PEOPLE'S REPUBLIC OF BANGLADESH

Ministry of Irrigation, Water Development and Flood Control

Bangladesh Water Development Board

CYCLONE PROTECTION PROJECT II - FAP 7
FEASIBILITY AND DESIGN STUDIES

FINAL PROJECT PREPARATION REPORT
APPENDIX C - EMBANKMENT DESIGN

May 1992

Joint Venture of
KAMPSAX INTERNATIONAL A/S,
BCEOM
DANISH HYDRAULIC INSTITUTE
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2

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REPORT VOLUMES

The present Report Volume is part of the

CYCLONE PROTECTION PROJECT II - FAP 7
FEASIBILITY AND DESIGN STUDIES
BWDB COMPONENT
FINAL PROJECT PREPARATION REPORT

Consisting of the following Volumes :

Volume 1	-	Main Report
Volume 2	-	Annexes I - XI, XIII
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Appendix A	-	Hydraulic Studies
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1. INTRODUCTION

The present report is Appendix C to the Preparation Report prepared as a conclusion of the feasibility study conducted under Cyclone Protection Project II.

The report contains the background for the Consultant's recommendations on the design of sea facing embankments to be constructed under the Mid Term Programme.

Embankments included in Phase 1 of this programme - the Emergency Cyclone Protection Project - have been designed in detail by the Consultant and Tender Documents have been prepared for International Competitive Bidding in Februar 1992.

Main findings, conclusions and recommendations included in the present report are presented in outline form in the Main Report and in its Annex II.

A separate analysis of alternatives for the embankment in polder 62, Patenga is found in Appendix I to the Main Report.

2. SUMMARY AND RECOMMENDATIONS

2.1 Existing Embankments

- The poor status of the existing sea facing earth embankments is mainly a result of erosion of the outer slope, the crest and the inner slope of the embankments. The wave erosion of grass covered outer slopes is a result of frequent excessive run-up and run-down velocities on too steep slopes that are poorly compacted. Erosion of crest and inner slope is mainly due to overtopping by waves in extreme situations. Lack of repair of the erosion damages has resulted in progressive deterioration of the embankments.

  The embankment slopes are originally designed 1:7 but have been steepened partly due to erosion and partly due to human activities.

- Protective works consisting of placed concrete blocks are generally underdesigned and most of them have suffered from extensive damages caused by extreme monsoon waves. The April 1991 cyclone has left the majority of the protective works completely destructed. The placed concrete units are not stable to even moderate wave action if (and when) they are dislocated due to settlements and erosion of the filter materials.

Further background is given in section 3 and 4 of the present report.

2.2 Repair and Strengthening of Embankments

- Armoured embankments for protection of agricultural land and the population living in these areas is generally not economically feasible or justifiable.

- The Consultant recommends the main protective embankments generally to be constructed as compacted earth embankments with a protective layer of well compacted clay soil with grass turfing and minimum 100-200 m foreshore or foreland with afforestation where-ever possible.

- It is recommended that slopes be 1:7 on the seaward side and 1:3 on the country side and that crest levels be established according to a technical /economical optimization.

- The optimum crest levels are obtained by applying the following design criteria:

  All polders in the emergency construction programme, except Polder 62:

- The return period of **monsoon** design condition has been set to 5 years. 'No' overtopping should occur in this situation (only 13 % of the waves should overtop).

- The return periods of **cyclonic storm** design conditions has been set to:

  - 20 years, where flooding due to wave overtopping of the sea facing embankment should not result in average water depth in the polder exceeding 1.0 m

  - 40 years, where the crest level should not be lower than the still water level

For Polder 62, Patenga:

- 'No' overtopping shall take place in **cyclonic storm** with return period 40 years (only 13 % of the waves should overtop)

- Resulting crest levels vary between 6.0 and 8.5 m above Public Works Datum (PWD)

- Embankment heights vary between 4 and 6 m.

- Materials for embankment core should be silt and clay materials. Hydraulic fill will be allowed for the Patenga embankment only.

- Materials for cover layer (1 m thickness on outer slope) should be good quality clay materials.

- Protective works are proposed only on eroding coasts in embankment sections where retirement is not possible.

- Protection works are proposed as revetments consisting of stone or concrete blocks in random placement.

Further background is given in section 6, 7 and 8 of the present report.

2.3 Erosion of Embankments

- The yearly average erosion of outer slope, crest and inner slope is estimated to 2.5 m^3/m embankment for all sea facing embankments under the Emergency Cyclone Protection Project (except 62, Patenga).

  For Patenga South the average erosion is estimated to 0.4 m^3/m.

- The yearly average erosion of outer slope is estimated to 1 m^3/m embankment for polders of the phase 2 programme outside the most cyclone prone area.

- The average yearly damage to protective works is estimated to 5% of the armour layer (0.5 m^3/m).

- The erosion will be reduced by approximately 10-20% if a 200 m wide belt of afforestation be established on the foreshore.

Further background is given in section 9 of the present report.

3. PRESENT EMBANKMENT DESIGN

3.1 Earth Embankments

The existing coastal embankments in the project area have mainly been constructed since 1961 under CEP (Coastal Embankment Project).

The design characteristics are described in the report "Coastal Embankment Project, Engineering and Economic Evaluation", prepared in 1968 by Leedshill-DeLeuw Engineers [1].

Three types of embankments have been designed (standard cross-sections are shown in Enclosure 1):

Sea Dikes are used at locations where high waves may be expected i.e. at locations facing the Bay of Bengal and along the banks of major rivers or channels.

Design :
- Sea side slope 1:7
- Country side slope 1:2
- Crest width 4.27 m (14 feet)
- Crest level equal to normal maximum recorded water stage plus 1.52 m (5 feet).
- Set back distance of sea side embankment toe: minimum 76 m (250 feet)

Interior Dikes are provided at relatively protected locations along major streams or along exposed sections of secondary streams or interior khals.

Design :
- Sea side slope 1:3
- Country side slope 1:2
- Crest width 4.27 m (14 feet)
- Crest level equal to normal maximum recorded water stage plus 0.91 to 1.52 m (3 to 5 feet), depending on actual exposure to waves
- Set back distance of sea side embankment toe: minimum 53 m (175 feet)

Marginal Dikes are used on locations on interior khals where the current and wave action is mild.

Design :
- Sea side slope 1:2
- Country side slope 1:2
- Crest width 2.44 m (8 feet)
- Crest level equal to normal maximum recorded water stage plus 1.52 m (3 feet)
- Set back distance of sea side embankment toe: minimum 38 m (125 feet)

Crest elevations for these embankments have been set in relation to the **maximum normal high tide** as recorded at a number of stations in the period 1960 to 1968. Records of unusually high water stages that is known to have ocurred during a cyclonic surge have been excluded.

Embankment material has been placed by hand and no compactive effort, other than incidentally by workmen during construction, has been applied. The embankments were expected to consolidate naturally over a period of two to three years by the rise and fall of the tides and the action of rainfall percolating downward.

Embankments have been intended constructed to a 20 % greater height than required by the design crest elevation to allow for settlement and consolidation.

Materials have been taken from borrow pits located on the sea side of the embankment (see Enclosure 1). Organic and sandy materials have been avoided for embankment use *if possible*. Where sandy material has been used, efforts have been made to obtain and place a blanket of cohesive earth over the sand embankment to encourage good turfing and reduce surface erosion.

Grass turfing, using locally established grasses, has been specified for the slopes and tops of all embankments.

3.2 Protective Works

The design prepared by Leedshill-DeLeuw did not include hard materials for protection of the embankment slopes against wave erosion.

However, the following methods were proposed for protection of already constructed embankments threatened by erosion from waves:

- Soil cement mixture placed in jute bags, thoroughly compacted, and placed in layers on a prepared embankment slope and into a cut-off trench at the embankment toe.

- Timber or bamboo piling spaced closely together and driven into the ground at the toe or along the slope of the embankment.

- Broken or otherwise damaged concrete pipe placed along the toe of the slope and allowed to become filled with soil

Due to poor performance, none of these methods have however found widespread use and types of protective works found in existing coastal embankments are mainly of the following types:

- Revetment consisting of precast concrete blocks, size 0.63 x 0.63 x 0.39 m placed on a geotextile and a khoa filter of 0.23 m thickness (crushed bricks). The concrete blocks are covering the entire seaward slope

from toe to crest. Slopes vary from 1:3 to 1:5.
The toe is in some cases protected by a toe wall and in situ concrete apron blocks of size 1.27 x 1.27 x 0.61 m.

According to the design, the joints between the individual blocks are open. In some cases, however, the joints have been sealed with cement mortar.

Typical cross-section of the revetment designed for use in Polder 62, Patenga is shown in Enclosure 2 [2].

- Revetment consisting of a slab constructed by 3-4 layers of bricks placed in cement mortar. The slabs are in some cases extending from toe to crest of the embankment (Polder 70 Matherbari and Polder 64/1A Banskhali), but in other cases the revetment is covering the lower part of the slope only (in Polder 66/3 Cox's Bazar up to level +3.6 m).

  The brick slab is compartmentalized by construction of vertical masonry walls in a grid of size approximately 8 x 8 m. The revetment toe consists of a masonry wall being part of this grid.

  Sea side slope is typically 1:3.

- Revetment consisting of rubble stones placed on the lower part of the sea side slope (Polder 61/1 Sitakunda and Polder 71 Kutubdia). Stone sizes vary widely up to diameters approximately 0.4 m.

  In general no graded stone filter or geotextile has been applied.

4. TECHNICAL EVALUATION OF THE PERFORMANCE OF EXISTING EMBANKMENTS

The performance of the existing coastal embankments - as related to current design and construction/maintenance practice - has been evaluated in the following.

The Consultant has monitored the condition of the coastal embankments through field surveys conducted in the period April 1990 to March 1991. The results of these surveys are presented in Volume 2 Annex I to the main report.

For some of the polders the condition of the embankments changed radically during the cyclone on 29 April 1991. After the cyclone field surveys have been conducted in Polder 62 Patenga only. For other polders heavily affected by the cyclone, the changes in condition have been roughly assessed on basis of field inspections and aerial reconnaissances from helicopter.

4.1 General on Modes of Embankment Failure

In general the following main events typically leads to embankment failure:

- Erosion of the outer slope of grass covered earth embankments
- Loss of stability of revetment
- Erosion of the inner slope
- Geotechnical failures

Two different degrees of failure is defined:

- **limited failure**, where the embankment itself is damaged but no unacceptable inundation of the polder takes place. The type and extent of damage will depend of the duration and degree of overloading taking place.
- **ultimate failure**, where the overloading leads to breaching of the embankment cross section followed by unacceptable inundation of the polder. The ultimate failure can be the result of extreme overloading but will often be the result of a number of repeated limited failures remaining unrepaired.

Damages to an embankment will occur if the external load on the embankment exceeds the actual strength of the embankment structure.

The actual strength is defined by the actual geometry of the embankment (original design and present state of maintenance) and the properties of the materials involved (embankment material and sub-soil).

4.2 Erosion of the Outer Slope of Grass Covered Earth Embankments

Erosion of the outer slope due to wave rush-up and rush-down has been found to be a major cause for failure of many of the embankments surveyed.

4.2.1 Location of Erosion

The area that is exposed to wave loading is bounded by the highest up-rush and the lowest down-rush point for the waves. This zone obviously varies with the tide.

The coastal embankments are all located in the tidal region and the joined frequency distribution of water levels and waves is therefore most relevant when evaluating the distribution of wave erosion on the outer slope (see Section 7.1).

The erosion takes place in the following approximate zones (see Figure 4.1), defined by the still water levels in front of the embankment:

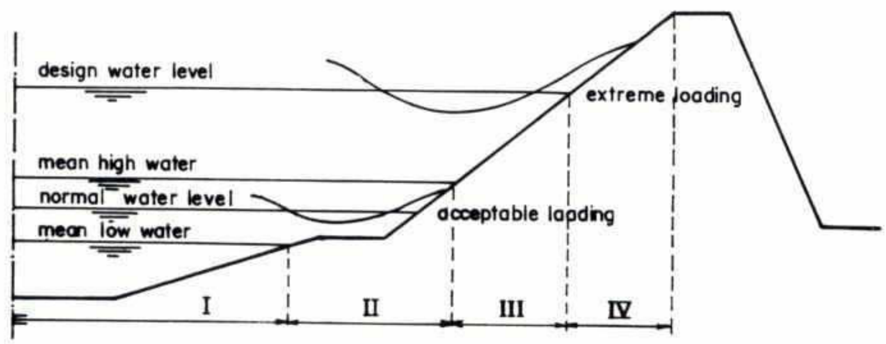

Figure 4.1 : Damage zones.

I The zone permanently submerged.

Due to small waterdepths the direct wave loading is moderate.

Not present in the case of a high foreshore. Embankments on generally eroding coasts may originally have had high foreshore that has now vanished. The lower embankment slope has thus become part of the beach and the erosion of it then a part of the general beach erosion - it being rapid or slow.

Limited failures in some cases, but in other cases developed into ultimate failures where beach erosion could not be stopped (polders 62 Patenga, 71 Kutubdia and 72 Sandwip and others).

II The zone between MLW and MHW.

If no high foreshore exists the ever present wave loading of relatively low intensity is of importance for the long-term behaviour of the embankment.

Limited failures (polder 59/3B) that have developed into ultimate failures since not repaired (polders 72 Sandwip, 64/1A Banskhali and 63/1A Anowara).

III The zone between MHW and the design water level.

The intensity of wave attacks in this zone can be heavy but the frequency of such attacks reduces when going up the slope.

Limited or ultimate failure depending on the duration and the intensity of the wave attack.

IV The zone above design water level.

In the design situation there will only be wave run-up in this zone. In case of exceedance of the design situation the wave attack can be heavy due to great water depths and corresponding big waves.

The design water level for the existing CEP embankments is 'maximum normal high tide' as registered over a certain period (1960-1969). The frequency of exceeding this level **under normal monsoon conditions** is estimated to approximately 3 hours per 20 years on average.

Limited failure occurs if the water level is extremely high (overtopping and only little wave run-down) and if the duration of the exposure is relatively short. In this case the major damages occur to the crest and to the inner slope (see 4.3 below).

This situation was experienced during the April 1991 cyclone where some embankments were found stripped for grass cover at the crest but with only minor erosion to the outer slope itself (polder 63/1A Anowara, chainage 41 to 43 km).

Ultimate failure will occur if erosion in this zone progresses due to long duration of wave attack at a water level between the design water level and the crest level. Heavy overtopping and later overflow will result in complete breaching as described in 4.3 below.

4.2.2 Depth of Erosion

No reliable formula is available for calculation of the depth of erosion in a certain point of an embankment slope being exposed to waves. Although the design shows proper grass covering of the slopes, it is often found that this cover is in very poor shape due to human activities and lack of maintenance.

Full scale tests has been carried out in the Netherlands with grass mats on clay on slope 1:4 and with unprotected clay surfaces with slope 1:3.5 [3].

All the tests indicated that the strength of the grass slopes is strongly affected by the quality of the clay and the condition of the grass and its rooting.

For the grass mat the tests showed erosion speeds of 1 to 2 mm per hour during up to 20 hours testing (for average water velocities 2 m/s and maximum velocities 4 m/s). After 20 hours of loading the erosion speed started to grow very progressively for a bad quality grass mat. Similar process took place for a good quality grass mat, but after 40 hours of loading.

After 5 hours loading of the unprotected clay surface with water velocities up to 3 m/s some 2-3 cm erosion was experienced in the upper part of the slope. Below still water level the depth of erosion was 7 cm for good clay, while it was 40 cm for a lean clay.

4.2.3 Maximum Non-scouring Velocities

Pilarzcyk [3] recommends that the limit water velocity in wave up-rush for non-scouring condition be set to 2 m/s for grass mat on poor clay and 3 m/s for grass mat on proper clay.

The maximum velocity in the up-rush (appearing in the area where the water particles passes the position of still water level) can be estimated by the following formula for smooth slopes [3]:

$U_{max} = A\, \xi^{0.5} \sqrt{2gH}$, where

A = Coefficient ( )
ξ = Breaker index ( )
g = Acc. of gravity (m/s^2)
H = Wave height (m)

The breaker index ξ, is defined by the formula:

$\xi = (H/L_o)^{-0.5} \tan\alpha$, where

L_o = Wave length at deep water (m)
α = Slope angle

The following average values of the coefficient A can be used [4]:

Slope	1:3	1:5	1:7
A	0.6	0.5	0.5

8

Three examples are given in the following to demonstrate that the erosion experienced on the seaward slopes of many embankments is due to too steep seaward slopes which result in too high up-rush water velocities.

In all three examples the fully developed wave heights are estimated as 80% of the actual water depth. Wave period 8 s has been applied.

It should be noted that due to erosion many sea dikes have developed seaward slopes 1:3 to 1:5 in spite of the design showing 1:7 slopes.

Example 1: Polder 64/1A, Banskhali

Design water level for existing embankment : + 4.6 m (PWD)
Mean High Water Springs : + 2.4 m
Existing seaward slope 1:3 and 1:5
Foreshore ground level + 1.7 m

The seaward slopes of this embankment was heavily eroded prior to the April 1991 cyclone. Protective works were being carried out in order to stop this erosion. During the cyclone the embankment suffered ultimate failure in many places.

Figure 4.2 to 4.4 below show the maximum up-rush velocities as calculated by the above formulas for different water levels varying between MHWS and present design water level.

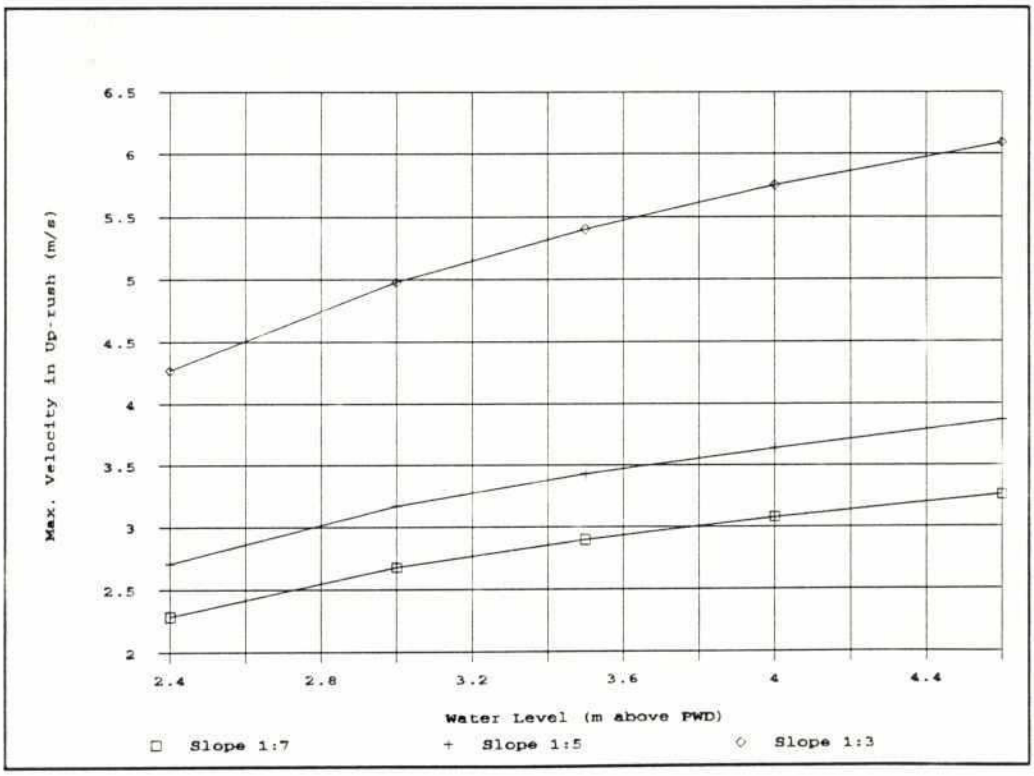

Figure 4.2: Maximum Up-rush Velocities, Banskhali

For slope 1:3 the limit velocity is heavily exceeded for all water levels and wave erosion in the grass cover will take place in all zones on the slope.

For slopes 1:5 the limit velocity for poor clay (2 m/s) is exceeded in all zones, whereas proper clay will only be eroded above level + 2.8 m (zone III and IV).

For slopes 1:7 the limit velocity for poor clay is exceeded in all zones, whereas proper clay will only be eroded above level + 3.8 m (zone III and IV).

Example 2: Polder 61/1, Sitakunda, Chainage 0 to 21 km

Design water level for existing embankment: + 5.2 m
Mean High Water Springs : + 2.9 m
Existing seaward slope 1:5
Foreshore ground level + 2.2 m

The seaward slope was in fairly good shape prior to the April 1991 cyclone, and damage during the cyclone was mainly appearing in the upper zone (zone IV). There were no ultimate failures due to erosion of seaward slope.

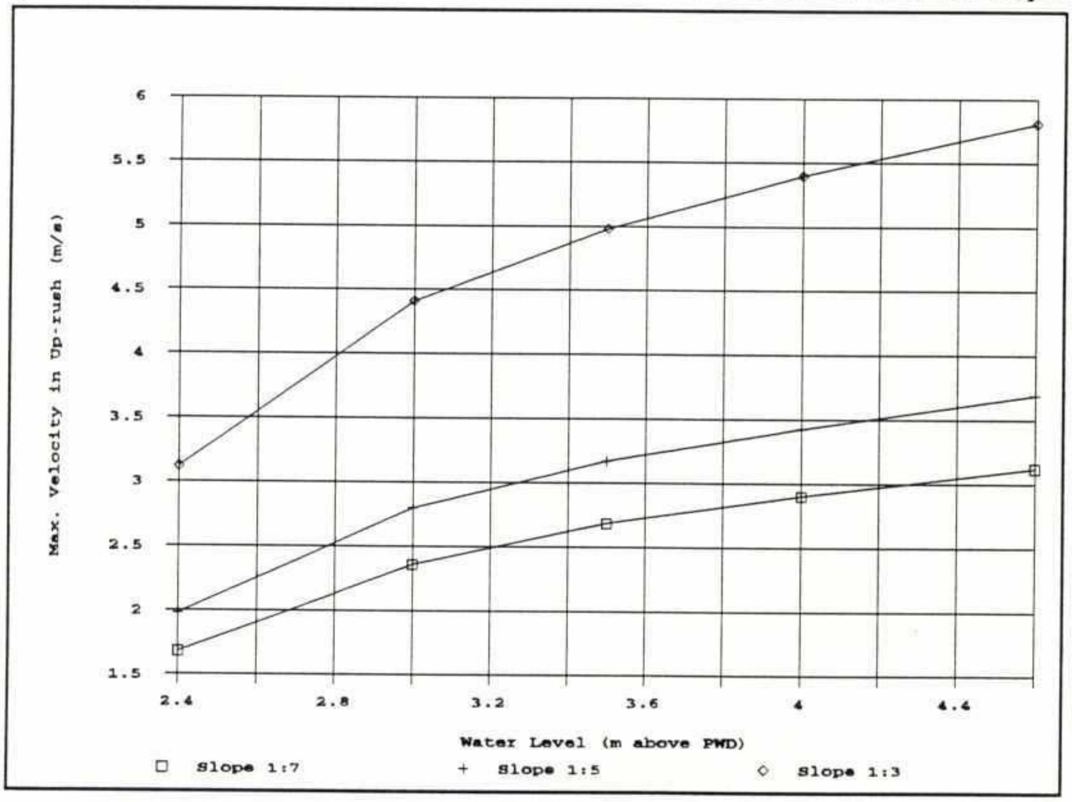

Figure 4.3: Maximum Up-rush Velocities, Sitakunda

For slope 1:3 the limit velocity is heavily exceeded for all water levels and wave erosion will take place in all zones on the slope.

For slopes 1:5 the limit velocity for poor clay (2 m/s) is exceeded in all zones, For slopes 1:5 the limit velocity for poor clay (2 m/s) is exceeded in all zones, whereas proper clay will only be eroded above level + 3.2 m (zone III and IV).

For slopes 1:7 the limit velocity for poor clay is exceeded above level +2.7 m, whereas proper clay will only be eroded above level + 3.8 m (zone III and IV).

