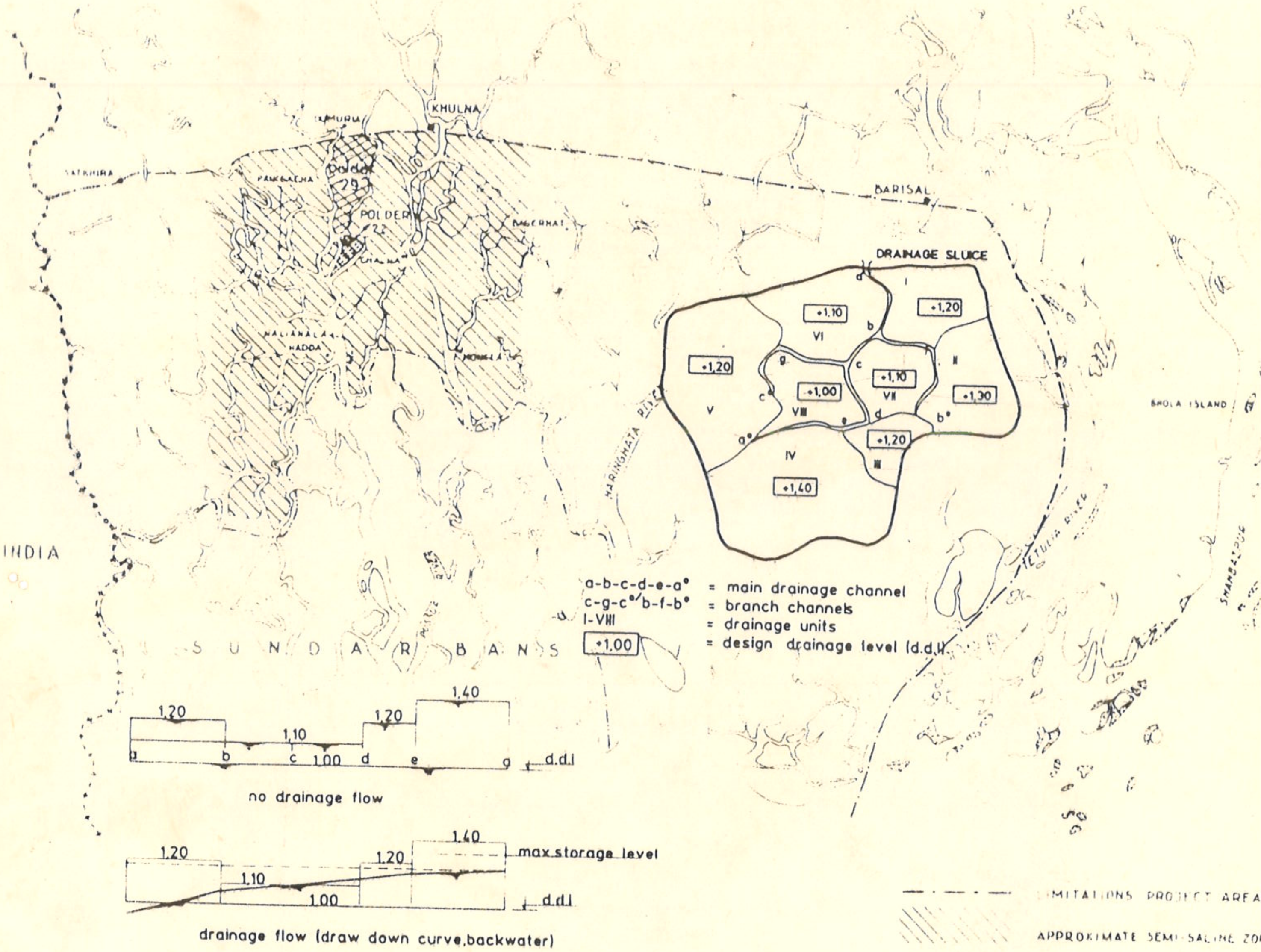
DESIGN MANUAL

FOR POLDERS IN SOUTH-WEST BANGLADESH



VOL. III

DELTA DEVELOPMENT PROJECT
BANGLADESH-NETHERLANDS JOINT PROGRAMME
UNDER BWDB

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VOLUME III DESIGN OF EMBANKMENTS

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VOLUME III. DESIGN OF EMBANKMENTS

Chapter 1. Criteria for Embankment Design

1.1 Introduction

The criteria on which the design is to be based, depend on the requirements of the embankment. The question to be answered is : How safe has the embankment to be, and what damages with respect to total costs and recurrence period, are acceptable. The total costs comprise the construction costs, maintenance and repair cost, and economic losses (cost) due to damage to land, crop, structures and population in case of failure of the embankment.

These total costs can be split up in construction and maintenance costs, which will increase with the height of the embankment, and anticipated damages as specified above.

It is rather difficult to assess how these anticipated damages will decrease if the height of the embankment increases. One may conclude that in case the embankment provides higher safety, it will stimulate higher investments in the protected area, which will result in higher damages in case of failure of the embankment. What really counts is the product of chance of failure times the anticipated damages in case of this failure, capitalized in a sum which has to be available to pay all costs during the lifetime of the embankment.

As already mentioned, it is very difficult to calculate the anticipated damages in case of embankment failure. Therefore it is adopted that generally a flood level with a return period of 20 years (i.e. chance of occuring once every 20 years), is the criteria for the crest height of the embankment.

In special areas, where significant investments are planned with regard to socio-economic and agricultural development this criteria is set at a return period for floods of once every 50 years.

1.2 Design Criteria

1.2.1 General

The aspects for which criteria have to be set are all related to the stability of the structure which should be safeguarded under all "design-load" conditions. For these load conditions distinction has to be made between sea dykes and river embankments.

The design load can be considered to comprise :

- duration and height of flood level
- wave action
- wind set-up
- currents

In the following paragraphs this will be further elaborated. However, the criteria presented should not be looked at as coersive for all embankments and dykes. For every case the possibilities and applicabilities have to be checked according to the methods elaborated further on.

1.2.2 Design floodlevel

The design floodlevel, which the embankment has to retain, is the major criteria for the design of the embankment. As highlighted in paragraph 1.1, it is recommended that the design flood level is the floodlevel which occurs once in 20 years for normal conditions and once in 50 years for special areas where the increased construction costs can be justified because of higher investment prospects.

1.2.3 Design crest level

The other design loads specified under paragraph 1.2.1 will contribute to the establishment of the design crest level, which in its turn is depending on the gradient of the river side slope of the embankment.

In most cases the influence of waves and wind will be such that precise calculation will yield only marginal increases to be added to the design flood level. In such cases an additional freeboard of 1 m is to be applied and design crest level will be the sum of design flood level plus 1 m.

For locations where wave run-up and wind set-up are appreciable, additional crest height has to be calculated as indicated in paragraph 3.3 and 3.4.

For wind set-up and wave heights, at least a period of 24 hours has to be considered. The average 24 hours wind velocity has to be analysed in the same way as is indicated in Volume IV - Chapter 2.3 for rainfall data.

1.2.4 Slope Gradients

In designing slopes of embankments, distinction is to be made between slopes of river embankments and slopes of sea dykes. River embankments have to resist floodlevel during longer periods than sea dykes, that are affected by flood levels during short periods only.

Therefore in a river embankment full saturation of the earth can develop and the c/s slope should be designed in such a way that any seepage flow will not cause damages. Sea dykes c/s slopes can be steeper because flow is less important. Seadykes however may have to face severe wave attack, for which a proper design of the r/s (arbitrarily the term river/side is also applied here to indicate sea side) has to be made.

Both types of embankment may have to endure attack by currents, which might compel revetments to avoid erosion.

The crest height criteria does not prevent any embankment overtopping. In order to reduce immediate and serious damage of the c/s slope by over flowing water, even in case this over topping is only minor, c/s slopes should never be less than 1 into 2 (horizontal), even if stability calculations might indicate steeper permissible gradients.

1.2.5 Design crest width

The crest width of an embankment has to have a minimum dimension of 2.5 m. Smaller widths will make the embankment more vulnarable in case of overtopping which will cause damages (erosion) to the inner slope and crest. In case inspection by vehicle is to be considered or if a road has to be constructed on top of the embankment, then additional width is to be provided. For off road inspection by fourwheel drive vehicle the width should be 4 m. In case of a road, the crestwidth should be the width of the road plus a 1.5 m berm at both sides.

1.2.6 Berms

Under certain conditions the application of a berm in the embankment may be usefull.

In Chapter 3.4 of this Volume it is highlighted that a berm might be effectively applied to reduce the wave run-up against an embankment. The width of such a berm may be calculated according to the formula presented and the extra cost of the berm should be evaluated against the cost of other means to reduce wave run-up.

In other cases, where artificial protection of the r/s slope have to be designed, a small berm is applied to facilitate the transition between two different types of slope revetments. Details of such construction berms are presented in Chapter 5.5 of this Volume.

In constructing closure dams, berms are applied with a width of 10 m at the level of H.W.S. to provide extra safety. More details are presented in Chapter 8.4 of this Volume.

1.2.7 Set back

The criteria on which the set back of the embankment (i.e. space between actual riverbank and r/s toe of the embankment) is to be designed can be formulated as follows.

- fore land should be big enough to locate the excavation pit for the embankment construction
- there should be sufficient space to increase the height of the embankment by putting additional material against the

outer slope

- for sea dykes the fore land should be big enough to locate a second borrow pit to be used for repairs. The area has to be afforested with adequate plants to reduce wave actions during flood conditions
- at places where erosion has taken place regularly over the past years, a set-back may be based on this erosion rate. An extra margin, equivalent to 10 years of the present erosion rate, should be added to the minimum setback to be applied.

Chapter 2. The Alignment of Embankments

The determination of the alignment of an embankment is governed by technical, economical and morphological considerations. Economically the best embankment is that, which can be built as efficient and cheap as possible, requires the least land acquisition, is made of the best available material, and encloses as much land as possible.

With regard to morphology of nearby rivers and tidal estuaries a sufficiently wide fore land has to be provided. Scouring of river banks has to be studied from maps and aerial photos in order to determine whether rivers tend to be moving towards the planned embankment and at what speed and whether this will result in a dangerous situation. Set-back criteria as mentioned in paragraph 1.2.7 should be considered. In any case sharp corners in the alignment have to be avoided as much as possible, especially at vulnarable locations.

