 curves from the top 6 meters. The results are presented in figure D-1 as a percentage of each group samples.

To allow proper choice of woven and/or non-woven filter fabric, to be placed directly in contact with the river bed sediments and the slope of the embankment, all available grain size analyses of river bed samples were reviewed giving following design characteristics (Table B - 3 ).
### TABLE: B - 3 Grain size characteristics.

<table>
<thead>
<tr>
<th>Boring</th>
<th>D&lt;sub&gt;50&lt;/sub&gt;</th>
<th>D&lt;sub&gt;15&lt;/sub&gt; in mm</th>
<th>D&lt;sub&gt;85&lt;/sub&gt;</th>
<th>D&lt;sub&gt;90&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>90</td>
<td>40</td>
<td>200</td>
<td>250</td>
</tr>
<tr>
<td>2</td>
<td>60</td>
<td>30</td>
<td>140</td>
<td>180</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>40</td>
<td>200</td>
<td>220</td>
</tr>
<tr>
<td>5</td>
<td>120</td>
<td>55</td>
<td>190</td>
<td>210</td>
</tr>
<tr>
<td>7</td>
<td>150</td>
<td>55</td>
<td>180</td>
<td>200</td>
</tr>
<tr>
<td>8</td>
<td>35</td>
<td>17</td>
<td>75</td>
<td>150</td>
</tr>
<tr>
<td>9</td>
<td>150</td>
<td>75</td>
<td>200</td>
<td>220</td>
</tr>
<tr>
<td>11</td>
<td>80</td>
<td>45</td>
<td>140</td>
<td>200</td>
</tr>
<tr>
<td>13</td>
<td>70</td>
<td>50</td>
<td>120</td>
<td>150</td>
</tr>
<tr>
<td>14</td>
<td>70</td>
<td>50</td>
<td>130</td>
<td>150</td>
</tr>
<tr>
<td>15</td>
<td>100</td>
<td>55</td>
<td>180</td>
<td>200</td>
</tr>
<tr>
<td>17</td>
<td>100</td>
<td>50</td>
<td>190</td>
<td>210</td>
</tr>
<tr>
<td>19</td>
<td>70</td>
<td>27</td>
<td>200</td>
<td>240</td>
</tr>
<tr>
<td>21</td>
<td>45</td>
<td>22</td>
<td>70</td>
<td>90</td>
</tr>
<tr>
<td>22</td>
<td>90</td>
<td>45</td>
<td>160</td>
<td>200</td>
</tr>
<tr>
<td>23</td>
<td>90</td>
<td>50</td>
<td>140</td>
<td>160</td>
</tr>
</tbody>
</table>

Average diameter: 88 44 158 189

Standard deviation: 32 15 44 41

#### 2.2 Triaxial testing.

Triaxial tests, consolidated undrained (CU) with pore pressure/measurement and consolidated drained tests (CD) yielded following results (Table B - 4).
**TABLE**: B-4. Triaxial test results.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Sample depth [m SCB]</th>
<th>Type of test</th>
<th>$q'_{c}$</th>
<th>$c'$</th>
<th>$[KN/m^2]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H2</td>
<td>9.4/- 10.0</td>
<td>CU + pp</td>
<td>20</td>
<td>10.0</td>
<td>**</td>
</tr>
<tr>
<td>H5</td>
<td>9.4/- 10.0</td>
<td>CU + pp</td>
<td>25</td>
<td>40.0</td>
<td>**</td>
</tr>
<tr>
<td>H5</td>
<td>16.4/- 17.0</td>
<td>CU + pp</td>
<td>21</td>
<td>33.5</td>
<td>**</td>
</tr>
<tr>
<td>H7</td>
<td>6.4/- 7.0</td>
<td>CU + pp</td>
<td>21</td>
<td>26.5</td>
<td>**</td>
</tr>
<tr>
<td>H11</td>
<td>15.6/- 16.2</td>
<td>CU + pp</td>
<td>32</td>
<td>10.0</td>
<td>**</td>
</tr>
<tr>
<td>H13</td>
<td>16.2/- 16.8</td>
<td>CU + pp</td>
<td>29</td>
<td>10.0</td>
<td>**</td>
</tr>
<tr>
<td>H17</td>
<td>17.5/- 18.1</td>
<td>CU + pp</td>
<td>8.5</td>
<td>50.0</td>
<td></td>
</tr>
<tr>
<td>H21</td>
<td>15.6/- 16.2</td>
<td>CU + pp</td>
<td>32</td>
<td>20.0</td>
<td>**</td>
</tr>
<tr>
<td>H8</td>
<td>0.6/- 1.2</td>
<td>CD</td>
<td>38</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>H8</td>
<td>3.6/- 4.2</td>
<td>CD</td>
<td>33</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td>H13</td>
<td>0.7/- 1.3</td>
<td>CD</td>
<td>41</td>
<td>47.0</td>
<td></td>
</tr>
<tr>
<td>H13</td>
<td>3.7/- 4.3</td>
<td>CD</td>
<td>40</td>
<td>25.0</td>
<td></td>
</tr>
<tr>
<td>H15</td>
<td>3.7/- 4.3</td>
<td>CD</td>
<td>30</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td>H20</td>
<td>4.0/- 4.6</td>
<td>CD</td>
<td>32.5</td>
<td>35.0</td>
<td></td>
</tr>
</tbody>
</table>