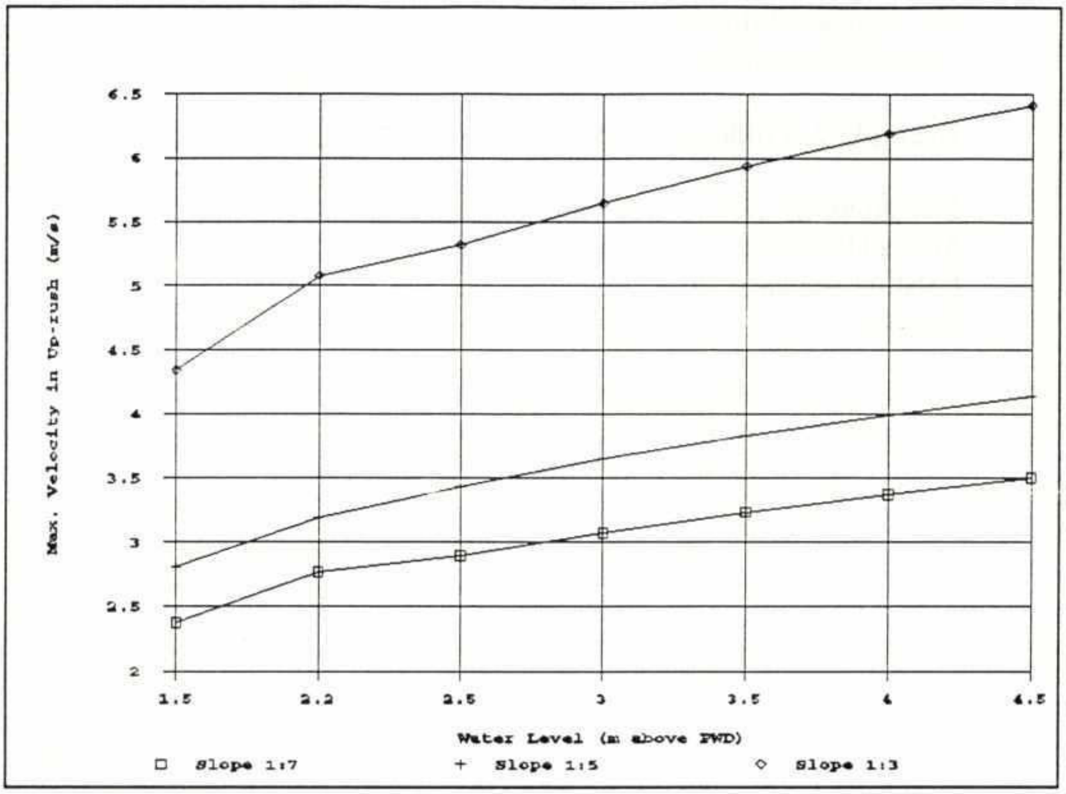

Figure 4.4 : Maximum Up-rush Velocities, Moheskhali

Example 3: Polder 70, Matherbari

Design water level for existing embankment: + 3.8 m
Mean High Water Springs : + 2.2 m
Existing seaward slope 1:3 and 1:5
Foreshore ground level + 0.7 m

The embankment of this polder was heavily eroded prior to the April 1991 cyclone. During the cyclone the embankment was completely washed away over long stretches. The very low foreshore results in relatively high waves even during moderate high tide.

For slope 1:3 and 1:5 the limit velocities for both poor and proper clay are exceeded for all water levels and wave erosion will take place in all zones on the slope.

For slopes 1:7 the limit velocity for poor clay is exceeded for all water levels, whereas proper clay will only be eroded above level + 2.7 m (zone III and IV).

4.3 Loss of Stability of Revetments

By definition a revetment is a slope protection designed to protect and stabilize a slope that may be subject to action by water currents and waves.

4.3.1 General Requirements

The following aspects will have to be considered in revetment design:

- stability (toplayer, sublayer, subsoil, foundation)
- flexibility (i.e. following the settlement without influencing the stability)
- durability
- possibility of inspection of damage
- easy placement and repair
- low cost (construction and maintenance)
- additional functional requirements (i.e. special measures for reduction of wave run-up, berms for maintenance roads etc.)

For dimensioning the following general criteria can be set out:

- Sliding criteria. The revetment should not slide under frequently occurring loading situations

- Equilibrium criteria. The revetment including sublayers and subsoils must be in equilibrium as a whole.

- Uplift criteria. In extreme loading situations, such as storm surges, the component of the weight of the revetment normal to the slope should be greater than the uplift force caused by the water.

- Surface-resistance criteria. The surface particles of a revetment should have enough resistance against wave and current attack.

The composition of top layer and sublayer is essential for the performance of the revetment. The following principles should be considered:

- Stability of top layers strongly depend on the sort/composition of sublayers and the system must therefore be regarded as a whole.

- Instability (erosion) of sublayers and/or subsoil can lead to failure of top layer. The stability of top layers and sublayers must therefore be designed steadily (with same safety against failure).

- A good tuning of the permability of the top layer and sublayers (including geotextiles) is essential.

4.3.2 Placed Concrete Block Revetments

This type of revetment as frequently used in the coastal area is described in section 3.2.

Damages to the revetments are typically:

- Settlements (shrinkage) in embankment core and failure in sublayers being washed out has resulted in dislocation of blocks. Sublayer which is hereafter directly exposed to wave action is eroded and blocks will no longer be stable against even moderate wave action. Since no reserve stability is built-in, the damage progresses very fast if not repaired. This type of (local) damage is very frequent also during normal monsoon conditions.

- Displacement of single blocks due to uplift during storm surges. This type of damage has caused ultimate failure of most of the revetments during the April 1991 cyclone.

Indicative stability criteria for wave attack has been given i [3].

The formula reads:

D [illegible] $(\xi_z)^b$ / (Δ ψ_u ϕ $\cos\alpha$), where

[illegible] breaker similarity index on the slope
= $(H_s/L_o)^{-0.5} \tan\alpha = 1.25\ T_z\ (H_s)^{-0.5} \tan\alpha$

D = thickness of protection unit (m)
H_s = significant wave height (m)
b = exponent related to the interaction process between waves and revetment. b = 1 for smooth, placed block revetments.
[illegible]ive density of unit ( )
[illegible]m determined (empirical) stability upgrading factor ( ). For [illegible]es see Table 4.1
ϕ = stability factor or stability function for incipient of motion, defined at ξ_z = 1 (= 2.25).
α = slope angle ( )

Type	Sublayer	ψ_u ( )
Loose, placed blocks	Granular	1.50
Loose, placed blocks	Geotextile on clay	2.00
Riprap (reference)	Granular	1.0

Table 4.1: Stability upgrading factors, ψ_u

Applying the above criteria to the revetments as constructed shows that the block sizes generally used on slopes 1:4 are far too small.

Figure 4.5 and 4.6 show the required block thickness as function of design water level (and corresponding fully developed waves) for loose, placed blocks on granular filter. In the figures are indicated the statistical return periods for the water levels in the monsoon period and during severe cyclonic storms.

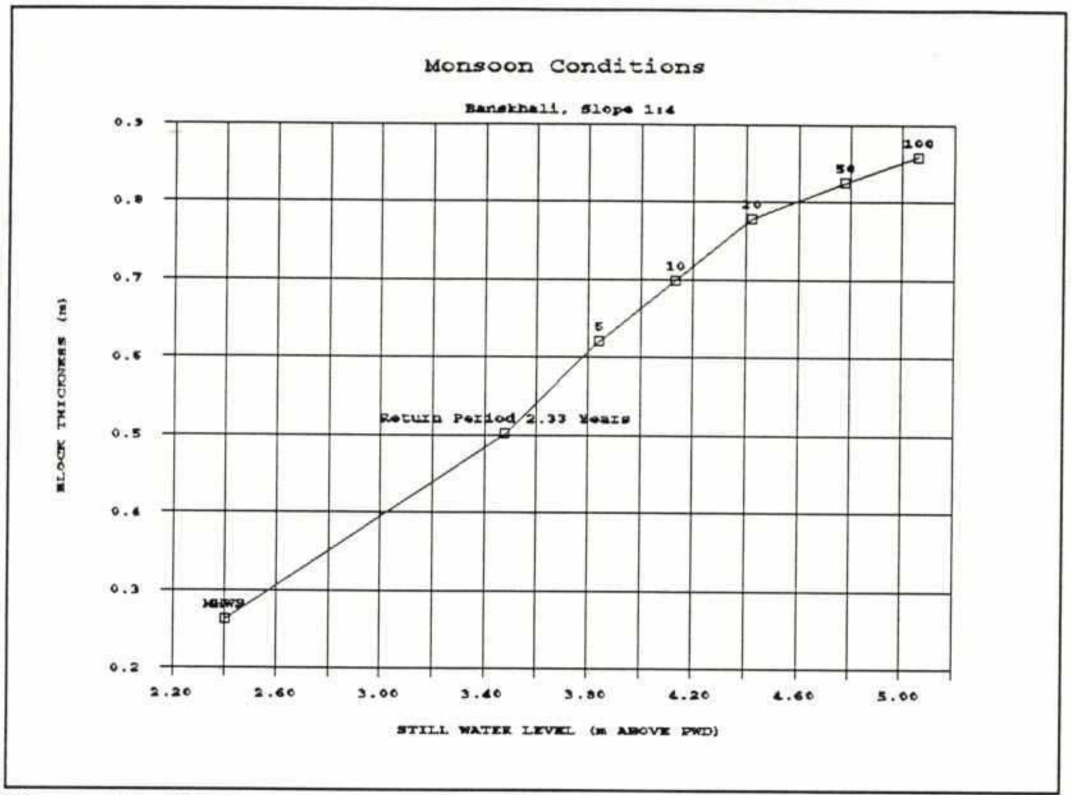

Figure 4.5: Necessary Block Thickness for Revetment, Monsoon

Figure 4.5 shows that the necessary block thickness for monsoon design conditions with return period 20 years would be approximately 0.8 m, which shall be compared to the present 0.39 m block thickness.

It can further be seen that the presently used blocks are stable for fully developed waves at water levels lower than approximately 3.0 m. Since the area of maximum impact is situated around still water level, it can be concluded, that blocks situated lower than level approximately + 3 m will be stable for all water levels. This is supported by the damage pictures from the April 1991 cyclone, where the lower parts of the revetments sustained the wave action.

Figure 4.6 shows that blocks with thickness approximately 0.9 m would have been stable during the April 1991 cyclone (estimated return period 40 years).

It should be noted that a block revetment on a sublayer of proper clay provides better stability than one on a permeable sublayer. Proper compaction and properly graded (smooth) clay surface is essential for this solution. The required block thickness for monsoon conditions with return period 20 years is approximately 0.6 m on slope 1:4.

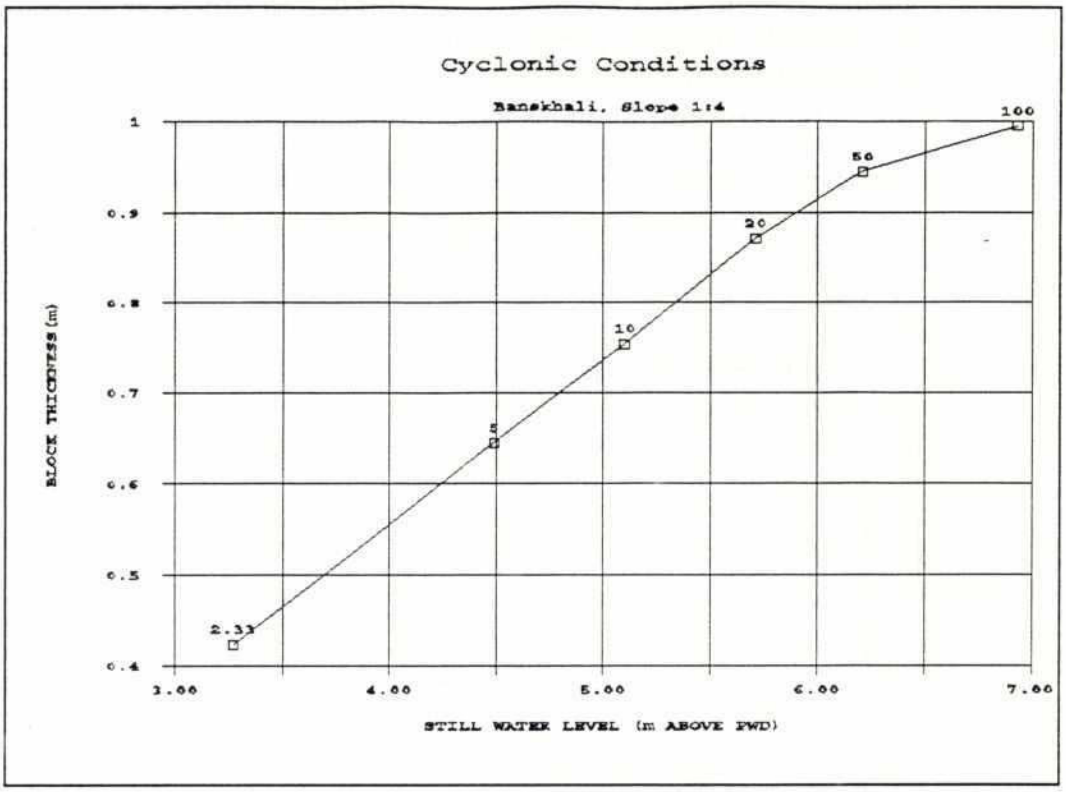

Figure 4.6: Necessary Block Thickness for Revetment, Cyclone

4.3.3 Brick Slab Revetment

Damage to this type of revetment is mainly due settlements in the sublayer causing cracking of the slab and following disintegration during wave action.

The compartmentalization of the embankment limits the damage to the compartment where it is initiated.

Lack of maintenance of minor damages has resulted in a rapidly progressing deterioration of these revetments where the reserve stability is very minor as soon as the brick slab has been opened.

Revetments that has been properly maintained and which are situated on non-eroding coasts have sustained reasonably well (polder 66/3, Cox's Bazar).

4.4 Erosion of Crest and Inner Slope

Erosion of the embankment crest and the inner slope is mainly caused by

- overflow (continuous flow across the embankment during flood level higher than embankment crest) or
- overtopping by waves (non-continuous flow across the embankment because of wave run-up higher than embankment crest) or
- erosion from run-off during heavy rain fall.

4.4.1 Erosion from Overflow

The majority of the CEP embankments will not be subject to overflow during monsoon but only during severe cyclones - provided the design crest level is maintained.

Table 4.2 shows that the crest level is higher than the monsoon water level with return period 100 years for all embankments under the Emergency Programme, except that of polder 70 Matherbari, where the crest level corresponds to water level with return period approximately 20 years only.

It can further be seen that overflow during cyclonic storms can be expected each 40 years on average for most of the embankments, whereas it can be expected as often as each 5-20 years in the southern polders (Moheskhali and Cox's Bazar) and only each 70 years for polder 62.

POLDER	Crest Level Design CEP (m PWD)	Monsoon Water Level Return Period 100 Years (m PWD)	Approx. Return Period for Cyclonic Surge Level = Crest Level (Years)
61/1 SITAKUNDA	6.4/6.7	5.55	30-40
62, PATENGA	6.7	5.14	70
63/1A ANOWARA	6.1	5.1	40
64/1A BANSKHALI	6.1	5.06	40
64/1C CHANUA	6.1	5.02	40
64/2B CHOKORIA	6.1	5.01	40
66/3 COX's BAZAR	4.9	4.81	10
69 MOHESKHALI	4.9/5.5	4.83	10-20
70 MATHERBARI	4.3	4.88	5
71 KUTUBDIA	Uncertain	4.99	
72 SANDWIP, S 72 SANDWIP, W	6.7	6.09 7.03	30 20
73/1B HATIA	6.1	5.1	30

Table 4.2: Crest Levels and Return Periods for Overflow

Since most of the embankments had breaches (or were breached) and in some sections had lower crests than designed, overflow took place in all of the listed polders during the April 1991 cyclone (estimated to have return period approximately 40 years).

Even small overflow of the embankment will result in heavy erosion of the inner slope if not protected by hard materials.

Maximum velocities in the flow can be roughly estimated by assuming critical waterdepth on the crest and natural water depth on the lower parts on the slope - see Figure 4.7.

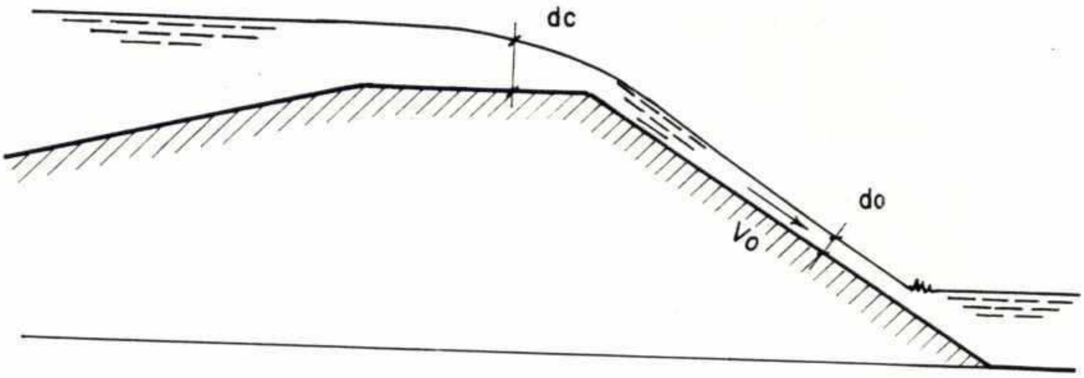

Figure 4.7 : Overflow of Embankment

In this case the discharge per metre width of embankment can be roughly estimated as [5]

$q = 0.38\ h\ (2\ g\ h)^{0.5}$, where

q = discharge per metre width ($m^3/s/m$)
g = acceleration of gravity (m/s^2)
h = height of water above crest level (m)

Figure 4.8 shows the discharge across the embankment as function of height of water above the crest for small values of h.
Overflow will result in erosion of the crest and growing values of h will appear locally, leading to progressive erosion and final breaching of the embankment. Figure 4.9 shows the discharge for bigger values of h.

It should be noted that the discharges as shown correspond to the free flowing water only and hence do not include waves which will increase the discharge considerably.
The maximum velocity (not considering waves) on the inner slope can be roughly estimated by the following formula [5]:

$V_{max} = V_o = q^{0.4}\ M^{0.6}\ (\sin\alpha)^{0.3}$, where

M = Manning Number ( )
α = Inner slope angle ( )

Figure 4.10 shows the maximum velocities as calculated for slopes 1:2 and 1:3 for Manning number 30.

It is seen that the limit velocity 2 m/s for start of erosion in a grass mat on poor clay will be exceeded as soon as the water level is more than 0.1 m higher than the crest and that the water level shall be only 0.2 m above the

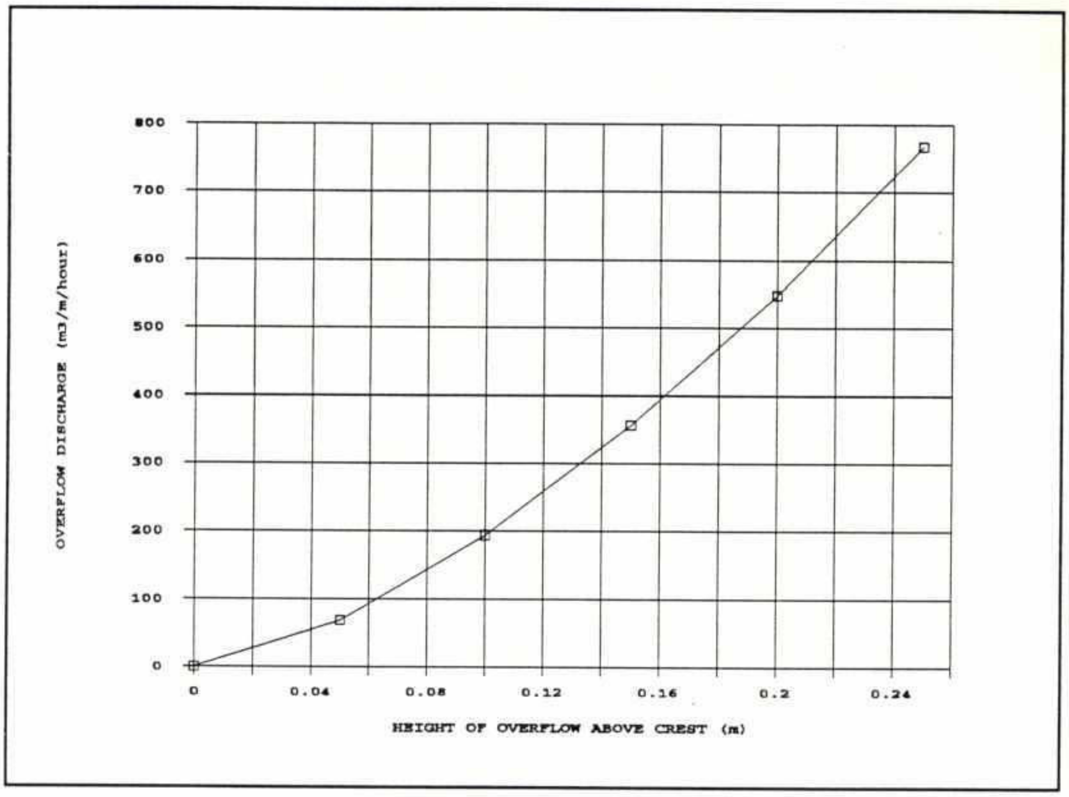

Figure 4.8: Discharge During Overflow

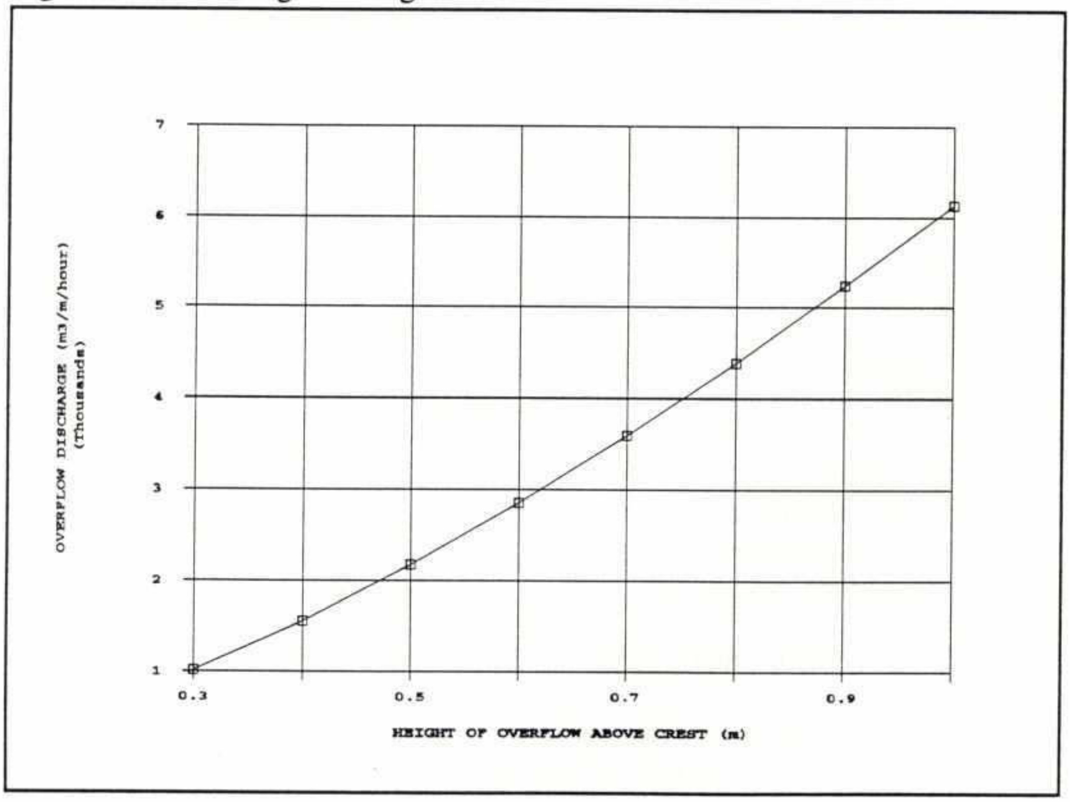

Figure 4.9: Discharge During Overflow

crest when erosion of grass on proper clay will start eroding (limit velocity 3 m/s).