For the alignment a soil survey has to be executed to make an inventory of the available soils to be used for the embankment construction. If the alignment allows, preference should be given to soils with a fair portion of clay, to provide water tightness of the embankment. Clay content should not be too high because a fresh embankment of clay will form cracks when it dries.

The sub-soil should not consist of peat or peaty soils, as they will result in unacceptable settlements. If no alternative is available, peaty soils should be removed. If the preliminary alignment has been determined, a soil survey should be executed to check if the soil conditions are satisfactory. Auger borings to a depth of 4 m should be made along the proposed alignment to investigate subsoil conditions for the foundation of the embankment and the material available in the proposed borrow pits. Borings should be made every 250 m along the alignment. Simple field test should provide information and general characteristics regarding silt and clay content. The soil conditions may give reason to change the alignment or to take due consideration of the earthmoving quantities with respect to settlements, shrinkage and transport distances.

The orientation of an embankment respective to prevailing wind-direction during flood conditions is important. An embankment that runs perpendicular to this wind direction may require a higher crest level considering higher waves and wave run-up.

Existing transportation systems may be affected by the embankment, that may block roads and rivers. Due consideration should be given to alternative solutions and their implications.

Chapter 3. The Height of the Embankment

3.1 General

The height of an embankment is determined by :

- the maximum water level that has to be retained:
  the "design flood level".
- an additional height for setup of water due to wind.
- an additional height for wave run-up.

Furthermore, allowance has to be provided for the shrinkage of the freshly built embankment body and the settlement of the subsoil due to the surcharge. However this additional height is not considered part of the design crest height. Allowance for shrinkage and settlement is further discussed in paragraph 3.5.

In addition an extra freeboard of 1 m is to be applied to arrive at the Design Crest Level.

The Design Crest Level is thus composed of :

$$H_{DES} = H_{FL} + H_w + H_z + 1 \text{ m.}$$

in which
H_{DES} = design crest level.
H_{FL} = design flood level
H_w = wind set-up
H_z = wave run-up
1 m = safety margin.

3.2 Design Flood Level

The determination of the design flood level can be done by frequency analysis of maximum yearly flood levels. In fact it would even be more accurate to use maximum yearly discharges for this analyses. Contrary to water levels, discharges are not influenced by morphological changes like changing riverbed, short cutting of meanders. However for most rivers morphological changes are very slow processes and therefore the method using frequencies of maximum floodlevels as defined by Gumbel, is adopted in this manual.

For the analyses of extreme water levels Gumbel assumed that a probability distribution exists to describe the phenomena of maximum yearly floodlevels. It should however realised that the number of records, or the duration of the period over which data is available, is often too short to apply statistical laws describing the phenomena with sufficient reliability and accuracy. Nevertheless a method was derived that approximates the probability distribution of maximum water levels. The probability that a yearly maximum with value x is reached, is given by:

$$P(x) = e^{-e^{-y}}$$

in which y is a reduced or standarised variable. The probability that a yearly maximum of value x is exceeded is then 1-P(x), and the probability that this maximum is exceeded in one specific year is $\frac{1}{1-P(x)}$. This value is called the expected return period T of the maximum value x. In a Gumbel graph the extreme value of water level or discharge is plotted against this return period T (see Figure II - 3.1). In fact the graph represents a plotting of the formula $P(x) = e^{-e^{-y}}$ for which x is the vertical axis (Q or H) and y is the horizontal axis, both in a lineair scale.

In the graph the plotting of extreme values against their return period has to be done in a special way. In stead of plotting the probability function P(x), Gumbel defined the plotting of the i-st smallest value as $\varphi = \frac{i}{N + 1}$ in which N is the total number of yearly maximums that is available, and i is the sequence order number of the data, arranged in magnitude.

Example:

For ten consecutive years, the maximum recorded levels are as shown in column (1) and (2) of Table III - 3.1. In column (3) these maximum are ordered from smallest value to highest value. In column (4) the plotting positions is calculated using $\varphi = \frac{i}{N + 1}$; φ is multiplied by 100 to yield percentages. On the Gumbel paper the maximum values of column (3) are plotted

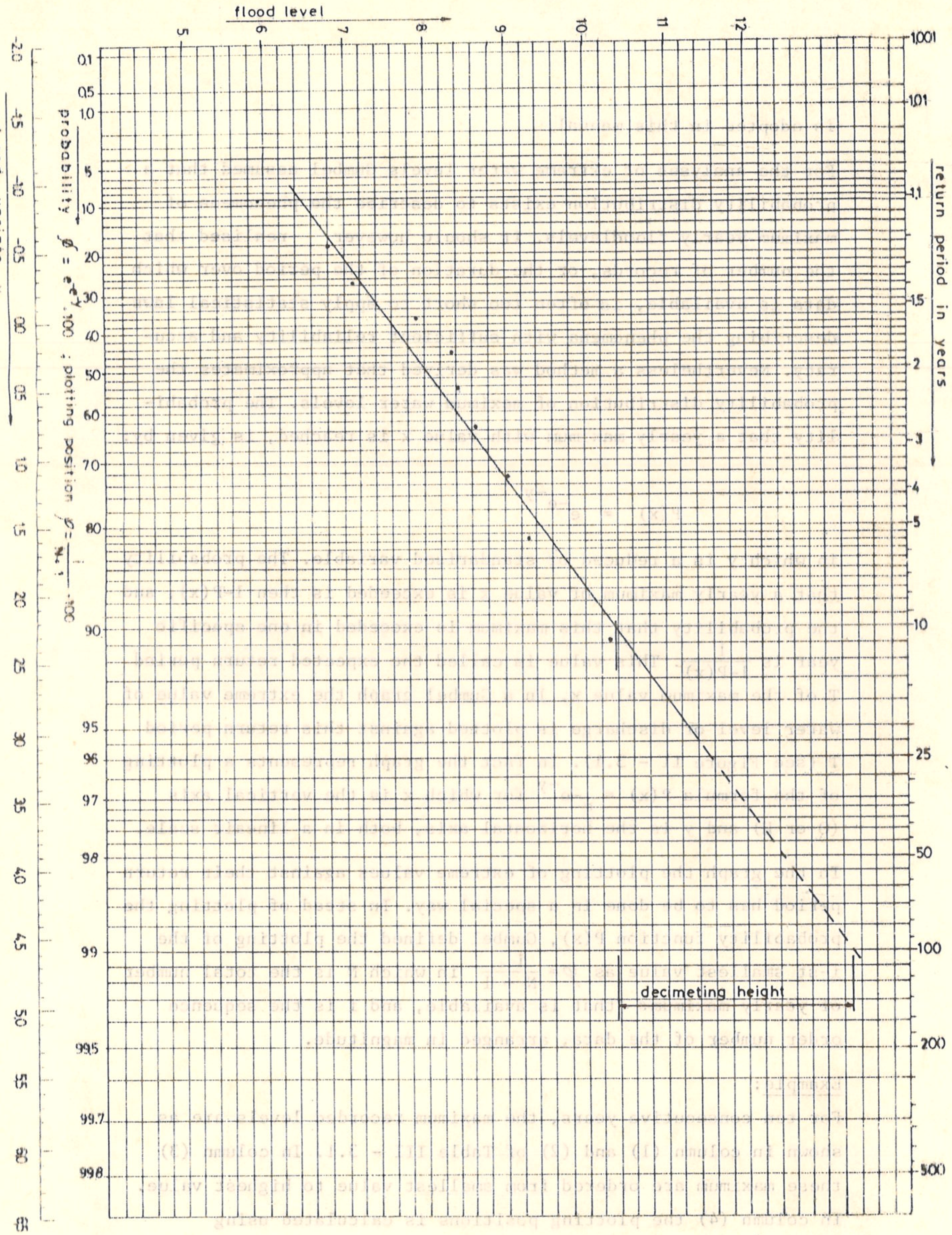


Figure III - 3.1 Gumbel graph.

against the values of column (4). (see Figure III - 3.1). Through the plots a strait line can be drawn by linear regression. From the graph the recurrence interval of any desired floodlevel can then be approximated.

Table IV - 3.1 Plotting positions of maximum floodlevels

	Year	H	H(ordered)	$\varphi = \frac{i}{N+1} \times 100$
column no.	(1)	(2)	(3)	(4)
	61	8.41	5.95	9.1
	62	9.32	6.80	18.2
	63	6.80	7.12	27.3
	64	7.91	7.91	36.4
	65	9.01	8.37	45.5
	66	10.32	8.41	54.5
	67	5.95	8.64	63.6
	68	8.37	9.01	72.7
	69	8.64	9.32	81.8
	70	7.12	10.32	90.9

In the report prepared by International Engineering Consultants IECO, December 1980, for the South West Regional Plan, frequency analyses of maximum flood levels of 46 gauging stations are presented. From these analysis it can be concluded that the increase in maximum flood level, in case a higher return period is chosen. (i.e. once per 100 year in stead of once per 10 year) is ranging from 0.3 m upto 1.0 m. This extra height is called the decimating height (see Figure III - 3.1) and is defined as the increase in floodlevel resulting in a increase of return period with a factor 10.

In view of the rather low decimating height, one is inclined to choose the higher returnperiod (once in 100 year) since additional height of embankment to gain additional safety against flooding is relatively low. However if a gauging station has only 20 years of water level records, these data may not be extrapolated to arrive at a floodlevel of once per 100 year occurance. Therefore a return period of design floods chosen of 20 years.

For areas with significant development investments, this returnperiod is to be taken once in 50 years.

3.3 Wind setup

If wind is blowing long enough from one direction over a significant water surface, a rise of the water level will result at the down-wind side. This wind setup can be calculated as follows.

The force that the wind is exerting on the watersurface is expressed as :

$W = p.F.w^2$, in which p = constant factor
F = area of water affected by the wind
w = wind velocity (m/s) at 6 m above water level.