* Clay sample at greater depth.

The test results of the CU - tests also show that practically all samples developed negative pore pressures at strains of 2-5%. A number of samples clearly developed positive pore pressure with a maximum at 10-15% strain. They are indicated with **.

2.3 Critical Density.

Critical densities, sometimes expressed as the complementary function "critical void ratio", have been established for 5 samples. The results could be compared with the in-situ densities of undisturbed samples. Though some disturbance will always happen it is consultants opinion that sampling procedures, use of drilling equipment and sample recovery in the laboratory were carried out professionally resulting in negligible sample disturbances.

The results of the comparative tests are presented in following table: (Table B-5).
TABLE: Critical density verses in situ density.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Sample depth [m SOB]</th>
<th>Density [KN/m³]</th>
<th>Critical at 25 KN/m²</th>
<th>Critical at 50 KN/m²</th>
<th>In-situ</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>H3</td>
<td>0.5/-1.1</td>
<td>12.46</td>
<td>13.03</td>
<td>11.97</td>
<td>&lt; crit.*</td>
<td></td>
</tr>
<tr>
<td>H3</td>
<td>7.0/-7.6</td>
<td>15.36</td>
<td>16.02</td>
<td>12.34</td>
<td>&lt; crit.*</td>
<td></td>
</tr>
<tr>
<td>H11</td>
<td>3.7/-4.3</td>
<td>13.31</td>
<td>13.36</td>
<td>13.29</td>
<td>&lt; crit.*</td>
<td></td>
</tr>
<tr>
<td>H11</td>
<td>6.7/-7.3</td>
<td>13.94</td>
<td>14.18</td>
<td>15.25</td>
<td>&gt; crit.</td>
<td></td>
</tr>
<tr>
<td>H19</td>
<td>8.0/-2.6</td>
<td>13.48</td>
<td>13.78</td>
<td>14.05</td>
<td>&gt; crit.</td>
<td></td>
</tr>
</tbody>
</table>

Note: * The results confirm the density indications on the bore logs, denoting these layers as loose and very loose.

2.4 Atterberg Limits and compaction testing.

Attenburg Limits were established for two samples only as previous studies for structures in this area already indicated the non-plasticity of the prevailing fine sand and silt layers.

Both tests, taken from the top and near the foot of the existing ring bund, gave a plasticity Index of 11. Standard compaction on tests were carried out on the same samples giving a practical identical optimum moisture content (OMC) of 19% and a maximum dry density of respectively 16.3 and 16.9 KN/m³.
3. SEISMICITY AND LIQUEFACTION POTENTIAL.

3.1 Introduction.

Reference being made to various codes and geo-technical publications (Section 2.2) seismicity has to be allowed for. The design parameter for the seismicity evaluation is called the "Basic Seismic Coefficient (Z)".

According the seismic Zoning Map of Bangladesh Perni is located in Zone II with Z = 0.05.