The overflowing discharge corresponding to start of erosion in grass on proper clay can be found from Figure 4.8 to be 550 $m^3/m/hour$.

Since the evaluated velocities are obtained even without taking waves and winds into consideration, it can be concluded that overflow of embankments should not be allowed in the design situation and that heavy erosion of the inner slope will occur whenever it happens.

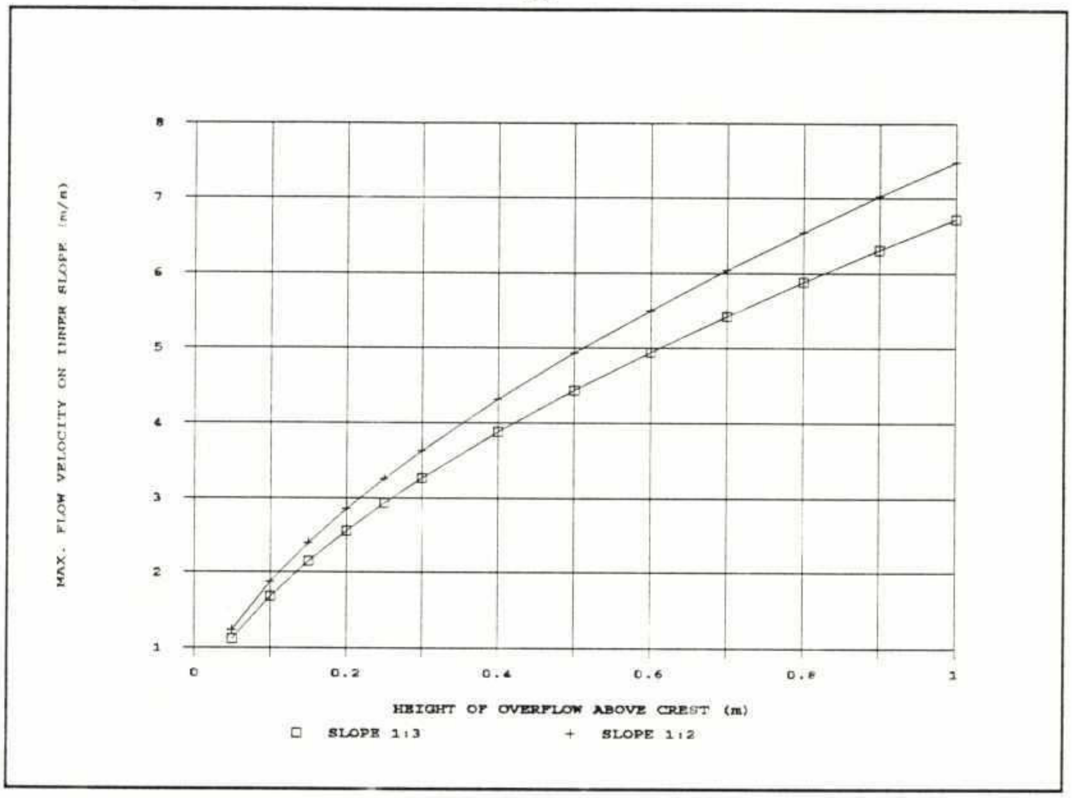

Figure 4.10: Maximum Velocities in Overflow

Erosion of the inner slope during overflow could be avoided through protection by hard materials.

According to formula presented by Krauss [6] placed concrete blocks as protection of inner slopes should have thickness approximately 0.2 m to be stable for overflows of height 0.5 m and approximately 0.1 m for overflows of height 0.25 m (on slope 1:2).

4.4.2 Erosion from Overtopping by Waves

Wave overtopping occurs when the run-up of the waves exceeds the embankment freeboard at still water level - see definitions in Figure 4.11.

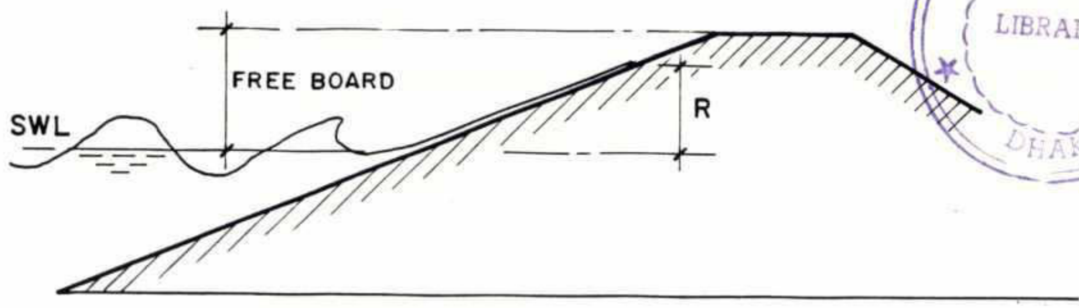

Figure 4.11 : Definition of Wave Run-up, R.

The run-up can be defined as [3] :

R	=	$R_n\ \gamma_R\ \gamma_B\ \gamma_\beta$, where

R	=	The effective run-up (m)
R_n	=	Run-up on smooth plane slopes. n = index of exceedance percentage (m).
γ_R	=	Reduction factor due to slope roughness and permeability (= 0.95 for grass mats)
γ_B	=	Reduction factor due to berm
γ_β	=	Reduction factor due to oblique wave attack (= $\sin(\beta - 10°)$), where β = angle of wave attack

For random waves R_n can be expressed by

R_n	=	$2.5\ C_n\ \xi_p$, where

ξ_p	=	$1.25\ T_p\ (H_s)^{0.5}\ \tan\alpha$ ( ) (breaker index)
C_n	=	Constant depending on the type of wave spectrum and exceedance percentage ($C_{2\%}$ = 0.6 for a narrow wave spectrum)
H_s	=	Significant wave height (m)
T_p	=	Peak period (s)
α	=	Slope angle

The Rayleigh distribution of waves provides

R_n	=	$0.77\ R_{2\%}\ (2-\log_{10}n)^{0.5}$ resulting in
$R_{s(13\%)}$	=	$R_{2\%}/1.4$

Figure 4.12 shows the wave run-up level (exceeded by 13% of the waves) on a grass covered slope for varying water level and varying slope. Fully developed wave conditions at the given water level have been used for polder 64/1A as an example.

The figure shows that for all the slopes applied overtopping of the embankment will take place in the design situation (water level 4.6 m).

For slope 1:7 overtopping by more than 13% of the waves will take place when the water level is higher than 4.1 m. This will occur in the monsoon

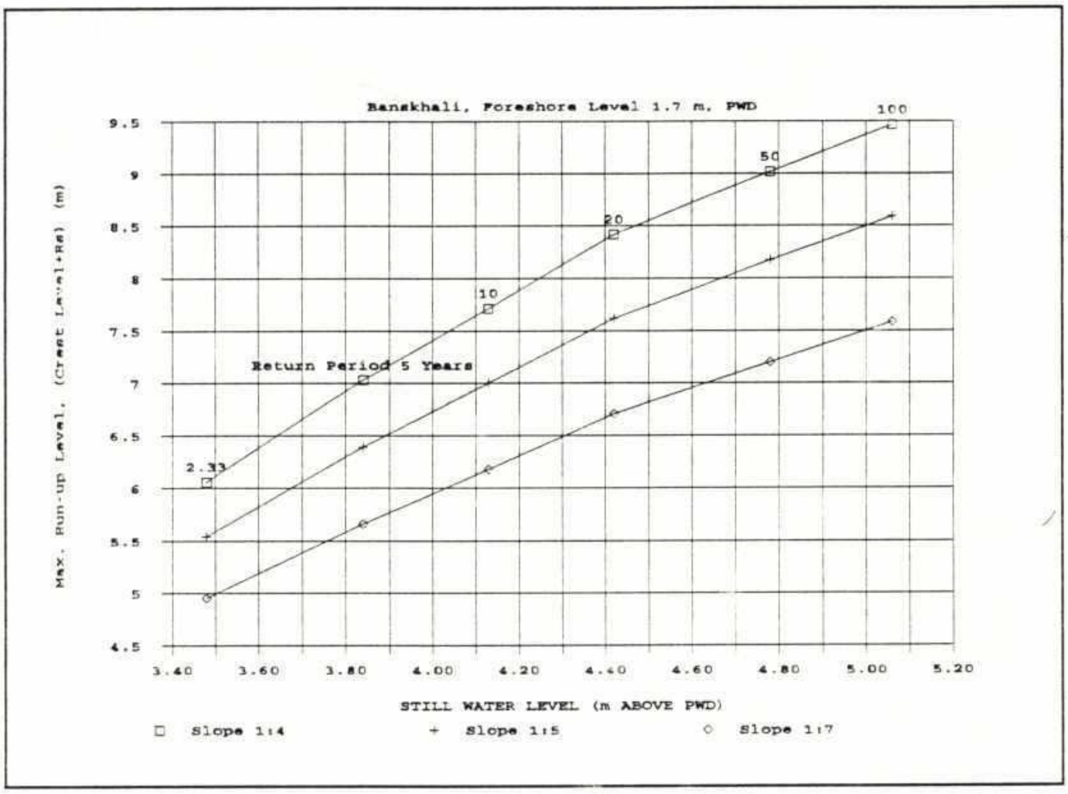

Figure 4.12: Run-up Levels (Still Water Level $+R_s$)

period on average each 10 years, approximately.

For slope 1:4 the same will happen if the water level is higher than 3.5 m only. This will occur in the monsoon period on average each 2.33 years, approximately.

Assuming Rayleigh distributed waves the formulas above show that an embankment with slope 1:7 will by overtopped by approximately 40% of the waves in the design situation and that 75% of the waves will overtop with the 1:4 slope.

There are no generally valid recommendations for acceptable levels of overtopping for embankments and the optimum should be found through a cost-benefit analysis taking into account the consequences of ultimate failures and the maintenance required for limited failures (see section 9).

In the Netherlands 2% exceedance is applied when establishing crest levels.

In the standard Dutch practice a safe (no erosion) overtopping value of 0.002 m^3/s (7 m^3/hour) for grassed crest and inner slope is recommended. Recent experience provides indication that this value can be increased to 0.01 m^3/s (36 m^3/hour) for proper quality grass mat on clay sublayer [3].

It should be noted that this overtopping discharge is a time-averaged value. The critical overtopping discharge initiating erosion is related to a single, characteristic wave.

A rough estimate of the time-averaged overtopping amount can be obtained by the following formula [3]:

$$\overline{q} = 0.1\, y\, \overline{H}\, L_o\, (\overline{T} \cot\alpha)^{-0.5}, \text{ where}$$
$$y = e^{-2.5x} \text{ and}$$
$$x = R_c \cot\alpha\, (\overline{H}\, L_o)^{-0.5}$$

Here

$\overline{q}$	=	time-averaged discharge over crest (m^3/s)
$\overline{H}$	=	mean wave height (m)
L_o	=	deep water wave length (m)
$\overline{T}$	=	mean wave period (s)
α	=	slope angle
R_c	=	height of crest above still water level (m)

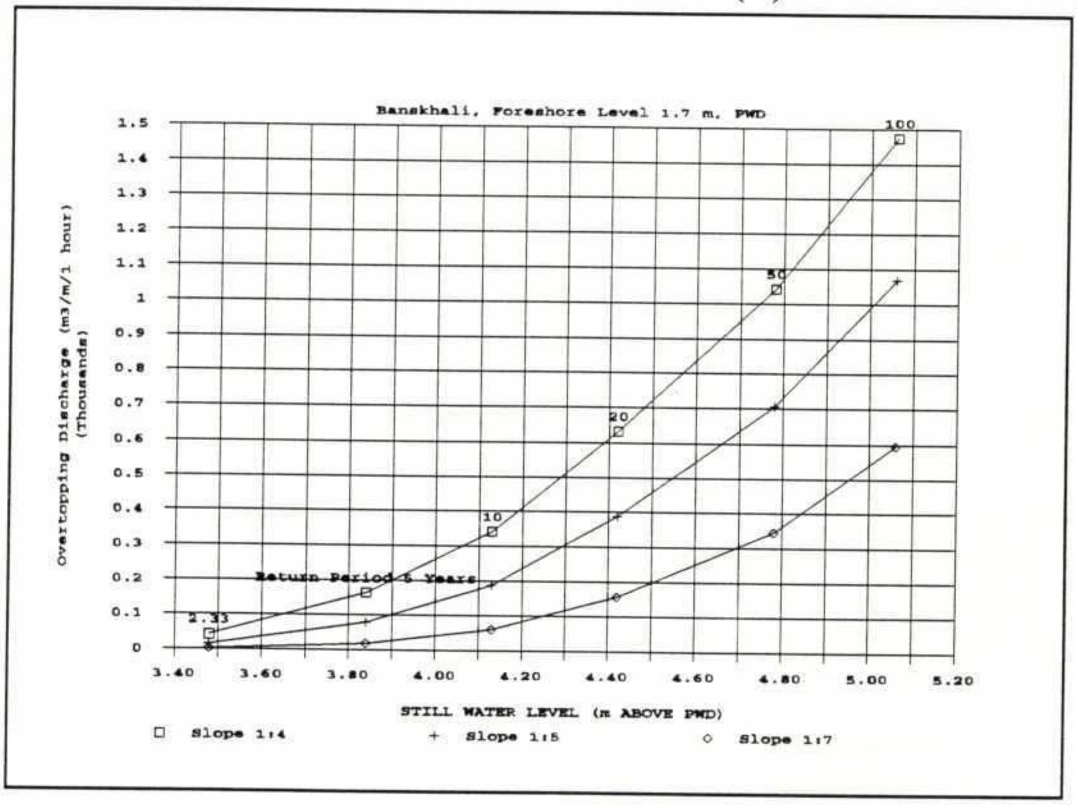

Figure 4.13: Time-averaged Wave Overtopping Discharge

Figure 4.13 shows the time-averaged discharge to be used in a rough evaluation of inundation of the polder under monsoon conditions.

For polder 64/1A having an sea facing embankment length 22 km and polder area 47 km^2 the average rising velocity of the water depth in the polder due to overtopping of waves is estimated as shown in Table 4.3 for still water level 4.4 m (once each 20 years on average).

It is concluded that inundation of the polder due to overtopping by waves is moderate even for slopes 1:4 (provided that the overtopping does not erode the crest leading to progressive overtopping).

Slope	Overtopping (m^3/m/hour)	Water Level Rising Velocity (m/hour)
1:4	625	0.3 m
1:5	400	0.2 m
1:7	150	0.1 m

Table 4.3: Rate of Water Level Rise, Monsoon.

For a given discharge the erosive effects of the overtopping can roughly be estimated by applying considerations similar to those outlined above for overflow. In this case only the higher waves are considered. As a first approximation the characteristic wave height H_s is applied and the erosion for a certain sea state is evaluated for overtopping by waves higher than H_s.

Figure 4.14 below shows the overtopping discharge, q_s corresponding to overtopping by more than 13% of the waves under monsoon conditions.

From Figures 4.8 and 4.10 above it can be seen that the limit velocity in the run-down on the inner slope with grass mat on proper clay (3 m/s) will be exceeded for overtopping discharge approximately 550 m^3/m/hour.

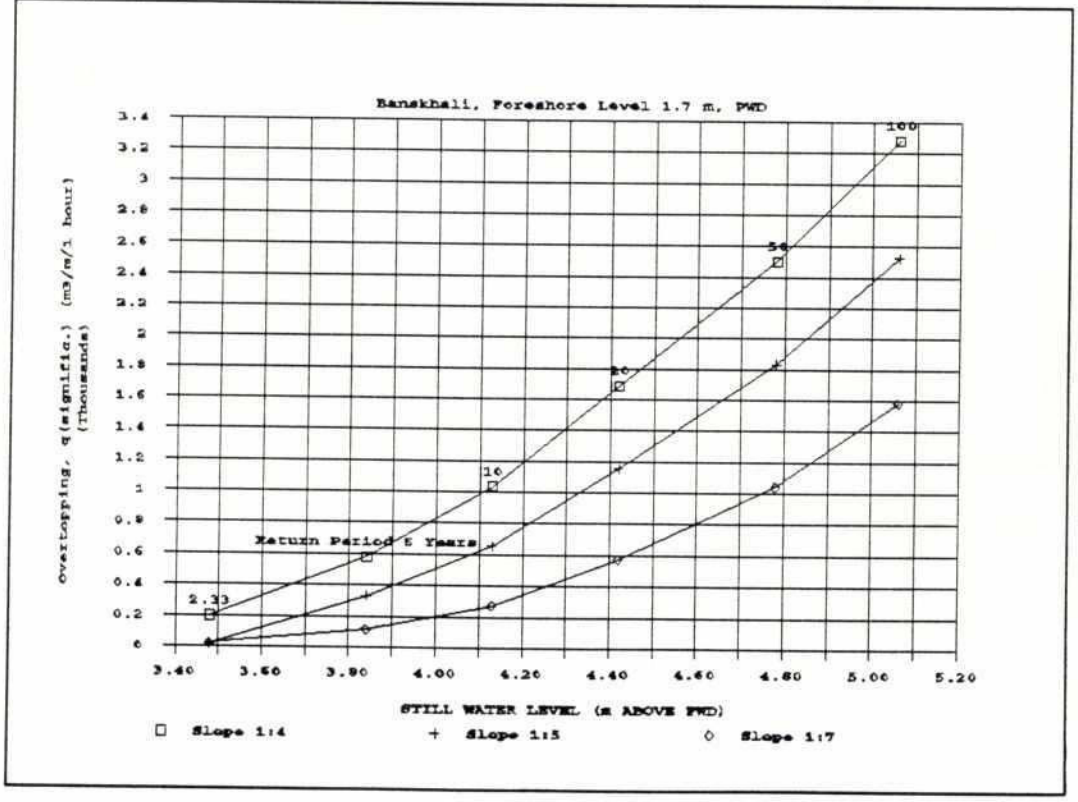

Figure 4.14: Overtopping Discharge, q_s in Monsoon Conditions.

Figure 4.14 shows that the discharge corresponding to exceedance of the limit velocity (550 m^3/m/hour) will be exceeded in the monsoon period each 5 years on average for slope 1:4 and each 15 years on average for slope 1:7. Hence erosion of the inner slope can be expected with the same recurrence interval.

Figure 4.15 shows the time-averaged overtopping discharge to be used for a rough evaluation of inundation **under cyclonic storm conditions.**

For polder 64/1A having an sea facing embankment length 22 km and polder area 47 km^2 the average rising velocity of the water depth in the polder due to overtopping of waves is estimated as shown in Table 4.4 for still water level 5.7 m (once each 20 years on average).

Slope	Overtopping (m^3/m/hour)	Water Level Rising Velocity (m/hour)
1:4	2600	1.2 m
1:5	2100	1.0 m
1:7	1500	0.7 m

Table 4.4: Rate of Water Level Rise, Cyclonic Conditions.

It is concluded that inundation of the polder due to overtopping by waves is severe even for slopes 1:7 and erosion will lower the crest leading to progressive overtopping. The rates as shown might therefore be exceeded extensively.

Figure 4.16 shows that the discharge corresponding to exceedance of the limit velocity (550 m^3/m/hour) will be exceeded during cyclonic storms each 3 years on average for slope 1:4 and each 6 years on average for slope 1:7. Hence erosion of the inner slope can be expected with the same recurrence interval.

Extensive erosion of inner slopes caused by wave overtopping has been experienced on many embankments after the April 1991 cyclone (still water levels 6 - 7 m.

4.4.3 Erosion from Rainfall

Steep country side slopes of embankments constructed of sandy materials or poorly compacted clay materials are very exposed to erosion by heavy rain fall.

During run-off the rain is forming small streams that very quickly develop into scouring currents.

This type of damages has been observed at the 1:2 country side slope of many embankments. Embankments with trees on the crest and slopes seems to be

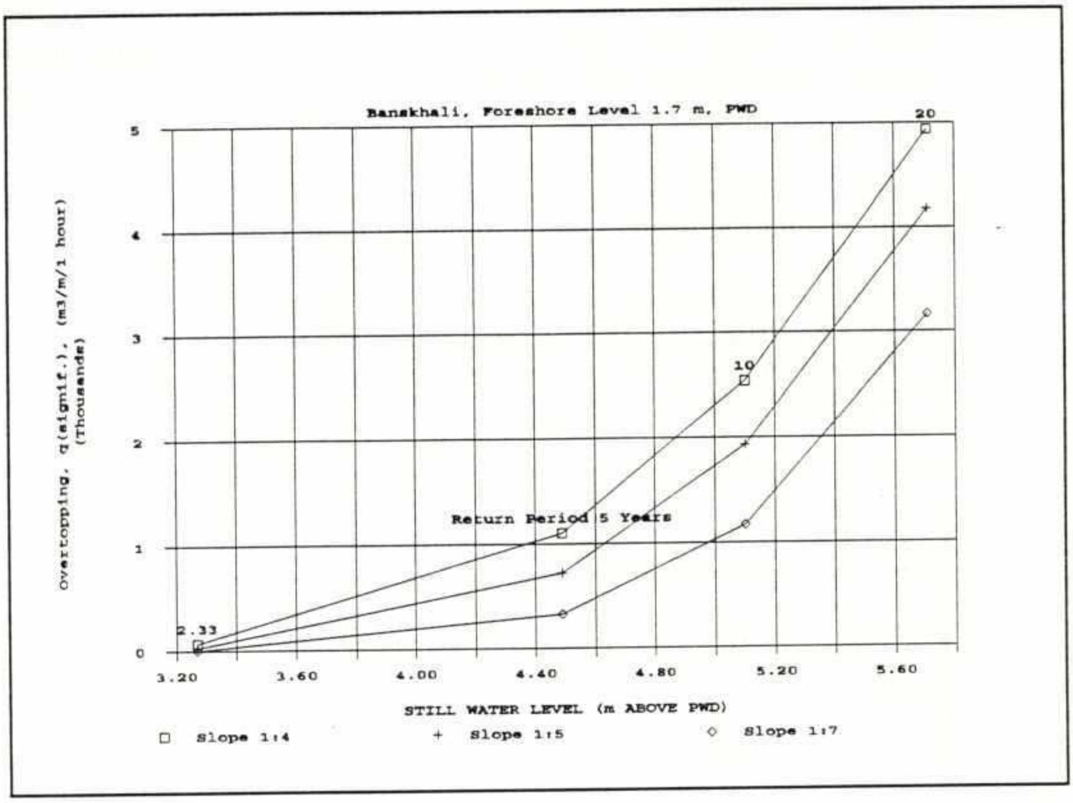

Figure 4.16: Overtopping q_s, Cyclonic Storm Conditions

less vulnerable to erosion from rainfall.

The damages require immediate repair which is normally not carried out with progressive damage as a consequence.

4.5 Geotechnical Failures

The main geotechnical limit states that are relevant in the design of embankments are:

- macro-instability of slopes due to failure along circular or straight sliding surfaces.

  Due to the relatively flat slopes there is generally no failures of the embankment slopes due to macro-instability.

  Janbu [10] has developed Slope Stability Charts for soft clay in undrained condition. The slope stability is dependent on the undrained shear strength, s_u, the total unit weight of the soil, γ, the height of the embankment, H, the slope angle, β and the thickness of the subsoil, D.

  The following critical embankment heights for sliding along toe circles can be found as function of s_u and slope (for $\gamma = 18$ kN/m^3):

	Undrained Shear Strength, s_u kN/m^2			
Slope	**10**	**15**	**20**	**25**
1:2	**3.8**	**5.7**	**7.6**	**9.4**
1:3	**4.1**	**6.1**	**8.1**	**10.1**
1:7	**5.1**	**7.7**	**10.2**	**12.8**

Table 4.5: Critical Embankment Heights (m)

Embankment heights are typically 3-4 m and macro-instability will not present a problem even for 1:2 country side slopes where s_u = 10 kN/m^2.

- settlements in embankment and in sub-soil due to the self weight of the embankment.

Due to poor compaction the shrinkage of the embankments can be quite substantial. Shrinkage may be 20% during the first monsoon period after construction of the embankment. If no overheight is given during construction or if no re-filling is carried out the embankment crest will be too low already after one year.

The shrinkage of a well compacted embankment is negligible.

The settlement in the sub-soil during construction is compensated for by extra filling of earth to reach the required crest level.