The wind setup will cause a slope of the water level and due to this slope, water will flow back along the bottom. This return current exerts a friction force equalling (see Figure III - 3.2) ;

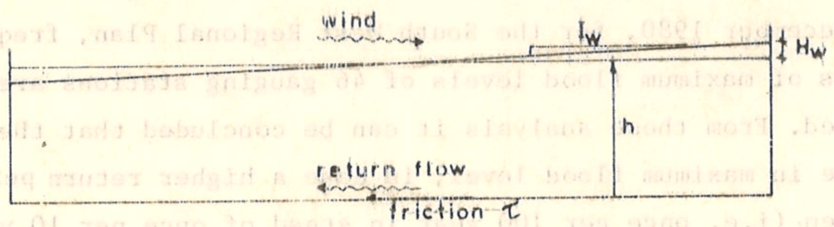


Figure III - 3.2 Wind setup

$\tau = \rho.g.h.I \times F \times q$, in which ρ = density of water
g = gravity acceleration
h = average waterdepth
I = slope
F = affected area
q = constant factor

In the equilibrium stage force W equals force τ and thus.

$$p.F.w^2 = \rho g\ h\ \ I.\ F.\ q$$

and $I = \frac{p}{\rho q} \cdot \frac{w^2}{gh}$ in which $\frac{p}{\rho q}$ is a dimensionless constant with value 4×10^{-6}

The wind setup gradient can thus be expressed as :

$$I_w = 4 \times 10^{-6} \times \frac{w^2}{gh}$$

The general formula for calculating the wind setup is then :

$$H_w = \frac{4\text{x}10^{-6} \times w^2 \times l \times \cos}{gh} \varphi$$

in which : w = wind velocity in (m/s) at 6 m above water level
l = length of water area over which the wind is blowing
g = specific gravity
h = average waterdepth along stretch l.
φ = angle at which the wind is approaching the coast.

Example :

$w = 40$ m/s
$h = 3$ m

$$I_w = 4 \times 10^{-6} \times \frac{1600}{9.8 \times 3} = 0.73 \times 10^{-4}$$

If the wind is acting over a length of 5000 m then the wind setup can be estimated to be :

$$H_w = 5000 \times 0.73 \times 10^{-4} = 0.36 \text{ m.}$$

If a restricted basin is considered, there will be wind draw-down at the lee-ward side of the basin and the fetch has to be taken half. For open locations with unrestricted supply of water, the full fetch-length has to be taken.

For bigger waterdepths the bottom return flow generated by the wind can develop better and consequently the wind setup will be less. In shallow waters the wind setup can be considerable. Wind setup however will only develop after some time, i.e. during wind conditions that last for at least 24 hours. Short storms of less duration hardly generate any significant wind setup.

The effect of wind setup may be included in the design calculations in case the water level data are from a temporary gauge or a gauge with restricted (< 10 years) records. Water level data from permanent gauge stations with many years recordings already include any effects of wind setup.

3.4 Wave run-up

With respect to embankment design, two aspects of waves have to be considered :

- the forces of breaking waves against the slope of an embankment, causing erosion if no protection is provided.
- the run-up of waves against a slope, which might cause over-topping of the embankment if the crest level is not high enough.

The protection of slopes against erosive forces of waves is discussed in paragraph 5.5. Wave run up will be dealt with in this paragraph.

Waves, generated by wind blowing over water, are increasing in height, if the fetch is increasing. A relation between fetch (F) wind velocity (w), wind duration (t), wave height (H) and wave period (T) is presented in Figure III - 3.3.

Example :

If : F = 5000 m	then :	H = 1.2 m
w = 25 m/s		T ≈ 4 sec.
t = 6 hours		

It should be noted that either the fetch (F) or the duration (t) are the limiting factor for the wave height and wave period. In the case of the example, where the fetch of 5000 m is the limiting factor, it can be seen that after 40 minutes the waves of 1.2 m have developed.

If the wind continuous for another 6 hours ~~this will not influ-~~ence the wave height. However if fetch is increased to 100 km, then after 6 hours the waves have reached a height of 6 m !

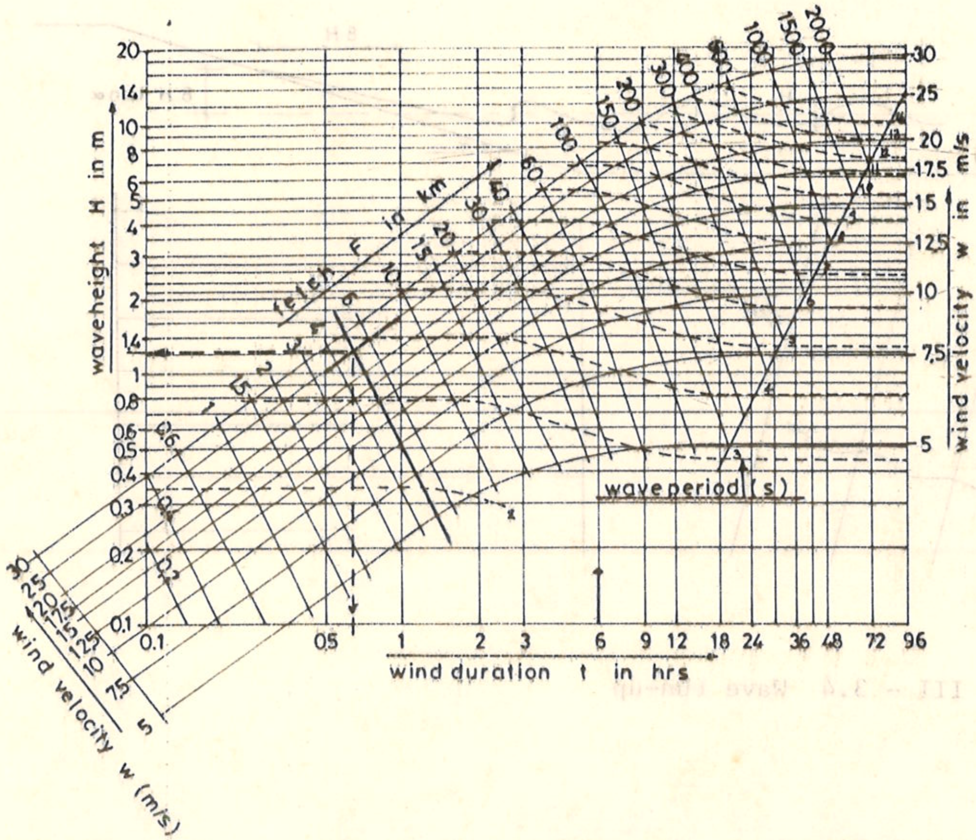


Figure III - 3.3 Relation between wind velocity, fetch and wave height.

Waves generated by the wind will proceed in the direction of the wind and will eventually approach the slope of an embankment where they will break. Water will be jetted against the slope by the influx of the breaking wave. The general expression for this wave run-up is (see Figure III - 3.4):

$$H_z = 8.\ f.\ H.\tan\alpha \cdot \sin\beta \cdot (1 - \frac{B}{L}) \qquad [m]$$

in which
- f = constant factor
- H = wave height
- α = slope of embankment
- β = direction of incedent waves
- B = berm width
- L = wave length

<u>Note</u>: factor $(1 - \frac{B}{L})$ may be omitted if no berm is applied.

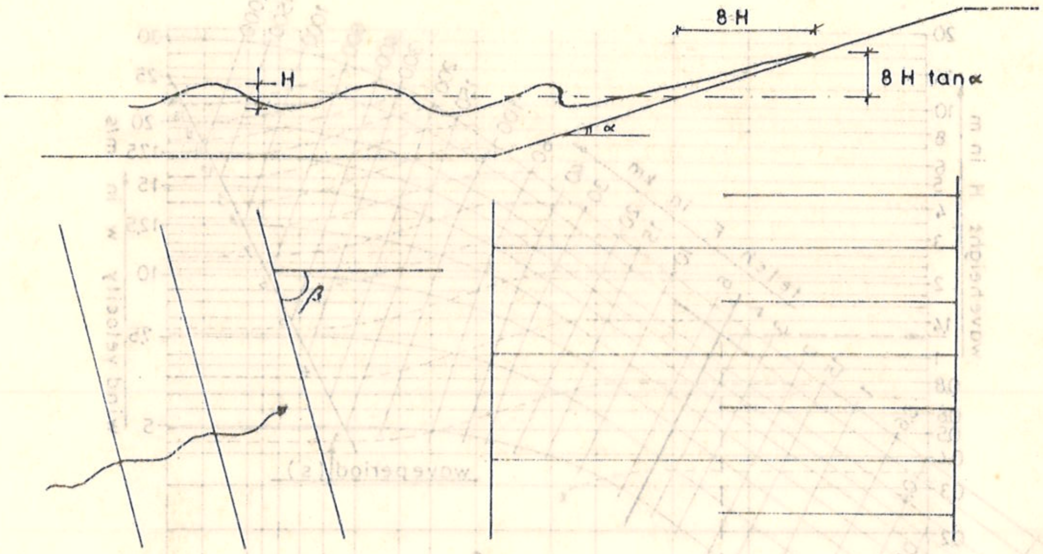


Figure III - 3.4 Wave run-up

The factor f depends on the smoothness of the slope surface and varies from 0.75 for very rough rip-rap, to 1.25 for very smooth asphalt-concrete slopes. For turfed slopes f = 1.1 can be assumed.

Furthermore it should be noted that the formula is only valid for slopes with $1/8 < \tan\alpha < 1/3$.

For the determination of the design waveheight (H), Figure III - 3.3 can be used, but if detailed wave information is available the significant wave height (H_s) is to be used. H_s is defined as being the average of the highest third of all waves in a certain wavespectrum. H_s is exceeded by 13.4% of the waves.

Application of a berm in the embankment is only to be considered for sea dikes or other dikes that are extremely exposed to high waves. For small waves the beneficiary effect of a berm is relatively small and application is not economically justified.

Closure dams are designed with a berm at both sides for increased stability and extra safety with respect to erosion.

Wave attack on an embankment can also effectively be reduced by designing the alignment in such a way that a relatively large fore-land remains in front of the embankment. Higher waves will break in this area (when flooded) and waves will be reduced before they reach the slope of the dike.