The Indian code (IS : 1975) does elaborate on dam design. Comparative values from this code give \( \xi \alpha = 0.05 \) and \( Fo = 0.25 \). The latter value must be used when a Response Spectrum method is used. The values \( Z \) are used for the Seismic Coefficient Method.

The attached plot ( Fig. 2.1) with epicentra indicates that earthquakes with a magnitude of Richter 5-6 have been monitored relatively frequently at a distance of some 80 km (north and north-east of PERNI) and that a magnitude 7.6 was monitored at some 160 km due north of PERNI (1918).

Taking into account the nearby epicentra, magnitude 5-6 has an intensity at the location of the epicentra of approximately 7 on the Mercalli scale. This intensity implies maximum accelerations of 0.5 - 1 m/s. Allowing for geometrical damping this intensity will cause horizontal accelerations of about 5% of this value at PERNI being 0.003 - 0.005 g. A magnitude 7.6 earthquake at the same distance would result in an acceleration of about 0.03 to 0.05 g. This results covers well the previously indicated values.

2.2 Conclusion

Based on a Basic Seismic Coefficient \( Z = 0.05 \) we will now establish the design accelerations of \( \xi \alpha \) taking into account soil condition(s) importance factor (I) for the Seismic Coefficient method and additionally, according to the Indian Code, for the Response Spectrum Method, the seismic zone factor (Fo) and the average acceleration coefficient for appropriate natural period and damping of the structure.

This approach yields following results for:

**Basic Coefficient Method**

\[ \xi \alpha = \text{SIX} \]

\[ S = 1.2 \text{ to } 1.5 \quad (\text{acc. table 4.5 of seismic Zon.Map of B.}) \]

\[ I = 1.5 \quad (\text{" } 4.2 \text{ " of B.}) \]

We then find

\[ \xi \alpha = 1.2 \times 1.5 \times 0.05 = 0.09 \text{ g} \]

\[ 1.5 \times 1.5 \times 0.05 = 0.11 \text{ g} \]

average \( \xi \alpha = 0.10 \text{ g} \)
Response Spectrum Method

The theoretical principles of this method are not fully met by the actual dam body characteristics as the dam cannot be regarded as completely rigid above its base; however, with reference to the earlier mentioned Indian Code (Par. 7.4), following analyses have been made:

Fundamental period of structure,

\[ T = 2.9 \frac{H_T}{\sqrt{g}} \]

in which

\( H_T \) = dam height

: say 10 meter.

\[ C_s = \sqrt{\frac{g}{\rho}} \] = Shearwave velocity

For a saturated sand, one can choose as maximum value for \( C_s \) a value of some 250 m/sec resulting in a \( T = 0.1 \) sec.

For a softer material, with E-values of 20 - 40 LN/m², the period will be in the order of \( T = 4 \) sec.

Allowing for 10% damping, the average acceleration coefficient will be respectively 0.15 and 0.03 resulting in design accelerations of

\[ \alpha_h = 1.2 \times 1.5 \times 0.25 \times 0.15 = 0.07 \text{ g} \]

\[ \alpha_h = 1.2 \times 1.5 \times 0.25 \times 0.03 = 0.014 \text{ g} \]

\[ \alpha_h = 1.5 \times 1.5 \times 0.25 \times 0.15 = 0.08 \text{ g} \]

\[ \alpha_h = 1.5 \times 1.5 \times 0.25 \times 0.03 = 0.016 \text{ g} \]

It can be concluded from the previous analyses that with a range of horizontal design parameters as already incorporated in previous studies, of 0.05 - 0.1 g seismicity influences are well covered.
The liquefaction potential seems to be most critical at the location of Boring H3 where loose and very loose layers have been encountered to a depth of some 10 meters with some N-values being zero. To allow a reasonable design procedure according H.B. Seed's paper (See Section 2.2.1) we have introduced a N-value of e.g. 4 at a depth of some 5 meters and with an effective stress of 45 kN/m².