The long term consolidation should be covered for by constructing the embankment with an over height. For soil conditions typical in the embankment areas the following long term consolidation has been estimated:

Embankment Height (m)	**3.0**	**4.0**	**5.0**	**6.0**
Consolidation (m)	**0.05**	**0.1**	**0.15**	**0.20**

Table 4.6: Consolidation in Sub-soil

For many CEP embankments lack of re-filling that is required due to sub-soil settlements has caused too low crest levels and thus more frequent overtopping by waves.

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5. OBJECTIVES OF THE COASTAL EMBANKMENTS

The overall objectives of a protection system to reduce the impact of cyclones can be categorized as follows:

- protection of human lives
- protection of standing crops
- protection of cropping potential
- protection of industrial production facilities
- protection of other infrastructure - buildings, roads, communication lines etc.

Experiences from different cyclones show that, depending on the severity of the cyclone and the time of its occurence, some of these objectives are fulfilled by the present protection set-up , whereas others are not reached at all.

The role of an embankment as a part of the cyclone protection system should be studied and clearly defined for each type of polder to be protected against the impact of cyclones.

The general objectives for the sea facing embankments included in CPP II have been set as:

- to **protect** the polder against inundation by saline water due to high tide and wave overtopping during monsoon conditions and

- to provide protection against loss of life and damages caused by cyclonic storm surges by **minimizing** flooding and water flow velocities in the polder during severe cyclonic storm conditions

For Polder 62, Patenga, which is characterized by the presence of the Export Processing Zone (EPZ) and other industrial areas, the following additional objective has been set:

- to **protect** the EPZ area, adjoining EPZ development area and other major industrial areas against inundation by saline water due to storm surge and wave overtopping during severe cyclonic storms .

For protection of human lives a coastal embankment that is breached during cyclonic conditions is worse than no coastal embankment. The embankment gives the residents behind it a (some times false) feeling of safety which in some cases makes them hesitate to take shelter in higher grounds. This will evidently result in great losses if the embankment breaches - or if it is even incomplete.

This effect may partly be the reason for the many human casualties in the Patenga area and in the Banskhali polder during the April 1991 cyclone.

If a very high coastal embankment is a part of the cyclone protection system for saving lives of the polder residents, then the **entire** polder must be embanked (without any missing link) and the embankments must be designed and maintained to a standard that implies a very low probability of breaching during severe cyclonic conditions. Moderate to severe overtopping of the embankment can be accepted as long as this will not lead to breaching of the embankment or to very high flood levels in the polder.

Protection of standing crop will require embankment of the entire polder and no breaching during severe cyclone conditions. Moderate overtopping can be accepted, but good drainage of the polder is required in cases where overtopping leads to extensive flooding.

Protection of cropping potential is relevant especially for cyclones occuring in the pre-monsoon period. In case of breaching immediate actions must be taken drain the polder and close the breaches to prevent inundation by saline water during the following monsoon period.

In the aftermath of the April 1991 cyclone actions were taken in order to locate and close breaches in embankments to secure the possibility of growing the aman crop.

Protection of Industrial Production Facilities by coastal embankments is relevant for a few polders only. In some cases individual embankments, not exposed to heavy wave attack, or construction on high ground is necessary. In cases where coastal embankments are to be preferred these should be designed and maintained to prevent breaching during severe cyclone conditions. The acceptable degree of overtopping is to be established on basis of the existing floor level for the actual industries.

Other Infrastructure, such as buildings and roads should not be considered as an objective alone for protection by coastal embankments. More valuable types of infrastructure should be designed to withstand the forces from flowing water and floor levels should be chosen high enough to avoid unacceptable inundation. Many damages to buildings are furthermore caused by winds and not by water.

6. DESIGN CRITERIA

6.1 Materials

On basis of the Consultant's study of existing embankments and from experience elsewhere in both developing - and developed countries it has been concluded, that armoured embankments for protection of agricultural land and the population living in these areas are generally not economically feasible or justifiable.

The cost of embankments armoured with concrete block - or brick block revetments of the type used hitherto in several polders in the study area (30-40,000 taka per meter length embankment) is 5-7 times the cost of well compacted earth embankments with grass turfing, which has proven to perform better than most of the block revetments and which are far cheaper and easier to repair and maintain.

The Consultant is therefore recommending the main protective embankments generally to be constructed as compacted earth embankments with a protective layer of well compacted clay soil with grass turfing and minimum 100-200 m foreshore or foreland with afforestation whereever possible.

6.2 Level of Protection

The level of protection is defined through the selection of hydraulic design conditions for which the embankment is designed.

These conditions have been chosen on basis of an optimization of the embankment design taking into account both technical and economical criteria - see section 10 of the present report. Only the technical criteria are dealt with in the present section.

The design criteria applied for all polders in the emergency construction programme, except Polder 62, are:

- The return period of **monsoon** design condition has been set to 5 years. 'No' overtopping should occur in this situation (only 13 % of the waves should overtop).

- The return periods of **cyclonic storm** design conditions has been set to:

  - 20 years, where flooding due to wave overtopping of the sea facing embankment should not result in average water depth in the polder exceeding 1.0 m

  - 40 years, where the crest level should not be lower than the still water level

For Polder 62, Patenga the design criteria has been set to:

- 'No' overtopping shall take place in **cyclonic storm** with return period 40 years (only 13 % of the waves should overtop)

The Consultant has analysed an emergency construction programme with embankments generally designed for 'no overtopping' during monsoon conditions with water levels and waves that on average will occur once every 20 years in the monsoon period (return period 20 years).

Application of this design criteria resulted in embankments with crest levels 1 - 2 meter higher than existing design crest levels.

This implies estimated construction and maintenance cost for embankments that would result in internal rates of return that would be unacceptably low and in some cases negative.

6.3 Crest Levels and Seaward Slopes

Reduction of construction costs can be obtained by reducing the cross sectional area of the embankment either by lowering the crest levels and/or by applying steeper slopes.

Lower crests will for identical seaward slope lead to more frequent overtopping and hence to increased erosion damages to the crest and the inner slope. Too low crests will result in overflowing, which leads to considerable erosion of the inner slope.

Steeper seaward slopes will result in higher velocities in the wave run-up and the slope will be subject to wave erosion even during moderate monsoon conditions. For identical crest level the frequency and the amount of overtopping of the embankment will be increased.

For a given overtopping criteria (and thereby given protection), it can be demonstrated that a flat slope (and corresponding low crest) is more cost effective than a steep slope (and a corresponding high crest).

Figure 6.1 shows that the net present value (NPV) of the investment in earth and land acquisition plus the maintenance of the embankment throughout the lifetime will decrease when the seaward slope is flattened (for estimation of maintenance requirements - see section 9).

Steeper inner slopes will be subject to increased scour in overtopping situations and the geotechnical stability is not acceptable for steep country side slopes.

The Consultant has therefore recommended that slopes be 1:7 on the seaward side and 1:3 on the country side and that crest levels be established according to an optimization as presented in section 10 of the present report.

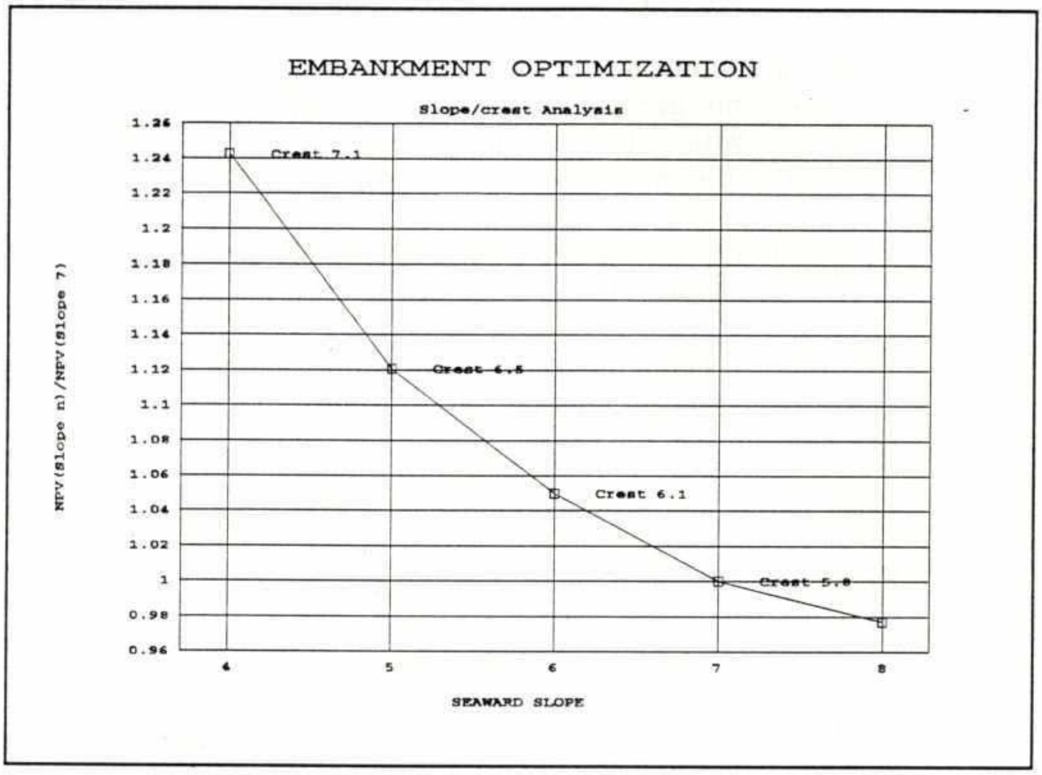

FIGURE 6.1: NPV for varying slope and crest level

6.4 Crest Width

The crest width of 4.3 m introduced in the original CEP design for sea dikes and interior dikes is recommended for the future coastal embankments.

This width allows for some scouring of the embankment shoulders before the embankment crest is reduced. Furthermore it will permit for traffic by inspection vehicles.

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7. NATURAL CONDITIONS

The following natural conditions are of importance for the embankment design:

- water levels
- ground levels
- waves
- coastal morphology
- geological and geotechnical conditions

The hydraulic conditions (water levels and waves) are described in a separate volume of the feasibility report - Appendix A, Hydraulic Studies. In the following are summarized the design conditions as used for embankment design.

The topographic, geological and geotechnical conditions as established through field surveys are reported in Appendix B, Report on Field Surveys. Only characteristics used for design of embankments are summarized below.

7.1 Water Levels

7.1.1 Reference Datum

All levels in the present report refer to Public Works Datum (PWD). The relation to other datums appear from Figure 7.1 below.

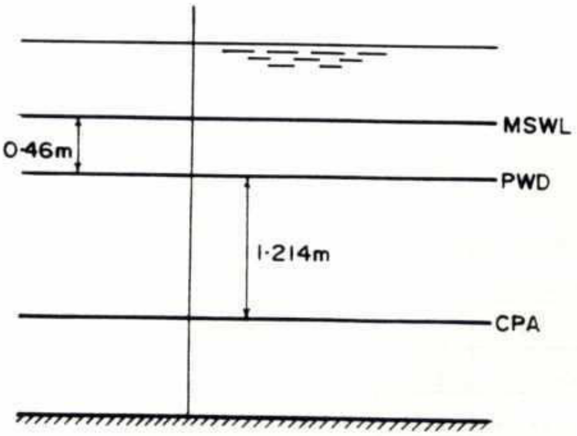

Figure 7.1 : Reference Datums. CPA = Chittagong Port Authority
MSWL= Mean Sea Water Level

7.1.2 Monsoon Water Levels

Water levels recorded at 24 stations in the period 1959 to 1988 have been analyzed in order to establish statistics for maximum levels occurring during the monsoon season - excluding cyclonic surges. The recorded levels include components of astronomical tide, wind set-up and wave set-up.

A Gumbel analysis has been performed for the annual maximum values recorded in the monsoon period (June to September). Extreme values practically occur only in July and August. Hence records do not include surges due to cyclonic storms.

The results are presented in Appendix A, Hydraulic Studies.

Level gauges are often located in river mouths and during high river discharges the water levels recorded here can be considerably higher than those relevant for the coast outside the embankments. For polders covered by the emergency programme the gauges at Sandwip, Chittagong and Cox's Bazar have been utilized and levels for other polders have been established through interpolation between these three gauges.

The gauge at Chittagong is located in the Karnaphuli river, approximately 3 km from the river mouth, whereas the gauge at Patenga is located in the river mouth and therefore reflects coastal conditions better. However, the gauge at Patenga has a very short record (10 years only and missing the period 1982-1988 where high levels have been recorded at other gauges). The Chittagong gauge has therefore been preferred.

A correlation analysis have shown that the maximum levels at Patenga generally can be set 0.2 m lower than those at Chittagong. Hence the water levels applied for Polder 62, Patenga are those recorded at Chittagong minus 0.2 m.

Table 7.1 below shows the water levels applied for design of embankments under the emergency programme:

POLDER	RETURN PERIOD (YEARS)							
	2.33	5	10	20	40	50	100	200
61/1 SITAKUNDA	3.98	4.33	4.62	4.91	5.27	5.37	5.55	5.80
62 PATENGA	3.57	3.92	4.21	4.50	4.76	4.86	5.14	5.39
3/1A ANOWARA	3.53	3.88	4.17	4.46	4.72	4.82	5.10	5.35
64/1A BANSKHALI	3.49	3.84	4.13	4.42	4.68	4.78	5.06	5.31
64/1C CHANUA	3.45	3.80	4.09	4.38	4.63	4.73	5.02	5.27
64/2B CHOKORIA	3.44	3.79	4.08	4.37	4.62	4.72	5.01	5.26
66/1 COX's BAZAR	3.24	3.59	3.88	4.17	4.43	4.53	4.81	5.13
66/3 COX's BAZAR	3.26	3.61	3.90	4.19	4.45	4.55	4.83	5.15
69 MOHESKHALI	3.26	3.61	3.90	4.19	4.45	4.55	4.83	5.15
70 MATHERBARI	3.31	3.66	3.95	4.24	4.5	4.60	4.88	5.20
71 KUTUBDIA	3.42	3.77	4.06	4.35	4.61	4.71	4.99	5.24
72 SANDWIP, S 72 SANDWIP, W	4.34 5.11	4.74 5.55	5.07 5.91	5.38 6.25	5.68 6.59	5.78 6.69	6.09 7.03	6.40 7.40
73/1B HATIA	3.57	3.92	4.21	4.50	4.76	4.86	5.14	5.39

Table 7.1: Maximum water levels in monsoon period (m above PWD)

For polders included in the Mid Term Programme the water level statistics for the relevant gauges has been applied as follows:

POLDER	Gauge No.	RETURN PERIOD (YEARS) 2.33	5	10	20	50	100
5 KALIGANJ	1	3.29	3.47	3.61	3.75	3.93	4.07
7/1+7/2 ASASUNI	2	2.47	2.74	2.95	3.15	3.42	3.62
10-12 PAIKGACHA	3	2.43	2.72	2.96	3.19	3.49	3.71
14/1+14/2 KOYRA	2	2.47	2.74	2.95	3.15	3.42	3.62
15 SHAMNAGAR	2	2.47	2.74	2.95	3.15	3.42	3.62
35/1 SHARANKHOLA	5	3.24	3.38	3.50	3.61	3.76	3.87
40/2 PATHERGHATA	6	2.72	3.01	3.25	3.48	3.77	4.00
48 KUAKATA	8	2.20	2.50	2.75	2.98	3.29	3.52
56+57 BHOLA, N BHOLA,South	12 *)	3.99 3.39	4.22 3.62	4.41 3.81	4.59 3.99	4.83 4.23	5.00 4.40
59/3B SUDHARAM	**)	5.41	5.85	6.21	6.55	6.99	7.33
59/3C COMPANI-GANJ	**)	5.41	5.85	6.21	6.55	6.99	7.33
60 SONAGAZI	**)	5.41	5.85	6.21	6.55	6.99	7.33
66/1 RAMU	***)	3.59	3.88	4.17	4.53	4.81	5.13
68 TEKNAF	24	1.84	2.00	2.13	2.26	2.42	2.54
70 MATHERBARI	***)	3.31	3.66	3.95	4.24	4.6	4.88

Table 7.2: Maximum water levels in monsoon period (m above PWD).
*) : gauge 12 minus 0.6 m,
**): gauge 16 plus 0.3 m
***) interpolation between Chittagong and Cox's Bazar

The isolines for annual maximum high water as presented in [1] has been used as basis for extrapolations as indicated.

7.1.3 Cyclonic Storm Surge Levels

A cyclonic surge is the rise in sea level during a cyclone above that produced by astronomical tides. The surge is due to wind stresses, pressure deficits and bathymetric effects.

The maximum surge level is (here) defined as the maximum still water level during a cyclonic storm.

Confusion and uncertainty exists about the values of storm surges reported. Different values for the same cyclone are given in various sources and it is often unclear whether the value quoted represents the rise above astronomical tide or it is an elevation above mean sea level.

Hay & Co.[7] has established a joint tide-surge probability function the

Chittagong area by combining observed surge heights with tide height frequences for May, June October and November which are the most probable months for cyclones to appear. Curves were derived for three assumed distributions of surge frequency and the following design curve was arrived at:

Return Period (Years)	2.5	3.5	5	10	20	40	100	200
Water Level (m PWD)	3.6	4.3	4.9	5.5	6.1	6.7	7.3	7.6

Table 7.3: Cyclonic Surge Frequencies after [7]

The Consultant has investigated surge levels as observed during the April 1991 cyclone. The results are reported in Annex II of the Main Report. For the Chittagong area surge levels between 6.2 and 7.0 m have been reported. According the above frequency distribution this corresponds to a return period around 40 years.

On basis of this distribution the surge level statistics for polders covered by the Emergency Programme has been prepared by reducing the levels proposed by Hay & Co. by 0.3 m to obtain the values applicable for polder 62, Patenga. Extrapolation from Patenga to other polders has been done by assuming water level differences identical to those applied for monsoon conditions.

POLDER	RETURN PERIOD (YEARS)						
	5	10	20	40	50	100	200
61/1 SITAKUNDA	4.98	5.59	6.20	6.70	6.90	7.42	7.72
62 PATENGA	4.57	5.18	5.79	6.29	6.49	7.01	7.31
63/1A ANOWARA	4.53	5.14	5.75	6.25	6.45	6.97	7.27
64/1A BANSKHALI	4.49	5.10	5.71	6.21	6.41	6.93	7.23
64/1C CHANUA	4.45	5.06	5.67	6.17	6.37	6.89	7.19
64/2B CHOKORIA	4.44	5.05	5.66	6.16	6.36	6.88	7.18
66/3 COX's BAZAR	4.26	4.87	5.48	5.98	6.18	6.70	7.00
69 MOHESKHALI	4.26	4.87	5.48	5.98	6.18	6.70	7.00
70 MOHESKHALI	4.31	4.92	5.53	6.03	6.23	6.75	7.05
71 KUTUBDIA	4.42	5.03	5.64	6.14	6.34	6.86	7.16
72 SANDWIP, S	5.33	5.94	6.55	7.05	7.25	7.77	8.07
72 SANDWIP, W	5.58	6.19	6.80	7.30	7.50	8.02	8.32
73/1B HATIA	4.58	5.19	5.80	6.30	6.50	7.02	7.32

Table 7.4: Cyclonic surge levels (still water levels) in m above PWD

No statistics are available for other districts. The highest reported surge level for Khulna is 4.6 m above PWD.

7.2 Ground Levels and Water Depths

Data on ground levels and water depths are required for evaluation of wave conditions and for calculation of embankment volumes.

In calculation of wave heights the foreshore level at the seaward toe of the embankment has been determined as the average of the ground levels surveyed at distances 50, 75 and 100 m from the centre line of the existing embankment.

The ground levels used for estimation of volumes in retired embankments have been calculated as the average of surveyed levels in distances 50, 75 and 100 m inland the centre line of the existing embankment.

The nearshore beach slope has been evaluated on basis of bathymetric charts supplemented by the survey results.

All surveys were carried out prior to the April 1991 cyclone but generally only small changes have occurred to the levels mentioned above.

The following levels have been used for polders under the emergency programme:

Polder	Chainage (km)	Average levels (m)	
		Backshore	Foreshore
61/1 Sitakunda	1 - 2.6	2.52	1.81
	0.5E - 2.16E	3.47	2.98
	2.16E - 4.575E*	3.23	2.71
63/1A Anowara	28.5 - 33.9	2.07	1.00
	33.9 - 36.8*	2.95	1.91
	36.8 - 38.1	2.86	1.69
	38.1 - 41.75*	2.73	1.63
	41.75 - 43	2.90	2.85
64/1A Banskhali	83.2-85.8*	2.70	0.72
	85.8 - 92*	2.04	1.47
	92 - 100.45	2.47	1.76
	100.45-108.35*	2.46	1.41
	108.35 - 113	3.20	1.66
64/2B Chokoria	116.8 - 128.4	2.34	2.45
64/1C Chanua	0 - 10	2.34	2.45
66/3 Cox's Bazar	44.3 - 46.3*	2.37	2.05
	46.3 - 49.3	3.33	2.12

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Polder	Chainage (km)	Average levels (m)	
		Backshore	Foreshore
69 Moheskhali	0 - 12.9	1.69	1.60
71 Kutubdia	9 - 12	3.19	2.15
	12 - 20*	3.48	2.98
	20 - 25*	3.82	2.54
	25 - 28.5	4.25	3.45
72, Sandwip	8.5 - 14.5*	3.52	3.45
	14.5 - 16	3.51	3.23
	16 - 18.8*	3.62	3.37
	18.8 - 20	3.13	3.09
	20 - 21.0*	3.05	2.99
	21.0 - 45	3.58	3.4

* Retired section.

Table 7.5: Ground Levels (m PWD)

7.3 Waves

7.3.1 Assessment of Extreme Wave Conditions

Waves are generally described in Appendix A to the Main Report-Hydraulic Studies.

No reliable wave measurements have been available for the present study. Measuring programmes carried out as part of other projects cover limited periods only.

Estimation of wave heights and periods have therefore been based on theoretical considerations.

Due to the shallow coastal areas wave breaking will allways occur under the extreme conditions of interest for the embankment design.

Two sources for generation of the nearshore waves have been considered:

- offshore waves generated in the Bay of Bengal
- waves generated by travelling depressions (cyclones of varying intensity)

The offshore waves are in both cases transformed to nearshore waves by shoaling,refraction and wave breaking. For the water depths in question refraction and shoaling have only small effect on the wave heights compared to wave breaking.

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The main difference between nearshore waves generated by the two mechanisms is the wave period.

The waves generated in the Bay of Bengal are always generated in deep water and the significant wave periods will here typically be 11 to 13 sec.. When these waves travel a long distance over shallow water the wave breaking will cause a lowering of the significant wave period.

The waves generated by travelling depressions will often be generated in shallow waters which will limit the significant wave period. Significant wave periods will typically be 7 to 9 sec..

An approach described by Goda [8] has been used for calculation of nearshore wave heights.

In these calculations a foreshore bottom slope 1:200 has generally been applied and the following offshore wave heights and nearshore significant wave periods have been used (see Appendix A). The duration of the sea-state with the shown return period is 12 hours.

Return Period (Years)	2.5	5	10	20	50	100
Offshore Significant Wave Height (m)	6.9	7.6	8.2	8.8	9.6	10.2
Offshore Significant Wave Period (s)	11.1	11.7	12.2	12.5	13.1	13.6

Table 7.6: Offshore Significant Wave Heights and Wave Periods

The wave heights in front of the embankments will be of the order of magnitude as shown i Table 7.7 - depending of the water depth defined as height of still water level above average ground level.

Water Depth (m)	1.0	2.0	3.0	4.0	5.0	6.0
Nearshore Significant Wave Height (m)	0.8	1.50	2.10	2.70	3.3	3.6
Nearshore Significant Wave Period (s)	7	8	8.5	9	9	9

Table 7.7: Approximate Nearshore Significant Wave Heights

The frequency distribution of wave heights are based on a joint probability analysis of extreme water levels and waves.

The nearshore wave heights are depth-limited and the frequency distribution for the water levels will therefore be governing for the frequency distribution of nearshore waves.

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As furthermore extreme high water occurs as a combination of high tide and strong winds (generating wave set-up or surge) a strong correlation between extreme high water and waves is found. In order to ensure that the sea is fully developed in this situation a duration of 12 hours has been used in the above frequency distribution for offshore waves.

The frequency distribution for waves has been established for each individual polder on basis of the water level statistics as shown in Tables 7.2 and 7.4.