Also the topography of the river bottom or sea bed is of importance in this respect. Shallow fore shore not only can force waves to break but also can cause refraction of waves. Refraction of waves is caused by the phenomena that wave propagation velocity is smaller in shallow water than in deep water. If a wave front is not running paralled towards parallel depth contours, the wave frontwill bent and wave heights will change. Another phenomena is the diffraction of waves which occurs if waves are obstructed by a protruding land tongue, groyne, harbor mole, ferry dike and the like. In the "shadow area" behind the obstruction waves will spread in a certain pattern and wave heights will be influenced. For more details and calculations of increased/ decreased wave heights, reference is made to :

"Shore Protection Manual"
U.S. Army Coastal Engineering Research Centre
U.S. Government Printing Office, Washington D.C.

3.5 Shrinkage and Settlements

It is to be estimated how much the design crest level should be raised to allow for shrinkage and settlements. Experience has learned that shrinkage of a fresh embankment averages 10% of the maximum fill height. So, if the ultimate crestlevel (= design crestlevel) should be 4.0 m above the original groundlevel, the height of fill immediately after construction should be 4.40 m.

In non-organic subsoils, settlements of the subsoil will be of a lower order of magnitude than this 10% and are therefore considered to be incorporated in this 10% allowance. Minimum allowance should be 0.20 m. In case of organic subsoil layers, a settlement of these layers of 50% has to be considered and should be added to the 10% allowance for fill-shrinkage. If the compressible layer is composed partly of peaty soils, than the percentage of 50% may be reduced proportionally. Peat should not be used for embankment construction.

example : embankment 3.60 m above present groundlevel, with a well defined peat layer of 1.2 m thickness in the subsoil.

Total allowance for shrinkage and settlement should be :

$(0.1 \times 3.60) + (0.5 \times 1.2) = 0.96$ m.

Chapter 4. Crest Width

4.1 New embankments

Minimum crest width for embankment should be 2.5 m.

Smaller crest widths will make the embankment more vulnarable in case of overtopping which will cause erosion of inner slope and crest.

For the determination of an appropriate minimum crest width of embankments the following empirical rule can be used :

$H \leqslant 2$ m	$B = 2.50$ m
$H \geqslant 5$ m	$B = 6.$m
$2 < H < 5$ m	B is chosen at a proportionate size in between 2.5 and 6 m, rounded of to the nearest 0.5 m.

in which :

$H = H_{DES}$ minus existing groundlevel,or

$H = H_{DES}$ minus berm level (if available).

In case inspection of the embankment is to be executed with a vehicle (four wheel drive, off road), the minimum crestwith should be 4 m.

If a metalled road is to be constructed following the same alignment as the embankment, it is to be considered whether this road should be on crest level or that a berm of the c/s is to be made to accommodate the road.

For a road at crest level a road-berm has to be provided of minimum 1.5 m width at both sides of the road. If the embankment is rather high, then it might be more economic to reduce crest width and construct an extra c/s berm at 0.5 m above maximum spring tide level, (H.W.S.) (see Volume V) thus reducing earthwork. In case of embankment failure and inundation of the protected area, the road still would be above water level after the flood has passed and water level in the polder is following the tidal movement.

The matter is illustrated in Figure III - 4.1.

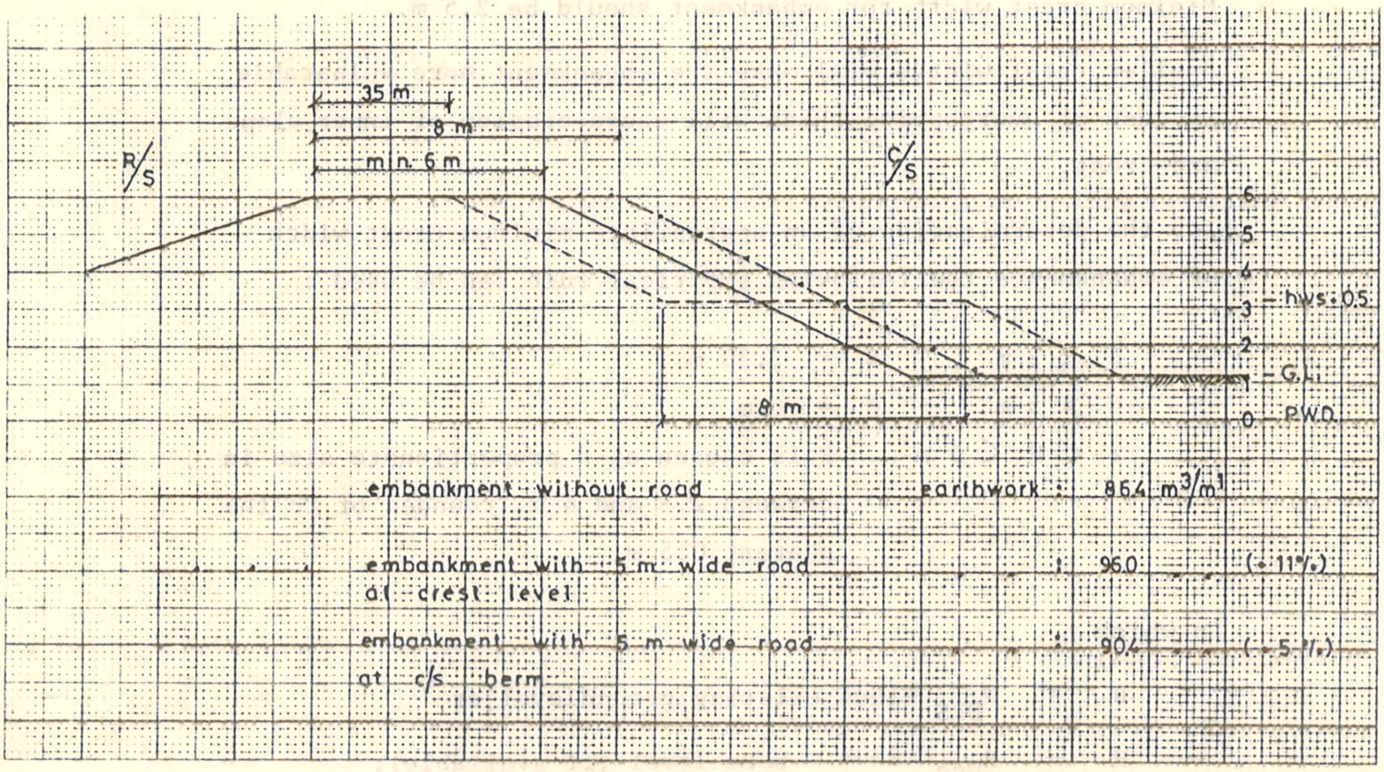


Figure III - 4.1 Crest width of embankment with road.

4.2 Embankment Rehabilitation

For repair and/or rehabilitation work on existing embankments the availability of sufficient earth at the r/s might be a problem which only is to be solved by acquiring additional land at the c/s.

In order to reduce earthwork an alternative design for the cross-section of the embankments as shown in Figure III - 4.2 can be applied which might save upto 60% of earthwork. Minimum dimensions to be maintained are mentioned in the figure. The reasoning is that the additional 1 m free board, applicable to embankments, is only water retaining in exceptional cases. Therefore the crest bund can be of dimensions as shown: "Whether the application of split level crests on embankments will be introduced on a larger scale is a matter of evaluating construction costs and maintenance costs to keep the embankment at design dimension. The experiences in Polder 22, where the crest bund was introduced on an experimental basis, indicate that maintenance of this type of embankment is not likely to costs more than the maintenance of conventional embankments.

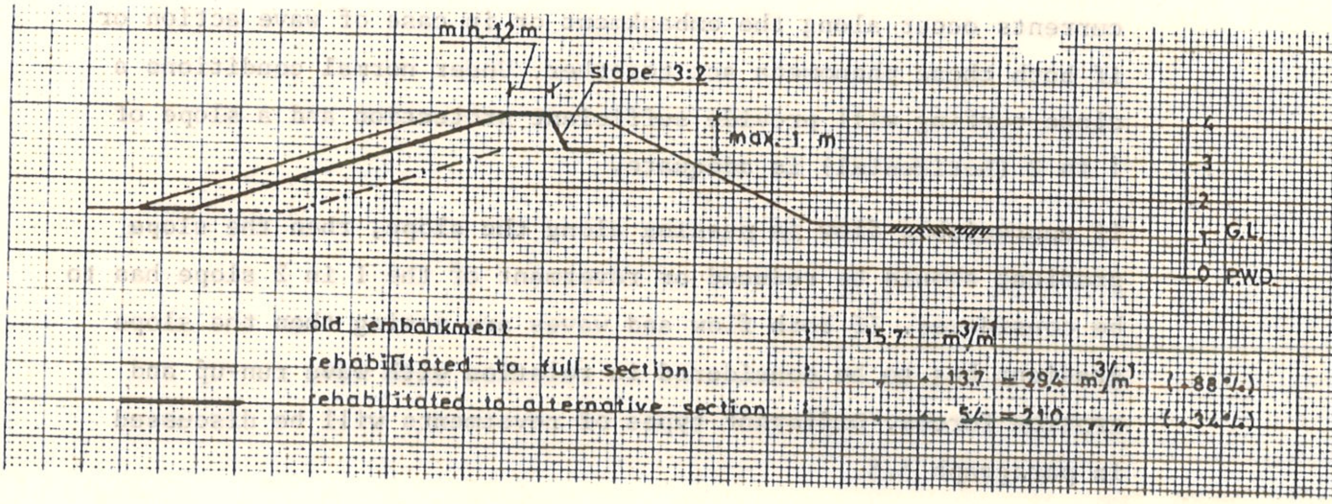


Figure III - 4.2 Embankment rehabilitation with application of crest bund.

Chapter 5. Slopes

5.1 River Embankment

Figure III - 5.1 shows a typical cross-section of a river embankment. The r/s slope is steeper than the c/s slope. The slope at r/s is designed as steep as the stability of the earth body allows, in order to reduce cost of revetment if necessary. Artificial revetment only will be necessary if at flood stages high currents occur along the embankment or in case of wave action or if both these phenomena act together. Under normal conditions a close turfing will provide sufficient protection and a slope of 1 in 2 (horizontal) is adequate.

If appreciable flow is passing along the slope, then the slope gradient should be reduced or revetment of the 1 in 2 slope has to be considered. If both flow and waves are acting then the slope gradient has to be considered in relation with wave run-up and erosion forces. Different types of revetments will be discussed in paragraph 5.5.