According figure 5 of said publication the N-value should be corrected for an overburden (effective) stress of 100 kN/m² giving:

\[ N^1 = 1.4 \times 4 = 5.6 \]

The cyclic stress ratio causing liquefaction can be determined from

\[ \frac{\sigma_{rav}}{\sigma'_0} = 0.65 \frac{\alpha_h}{g} \times \frac{\sigma_v}{\sigma'_0} \times r_d \]

in which

\[ \alpha_h = \text{Maximum acceleration} \]

\[ \sigma'_0 = \text{Total overburden stress on sand layer under consideration} \]

\[ \sigma'_0 = \text{Effective overburden stress on sand layer under consideration.} \]

\[ r_d = \text{Stress reduction factor, between 0.9 and 1.0.} \]

The stress ratio \( \frac{\sigma_v}{\sigma'_0} \) taking into account the established wet and dry densities, is of the order of 1.35.

We then find for \( \alpha_h = 0.1 \ g \)

\[ 0.65 \times 0.1 \times 0.95 \times 1.35 = 0.083 \]

Taking into account the lower bound values in Fig. 24 of earlier mentioned publication it can be concluded that the above ratio does exceed the lower bound values dramatically.

Conclusively it means that liquefaction has to be expected in certain parts of the subsoil depending on intensity, damping, etc., etc.
ANNEX - C

BED PROTECTION AND SILL DETAILS.


The joint system of bed protection mattress and sill should be able to withstand the maximum current velocities during the construction period. These maximum velocities occur during a severe winter spring tide, which has been assumed to reach a level of SOB + 4.60 m.

The mathematical model has provided the specific discharge \( q \) and water height \( h_b \) above the "starting point" of the sill as indicated in the sketches below.

---

![Sketch of Sill on Shoal](image)

![Sketch of Sill in Gully](image)
The velocity at the "starting point" of the sill can easily be calculated by dividing $q_\perp$ by $h_{\perp}$. The velocity at the end will be higher, as he is (always) smaller than $h_{\perp}$.

Though he can reach very low values, the maximum current velocities at the "end point" of the sill always occur when he has a substantial height. (region of 1.50 m).

The calculated maximum velocities do not exceed 1.8 m/s and 2.1 m/s at any point on the shoal and on the gully mattress/sill system respectively.
2. **Size of ballast.**

The size of individual ballast elements, able to withstand current forces by their own weight only, depends on:

- current velocity
- water depth
- specific density of ballast material.

Stone diameters may be calculated with the following formula:

\[
D_{50} = \frac{\alpha \sqrt{V}}{\Delta 2 \gamma}
\]

in which,

\[
\begin{align*}
D_{50} & = \text{stone diameter exceeded by 50\% of the stones.} \\
V & = \text{current velocity} \quad (\text{m/s}) \\
\Delta & = (\rho_s - \rho_w) / \rho_w \quad (\text{1m}) \\
\rho_s & = \text{specific density of stone} \quad (\text{kg/m}^3) \\
\rho_w & = \text{specific density of (sea) water} \quad (\text{kg/m}^3) \\
\alpha & = \text{turbulence factor} \quad (1) \\
\gamma & = \text{acceleration of gravity} \quad (\text{m/s}^2)
\end{align*}
\]

For \( V = 2 \text{ m/s} \) and \( \rho_s = 2650 \text{ kg/m}^3 \) (for boulders), the required size of stones is:

\[
D_{50} = 0.20 \text{ m}
\]
### Table D-1. Return Periods of Highest Annual Water Levels near Peninsula Site

<table>
<thead>
<tr>
<th>Return Period in Years</th>
<th>Water Level in ft. + SOB</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.6</td>
</tr>
<tr>
<td>2</td>
<td>22.12</td>
</tr>
<tr>
<td>5</td>
<td>24.57</td>
</tr>
<tr>
<td>10</td>
<td>26.44</td>
</tr>
<tr>
<td>20</td>
<td>29.15</td>
</tr>
<tr>
<td>50</td>
<td>32.87</td>
</tr>
</tbody>
</table>

### Table D-2. Normal Operating Reservoir Level

<table>
<thead>
<tr>
<th>Water Level in m + SOB</th>
<th>SOB + 6.5 ft, ( = 21.8 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.99</td>
<td></td>
</tr>
<tr>
<td>7.05</td>
<td></td>
</tr>
<tr>
<td>7.49</td>
<td></td>
</tr>
<tr>
<td>8.06</td>
<td></td>
</tr>
</tbody>
</table>

### Annex D. Calculations of Size of Slope Protection

#### General

Data relevant to the design of the slope protection system is as follows:

- Data was collected for the period 1957 - 1977. The highest tide level observed in June (20.7 ft) and October (25.4 ft).
- The peak discharges at Vizhinjam are not available. The discharge at Vizhinjam is the result of large river and stream contributions.
- Therefore, statistics of highest annual water levels are given in Table D-1.