7.3.2 Reduction of Wave Heights by Afforestation

When waves travel across a shallow flooded area, the initial heights and periods of waves will decay when the frictional stress of the ground and vegetation underlying the shallow water exceeds the wind stress.

An approximate method for estimating the decay of wind waves passing over areas with high values of bottom friction is presented in [9].

Figure 7.1 shows as an example the reduction of the significant wave height for waves travelling over an area grown by a dense stand of trees with little undergrowth. The water depth is 2.5 m and the significant wave height is 1.8 m on the seaward edge of the forest. The significant wave period is 8 sec..

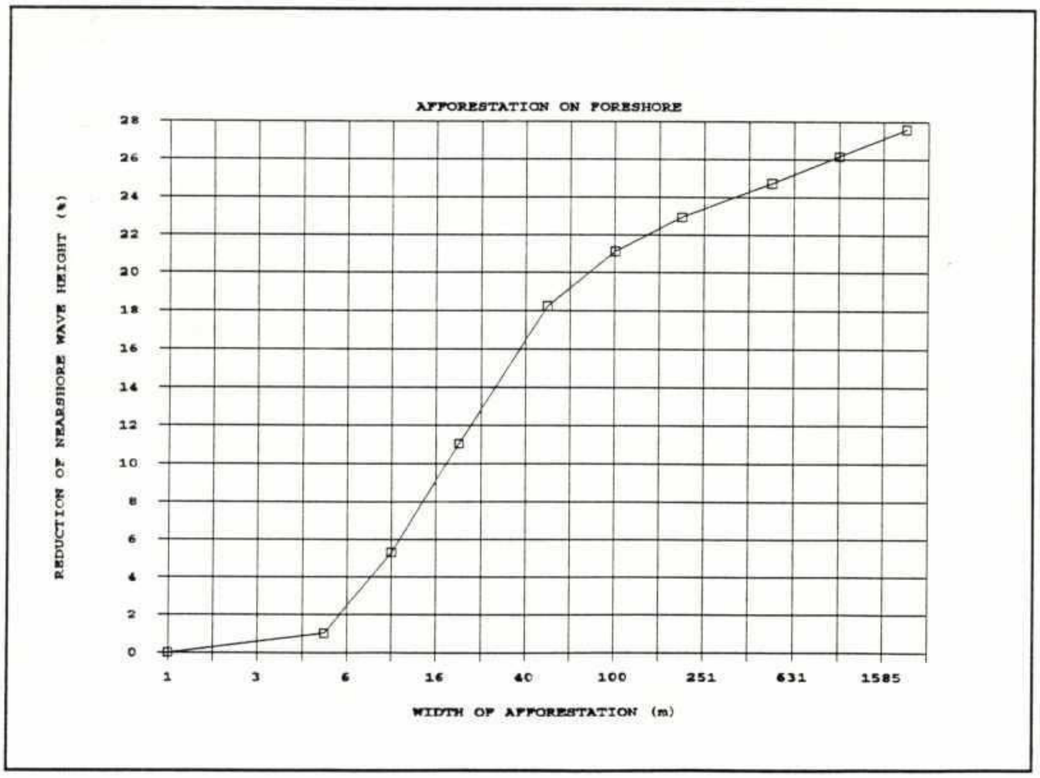

Figure 7.1: Wave height reduction due to afforestation on foreshore

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The figure shows that the decay is constant for wave travelling length up to approximately 100 m where the wave height is reduced by approximately 20%. After this the wave height is so much reduced that only a minor damping of the waves is achieved if the forest is widened.

As the method is rather approximate it is concluded that dense afforestation over a width of 100-200 m seawards of an embankment will reduce the significant wave height at the toe of the embankment by approximately 20 - 25%.

Since the establishing of afforestation and the maintenance of it cannot be assured this effect has not been accounted for in the calculation of crest heights. If afforestation be established and properly maintained, the maintenance of the embankment will be reduced and the lifetime of the embankment will be extended.

7.4 Coastal Morphology

7.4.1 The Southwestern Region

From a physical point of view the **southwestern region** of Bangladesh is now in a state of equilibrium. This is partly due to natural processes and partly due to human activities. The natural process of decay started in the 19th century when the local streams were cut of from the Ganges, their parent and perennial source. Subsequently, they only carried local drainage generated by rainfall and some base flow in some cases. As a result the region degenerated into a moribund delta. The process has accelerated in recent years due to further reduction in Ganges flow during the dry months. It is a stable region covered by dense mangrove forests which prevent bank erosion. On the other hand there is not much accretion either.

The coast from Tetulia river in the west to the Chittagong coast in the east is an area with active delta building process. According to some experts, it is preferable to sub-divide this part into two :

- the active part from Tetulia river to Feni river
- the part from Feni to the more stable and regular Chittagong coast.

The active part is known as the lower Meghna estuary and it is the outlet for discharge of combined flows of Ganges, Brahmaputra and Meghna into the Bay of Bengal. This is the most dynamic region and most of the erosion and accretion of the entire coastal area occurs in this part. Some extreme conditions such as high upland discharge with heavy sediment load, severe tidal activity, swelling of the Bay by monsoon winds etc. are prevalent here inducing rapid morphological instability.

Morphological behaviour of the estuary can be described in broad lines as it is yet to be well understood supported by adequate data, research and studies.

Morphological changes may be generated by a complex combination of several variables. These may be described as, inter alia :

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- water movement governed by tide, upland discharge and monsoon winds;
- sediment load of the upland and tidal flows;
- salinity incursion;
- wave action;
- bottom topography;

Undercutting of the river bank by strong currents and the consequent bank slumping are the main causes of estuarine bankline erosion. At spring tides during monsoon strong flow velocities, exceeding 3m/s develop and cause severe erosion. Erosion caused by wave action is relatively less severe and is mainly concentrated near the southern part of Sandwip.

The coastal changes of Sandwip is depicted in Figure 7.2 [10] in different year from 1913 to 1984. Between 1913 and 1963 about 10 km of landmass at the north tip of the island has been washed away with an average rate of about 200/m year.

Between 1963 and 1984 another 7 km gave in with a rate of 350m/year. In all cases rate of erosion is higher during the monsoon. The severe impact of erosion can be gauged from the fact that the Sandwip Island which was about 650 km^2 in 1789 has been stripped to 250 km^2 in 1989 and at present rendering about 200 to 100 families homeless every year. Out of 15 Union Parishads, 8 have been affected by erosion. Some of the homeless families settle on BWDB embankments while others join exodus to cities.

The serious erosion problems around the township of Sandwip needs further study before a proper design of the embankment in this area is carried out. This study is proposed to be part of the detailed engineering of the projects of Mid Term Programme - Phase II.

7.4.2 Eastern Region

The erosion of coasts in the eastern region is mainly a result of the longshore transport of sediments set up by waves.

During the monsoon period the wind direction is mainly SSW. The wave breaking together with the incoming tide sets up a longshore north-going current resulting in a littoral drift of same direction.

In the dry season the northly winds sets up a south-going littoral drift. However, due to higher waves and more consistent direction during the monsoon period, the net littoral transport is north-going.

A number of the original CEP embankments have been retired due to erosion of the foreland and subsequent erosion of the embankment.

Embankments in the following polders in the eastern region are situated on eroding coasts. The rate of erosion is estimated very roughly on basis of study of old maps.

POLDER	CHAINAGE (km)	APPROXIMATE RATE OF EROSION (m/year)
62 PATENGA	2.5 - 5.0	15-25
63/1A ANOWARA	35 - 40	5 - 10
64/1A BANSKHALI	87 - 91 & 100 - 108	10-15
71 KUTUBDIA	12 - 34	10 - 15

Table 7.8: Eroding Coasts, Eastern Region

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8. DESIGN

8.1 Earth Embankments

Retirement of the embankment is proposed on eroding coasts whereever possible. Afforestation of foreland and embankment is further proposed in order to reduce the wave impact on the slopes and to reduce the coastal erosion.

The earth embankments included in the Emergency Programme and the sea facing embankments included in Mid Term Programme Phase 2 have generally been designed according to the design criteria outlined in section 6 and for the natural conditions as described in section 7 above.

The river embankments included in Mid Term Programme Phase 2 are to be designed more individually under consideration of the local current conditions and wind fetch for wave generation. River side slopes will vary as a consequence hereof.

A general embankment cross section is shown in Enclosure 4 to the present report.

8.1.1 Crest Levels

The crest levels arrived at for embankments in the Emergency Programme is listed in Table 8.1 below where also is listed the crest levels required to fulfill the different criteria. The present design crest level is included as well. The embankments will be constructed with over height to cover for long term consolidation in the sub-soil. Over height is 0.1 m for resectioned embankments and 0.20 m for retired embankments.

	CREST LEVELS AND WATER LEVELS (m PWD)							
	'No overtop.' in 5 Years Monsoon		Max. 1 m water depth 20 Years Cyclone		'No over-flow' in 40 Years Cyclone		Design	
							Re-comm.	Present
POLDER	Water Level	Crest Level	Water Level	Crest Level	Water Level	Crest Level	**Crest Level**	Crest Level
61/1 SITAKUNDA	4.33	5.6	6.20	< 6.7	6.70	6.7	**6.7**	6.4/6.7
62, PATENGA							**8.5/ 7.0**	6.7
63/1A ANOWARA	3.88	5.2	5.75	6.3	6.25	6.3	**6.3**	6.1
64/1A BANSKHALI	3.84	5.5	5.71	6.3	6.21	6.2	**6.3**	6.1
64/1C CHANUA	3.80	5.1	5.67	< 6.2	6.17	6.2	**6.2**	6.1
64/2B CHOKORIA	3.79	5.1	5.66	< 6.2	6.16	6.2	**6.2**	6.1
66/3 COX's BAZAR	3.61	4.9	5.48	< 6.0	5.98	6.0	**6.0**	4.9
69 MOHESKHALI	3.61	4.9	5.48		5.98	6.0	**6.0**	4.9/5.5
71 KUTUBDIA, W	3.77	4.9	5.64	5.8	6.14	6.2	**6.3**	Uncer-tain
72 SANDWIP, S	4.74	5.9	6.55	6.7	7.05	7.1	**7.0**	6.7

Table 8.1: Crest Levels, Emergency Programme (m above PWD)

The table shows that the 'non overflowing' condition during a cyclone with 40 years return period is defining the crest level in most of the polders.

The amount of water entering into the polder due to wave overtopping during cyclonic conditions with 20 years return period has been calculated by the formulas for wave overtopping as presented in section 4.4.2 above.

In the calculation a duration of 1 hour for the maximum level during a cyclone has been applied together with a total duration of 3 hours for the overtopping of the embankment. During the hour before and after the peak a level 0.3 m lower than the maximum level has been applied.

An example of the detailed calculation is included in Enclosure 6 to the present report.

The preliminary design characteristics arrived at for embankments in the Mid Term Programme Phase 2 is shown in Table 8.2.

POLDER NO	CHAINAGE	EMBANKMENT WORKS					
		RESEC-TIONING (KM)	NEW RE-TIRED (KM)	EXISTING DESIGN (LDL)		RECOMMENDED DESIGN	
				CREST LEVEL (M)	RS/CS SLOPE	CREST LEVEL (M)	RS/CS SLOPE
35/1 SHARANKHOLA	1.6-4.1 6.3-8.1	2.5 1.8		4.9	3/2	4.9	5/2
40/2 PATHERGATA	10.0-15.0 &18.5-23	9.5		5.2	5/2	5.2	7/3
48 KUAKATA	26.0-35.0	9.0		6.1 ?	5/2	6.1	7/3
56/57 BHOLA	63.7-67.5 67.5-76.0 76.0-80.5 80.5-126	3.8 45.5	 8.5 4.5	5.8 5.8 5.5 5.5	7/2	6.1	7/3
59/2	121-124.0 124-126.5 126.5-135	3.0 8.5	2.5	7.0	7/2	7.0	7/3
59/3B SUDHARAM	19.5-42.0 60.8-69.8	22.5 9.0	0.5	7.0/7.6	7/2	7.6	7/3
59/3C COMPANIGANJ	11.0-14.0 14.0-21.0 21.0-27.0	 7.0	3.0 6.0	7.0/7.6	7/2	7.6	7/3
60 SONAGAZI	15.0-21.0 & 25.0-27	8.0		7.0/7.6	7/2	7.6	7/3
66/1 RAMU	0-5.0	5.0		4.9	7/2	6.0	7/3
68 TEKNAF	11.4-16.4 16.4-18.3 18.3-23.5 23.5-28.4	5.0 5.2	 1.9 4.9	5.2	7/2	5.2	7/3
70 MATHERBARI	0-6.5 6.5-26.0	6.5	2.5	4.3	7/2	6.3 5.7	7/3 3/3
72 SANDWIP	0-3.0 3.0-8.5 46.0-51.0 51.0-52.5 52.5-57.5 57.5-59.5 59.5-60.2	 5.5 5.0 5.0 0.7	3.0 1.5 2.0	6.7	7/2	7.0 7.0 6.7 6.7 6.7 7.0	7/3 7/3 5/2 5/2 5/2 7/3
73/1B HATIA	51.5-56.5		5.0	6.1	5/2	6.3	5/2
TOTAL		115.7	27.9				

R/S : River Side   C/S : Country Side

Table 8.2 : Crest level, Phase 2 Projects

8.1.2 Materials and Compaction

The embankments are generally designed as earth embankments with a clay cover layer. A typical cross section is shown in Enclosure 3.

The **core** material (Class I) is silt and clay materials with sand content less than 40 %. Plasticity index 10 - 35%.

Use of hydraulic fill is generally not accepted due to the relatively low crest levels that will result in frequent wave overtopping. For polder 62

Patenga use of hydraulic fill can be accepted due to the low frequency of overtopping (see section 9.2). The hydraulic fill shall in this case consist of non cohesive sand or silty sand.

The core will be compacted to minimum 80 % Standard Proctor.

The **cover** layer (Class II) is clay material with content of clay 10 - 30%, content of sand less than 40% and plasticity index 20 - 35%..

The cover layer will be compacted to minimum 90% Standard Proctor.

The cover layer will be protected by a grass cover.

8.2 Protective Works

Retirement of embankments on eroding coasts is proposed where ever possible.

Retirement will not be possible in areas with dense habitation (Anowara and Kutubdia) and has not been accepted in areas where the hinterland provides potential industrial areas (Patenga).

Protection of the embankment toe by hard, flexible materials is proposed in these areas.

The cross section proposed is shown in Enclosure 4 to the present report.

The purpose of the protective works is to stop the erosion of the lower seaward earth slope during monsoon conditions. The protection should not be exposed directly to breaking waves during the peak of extreme cyclonic surges where the still water level is high.

The upper limit for the protection is for that reason set to the still water level with return period 20 years during monsoon conditions, which approximately corresponds to cyclonic surge levels with return period 5 years.

The armour layer consists of rough angular stones or precast concrete blocks in random placement. The weight of the units has been calculated by Hudson's Formula [9].

The armour stones are designed for wave conditions with 20 years return period in the monsoon period. In the design situation 5-10% damage is to be expected. The expected damage in other situations is estimated in section 9 below.

The front slope is set to 1:3 in order to keep the stone size in a range that is available in Bangladesh.

The toe of the protection is buried in order to meet a possible erosion of the beach down to elevation 0.5 m (PWD). The amount of materials in the buried toe allows for self-stabilizing during the future erosion.

A geotextile is proposed between the sandy sublayer and the bedding stones in order to avoid wash-out of sand and subsequent settlements.

The berm at the upper limit of the protection reduces the run-up on the earth embankment during extreme conditions and it further provides a more stable intersection between stone material and earth slope.

9. MAINTENANCE OF EMBANKMENTS

9.1 General

The routines for maintenance of earth embankments has been described in Appendix G of the present feasibility report. The background for estimation of the average yearly maintenance of embankments constructed as per the design principles outlined above is given in the following.

The major maintenance activity on coastal embankments constructed with a proper clay cover layer is substitution of earth that has been removed from the embankment by waves eroding the seaward slope or by water eroding the inner slope when the embankment is overtopped.

Other maintenance required is the adding of earth on the embankment crest in order to maintain the design crest elevation and repair of scars created by human activities. These operations are of minor scale compared to the repair of erosion damages and should be carried out as regular maintenance.

Hence, the average yearly need for periodic maintenance is estimated as the *yearly average amount of earth eroded from the embankment by wave scour and wave overtopping.*

9.2 Amount of Earth on Crest and Inner Slope Eroded by Wave Overtopping

The calculation of eroded volumes can only be estimated roughly due to lack of proven calculation methods.

The frequency of loads (waves during a certain water level) is defined by the water level statistics as described in section 7.1 above.

The damage is depending of the rate of overtopping defined as the percentage of waves for which the run-up exceeds the free board at a certain water level. A method for estimating this rate of overtopping is outlined in section 4.4.2.

The response function (i.e. the relation between rate of overtopping and damage) has been established on basis of observed damages during the 1991 cyclone.

The following response function is reflecting that the damage increases more rapidly when heavy overtopping takes place and the cover layer is totally damaged.

The resulting average damage per m embankment per year (the most probable damage) is found as

$\sum f\,D$, where

summation is done for all actual loadings that result in damage and

f	=	frequency of occurence of a certain load
D	=	Damage corresponding to the actual load

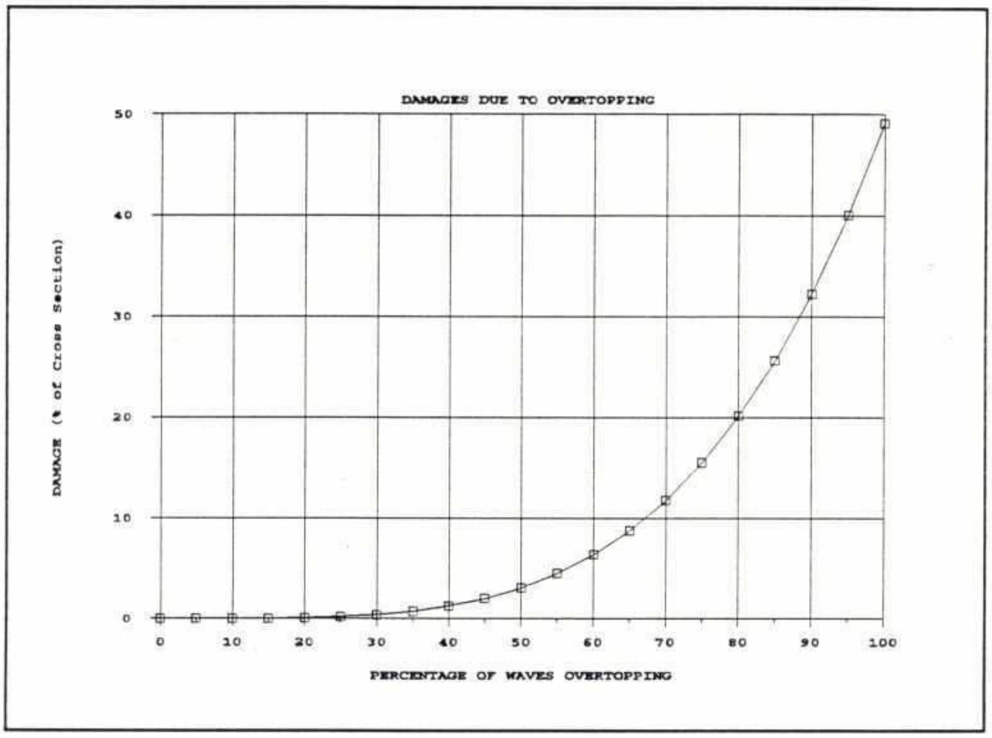

Figure 9.1: Damage due to Overtopping

The calculation is shown in detail in Enclosure 7 to the present Appendix for a selection of polders.

The calculation has been done for varying crest levels and the rate of damage has been used as input to the optimization analysis carried out in section 10 below.

The expected damage has been calculated for overtopping occurring during monsoon conditions and for overtopping occuring during cyclonic conditions.

Figure 9.2 shows the rate of damage as function of crest level for cyclonic and monsoon conditions for polder 64/1A Banskhali.

Table 9.1 shows the average yearly damage calculated for a selection of polders included in the Emergency Programme.

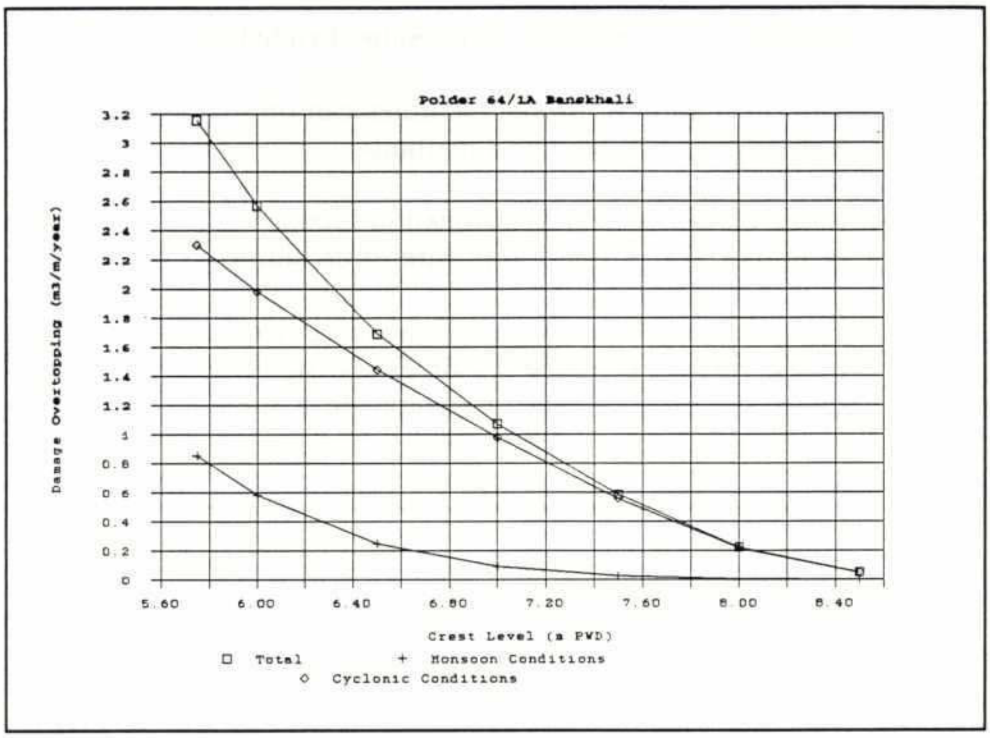

Figure 9.2: Damage for Different Crest Levels

POLDER	DAMAGE DURING CYCLONIC CONDITIONS ($m^3/m/year$)	DAMAGE DURING MONSOON CONDITIONS ($m^3/m/year$)	TOTAL DAMAGE TO CREST AND INNER SLOPE ($m^3/m/year$)
72 Sandwip, NW	3.05	1.85	4.90
72 Sandwip, S	2.42	0.18	2.60
62 Patenga, S	0.23	0.01	0.24
62 Patenga, N Crest 7.0 m Crest 8.5 m	 1.48 0.08	 0.10 0.0	 1.58 0.08
64/1A Banskhali	1.66	0.39	2.05
63/1A Anowara	1.88	0.30	2.18
71 Kutubdia	2.06	0.32	2.38

Table 9.1: Yearly Average Damage due to Overtopping

The small damages for Patenga reflects that the embankment for this polder is designed for 'no overtopping' during cyclonic conditions with return period 40 years whereas others are designed for allowance of overtopping resulting in 1 m water depth in the polder once each 20 years on average.

9.3 Amount of Earth on Seaward Slope Eroded by Waves

The calculation of eroded volumes can only be estimated roughly due to lack of proven calculation methods.

Based on the principles outlined in section 4.2 the following approach has been used for estimation of rate of erosion on front slope:

The **area** subject to erosion is the area around the actual still water level within vertical distance 0.5 times the significant wave height. Figure 9.3 shows the area subject to erosion on a slope 1:7 for varying significant wave height.