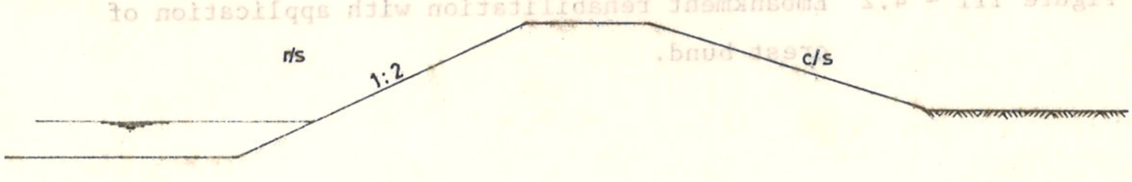


Figure III - 5.1 Typical section of river embankment.

The c/s slope of a river embankment has to be as flat as to prevent damage caused by seepage. Seepage flow through embankments can be checked with the procedure indicated in Volume VI - Foundation Design, Chapter 2. Generally a c/s slope of 1 into 3 (horizontal) will be adequate but in case of sandy material this might have to be reduced.

5.2 Sea dyke

Sea dykes, do not have to retain floodlevels as long as river embankments. Therefore the c/s slope does not require seepage control calculations.

For stability reasons and in order to reduce damage in case of over topping, the c/s slope is taken at 1 in to 3 (horizontal). The r/s slope has to be designed considering wave-action, wave run-up, wind setup and the features of the fore shore. Generally it will be flatter than the c/s slope (see Figure III - 5.2).

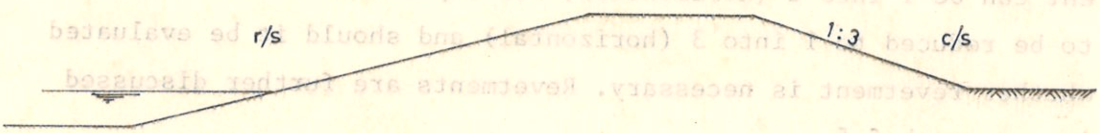


Figure III - 5.2 Typical section of sea dyke.

If a considerable fore land and/or shallow depths occur in front of the embankment, then wave height might only be moderate and sufficient protection could be provided by a close turfing on a flat (1:5 or 1:7) slope, depending on wave breaking facilities like afforestation or special groynes. In case of absence of fore land or any other protective facilities, the slope has to be protected rigorously implying high cost of materials and construction. In the latter case the r/s slope is made as steep as possible to reduce these costs, but additional berms may be necessary to reduce wave run-up.

Revetments for a seadyke are discussed further in paragraph 5.5.

5.3 Embankment in the delta area

In the delta area the character of the water to be retained has a seasonal and a tidal variation. The more land inward, the more the seasonal character will prevail, while towards the sea the tidal character will become predominant. Moreover, in the delta area the seasonal differences in water level will reduce because the water is spreading over numerous channels and creeks. Consequently the head of water to be retained by embankments in the delta area during flood conditions is less and seepage flow will thus be reduced. It is therefore adopted that in areas where the seasonal variation is less than 2.0 m, the c/s slope of embankments in the delta area can be attained at 1 into 2 (horizontal).

The r/s slope of embankments in the delta area depend fully on the mode of attack by waves and/or currents. For unexposed embankments along minor rivers or branches the r/s slope gradient can be 1 into 2 (horizontal), for exposed locations this has to be reduced to 1 into 3 (horizontal) and should it be evaluated whether revetment is necessary. Revetments are further discussed in paragraph 5.5.

On the drawing of Annex III - 1 it is shown which type of embankment is to be considered in case of several modes of attack, distinguishing riverine and sea-environments, currents and/or waves.

5.4 Slope Stability

Experience has indicated that failure of slopes in cohesive soils often takes place along circular slide-planes and trials have indicated that for slopes flatter than 53°, the critical sliding circle is generally tangent to a firm layer and is cutting the surface beyond the toe of the slope. Furthermore the centre of the sliding circle is laying on the vertical line through the middle of the slope. (see Figure III - 5.3).

This is illustrated if the slope is displaced Δx to the left or the right. The weight of the sliding earth mass is then resp. increased or decreased. However in both cases the momentum around the sliding circle centre is decreased. Thus it is shown that momentum is maximum if the centre of the sliding circle is above the middle of the slope.

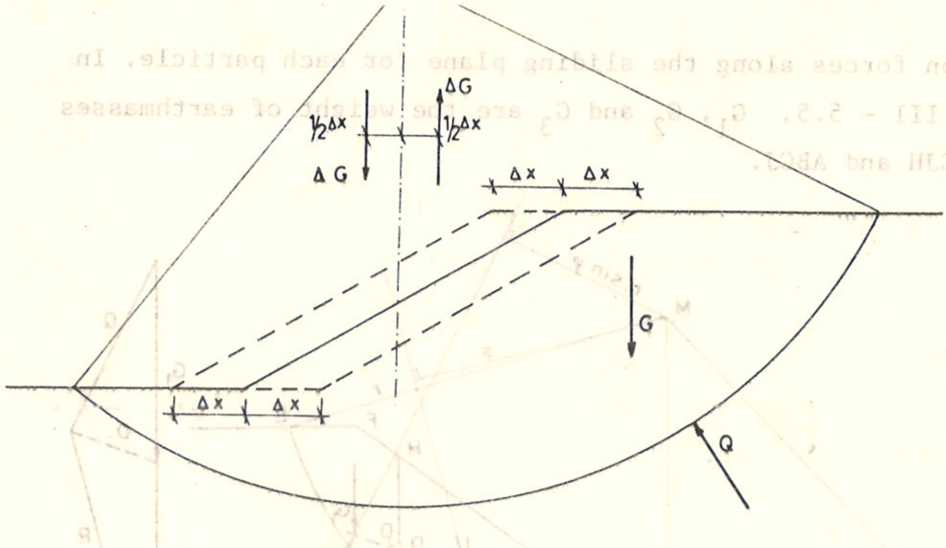


Figure III - 5.3 Mechanism of subsidence of embankments.

If an angle of internal friction (φ) is introduced, it can be approximated that the centre of the sliding circle is at distance (r. sinφ) from the vertical through the middle of the slope (see Figure III - 5.4).

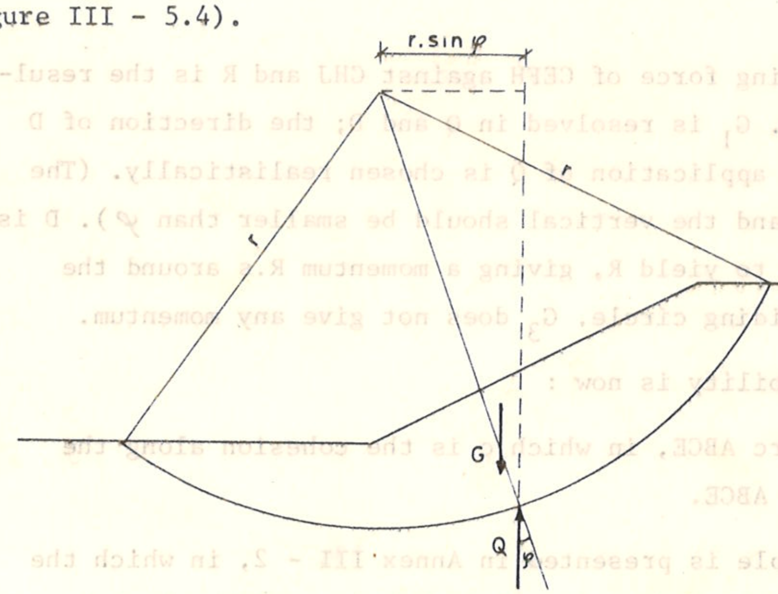


Figure III - 5.4 Effect of internal friction on sliding mechanism.

To calculate the stability, the sliding earthmass is divided into particles by drawing vertical lines at every point along the sliding plane where soil properties are changing. Stability can be checked graphically or analytically by calculating the weight and reaction forces along the sliding plane for each particle. In Figure III - 5.5, G_1, G_2 and G_3 are the weight of earthmasses CEFH, CJH and ABCJ.

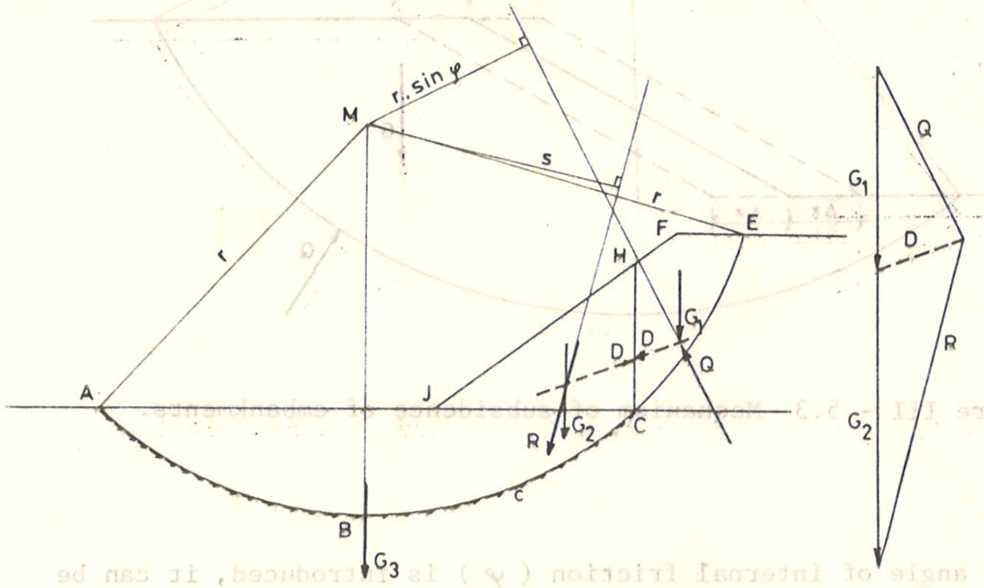


Figure III - 5.5 Principle of stability calculation.