#### Data

Data was collected for the period 1957 - 1977.

#### Water Levels at Sea Side of Dam

<table>
<thead>
<tr>
<th>Return Period in Years</th>
<th>Water Level in ft. + SOB</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.6</td>
</tr>
<tr>
<td>2</td>
<td>22.12</td>
</tr>
<tr>
<td>5</td>
<td>24.57</td>
</tr>
<tr>
<td>10</td>
<td>26.44</td>
</tr>
<tr>
<td>20</td>
<td>29.15</td>
</tr>
<tr>
<td>50</td>
<td>32.87</td>
</tr>
</tbody>
</table>

### Table D-2. Normal Operating Reservoir Level

<table>
<thead>
<tr>
<th>Water Level in m + SOB</th>
<th>SOB + 6.5 ft, ( = 21.8 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.99</td>
<td></td>
</tr>
<tr>
<td>7.05</td>
<td></td>
</tr>
<tr>
<td>7.49</td>
<td></td>
</tr>
<tr>
<td>8.06</td>
<td></td>
</tr>
</tbody>
</table>

### Minimum Operating Reservoir Level

SOB + 6.5 ft, ( = 21.8 m)
1.4 Waves at the sea side.

Calculation of the significant wave height to be expected in front of the Feni River Closure Dam has been made using data taken from the U.S. Navy, Marine climates Atlas of the World, Vol. III, Indian Ocean, as explained in *(1).*

It was thus explained that an offshore significant wave height $H_s$ of 12 ft. during June would reduce to 5.6 ft. approximately at a significant time period of 7 sec. at the mouth of the Feni River. At mid-tide (11.4 ft.) wave height $H_s$ would be 3.3 ft. at a period of 6 sec.

Also a calculation of the locally generated waves is given. A $H_s = 4.65$ ft. at a time period of 4.8 Sec. was found.

Consultant regrets that no direct wave observations and measurements have been made during the many years lapsed since M.I.P. was studied for the first time. Now he has no other option then to use the information contained in *(1)* which, as such, does not give reason for criticism other than that conclusions might well be considered to be conservative in some respects.

For the dam design a $H_s = 5.6$ ft. and a time period of 7 s will be taken into account. The orientation of the dam axis in relation to the expected direction of movement of waves is such that if the angle $\beta$ (See Section 2.1 of this Annex) at the old closure site was zero, it can be considered to be between 30° and 40° at the new closure site after taking into account refraction.

1.5 Waves at the river (= reservoir) side.

The information presented in *(1)* was studied and it was concluded that it would appear to be less realistic to combine flood stage of the reservoir with a high wind speed, i.e. high waves, because the rains causing the high water levels in the Feni Reservoir will be induced by winds from southern directions. In TABLE D-2 the final data to be used are presented.

<table>
<thead>
<tr>
<th>Reservoir level in S.C. (m)</th>
<th>Wind speed (MPH)</th>
<th>$H_s$ (m)</th>
<th>Period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.91</td>
<td>25</td>
<td>0.64</td>
<td>2.5</td>
</tr>
<tr>
<td>3.81</td>
<td>50</td>
<td>0.98</td>
<td>3</td>
</tr>
<tr>
<td>1.98</td>
<td>50</td>
<td>0.58</td>
<td>2.5</td>
</tr>
</tbody>
</table>
2. SLOPE PROTECTION SEA SIDE (MAIN DAM)

2.1 Wave run-up.

Wave run-up is calculated for the upper part of the slope by using data presented in sections 1.2 and 1.4 and by adopting the following formula

\[ Z = 3.9 \times f \times H_s \times \tan \alpha \times \cos \beta \]

in which:

- \( Z \) = Wave run-up in metres exceeded by 2\% of the waves;
- \( f \) = friction coefficient, which for concrete blocks is 0.95;
- \( H_s \) = Significant wave height in metres at foot of slope;
- \( \alpha \) = gradient of slope, taken at 1/6;
- \( \beta \) = angle between direction of wave front and perpendicular line on dam axis.