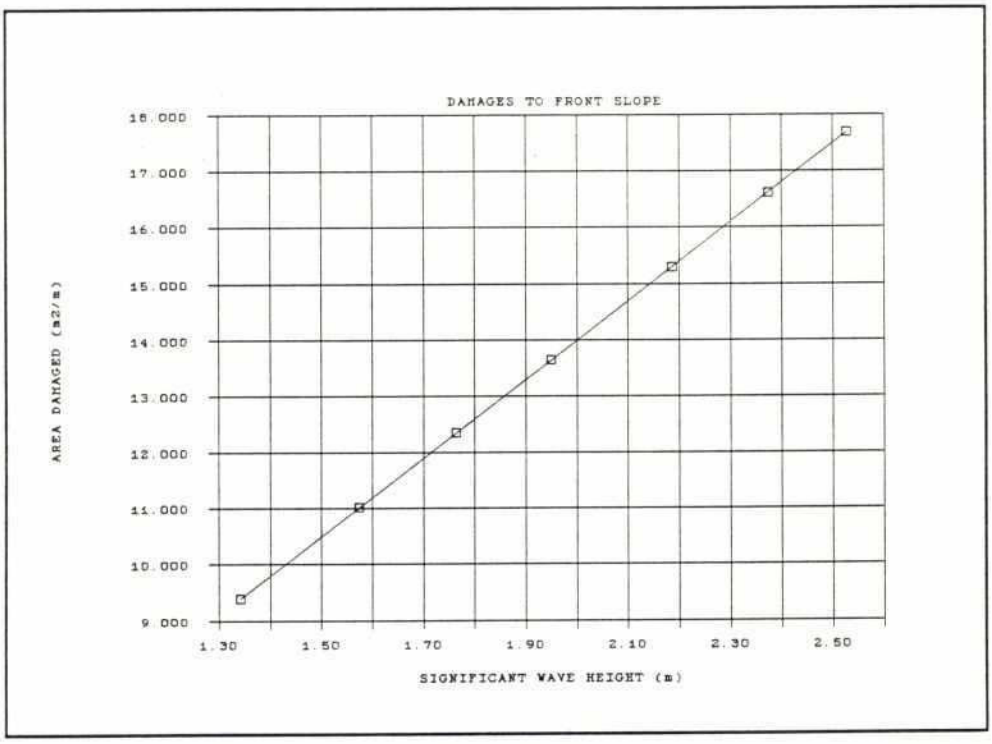

Figure 9.3: Area on Seaward Slope Subject to Erosion

The **depth** of erosion is evaluated on basis of the test results referred to in section 4.2.2. By assuming a proportionality to the square of the maximum uprush velocity of the waves the depth of erosion for 12 hours duration of storm is estimated as shown in Figure 9.4 below.

The **volume** of erosion as function of significant wave height is found by multiplying the average depth of erosion and the area exposed. Figure 9.5 shows the result. This function is the response function used in calculation of the expected average erosion per year.

The expected average volume of erosion per year has been calculated as stated in section 9.2 by taking into account the expected frequency of the loads. The total erosion on seaward front has been calculated as the sum

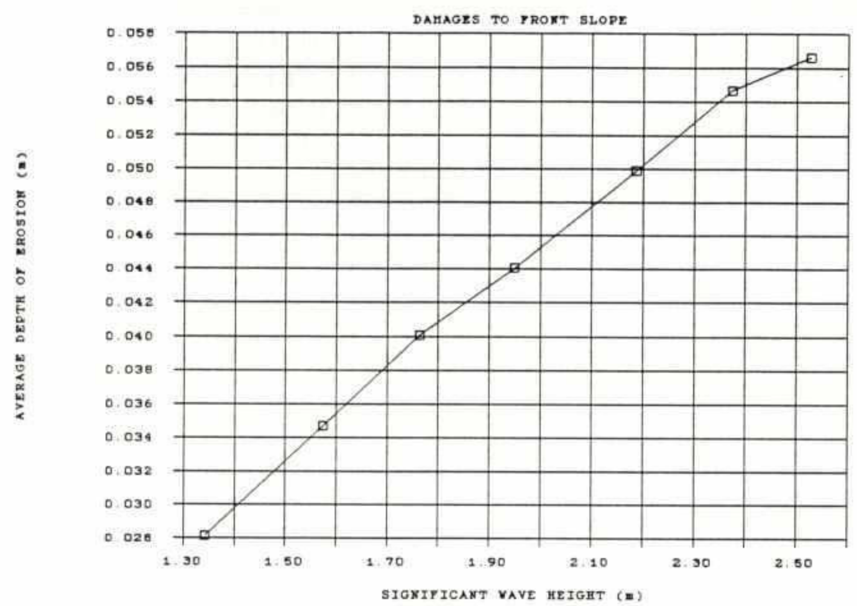

Figure 9.4: Depth of Erosion on Seaward Slope

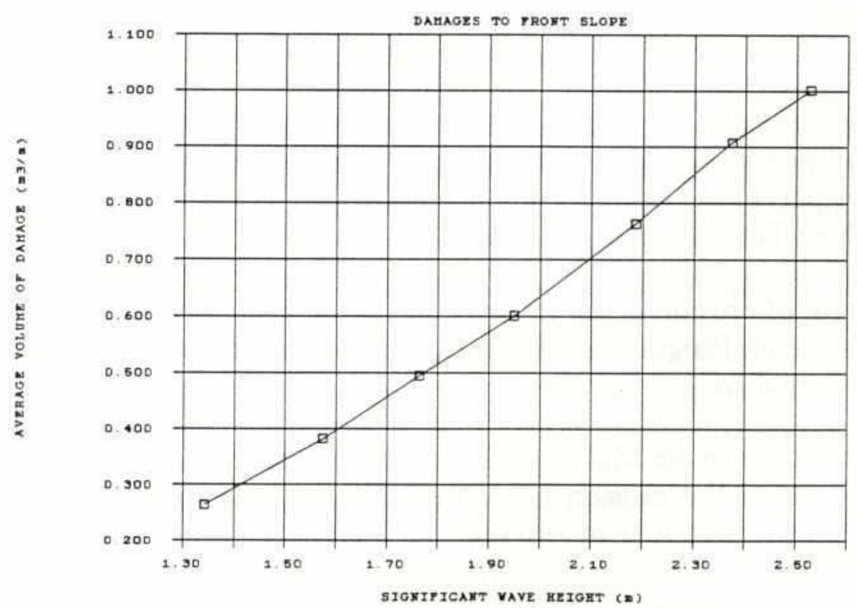

Figure 9.5: Volume of erosion on Seaward Slope

of the erosion during the monsoon period and the erosion during cyclonic storms. The frequency functions included in sections 7.11 and 7.1.2 have been applied.

Table 9.2 shows the rate of erosion calculated for a selection of polders under the Emergency Programme.

POLDER	DAMAGE TO SEWARD SLOPE ($m^3/m/year$)	DAMAGE TO CREST AND INNER SLOPE ($m^3/m/year$)	DAMAGE TO EMBANKMENT TOTAL ($m^3/m/year$)
72 Sandwip, NW	0.32	4.90	5.22
72 Sandwip, S	0.24	2.60	2.84
62 Patenga, S	0.08	0.24	0.32
62 Patenga, N			
Crest 7.0 m	0.26	1.58	1.84
Crest 8.5 m	0.26	0.08	0.34
64/1A Banskhali	0.36	2.05	2.41
63/1A Anowara	0.24	2.18	2.42
71 Kutubdia	0.26	2.38	2.64

Table 9.2: Yearly Average Damage due to Overtopping

The total rate of erosion amounts to approximately 2.5 $m^3/m/year$ on average for all polders except Patenga South and North, crest 8.5 m and Sandwip North West.

For this part of Patenga the design criteria are different and protective works reduces the erosion on seaward slope. For Sandwip NW the crest level proposed is slightly lower than required according to design criteria. This part of the embankment is proposed to be studied further as a project in Mid Term Programme Phase 2.

A rate of erosion of 2.5 $m^3/m/year$ has been applied to all polders in the Emergency Programme except Patenga, where the values of Table 9.2 have been applied.

For polders in the Mid Term Programme Phase 2 outside the most cyclone prone area the erosion is seaward front erosion only. This erosion has been estimated to 1 $m^3/m/year$ on average.

9.4 Damage to Protective Works

The response function used for calculation of the expected damage to new protective works at Patenga (3,500 m), Anowara (1,600 m) and Kutubdia (800 m) has been prepared on basis of the damage curves presented in [9].

The response function used is shown in Figure 9.6.
The very high waves are associated with high water levels during cyclonic storms. The embankment is overtopped in this situation and the down rush

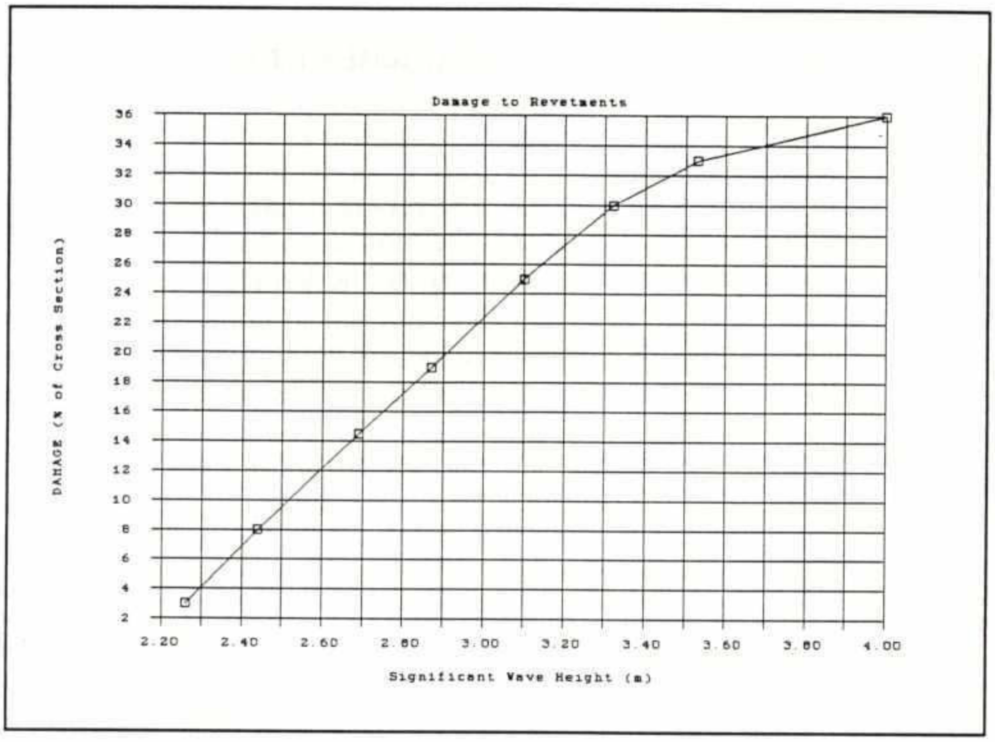

Figure 9.6: Response Function Revetments

of the waves will be weakened which results in less damage to the revetment than if no overtopping took place.

The expected average damage per year has been calculated as stated in section 9.2 by taking into account the expected frequency of the loads. The total damage has been calculated as the sum of the damage during the monsoon period and the damage during cyclonic storms. The frequency functions included in sections 7.11 and 7.1.2 have been applied.

For a cross sectional area of 9.5 m^3 the total average yearly damage has been calculated to 0.47 m^3/m (5%) out of which 0.08 m^3/m is the damage during monsoon and 0.39 m^3/m is the damage during cyclonic storms.

9.5 Embankment Erosion if Afforestation is Etablished

The significant wave height in front of the embankments can be reduced by approximately 20% if a dense mangrove afforestation be established in a 200 m wide belt on the foreshore (see section 7.3.2).

This reduction in wave impact will reduce the erosion of both the seaward slope, the crest and the inner slope.

By applying the methods outlined above the reduction in erosion is calculated to approximately 10% on average for all polders.

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10. OPTIMIZATION OF EMBANKMENT DESIGN

10.1 General

In section 6.3 of the present report it has been concluded that seaward slope shall be 1:7 and that the most economical crest level shall be chosen under due consideration to costs and benefits.

The most economical crest level is defined as the crest level resulting in the highest internal rate of return for the embankment project.

The benefits taking into account are those described in section 4 of the Main Report. The following analysis demonstrates the economic effect of reducing or increasing the crest level by 0.5 m relative to the crest levels as they have emerged from the design criteria applied.

The alternative crest levels correspond to the following return periods for the design situation where the water depth in the polder is limited to 1 m:

- 10 years for crest level lowered 0.5 m and
- 40 years for crest level increased by 0.5 m

A representative selection of polders included in the Emergency Programme have been analysed.

10.2 Estimated Costs

The total project cost for the base case (design period 20 years) has been estimated on basis of bill of quantities prepared during the detailed design.

The total costs of the projects with alternative crest levels have been estimated by adding/deducting the costs of the earth volumes involved and further adding 20% to other project costs for the 0.5 m higher embankments.

Table 10.1 shows the investment costs.

RETURN PERIOD	10 Years	20 Years	40 Years
CREST LEVEL INCREMENT	-0.5 m	0	+0.5
63/1A Anowara	150.8	172.5	200.7
64/1A Banskhali	220.0	267.4	316.3
71 Kutubdia	234.9	277.5	321.2
72 Sandwip, S	227.2	262.9	303.1

Table 10.1: Total Investment Costs (million Taka)

For embankments the maintenance cost in the base case has been estimated as shown in Appendix F to the Main Report on basis of the evaluation of erosion as presented in section 9 above. The total maintenance cost is calculated as a sum of routine maintenance, periodic maintenance and repair after cyclone damages.

For alternative crest levels the maintenance cost has been estimated on basis of a calculation of the reduced/increased damages due to reduced /increased overtopping of the embankment. The erosion of the seaward slope is almost independent of the crest level.

The total average maintenance cost per year is estimated as shown in Table 10.2.

RETURN PERIOD	10 Years	20 Years	40 Years
CREST LEVEL INCREMENT	-0.5 m	0	+0.5
Cost (Tk/m/year)	310	220	160

Table 10.2: Total Maintenance Cost, Embankment

Costs for maintenance of structures are to be added to the above figures in order to establish the total maintenance cost to be entered in the calculation of the internal rate of return.

10.3 Optimization

Details of the calculation of the internal rate of return is included in Enclosure 8 to the present report. The results are shown in Table 10.3 below.

RETURN PERIOD	10 Years	20 Years	40 Years
CREST LEVEL INCREMENT	-0.5 m	0	+0.5
63/1A Anowara	1.6	4.6	4.8
64/1A Banskhali	11.6	13.6	12.7
71 Kutubdia	3.8	6.8	6.2
72 Sandwip	21.4	22.2	18.3

Table 10.3: Internal Rate of Return for Different Design Return Periods

It is concluded that the optimum design return period , where the water depth in the polder should not exceed 1 m is 20 years.

11. REFERENCES

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[9] : Shore Protection Manual, 1984

[10] : Janbu, N.: Slope Stability Computations, 1968

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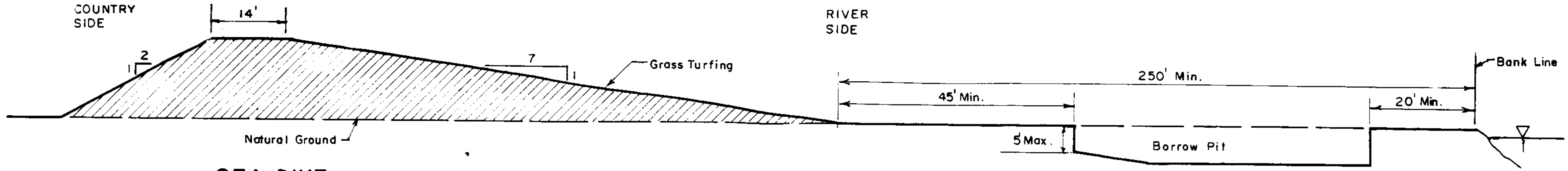
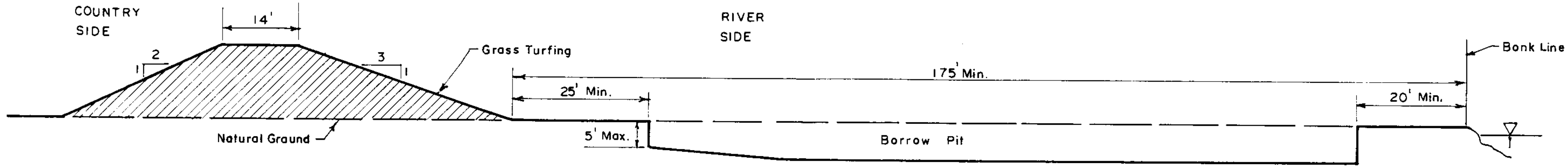
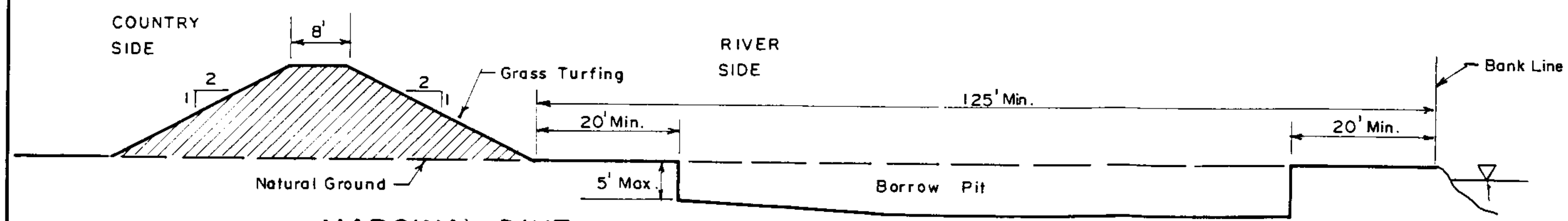


SEA DIKE



INTERIOR DIKE



MARGINAL DIKE

SECTION THROUGH DIKE & BORROW PIT

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BANGLADESH WATER DEVELOPMENT BOARD

CYCLONE PROTECTION PROJECT II

DRAFT PROJECT PREPARATION REPORT, APPENDIX C.

DESIGN CROSS SECTIONS, CEP EMBANKMENTS

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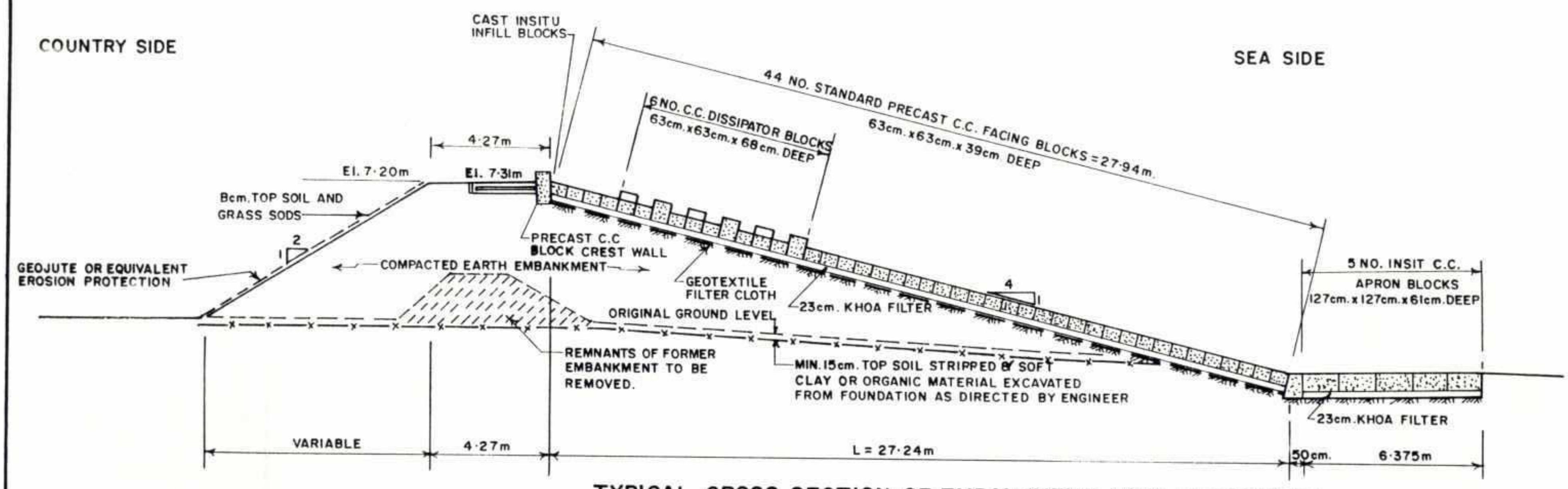
	ENCLOSURE 1



TYPICAL CROSS SECTION OF EMBANKMENT WITH REVETMENT

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BANGLADESH WATER DEVELOPMENT BOARD

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SLOPE PROTECTION, EXISTING DESIGN. PLACED CONCRETE BLOCKS

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ENCLOSURE 2

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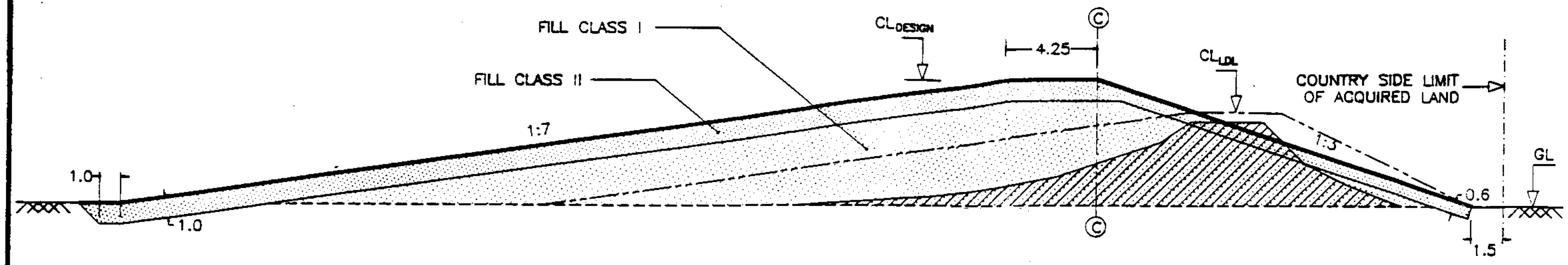
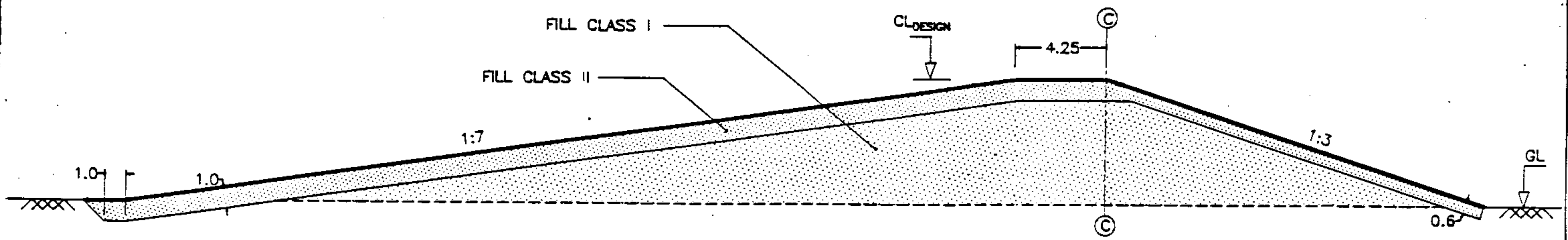
3 f

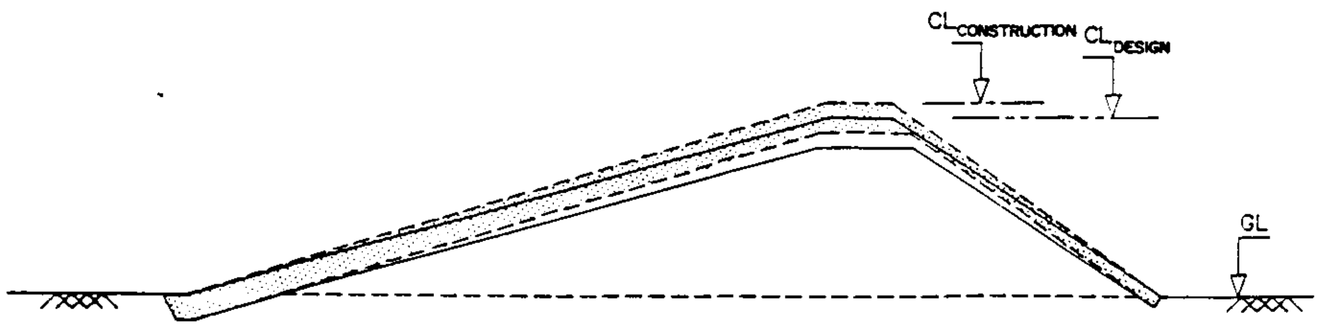
RESECTIONED EMBANKMENT
DESIGN CROSS SECTION



RETIRED EMBANKMENT
DESIGN CROSS SECTION



CONSTRUCTION CROSS SECTION
(NOT TO SCALE)



NOTES:

DIMENSIONS ARE IN METRES

- : NATURAL GROUND
- : EMBANKMENT FORMATION LEVEL
- ©——© : MAIN REFERENCE LINE
- : CORE
- : COVER LAYER
- : DESIGN PROFILE FOR EXISTING EMBANKMENT (LDL DESIGN)
- : EXISTING EMBANKMENT (SCHEMATIC)

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CYCLONE PROTECTION PROJECT II

DRAFT PROJECT PREPARATION REPORT, APPENDIX C.