D is the supporting force of CEFH against CHJ and R is the resultant of G_2 and D. G_1 is resolved in Q and D; the direction of D and the point of application of Q is chosen realistically. (The angle between D and the vertical should be smaller than φ). D is composed with G_2 to yield R, giving a momentum R.s around the centre of the sliding circle. G_3 does not give any momentum.

Condition of stability is now :

R.s $<$ c.r. arc ABCE, in which c is the cohesion along the sliding plane ABCE.

A numerical example is presented in Annex III - 2, in which the above method of calculation (Swedish Methods - Fellenius) is applied.

The effects of groundwater can easily be introduced since the unit weight of the soil segments is determined. Effects of groundwater flow are discussed in Volume VI, Chapter 6.3.

5.5 Slope revetments

5.5.1 General

For embankments at non exposed locations, where no significant waves or current is expected to attack the r/s slopes longer than a few hours, a proper turfing of the slopes will provide sufficient protection against serious damage. However, if wave and current attack are appreciable and/or will remain for longer periods, an adequate artificial protection has to be designed. Difference has to be made between that part of the slope which is submerged permanently and the upper part of the slope. The submerged part will mainly be attacked by current. If it is not possible to construct the under water slope protection in the dry the only way is to place current resistant material on the slope from a temporary scaffolding or from a barge.

5.5.2 Under water slope-protection

If the protection layer can be constructed in the dry, the current resistant material has to be applied on a filter construction, to prevent sinking in the existing bottom slope-material. For this filter construction, layers of fine sand-coarse sand-gravel, can be used. Details of such a filter construction are presented in Annex III - 3.

If the under water slope protection cannot be constructed in the dry, then it is very difficult to make a filter construction as indicated in Annex III - 3. The method that should be applied in this case is by sinking a mattress, made of bamboo and reed, with stones, brick-blocks or gunny bags filled with clay, sand-cement-mixture or rip rap. The mattress will then serve as a filterbed. For details of mattress construction and sinking operations, reference is made to the Interim Reports on Closures of Tidal Rivers of the Early Implementation Project (E.I.P.)

To sink a mattress a ballast weight of 120 kg/m^2 has to be applied. After sinking, this should be increased to 300 kg/m^2 for extra stability.

The mattresses have to be anchored on a berm of 2 m width, located at the lowest water level. In tidal areas this is taken at Low Water Spring tide level (L.W.S.). The anchoring consist of a row of bullah-pegs, placed at every crossing of the bamboo frame of the mattress.

Figure III - 5.6 illustrates the construction.

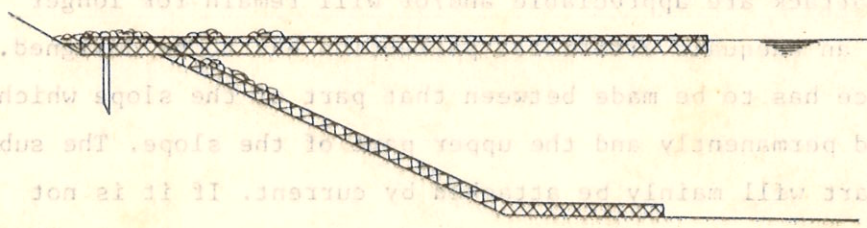


Figure III - 5.6 Shore protection with mattress.

To prevent the protection material from rolling down the slope, the slope gradient should not be steeper than 1:2½ (horizontal). The toe of the protection layer, whether made in the dry, or via mattress-sinking, should be placed at a level 1 m below the expected natural depth of the river along the embankment.

The size and weight of a ballast-unit that has to be applied in order to resist the current, can be determined from Figure III - 5.7 and 5.8. The diagram is based on the formula :

$$d \geqslant \frac{\alpha}{\Delta} \cdot \frac{v^2}{2g}$$, in which:

d = diametre of stone or unit

Δ = relative density of stone or unit

v = velocity

g = gravity acceleration

α = coefficient *)

*) U.S. Bureau of Reclamation recommends α = 1.4

5.5.3 Above water slope

The part of the slope that is exposed to air and water will be attacked by current and waves. If revetment is necessary, the