The crest height is now determined by combining the design water level \( (S_C + \text{8.1 m at a frequency of once in 50 years}) \) with the wave run-up for a \( H_s = 1.7 \text{ m} \). The formula mentioned above gives the following result for the wave run-up:

\[ Z = 8 \times 0.95 \times 1.7 \times 1/6 \times \cos 30^\circ \]

\[ = 1.96 \text{ m} \]

This means that a crest level of \( S_C + 10.0 \text{ m} \) has to be adopted.

It must be pointed out that the formula used for calculating the wave run-up is in fact valid only for relatively small wave periods. Accordingly, the wave run-up in reality may turn out to be higher. (It is, however, the intention to protect the whole dam "skin" by a stone cover more than 2 % over topping can be accepted *)

A calculation was made by using other wave run-up formulas to check the possible higher wave-run-up due to the higher wave period. However, no significant difference was found and, consequently, the crest level of \( S_C + 10.00 \text{ m} \) (= value after initial settlement of dam body) will be maintained.

When the angle \( \beta = 40^\circ \) instead of \( 30^\circ \) (Section 1.4 of this Annex,) the wave run-up will reduce to 1.65 m.

Finally, local wind set-up was calculated at 0.20 - 0.30 m which does not change the wave run-up considerably.

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Note: *) \( Z(2\%) \) has been derived from dykes covered by grass on clay.
2.2 **Block size.**

For calculating the block size on the upper part of the slope the formula of Hudson is used:

\[
G = \frac{\rho_b \times g}{(\rho_b - \rho_w)^3 \times k_d} \times H_s^3 \times \tan \alpha
\]

in which:

- \( G \) = Weight of blocks in Newtons
- \( g \) = acceleration of gravity
- \( \rho_b \) = specific density of blocks in kg/m³
- \( \rho_w \) = specific density of water in kg/m³
- \( k_d \) = coefficient depending on material used (concrete blocks: 8)
- \( H_s \) = significant wave height in metres
- \( \alpha \) = gradient of slope.

For \( H_s = 1.7 \) m (water level SOB + 8.1 m):

- \( 4g \times \alpha = 1/6 \)
- \( \rho_b = 2250 \) (concrete blocks, aggregate crushed boulders)
- \( \rho_w = 1016 \)
- \( G = 1260 \) Newton.

A block size of 0.50 x 0.50 x 0.25 m satisfies the requirements (\( G = 1380 \) N).

In case concrete blocks with chipped bricks would be taken the weight and size would be much larger as the specific density of these blocks is much lower (\( \rho_b = 1860 \) kg/m³; \( G \) would be 3200 N; size would be 0.65 x 0.65 x 0.40 m).

Concrete blocks with chipped bricks were in fact selected for the lower part of the slope. Blocks of 0.50 x 0.50 x 0.20 satisfy the requirements.
3. SLOPE PROTECTION RESERVOIR SIDE (MAIN DAM).

3.1 Wave run-up.

Water levels of the Reservoir in combination with the waves as presented in TABLE D - 2 are important for the design of the revetment. Following wave run-ups are found:

Flood stage $SGB + 4.90 \text{ m } Z 2\% = 1.2 \text{ m}$

Normal max. $SGB + 3.80 \text{ m } = 1.9 \text{ m}$

Normal min. $SGB + 2.00 \text{ m } = 1.1 \text{ m}$

Consequently, wave run-up will reach the following elevations:

Flood stage : $4.90 + 1.2 = SGB + 6.1 \text{ m}$

Normal max : $3.8 + 1.9 = SGB + 5.7 \text{ m}$

Normal min : $2.0 + 1.1 = SGB + 3.1 \text{ m}$

It can be concluded that wave attack is concentrated in the zone of $SGB + 4.50 \text{ m} \text{ to } SGB + 6.1 \text{ m}$, wave run-up is limited to levels between $SGB + 3.1 \text{ and } SGB + 6.1 \text{ m}$.