EARTH EMBANKMENT, PROPOSED DESIGN.

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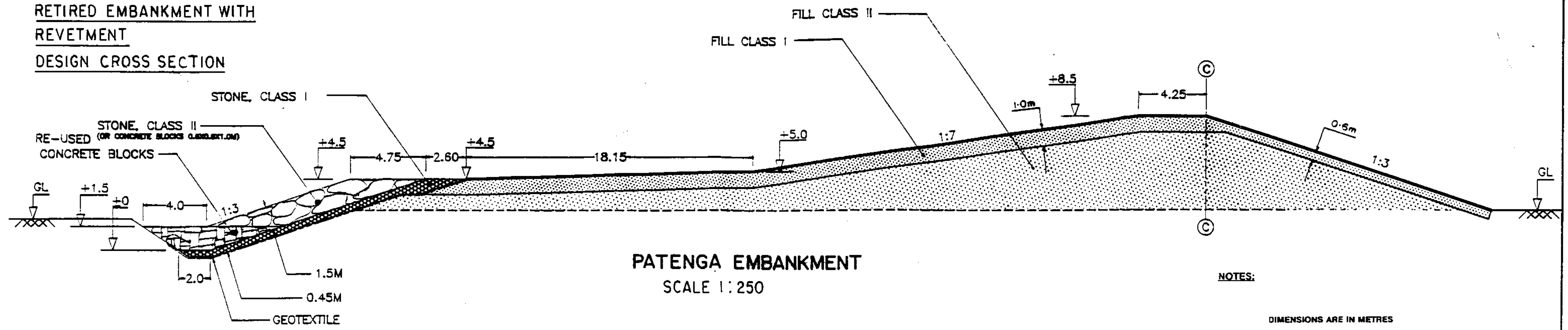
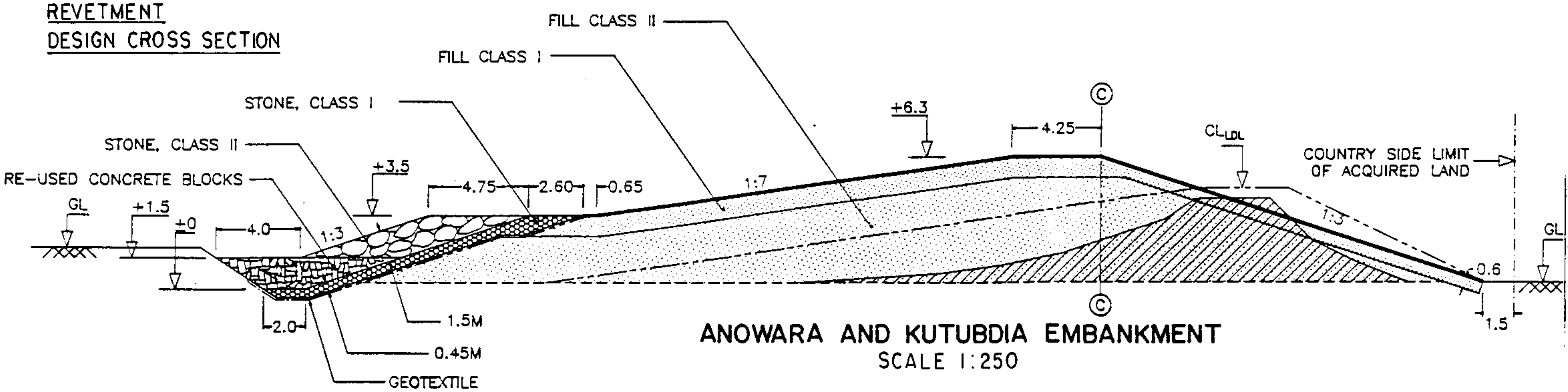
ENCLOSURE 3

RETIRED EMBANKMENT WITH REVETMENT
DESIGN CROSS SECTION



PATENGA EMBANKMENT
SCALE 1:250

RESECTIONED EMBANKMENT WITH REVETMENT
DESIGN CROSS SECTION



ANOWARA AND KUTUBDIA EMBANKMENT
SCALE 1:250

NOTES:

DIMENSIONS ARE IN METRES

- : NATURAL GROUND
- : EMBANKMENT FORMATION LEVEL
- Ⓒ—Ⓒ : MAIN REFERENCE LINE
- : CORE
- : COVER LAYER
- : DESIGN PROFILE FOR EXISTING EMBANKMENT (LDL DESIGN)
- : EXISTING EMBANKMENT (SCHEMATIC)

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CYCLONE PROTECTION PROJECT II

DRAFT PROJECT PREPARATION REPORT, APPENDIX C.

SLOPE PROTECTION, PROPOSED DESIGN

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	ENCLOSURE 4

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BANGLADESH WATER DEVELOPMENT BOARD

CYCLONE PROTECTION PROJECT II

DRAFT PROJECT PREPARATION REPORT, APPENDIX C.

CALCULATION OF WATERDEPTHS DURING OVERTOPPING

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ENCLOSURE **5**

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CALCULATION OF WATER DEPTH IN POLDER DURING OVERTOPPING

POLDER 63/1A, ANOWARA, CHAINAGE 33 to 42 KM

Foreshore Level 2.75

CYCLONIC CONDITIONS, 20 years Return Period

Time (Peak=0) Hours	-2	-1	0	1	2
Recorded Water Level, RWL	4.45	5.35	5.70	5.35	4.45
Ground Level, GL	2.75	2.75	2.75	2.75	2.75
Depth = RWL-GL	1.70	2.60	2.95	2.60	1.70
Depth incl. Wave Setup	1.70	2.60	2.95	2.60	1.70
Depth at SWL, h(m)	0.82	1.90	2.25	1.90	0.82
Slope	0.005	0.005	0.005	0.005	0.005
Tp (sec)	9	9	9	9	9
Lo (m)	126.47	126.47	126.47	126.47	126.47
H'o (m)	8.8	8.8	8.8	8.8	8.8
H'0/Lo	0.070	0.070	0.070	0.070	0.070
Beta o	0.078	0.078	0.078	0.078	0.078
Beta 1	0.531	0.531	0.531	0.531	0.531
Betao*H'o					
+Beta1*h = Hs (m)	1.12	1.69	1.88	1.69	1.12
H(mean)	0.70	1.06	1.17	1.06	0.70
Check:					
h/H'o	0.09	0.22	0.26	0.22	0.09
H'o/Lo	0.07	0.07	0.07	0.07	0.07
Eta/H'o	0.1	0.08	0.08	0.08	0.1
Wave setup	0.88	0.70	0.70	0.70	0.88
(Surf beat)	0.32	0.30	0.30	0.30	0.32
Hs/Depth	0.66	0.65	0.64	0.65	0.66

SEAWARD SLOPE 1:7

Return Period	-2	-1	0	1	2
Hs (m)	1.12	1.69	1.88	1.69	1.12
Tp	9	9	9	9	9
Tz	8.2	8.2	8.2	8.2	8.2
COT (alpha)	7	7	7	7	7
Ksip	1.52	1.24	1.17	1.24	1.52
Ksiz	1.38	1.12	1.07	1.12	1.38
Cn	0.60	0.60	0.60	0.60	0.60
Rs/H	1.63	1.32	1.26	1.32	1.63
Gammar	0.95	0.95	0.95	0.95	0.95

Wave Angle 45 Correction 0.707106

Wave Runup above RWL	1.22	1.50	1.59	1.50	1.22

CALCULATION OF WATER DEPTH IN POLDER DURING OVERTOPPING, PAGE 2

Overtopping:

	Crest level					
	6.00	1.554	0.654	0.304	0.654	1.554
	6.50	2.054	1.154	0.804	1.154	2.054
Height of	7.00	2.554	1.654	1.304	1.654	2.554
crest over	7.50	3.054	2.154	1.804	2.154	3.054
RWL	8.00	3.554	2.654	2.304	2.654	3.554
	8.50	4.054	3.154	2.804	3.154	4.054

Time (Peak=0) Hours		-2	-1	0	1	2
	Crest					
	6	58	585	1128	585	58
	6.5	23	274	550	274	23
Overtopping	7	9	129	268	129	9
(m3/m/1 hrs.)	7.5	4	60	131	60	4
	8	1	28	64	28	1
	8.5	1	13	31	13	1

	Crest	X=5	X=3
Overtopping	6	2413	2297
Total	6.5	1144	1099
after X hrs.	7	544	526
(m3/m)	7.5	259	252
	8	123	120
	8.5	59	58

Polder Area (m2) 12000000
Sea facing length (m) 8000

	Crest	X=5	X=3
Waterdepth	6	1.61	1.53
after X hrs.	6.5	0.76	0.73
	7	0.36	0.35
	7.5	0.17	0.17
	8	0.08	0.08
	8.5	0.04	0.04

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CYCLONE PROTECTION PROJECT II

FINAL PROJECT PREPARATION REPORT, APPENDIX C

CALCULATION OF DAMAGES TO EMBANKMENTS

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ENCLOSURE 6

POLDER 64/1A, BANSKHALI

Foreshore Level 1.7

WAVE CONDITIONS:
Cyclonic Conditions

Return Period	2.33	5	10	20	50	100	200
Recorded Water Level,RWL	3.27	4.49	5.10	5.71	6.41	6.93	7.23
Ground Level,GL	1.70	1.70	1.70	1.70	1.70	1.70	1.70
Depth = RWL-GL	1.57	2.79	3.40	4.01	4.71	5.23	5.53
Depth incl. Wave Setup	1.43	2.72	3.40	4.01	4.71	5.23	5.48
Depth at SWL, h(m)	0.88	2.11	2.74	3.31	4.04	4.57	4.87
Slope	0.005	0.005	0.005	0.005	0.005	0.005	0.005
Tp (sec)	7	8	8	9	9	9	9
Lo (m)	76.50	99.92	99.92	126.47	126.47	126.47	126.47
H'o (m)	6.9	7.6	8.2	8.8	9.6	10.2	10.2
H'0/Lo	0.090	0.076	0.082	0.070	0.076	0.081	0.081
Beta o	0.070	0.075	0.073	0.078	0.075	0.073	0.073
Beta 1	0.531	0.531	0.531	0.531	0.531	0.531	0.531
Betao*H'o +Beta1*h = Hs (m)	0.95	1.69	2.05	2.44	2.87	3.18	3.33
H(mean)	0.60	1.06	1.28	1.53	1.79	1.98	2.08
Check:							
h/H'o	0.13	0.28	0.33	0.38	0.42	0.45	0.48
H'o/Lo	0.09	0.08	0.08	0.07	0.08	0.08	0.08
Eta/H'o	0.08	0.08	0.08	0.08	0.07	0.065	0.06
Wave setup	0.55	0.61	0.66	0.70	0.67	0.66	0.61
(Surf beat)	0.22	0.24	0.25	0.28	0.29	0.30	0.30

CALCULATION OF RUN UP:

Seaward Slope 1:7

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.1	0.05	0.03	0.01	0.005
Hs (m)		0.95	1.69	2.05	2.44	2.87	3.18	3.33
Tp		7	8	8	9	9	9	9
Tz		6.4	7.3	7.3	8.2	8.2	8.2	8.2
COT (alpha)		7	7	7	7	7	7	7
Ksip		1.28	1.10	1.00	1.03	0.95	0.90	0.88
Ksiz		1.16	1 00	0.91	0.94	0.86	0.82	0.80
Cn		0.60	0.60	0.60	0.60	0.60	0.60	0.60
Rs/H		1.37	1.18	1.07	1.10	1.02	0.97	0.94
Gammar		0.95	0.95	0.95	0.95	0.95	0.95	0.95
Wave Runup above RWL	R13	1.24	1.89	2.08	2.56	2.77	2.92	2.99
	R2	1.74	2.65	2.92	3.58	3.88	4.08	4.18

OVERTOPPING:

		2.33	5	10	20	50	100	200
Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.1	0.05	0.03	0.01	0.005

	Crest level							
	5.75	2.48	1.26	0.65	0.04	0.00	0.00	0.00
	6.00	2.73	1.51	0.90	0.29	0.00	0.00	0.00
	6.50	3.23	2.01	1.40	0.79	0.09	0.00	0.00
Height of	7.00	3.73	2.51	1.90	1.29	0.59	0.07	0.00
crest over	7.50	4.23	3.01	2.40	1.79	1.09	0.57	0.27
RWL	8.00	4.73	3.51	2.90	2.29	1.59	1.07	0.77
	8.50	5.23	4.01	3.40	2.79	2.09	1.57	1.27

	5.75	0.0	41.3	82.3	99.9	100.0	100.0	100.0
	6.00	0.0	28.1	68.9	97.4	100.0	100.0	100.0
	6.50	0.0	10.6	40.7	82.6	99.8	100.0	100.0
Overtopp. %	7.00	0.0	3.0	19.1	60.2	91.4	99.9	100.0
	7.50	0.0	0.7	7.2	37.7	73.6	92.6	98.4
	8.00	0.0	0.1	2.1	20.3	52.0	76.4	87.7
	8.50	0.0	0.0	0.5	9.4	32.4	56.1	69.9

	5.75		20.67	61.79	91.11	99.97	100.00	100.00
	6.00		14.06	48.50	83.15	98.71	100.00	100.00
Overtopp. %	6.50		5.29	25.63	61.64	91.19	99.90	100.00
in interval	7.00		1.51	11.07	39.65	75.79	95.64	99.93
	7.50		0.33	3.90	22.41	55.62	83.08	95.49
	8.00		0.05	1.12	11.20	36.15	64.22	82.03
	8.50		0.01	0.26	4.94	20.36	44.22	62.99

RESPONSE (Damage Function):

Overtopp. (%)	0	5	10	15	20	25	30
Damage (%)	0.00	0.00	0.00	0.01	0.06	0.18	0.38
Overtopp. (%)	35	40	45	50	55	60	65
Damage (%)	0.72	1.24	2.00	3.06	4.48	6.35	8.76
Overtopp. (%)	70	75	80	85	90	95	100
Damage (%)	11.78	15.53	20.11	25.63	32.22	40.00	49.11

9y

DAMAGE TO CREST AND INNER SLOPE:

	Crest level	Interval (years) 2.3-5	5-10	10-20	20-50	50-100	100-200
	5.75	0.04	3.57	16.92	24.53	24.56	24.56
	6.00	0.00	1.35	11.73	23.31	24.56	24.56
Damage	6.50	0.00	0.10	3.54	16.98	24.45	24.56
in interval	7.00	0.00	0.00	0.60	8.10	20.54	24.49
(%)	7.50	0.00	0.00	0.05	2.34	11.69	20.42
	8.00	0.00	0.00	0.00	0.41	4.17	11.11
	8.50	0.00	0.00	0.00	0.04	0.93	3.86

Cross Sectional Volume of Embankment:

	Crest Lev	Vol.
	5.75	99.43
	6.00	110.94
	6.50	135.84
(m3)	7.00	163.24
	7.50	193.14
	8.00	225.54
	8.50	260.44

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.1	0.05	0.03	0.01	0.005
	5.75		0.04	3.55	16.82	24.39	24.42	24.42
Damage Vol	6.00		0.00	1.50	13.02	25.36	27.24	27.24
in interval	6.50		0.00	0.13	4.81	23.07	33.22	33.36
(m3/m/year)	7.00		0.00	0.00	0.98	13.22	33.53	39.98
	7.50		0.00	0.00	0.11	4.53	22.59	39.43
	8.00		0.00	0.00	0.00	0.93	9.41	25.06
	8.50		0.00	0.00	0.00	0.10	2.43	10.05
	5.75		0.01	0.36	0.84	0.73	0.24	0.12
Most prob.	6.00		0.00	0.15	0.65	0.78	0.27	0.14
Damage vol. in	6.50		0.00	0.01	0.24	0.69	0.33	0.17
intervals	7.00		0.00	0.00	0.05	0.40	0.34	0.20
per year	7.50		0.00	0.00	0.00	0.14	0.23	0.20
(m3/m/year)	8.00		0.00	0.00	0.00	0.00	0.09	0.13
	8.50		0.00	0.00	0.00	0.00	0.00	0.05

	Crest	Total
	5.75	2.30
	6	1.99
Most prob.	6.5	1.44
total damage	7	0.98
per year	7.5	0.56
(m3/m)	8	0.22
	8.5	0.05

	Crest	Total
	5.75	2.32
	6	1.79
Most prob.	6.5	1.06
total damage	7	0.60
per year in %	7.5	0.29
of cross section	8	0.10
(%)	8.5	0.02

POLDER 62, PATENGA SOUTH

Foreshore Level 0.5

WAVE CONDITIONS:
Cyclonic Conditions

Return Period	2.33	5	10	20	50	100	200
Recorded Water Level,RWL	3.35	4.57	5.18	5.79	6.49	7.01	7.31
Ground Level,GL	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Depth = RWL-GL	2.85	4.07	4.68	5.29	5.99	6.51	6.81
Depth incl. Wave Setup	2.85	4.07	4.68	5.29	5.99	6.51	6.81
Depth at SWL, h(m)	2.37	3.54	4.11	4.76	5.41	5.90	6.30
Slope	0.005	0.005	0.005	0.005	0.005	0.005	0.005
Tp (sec)	7	8	8	9	9	9	9
Lo (m)	76.50	99.92	99.92	126.47	126.47	126.47	126.47
H'o (m)	6.9	7.6	8.2	8.8	9.6	10.2	10.2
H'O/Lo	0.090	0.076	0.082	0.070	0.076	0.081	0.081
Beta o	0.070	0.075	0.073	0.078	0.075	0.073	0.073
Beta 1	0.531	0.531	0.531	0.531	0.531	0.531	0.531
Betao*H'o +Beta1*h = Hs (m)	1.74	2.45	2.78	3.21	3.59	3.88	4.09
H(mean)	1.09	1.53	1.74	2.01	2.25	2.43	2.56
Check:							
h/H'o	0.34	0.47	0.50	0.54	0.56	0.58	0.62
H'o/Lo	0.09	0.08	0.08	0.07	0.08	0.08	0.08
Eta/H'o	0.07	0.07	0.07	0.06	0.06	0.06	0.05
Wave setup	0.48	0.53	0.57	0.53	0.58	0.61	0.51
(Surf beat)	0.20	0.23	0.23	0.27	0.28	0.29	0.28

CALCULATION OF RUN UP:

Seaward Slope 1:7

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.1	0.05	0.03	0.01	0.005
Hs (m)		1.74	2.45	2.78	3.21	3.59	3.88	4.09
Tp		7	8	8	9	9	9	9
Tz		6.4	7.3	7.3	8.2	8.2	8.2	8.2
COT (alpha)		7	7	7	7	7	7	7
Ksip		0.95	0.91	0.86	0.90	0.85	0.82	0.79
Ksiz		0.86	0.83	0.78	0.82	0.77	0.74	0.72
Cn		0.60	0.60	0.60	0.60	0.60	0.60	0.60
Rs/H		1.01	0.98	0.92	0.96	0.91	0.87	0.85
Gammar		0.95	0.95	0.95	0.95	0.95	0.95	0.95
Wave Runup above RWL	R13	1.68	2.28	2.42	2.93	3.10	3.22	3.31
	R2	2.35	3.19	3.39	4.10	4.34	4.51	4.63

OVERTOPPING:

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.1	0.05	0.03	0.01	0.005
	Crest level							
	6.00	2.65	1.43	0.82	0.21	0	0	0
	6.50	3.15	1.93	1.32	0.71	0.01	0	0
Height of	7.00	3.65	2.43	1.82	1.21	0.51	0	0
crest over	7.50	4.15	2.93	2.32	1.71	1.01	0.49	0.19
RWL	8.00	4.65	3.43	2.82	2.21	1.51	0.99	0.69
	8.50	5.15	3.93	3.32	2.71	2.01	1.49	1.19
	6.00	0.7	45.7	79.7	99.0	100.0	100.0	100.0
	6.50	0.1	24.1	55.6	89.0	100.0	100.0	100.0
Overtopp. %	7.00	0.0	10.5	32.7	71.3	94.8	100.0	100.0
	7.50	0.0	3.8	16.3	51.0	81.0	95.5	99.3
	8.00	0.0	1.1	6.9	32.4	62.5	82.9	91.7
	8.50	0.0	0.3	2.4	18.4	43.5	65.5	77.4
	6.00		23.23	62.73	89.36	99.49	100.00	100.00
Overtopp. %	6.50		12.08	39.82	72.31	94.51	100.00	100.00
in interval	7.00		5.23	21.60	52.05	83.06	97.39	100.00
	7.50		1.88	10.02	33.63	66.00	88.28	97.44
	8.00		0.56	3.98	19.64	47.47	72.73	87.35
	8.50		0.14	1.35	10.41	30.94	54.49	71.44

RESPONSE (Damage Function):

Overtopp. (%)	0	5	10	15	20	25	30
Damage (%)	0	0	0.00	0.01	0.06	0.18	0.38
Overtopp. (%)	35	40	45	50	55	60	65
Damage (%)	0.72	1.24	2.00	3.06	4.48	6.35	8.76
Overtopp. (%)	70	75	80	85	90	95	100
Damage (%)	11.78	15.53	20.11	25.63	32.22	40.00	49.11

DAMAGE TO CREST AND INNER SLOPE:

		Interval (years)					
		2.3-5	5-10	10-20	20-50	50-100	100-200
	6.00	0.13	7.59	31.30	48.13	49.11	49.11
	6.50	0.00	1.22	13.41	39.18	49.11	49.11
Damage %	7.00	0.00	0.09	3.59	23.37	44.18	49.11
in interval	7.50	0.00	0.00	0.61	9.31	29.83	44.27
	8.00	0.00	0.00	0.06	2.48	13.73	28.59
	8.50	0.00	0.00	0.00	0.44	4.32	12.78

	Crest lev	Vol.
	6.00	174.90
	6.50	205.80
(m3)	7.00	239.20
	7.50	275.10
	8.00	313.50
	8.50	354.40

Return Period	2.33	5	10	20	50	100	200
Frequency of exceedence	0.429184	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)		2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval		0.229184	0.1	0.05	0.03	0.01	0.005

	6.00	0.23	13.28	54.75	84.17	85.90	85.90
Damage Vol	6.50	0.00	2.51	27.61	80.64	101.07	101.07
in interval	7.00	0.00	0.22	8.59	55.91	105.69	117.48
(m3/m)	7.50	0.00	0.00	1.69	25.60	82.06	121.77
	8.00	0.00	0.00	0.19	7.78	43.05	39.61
	8.50	0.00	0.00	0.00	1.55	15.30	45.30
	6.00	0.05	1.33	2.74	2.53	0.86	0.43
Most prob.	6.50	0.00	0.25	1.38	2.42	1.01	0.51
Damage vol. in	7.00	0.00	0.00	0.43	1.68	1.06	0.59
intervals	7.50	0.00	0.00	0.00	0.77	0.82	0.61
per year	8.00	0.00	0.00	0.00	0.00	0.43	0.45
(m3/m/year)	8.50	0.00	0.00	0.00	0.00	0.00	0.23

	Crest	Total		Crest	Total
	6	7.93		6	4.53
Most prob.	6.5	5.57	Most prob	6.5	2.71
total damage	7	3.75	total dam	7	1.57
per year	7.5	2.20	per year	7.5	0.80
(m3/m)	8	0.88	of cross	8	0.28
	8.5	0.23	(%)	8.5	0.06