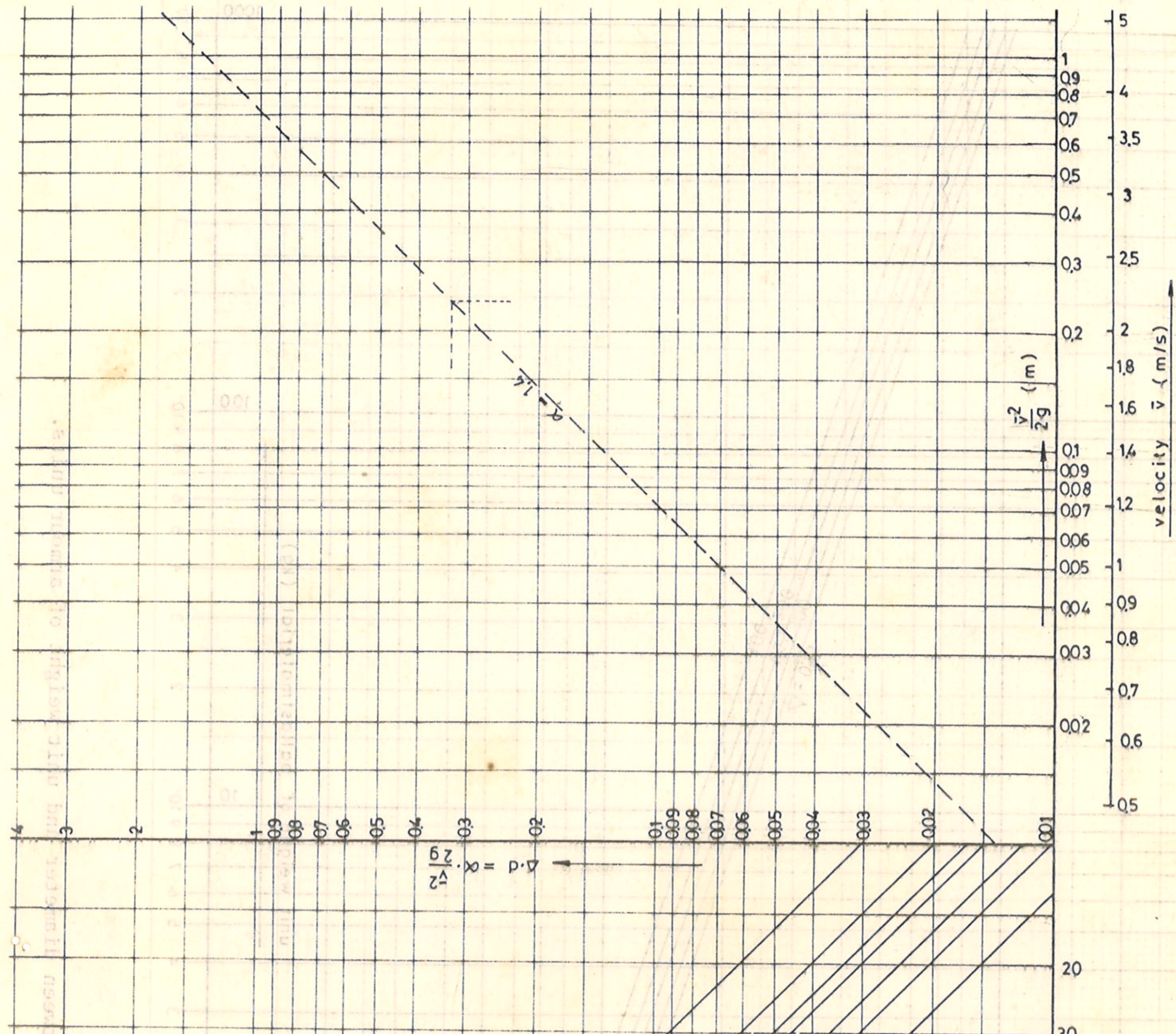


Figure III - 5.7 Relation between current velocity and size of

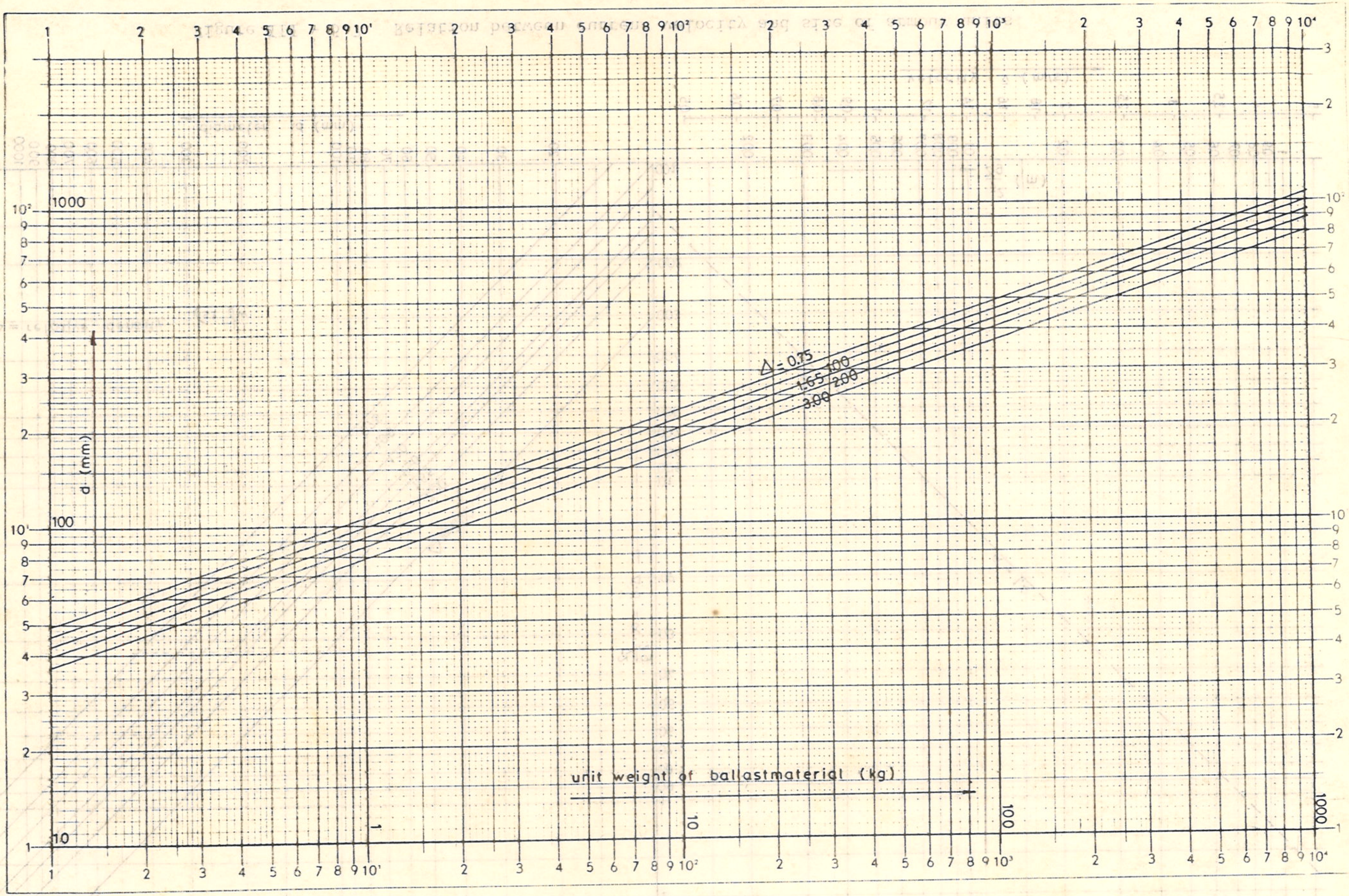


Figure III - 5.8 Relation between diameter and unit weight of armour units.

filter construction can be constructed in the dry and considerable gain in strength is obtained by placing the units as tight together as possible in a pattern. If special attention has to be paid with respect to wave run-up, the protection layer can be designed as a rough surface by placing alternating units "up-right".
In Annex III - 4 several alternative solutions are presented with specific details and application possibilities.

Annex III - 4.1 - strong current attack against dike with sufficient fore land.

Annex III - 4.2 - current and wave attack against dike with no fore land.

Annex III - 4.3 - armour units size and weight for slope revetments and mattress-ballasting.

Whatever revetment for a slope is being designed, some principles always have to be kept.

- every revetted surface needs to be fixed by means of an end stemming construction. At the toe of an embankment this is mostly done with a short sheetpiling of pegs or boards. At the top end, a concrete beam is casted or a small masonry wall is constructed to prevent erosion by rain water, waves and spray flowing down the slope.
- the resistance of ballast material or armour units against waves and current is considerably bigger than indicated in Figure III - 5.7 and 5.8 if the units are placed tight together in a closed pattern. Their lateral support will increase the stability significantly.

With respect to brickblock pitching on a proper filter layer it should be stressed that the best quality (i.e. close pitching) is achieved if the units are easy to handle. Therefore the weight of one unit should not exceed 27 kg. (see Annex III-4.3).

If only protection against flow is to be provided, then this unit weight is sufficiently heavy, even in case it is applied at the outfall of sluices. In case of appreciable wave attack, the armour units weight has to be checked with Irribarren's formula :

$$G \geqslant \frac{H^3 \cdot f \cdot \gamma st}{3\ (\cos\alpha - \sin\alpha)^3\ .\ \Delta^3}$$

in which G = armour-unit weight

H = wave height

γst = unit weight of armour material

Δ = $\gamma st - \gamma w$

α = slope gradient

f = coefficient depending on slope and ratio water-depth/wave length (see Figure III - 5.12).

Example:

if H = 0.6 m

γst = 2,100

α = 1:2½

$\left.\begin{array}{lll} h = \text{waterdepth} & = & 2\text{ m} \\ L = \text{wavelength} & = & 15\text{ m} \end{array}\right\}$ f = 0,032 (from Figure III - 5.12)

then: $G \geqslant \dfrac{0.032 \times 0.6^3 \times 2,100}{1.1^3 \times (0.1729)} = 63$ kg.

However this weight calculation is applicable for dumped stones or units without any lateral support. If brickblocks, placed in a tight pattern are considered, then the blockweight can be reduced considerably.

Annex III - 4.3 shows different modes of application.

On the drawing presented in ANNEX III - 1 a review is presented, showing solutions/principles for several "load" conditions. Details on actual slope gradient to be applied and type of protection to be used are only indicative and should be checked with formula's and figures presented in this Volume.

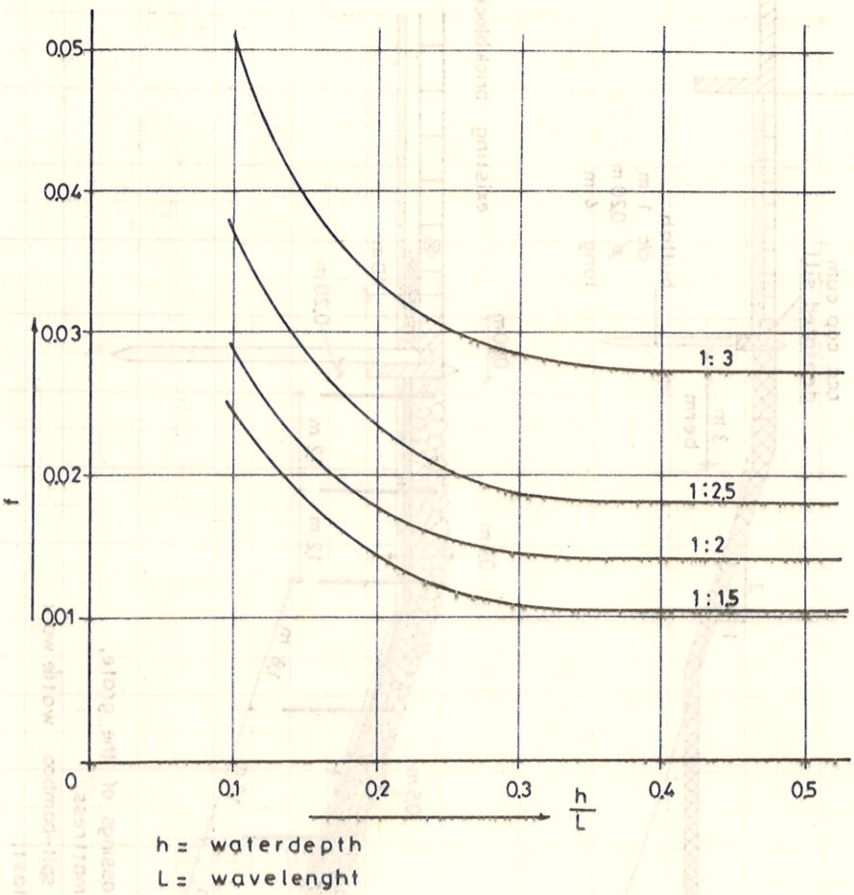


Figure III - 5.9 Irribarren's coefficient f against slope, depth and wavelength.

5.5.4 Slope and bottom protection at drainage sluice

For the brickblock protections as constructed at the drainage/ flushing sluices in the delta area, it is recommended to apply brickblocks with a weight of 27 kg per unit to be placed up right as indicated in Annex III - 4.3. Brickblocks weighing over 80 kg, as are prescribed regularly, can not be lifted easily and are therefore constructed on top of the sloping filterbed, which in most occasions, is thus badly damaged. If construction in the dry is not possible; the method discussed in paragraph 5.5.2, using mattresses can be applied.

This method is recommended in case repair works have to be executed to save a sluice from collapsing into the scouring holes which are developing at places with poor subsoil conditions

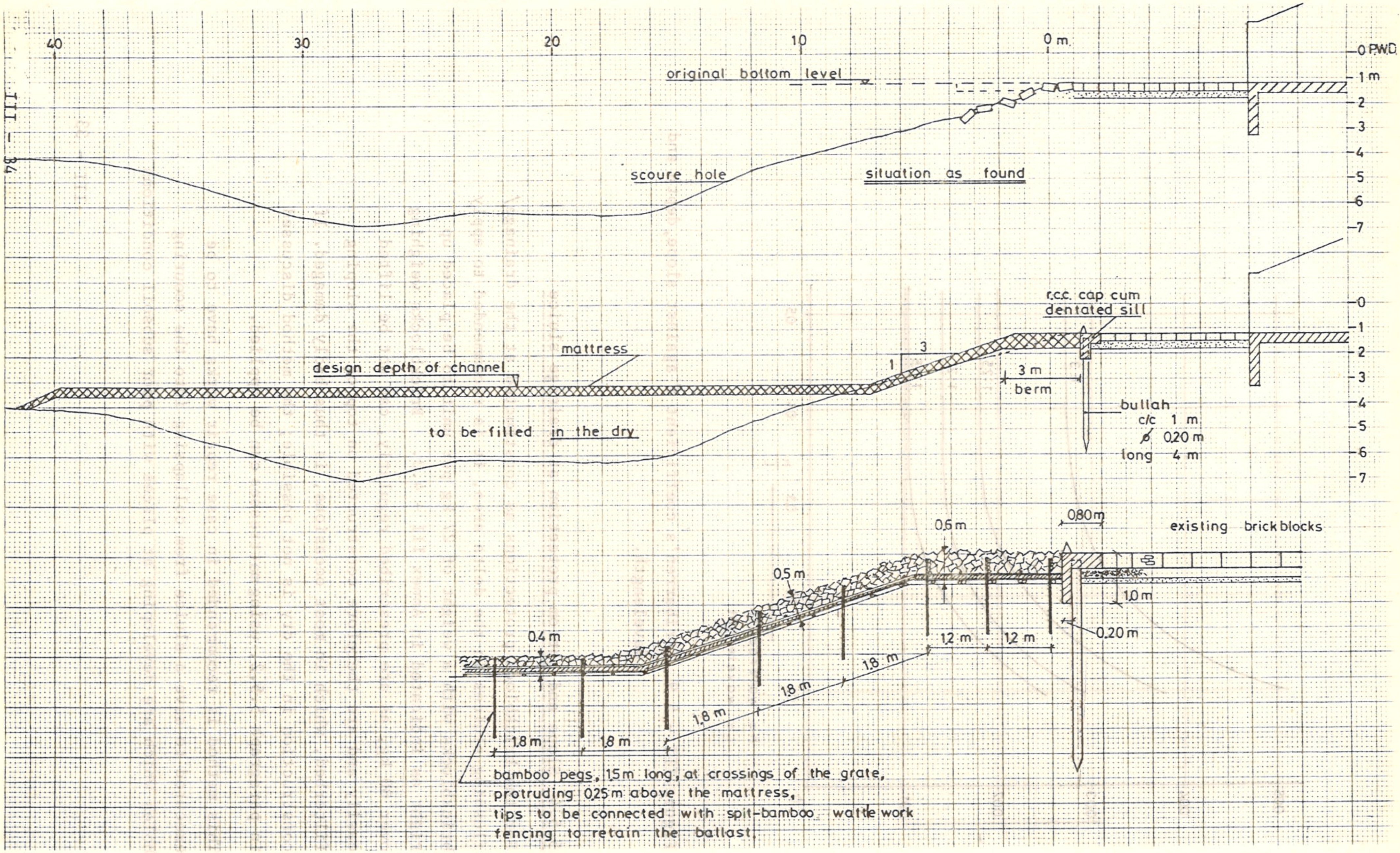


Figure III - 5.10 Repair of bottom protection at drainage/inlet sluice.

and mal-operation of the sluice. In such cases it is necessary to make a flexible ending of the bottom protection by means of a mattress, which can follow any future scour. The mattress construction might even be made in the dry, depending on the condition at the site. Figure III - 5.10 shows an alternative solution for the repair of bottom protection works at drainage/flushing regulators.