3.2 Revetment lower part.

As the lower part, say under $SGB + 3.00 \text{ m}$, has to be constructed under water a mattress with stones (rip-rap) will act as protection.

For $H_s = 1.00 \text{ m}$ and a water level of $SGB + 3.8$ the boulder weight is calculated by using the formula of Hudson (Section 2.2)

$$H_s = 1.00 \text{ m}$$

$$\phi = 9$$

$$\rho_b = 2650 \text{ (Specific density of boulders)}$$

$$\rho_w = 1005 \text{ (brackish water)}$$

$$K_d = 6 \text{ (damage percentage 1 - 5)}$$

It is found that:

$$J = 247 \text{ N}.$$

Boulders having a diameter of $D_b = 0.30 \text{ m}$ satisfy the criterium and the grading should be from $0.25 - 0.35 \text{ m}$ diameter.
3.2 Revetment upper part of slope.

For the upper part of the slope, a water level of $500 + 4.90$ m and a significant wave height $H_s = 0.64$ m is taken as design criterion.

When using chipped bricks as aggregate concrete blocks having a size of $0.50 \times 0.50 \times 0.20$ satisfy the requirements.

Because $H_s = 0.64$ m
$\gamma = \gamma$
$\rho_b = 1950$
$\rho_w = 1005$
$\rho_d = 3$

Which gives $G = 245$ N.
4. SLOPE PROTECTION FLANK EMBANKMENT (SEA SIDE)

4.1 Adjacent to main dam.

The average ground level is SGB + 4.25 m at the Left Bank and SGB + 3.35 m at the Right Bank.

The whole Flank embankment at the Left Bank and the part of the Flank embankment adjacent to the main dam at the Right Bank will be exposed to severe wave attack.

At design water level ( SGB + 8.1 m )
water depths are:

- **Left Bank**: SGB + 3.9 m
- **Right Bank**: SGB + 4.8 m

According to an (unpublished) report of Ryks water start (The Netherlands) at these depths maximum significant wave heights could be:

- **Left Bank**: \( H_s = 2 \) m
- **Right Bank**: \( H_s = 2.3 \) m

This means in fact that the wave height of 1.7 metres mentioned in Section 2.1 of this Report for the water level of SGB + 8.1 m is not reduced by limited depth and the revetment has to be same as for the main dam (See side).

4.2 Parallel to inlet channel to Regulator (Right Bank).

The direction of the embankment in relation to the waves is such that a slight slope protection is acceptable. It is therefore proposed to utilize a "local" slope protection system consisting of:

- 0.20 m fine filter
- 0.20 m coarse filter
- brick blocks 0.25 x 0.50 x 0.50

while the clay layer and the fabric can be omitted.

4.3 Adjacent to Regulator (Right Bank).

Because of the exposed location of this part of the flank embankment and the reflection of waves by the Regulator wave wall size and height of revetment blocks has to be as calculated for main dam (upper part, sea side) while also a toe protection is required.
Size of ballast on toe protection is determined by \( a H_s = 1.00 \) m at mid-tide level of \( S_{OB} + 3.48 \) m. It is further assumed that \( \lambda_d = 5 \), \( \rho_b \) for boulders is 2650, while slope of foreshore is not supposed to exceed 1 in 5.

For these conditions Hudson's formula gives a blockweight \( G = 214 \) Newtons, which means that boulders having a \( D_{50} = 0.25 \) m satisfy the requirements.
ANEX - B

REFERENCES


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(3) (Draft) Report on the redesign of the Feni River Closure Dam, June 1982 (Luhuri Irrigation Project, Design Cell II, BWDB).

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(4.1) First Interim Report, October 1976
(4.2) Second " " August 1977
(4.3) Third " " August 1978
(4.4) Fourth " " August 1979
(4.5) Fifth " " August 1980
(4.6) Sixth " " July 1982
(4.7) Design for closing the Chakamaya Khal in Patuakhali District, August 1978.
(4.9) Review of the Programme on the closure of tidal channels in Bangladesh, June 1982.

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