POLDER 62, PATENGA NORTH

Foreshore Level 2.5

WAVE CONDITIONS:
Cyclonic Conditions

Return Period	2.33	5	10	20	50	100	200
Recorded Water Level,RWL	3.35	4.57	5.18	5.79	6.49	7.01	7.31
Ground Level,GL	2.50	2.50	2.50	2.50	2.50	2.50	2.50
Depth = RWL-GL	0.85	2.07	2.68	3.29	3.99	4.51	4.81
Depth incl. Wave Setup	0.85	2.07	2.68	3.29	3.99	4.51	4.81
Depth at SWL, h(m)	0.23	1.46	2.11	2.76	3.41	3.90	4.20
Slope	0.005	0.005	0.005	0.005	0.005	0.005	0.005
Tp (sec)	7	8	8	9	9	9	9
Lo (m)	76.50	99.92	99.92	126.47	126.47	126.47	126.47
H'o (m)	6.9	7.6	8.2	8.8	9.6	10.2	10.2
H'0/Lo	0.090	0.076	0.082	0.070	0.076	0.081	0.081
Beta o	0.070	0.075	0.073	0.078	0.075	0.073	0.073
Beta 1	0.531	0.531	0.531	0.531	0.531	0.531	0.531
Betao*H'o +Beta1*h = Hs (m)	0.61	1.35	1.72	2.15	2.53	2.82	2.98
H(mean)	0.38	0.84	1.07	1.34	1.58	1.76	1.86
Check:							
h/H'o	0.03	0.19	0.26	0.31	0.36	0.38	0.41
H'o/Lo	0.09	0.08	0.08	0.07	0.08	0.08	0.08
Eta/H'o	0.09	0.08	0.07	0.06	0.06	0.06	0.06
Wave setup	0.62	0.61	0.57	0.53	0.58	0.61	0.61
(Surf beat)	0.23	0.25	0.26	0.29	0.30	0.31	0.30

CALCULATION OF RUN UP:

Seaward Slope 1:7

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.200	0.100	0.050	0.020	0.010	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.100	0.050	0.030	0.010	0.005
Hs (m)		0.61	1.35	1.72	2.15	2.53	2.82	2.98
Tp		7	8	8	9	9	9	9
Tz		6.4	7.3	7.3	8.2	8.2	8.2	8.2
COT (alpha)		7	7	7	7	7	7	7
Ksip		1.60	1.23	1.09	1.10	1.01	0.96	0.93
Ksiz		1.46	1.12	0.99	1.00	0.92	0.87	0.85
Cn		0.60	0.60	0.60	0.60	0.60	0.60	0.60
Rs/H		1.72	1.32	1.17	1.17	1.08	1.03	1.00
Gammar		0.95	0.95	0.95	0.95	0.95	0.95	0.95
Wave Runup above RWL	R13	0.99	1.69	1.91	2.40	2.60	2.75	2.82
	R2	1.39	2.36	2.67	3.36	3.64	3.85	3.95

OVERTOPPING:

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.200	0.100	0.050	0.020	0.010	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.100	0.050	0.030	0.010	0.005
	Crest level							
	6.00	2.65	1.43	0.82	0.21	0	0	0
	6.50	3.15	1.93	1.32	0.71	0.01	0	0
Height of	7.00	3.65	2.43	1.82	1.21	0.51	0	0
crest over	7.50	4.15	2.93	2.32	1.71	1.01	0.49	0.19
RWL	8.00	4.65	3.43	2.82	2.21	1.51	0.99	0.69
	8.50	5.15	3.93	3.32	2.71	2.01	1.49	1.19
	6.00	0.0	24.1	69.3	98.5	100.0	100.0	100.0
	6.50	0.0	7.5	38.7	84.1	100.0	100.0	100.0
Overtopp. %	7.00	0.0	1.6	16.4	60.4	92.7	100.0	100.0
	7.50	0.0	0.3	5.3	36.5	74.2	93.9	99.1
	8.00	0.0	0.0	1.3	18.6	51.3	77.3	88.8
	8.50	0.0	0.0	0.2	8.0	30.7	55.8	70.3
	6.00		12.04	46.69	83.90	99.25	100.00	100.00
Overtopp. %	6.50		3.74	23.07	61.36	92.03	100.00	100.00
in interval	7.00		0.82	9.03	38.40	76.53	96.34	100.00
	7.50		0.13	2.78	20.91	55.36	84.05	96.50
	8.00		0.01	0.67	9.95	34.96	64.32	83.07
	8.50		0.00	0.12	4.10	19.32	43.25	63.08

RESPONSE (Damage Function):

Overtopp. (%)	0	5	10	15	20	25	30
Damage (%)	0	0	0.00	0.01	0.06	0.18	0.38
Overtopp. (%)	35	40	45	50	55	60	65
Damage (%)	0.72	1.24	2.00	3.06	4.48	6.35	8.76
Overtopp. (%)	70	75	80	85	90	95	100
Damage (%)	11.78	15.53	20.11	25.63	32.22	40.00	49.11

DAMAGE TO CREST AND INNER SLOPE:

	Crest level	2.3-5	5-10	10-20	20-50	50-100	100-200
		Interval (years)					
	6.00	0.00	2.32	24.33	47.65	49.11	49.11
	6.50	0.00	0.13	6.95	35.22	49.11	49.11
Damage %	7.00	0.00	0.00	1.05	16.84	42.30	49.11
in interval	7.50	0.00	0.00	0.08	4.60	24.50	42.58
	8.00	0.00	0.00	0.00	0.72	8.39	23.38
	8.50	0.00	0.00	0.00	0.05	1.71	7.76

Cross Sectional Volume of Embankment:

	Crest lev	Vol.
	7.50	76.30
	8.00	97.20
(m3)	8.50	120.60
	0.00	146.50
	6.00	174.90
	6.50	205.80

Return Period		2.33	5	10	20	50	100	200
Frequency of exceedence		0.429	0.2	0.1	0.05	0.02	0.01	0.005
Interval (years)			2.3-5	5-10	10-20	20-50	50-100	100-200
Freq. of exceedence in Interval			0.229	0.1	0.05	0.03	0.01	0.005
	6.00		0.00	1.77	18.56	36.36	37.47	37.47
	6.50		0.00	0.12	6.76	34.23	47.73	47.74
Damage Vol	7.00		0.00	0.00	1.27	20.30	51.02	59.23
in interval	7.50		0.00	0.00	0.12	6.74	35.89	62.39
(m3/m)	8.00		0.00	0.00	0.00	1.26	14.68	40.90
	8.50		0.00	0.00	0.00	0.11	3.51	15.98
	6.00		0.00	0.18	0.93	1.09	0.37	0.19
Most prob.	6.50		0.00	0.01	0.34	1.03	0.48	0.24
Damage vol. in	7.00		0.00	0.00	0.06	0.61	0.51	0.30
intervals	7.50		0.00	0.00	0.00	0.20	0.36	0.31
per year	8.00		0.00	0.00	0.00	0.00	0.15	0.20
(m3/m/year)	8.50		0.00	0.00	0.00	0.00	0.00	0.08

	Crest	Total		Crest	Total
	6	2.76		6	3.61
Most prob.	6.5	2.09	Most prob	6.5	2.15
total damage	7	1.48	total dam	7	1.23
per year	7.5	0.87	per year	7.5	0.60
(m3/m)	8	0.35	of cross	8	0.20
	8.5	0.08	(%)	8.5	0.04

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PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION, WATER DEVELOPMENT AND FLOOD CONTROL
BANGLADESH WATER DEVELOPMENT BOARD

CYCLONE PROTECTION PROJECT II

FINAL PROJECT PREPARATION REPORT, APPENDIX C

OPTIMIZATION OF EMBANKMENT DESIGN, BENEFITS

KAMPSAX INTERNATIONAL A/S, BCEOM and DANISH HYDRAULIC INSTITUTE
in association with
DEVELOPMENT DESIGN CONSULTANTS LTD.
23, New Eskaton Road, Dhaka-1000, Tel. 405477, Fax 880 02 832951

DATE :01-02-1992

ENCLOSURE 7

POLDER 64/1a: Banskhali Length of embank Km: 27.4

Design Return Period 10 Years

Total financial cost: 220.0 Ec. Cost: 165.0 C. of Struc 8.0

YEAR	INVESTM O & M	TOTAL BENEFITS	Cash Flows	Agricul Benefits 20% reduct*	Non-agricult. benefits	Agric losses caused by cyclones	Inv O&M financ
1	65.99	0	(66.0)	0	0	0	38.0
2	98.99	0	(99.0)	0	0	0	132.0
3	6.43	15.0	3.6	4.3	7.4	3.3	6.1
4	6.43	19.6	13.2	8.7	7.6	3.3	6.1
5	6.43	24.2	17.8	13.0	7.9	3.3	6.1
6	6.43	26.7	20.3	15.3	8.1	3.3	6.1
7	6.43	29.3	22.8	17.6	8.3	3.3	6.1
8	6.43	29.5	23.1	17.6	8.6	3.3	6.1
9	6.43	29.8	23.3	17.6	8.8	3.3	6.1
10	6.43	30.0	23.6	17.6	9.1	3.3	6.1
11	6.43	30.3	23.9	17.6	9.4	3.3	6.1
12	6.43	30.6	24.2	17.6	9.7	3.3	6.1
13	6.43	30.9	24.4	17.6	10.0	3.3	6.1
14	6.43	31.2	24.7	17.6	10.3	3.3	6.1
15	6.43	31.5	25.1	17.6	10.6	3.3	6.1
16	6.43	31.8	25.4	17.6	10.9	3.3	6.1
17	6.43	32.1	25.7	17.6	11.2	3.3	6.1
18	6.43	32.5	26.0	17.6	11.5	3.3	6.1
19	6.43	32.8	26.4	17.6	11.9	3.3	6.1
20	6.43	33.2	26.7	17.6	12.2	3.3	6.1
21	6.43	33.5	27.1	17.6	12.6	3.3	6.1
22	6.43	33.9	27.5	17.6	13.0	3.3	6.1
23	6.43	34.3	27.9	17.6	13.4	3.3	6.1
24	6.43	34.7	28.3	17.6	13.8	3.3	6.1
25	6.43	35.1	28.7	17.6	14.2	3.3	6.1
26	6.43	35.5	29.1	17.6	14.6	3.3	6.1
27	6.43	36.0	29.5	17.6	15.1	3.3	6.1
28	6.43	36.4	30.0	17.6	15.5	3.3	6.1
29	6.43	36.9	30.5	17.6	16.0	3.3	6.1
30	6.43	37.4	30.9	17.6	16.5	3.3	6.1
NPV@12%	178.8	173.6	(5.2)	93.4	59.3	20.9	222.7

All calculations are based on a project period of 30 years.

EIRR base case 11.6 %

Benefits-cost ratio agricu' effects 0.64

Benefits-cost ratio non-agri effects 0.33

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POLDER 64/1a: Banskhali   Length of embank km: 27.4

Design Return Period 20 Years

Total   financial cost: 267.3   Ec. Cost: 200.5 C. of Struc   8.0

YEAR	INVESTM O & M	TOTAL BENEFITS	Cash Flows	Agricul Benefits 20% reduct*	Non-agricult. benefits	Agric losses caused by cyclones	Inv O&M financ
1	80.20	0	(80.2)	0	0	0	106.9
2	120.30	0	(120.3)	0	0	0	160.4
3	4.58	25.4	20.9	4.3	16.1	5.0	6.1
4	4.58	30.3	25.7	8.7	16.6	5.0	6.1
5	4.58	35.1	30.5	13.0	17.0	5.0	6.1
6	4.58	37.9	33.3	15.3	17.6	5.0	6.1
7	4.58	40.7	36.2	17.6	18.1	5.0	6.1
8	4.58	41.3	36.7	17.6	18.6	5.0	6.1
9	4.58	41.8	37.3	17.6	19.2	5.0	6.1
10	4.58	42.4	37.8	17.6	19.8	5.0	6.1
11	4.58	43.0	38.4	17.6	20.4	5.0	6.1
12	4.58	43.6	39.0	17.6	21.0	5.0	6.1
13	4.58	44.3	39.7	17.6	21.6	5.0	6.1
14	4.58	44.9	40.3	17.6	22.2	5.0	6.1
15	4.58	45.6	41.0	17.6	22.9	5.0	6.1
16	4.58	46.3	41.7	17.6	23.6	5.0	6.1
17	4.58	47.0	42.4	17.6	24.3	5.0	6.1
18	4.58	47.7	43.1	17.6	25.0	5.0	6.1
19	4.58	48.4	43.9	17.6	25.8	5.0	6.1
20	4.58	49.2	44.6	17.6	26.6	5.0	6.1
21	4.58	50.0	45.4	17.6	27.4	5.0	6.1
22	4.58	50.8	46.3	17.6	28.2	5.0	6.1
23	4.58	51.7	47.1	17.6	29.0	5.0	6.1
24	4.58	52.5	48.0	17.6	29.9	5.0	6.1
25	4.58	53.4	48.9	17.6	30.8	5.0	6.1
26	4.58	54.4	49.8	17.6	31.7	5.0	6.1
27	4.58	55.3	50.7	17.6	32.7	5.0	6.1
28	4.58	56.3	51.7	17.6	33.6	5.0	6.1
29	4.58	57.3	52.7	17.6	34.7	5.0	6.1
30	4.58	58.3	53.8	17.6	35.7	5.0	6.1
NPV@12%	196.7	254.0	57.3	93.4	128.7	31.9	262.2

All calculations are based on a project period of 30 years.

EIRR base case   15.7 %

Benefits-cost ratio agricul effects   0.64

Benefits-cost ratio non-agri effects   0.65

POLDER 64/1a: Banskhali Length of embank km: 27.4

Design Return Period 40 Years

Total financial cost: 356.0 Ec. Cost: 267.0 C. of Struc 8.0

YEAR	INVESTM O & M	TOTAL BENEFITS	Cash Flows	Agricul Benefits 20% reduct*	Non-agricult. benefits	Agric losses caused by cyclones	Inv O&M financ
1	106.79	0	(106.8)	0	0	0	142.4
2	160.18	0	(160.2)	0	0	0	213.6
3	3.35	26.1	22.7	4.3	16.1	5.7	6.1
4	3.35	30.9	27.6	8.7	16.6	5.7	6.1
5	3.35	35.8	32.4	13.0	17.0	5.7	6.1
6	3.35	38.6	35.2	15.3	17.6	5.7	6.1
7	3.35	41.4	38.0	17.6	18.1	5.7	6.1
8	3.35	41.9	38.6	17.6	18.6	5.7	6.1
9	3.35	42.5	39.1	17.6	19.2	5.7	6.1
10	3.35	43.1	39.7	17.6	19.8	5.7	6.1
11	3.35	43.6	40.3	17.6	20.4	5.7	6.1
12	3.35	44.3	40.9	17.6	21.0	5.7	6.1
13	3.35	44.9	41.5	17.6	21.6	5.7	6.1
14	3.35	45.5	42.2	17.6	22.2	5.7	6.1
15	3.35	46.2	42.9	17.6	22.9	5.7	6.1
16	3.35	46.9	43.5	17.6	23.6	5.7	6.1
17	3.35	47.6	44.3	17.6	24.3	5.7	6.1
18	3.35	48.3	45.0	17.6	25.0	5.7	6.1
19	3.35	49.1	45.7	17.6	25.8	5.7	6.1
20	3.35	49.9	46.5	17.6	26.6	5.7	6.1
21	3.35	50.6	47.3	17.6	27.4	5.7	6.1
22	3.35	51.5	48.1	17.6	28.2	5.7	6.1
23	3.35	52.3	49.0	17.6	29.0	5.7	6.1
24	3.35	53.2	49.8	17.6	29.9	5.7	6.1
25	3.35	54.1	50.7	17.6	30.8	5.7	6.1
26	3.35	55.0	51.7	17.6	31.7	5.7	6.1
27	3.35	56.0	52.6	17.6	32.7	5.7	6.1
28	3.35	56.9	53.6	17.6	33.6	5.7	6.1
29	3.35	57.9	54.6	17.6	34.7	5.7	6.1
30	3.35	59.0	55.6	17.6	35.7	5.7	6.1
NPV@12%	244.3	258.1	13.7	93.4	128.7	36.0	336.3

All calculations are based on a project period of 30 years.

EIRR base case 12.7 %

Benefits-cost ratio agricul effects 0.53

Benefits-cost ratio non-agri effects 0.53

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Estimation of losses due to inundation caused by cyclones

Design Return Period 10 years

Category III Sandwip

Crop	Value per ha	cult. land %	Losses due floods 10 yrs %	Losses due floods 20 yrs %	value of losses 10 yrs Tk	value of losses 20 yrs Tk	Accum losses Tk
B. Aus	6,415	8.5	40	10	10.9	1.4	12.3
T. Aus	8,370	10.0	40	10	16.7	2.1	18.8
T. Aman L	5,984	38.3	40	10	45.8	5.7	51.6
T. Aman HYV	9,005	15.3	40	10	27.6	3.4	31.0
Boro	11,286	5.6	40	10	12.6	1.6	14.2

Accumulated losses per cultivated ha per year 127.9

Accumulated losses per physical ha per year 230.2

Category VIBAnskhali,Anowara,Kutubdia

Crop	Value per ha	cult. land %	Losses due floods 10 yrs %	Losses due floods 20 yrs %	value of losses 10 yrs Tk	value of losses 20 yrs Tk	Accum losses Tk
B. Aus	7,247	10.5	40	10	15.2	1.9	17.1
T. Aus	13.365	13.6	40	10	36.4	4.5	40.9
T. Aman L	11,286	30.3	40	10	68.4	8.5	76.9
T. Aman HYV	16,632	20.3	40	10	67.5	8.4	76.0
Boro	19,008	10.7	40	10	40.7	5.1	45.8

Accumulated losses per cultivated ha per year 256.7

Accumulated losses per physical ha per year 469.7

Category VI Indirect protected Value

Crop	Value per ha	cult. land %	Losses due floods 20 yrs %	Losses due floods 40 yrs %	value of losses 20 yrs Tk	value of losses 40 yrs Tk	Accum losses Tk
B. Aus	7,247	10.5	30	0	5.7	0.0	5.7
T. Aus	13,365	13.6	30	0	13.6	0.0	13.6
T. Aman L	11,286	30.3	30	0	25.6	0.0	25.6
T. Aman HYV	16,632	20.3	30	0	25.3	0.0	25.3
Boro	19,008	10.7	30	0	15.3	0.0	15.3

Accumulated losses per cultivated ha per year 85.6

Accumulated losses per physical ha per year 156.6

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Estimation of losses due to inundation caused by cyclones

Design Return Period 20 Years

Category III Sandwip

Crop	Value per ha	cult. land %	Losses due floods 10 yrs %	20 yrs %	value of losses 10 yrs Tk	20 yrs Tk	Accum losses Tk
B. Aus	6,415	8.5	50	40	13.6	5.5	19.1
T. Aus	8,370	10.0	50	40	20.9	8.4	29.3
T. Aman L	5,984	38.3	50	40	57.3	22.9	80.2
T. Aman HYV	9,005	15.3	50	40	34.4	13.8	48.2
Boro	11,286	5.6	50	40	15.8	6.3	22.1

Accumulated losses per cultivated ha per year 198.9

Accumulated losses per physical ha per year 358.1

Category VIBAnskhali,Anowara,Kutubdia

Crop	Value per ha	cult. land %	Losses due floods 10 yrs %	20 yrs %	value of losses 10 yrs Tk	20 yrs Tk	Accum losses Tk
B. Aus	7,247	10.5	50	40	19.0	7.6	26.6
T. Aus	13,365	13.6	50	40	45.4	18.2	63.6
T. Aman L	11,286	30.3	50	40	85.5	34.2	119.7
T. Aman HYV	16,632	20.3	50	40	84.4	33.8	118.2
Boro	19,008	10.7	50	40	50.8	20.3	71.2

Accumulated losses per cultivated ha per year 399.3

Accumulated losses per physical ha per year 730.7

Category VI Indirect protected Value

Crop	Value per ha	cult. land %	Losses due floods 20 yrs %	40 yrs %	value of losses 20 yrs Tk	40 yrs Tk	Accum losses Tk
B. Aus	7,247	10.5	30	30	5.7	2.9	8.6
T. Aus	13,365	13.6	30	30	13.6	6.8	20.4
T. Aman L	11,286	30.3	30	30	25.6	12.8	38.5
T. Aman HYV	16,632	20.3	30	30	25.3	12.7	38.0
Boro	19,008	10.7	30	30	15.3	7.6	22.9

Accumulated losses per cultivated ha per year 128.3

Accumulated losses per physical ha per year 234.9

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Estimation of losses due to inundation caused by cyclones

Design Return Period 40 Years

Category III Sandwip

Crop	Value per ha	cult. land %	Losses due floods 10 yrs %	20 yrs %	value of losses 10 yrs Tk	20 yrs Tk	Accum losses Tk
E. Aus	6.415	8.5	50	50	13.6	6.8	20.4
T. Aus	8.370	10.0	50	50	20.9	10.5	31.4
T. Aman L	5.984	38.3	50	50	57.3	28.6	85.9
T. Aman HYV	9.005	15.3	50	50	34.4	17.2	51.7
Boro	11.286	3.6	50	50	15.8	7.9	23.7

Accumulated losses per cultivated ha per year 213.1

Accumulated losses per physical ha per year 383.7

Design Return Period 40 Years

Category VI Banskhali, Anowara, Kutubdia

Crop	Value per ha	cult. land %	Losses due floods 10 yrs %	20 yrs %	value of losses 10 yrs Tk	20 yrs Tk	Accum losses Tk
B. Aus	7,247	10.5	50	45	19.0	8.6	27.6
T. Aus	13,365	13.6	50	45	45.4	20.4	65.9
T. Aman L	11,286	30.3	50	45	85.5	38.5	124.0
T. Aman HYV	16,632	20.3	50	45	84.4	38.0	122.4
Boro	19,008	10.7	50	45	50.8	22.9	73.7

Accumulated losses per cultivated ha per year 413.6

Accumulated losses per physical ha per year 756.8

Category VI Indirect protected Value

Crop	Value per ha	cult. land %	Losses due floods 20 yrs %	40 yrs %	value of losses 20 yrs Tk	40 yrs Tk	Accum losses Tk
B. Aus	7,247	10.5	40	35	7.6	3.3	10.9
T. Aus	13,365	13.6	40	35	18.2	8.0	26.1
T. Aman L	11,286	30.3	40	35	34.2	15.0	49.2
T. Aman HYV	16,632	20.3	40	35	33.8	14.8	48.5
Boro	19,008	10.7	40	35	20.3	8.9	29.2

Accumulated losses per cultivated ha per year 164.0

Accumulated losses per physical ha per year 300.1

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ANNUAL CUMULATED BENEFIT

POLDER 64/1A BANSKHALI

Design Return Period 10 Years

Return period	Estimated damage without project	Frequency	Annual cost frequency	Estimated damage of protection	Annual cumulated benefits
5	0	0.80		1.00	
			3.2		3.2
10	71.344	0.90		0.90	
			3.7		7.0
20	142.688	0.95		0.60	
			2.0		8.9
40	356.72	0.975		0.20	
			0.5		9.5
100	535.08	0.990		0.00	
			0.0		9.5
200	713.44	0.995		0.00	

Design Return Period 20 Years

Return period	Estimated damage without project	Frequency	Annual cost frequency	Estimated damage of protection	Annual cumulated benefits
5	0	0.80		1.00	
			3.6		3.6
10	71.344	0.90		1.00	
			5.0		8.6
20	142.688	0.95		0.90	
			4.3		12.8
40	356.72	0.975		0.60	
			2.4		15.2
100	535.08	0.990		0.20	
			0.3		15.5
200	713.44	0.995		0.00	

Design Return Period 40 Years

Return period	Estimated damage without project	Frequency	Annual cost frequency	Estimated damage of protection	Annual cumulated benefits
5	0	0.80		1.00	
			3.5		3.5
10	70.98	0.90		1.00	
			5.3		8.9
20	141.96	0.95		1.00	
			5.8		14.6
40	354.9	0.975		0.90	
			4.8		19.4
100	532.35	0.990		0.60	
			1.2		20.6
200	709.8	0.995		0.20	