Chapter 6. Setback, borrow pits

To provide a safety against erosion of unprotected embankments by degradating river banks the alignment is to made at a certain distance from the actual riverbank or shore line.

The other reason for the set back is the location of the borrow pits. The borrow pits have to be made at the r/s so that the least valuable land is used. An alternative could be in case the farmers are interested, to strip a thin layer of earth from the c/s land if this is high enough. In this way the land would become easier to irrigate.

Borrowpits should be excavated in such a way that a berm is left between the toe of embankment and the edge of the excavation. The berm width should be 10 m for river embankments and 20 m for sea dykes. The depth of the borrowpits should not exceed 2.0 m below ground level.

Between excavation and actual riverbank a berm of 6 m has to be left. Cross berms, perpendicular to the embankment, have to be left in the excavation every 30 metre measured along the embankment. These berms have to have a width of 6 meter and are to prevent the development of flow concentration during flood conditions.

At locations, near villages or otherwise valuable land, where appropriate set back can not be maintained and the dike has to be constructed at the existing shoreline, extra protection measures for under and abovewater slope and excavation pits at the country side, have to be considered.

Irrespective of the borrowpits, the minimum setback to be applied is:

for river and delta area embankments	:	50 m
for sea dykes	:	100 m

Figure III - 6.1 illustrates the set-back to be applied.

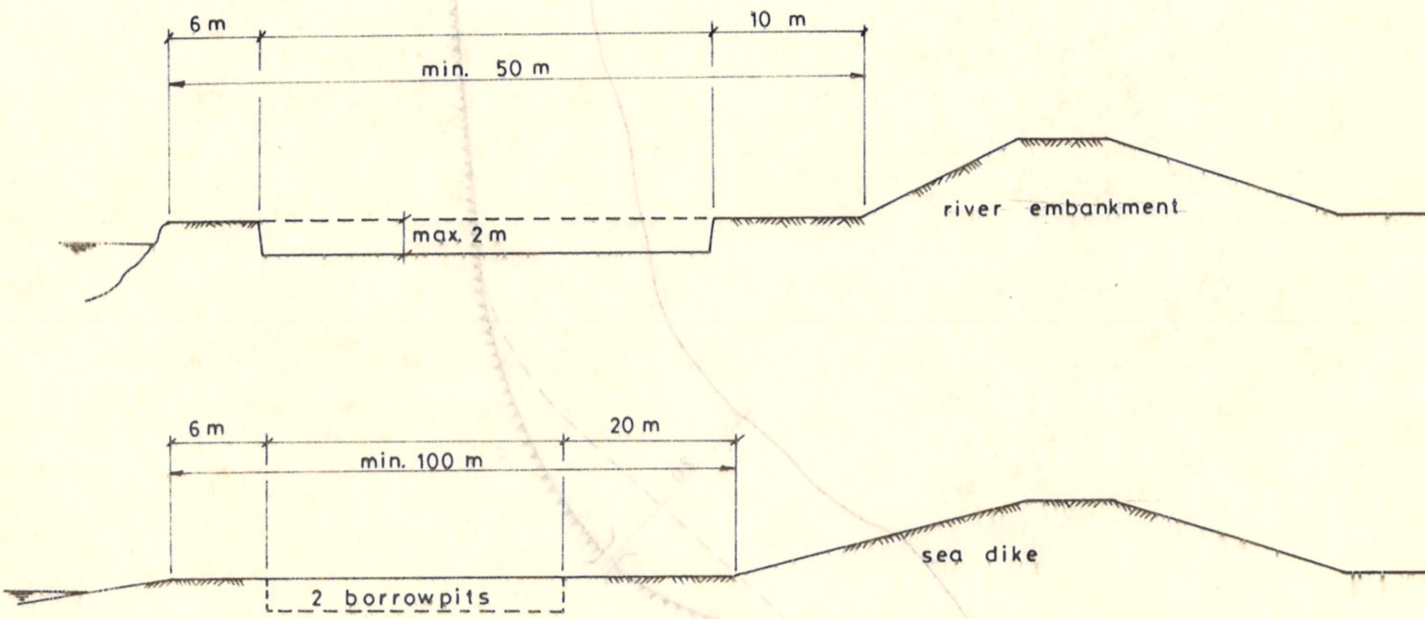


Figure III - 6.1 Set back for embankments and sea dykes.

In case of exposed locations or if erosion of the fore land is acute, the set back is to be increased accordingly. An extra margin allowing for 10 years erosion at the average occuring rate should be made.

At locations where a watercontrol structure is to be build, the setback should be increased. If erosion is going to progress at such a rate that the embankment will be affected, at least the costly structure will not be damaged immediately and protective measures can be taken. The setback at a water control structure should be at least be 2 x the minimum setback for the adjacent embankments (see Figure III - 6.2).

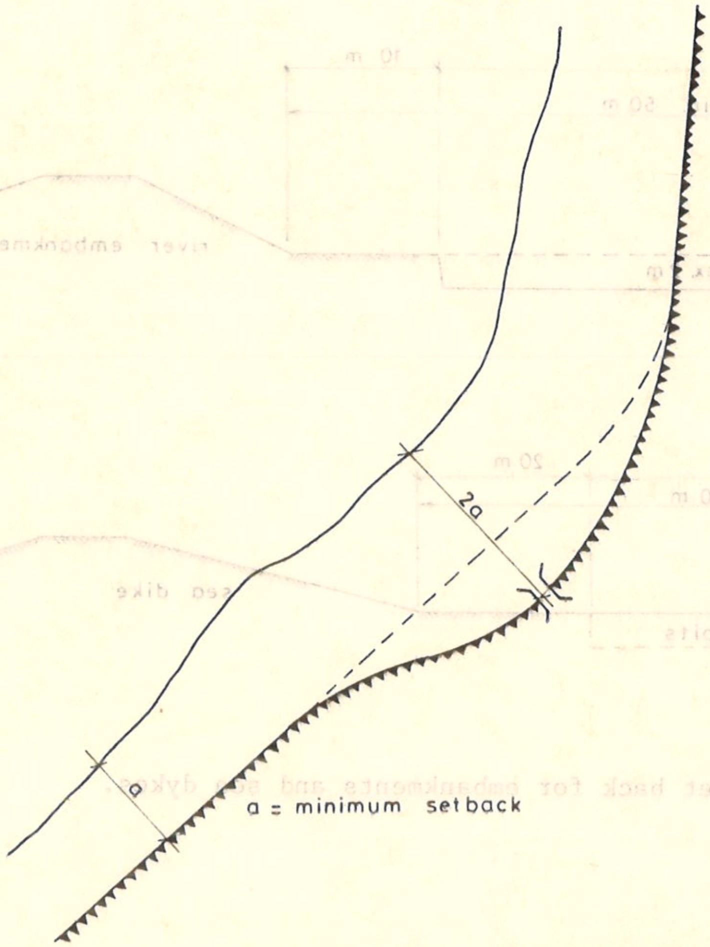


Figure III - 6.2 Additional set back at sluices.

Chapter 7. Embankment repair and rehabilitation

Subsiding of slopes, excessive settlements, erosion by waves and/ or currents or damages due to overtopping and breaches, necessitate repair and/or rehabilitation works. Several possibilities can be considered to upgrade the cross-section upto design standard. In case crest level is still adequate, the stability of the embankment slopes can be improved by making the slopes less steep. This can be done by increasing the base width of the embankment. By reducing the outer slope gradient, automatically wave run-up will be reduced which in its term might decrease required crest height. In this respect outer slope gradient reduction has the same effect as crest height increment.

Figure III - 7.1 (left) demonstrates how crest height can be increased if crestwidth is still sufficient. Seepage through the joint between old and new fill material is to be prevented by excavating and refill part A-B of the old embankment.

In case crestwidth has to be maintained the fill should be placed as indicated in Figure III - 7.1 (right).

Figure III - 7.1 Modes of repairs.

It should be tried, as much as possible, to place any additional fill against the outer slope of an embankment. If placed against the inner slope, waterpressure caused by seepage might press off the new fill. Additional fill against the outer slope will be pressed against the old embankment.

There will however be situations where the extension works cannot be constructed against the outer slope, or only with high costs, i.e. in case old borrow pits are located close to the toe and in case the outer slope is revetted. In such cases the additional fill can be applied at the c/s but due attention should be given to the joint between old and new fill.

Due to overtopping scour holes can develop at the country side of the embankment, and if overtopping lasts long enough, the erosion of the inner slope and the scour holes can completely disrupt the embankment, and a breach will be formed.

In case of such a breach, scour holes may develop to such a size that re-filling requires more material than building a new embankment. A new embankment alignment has to be designed then, to bypass the scour holes.

Whether this alignment is enclosing or excluding the breach depends mainly on available land for construction and borrowpits and provisions for set-back.

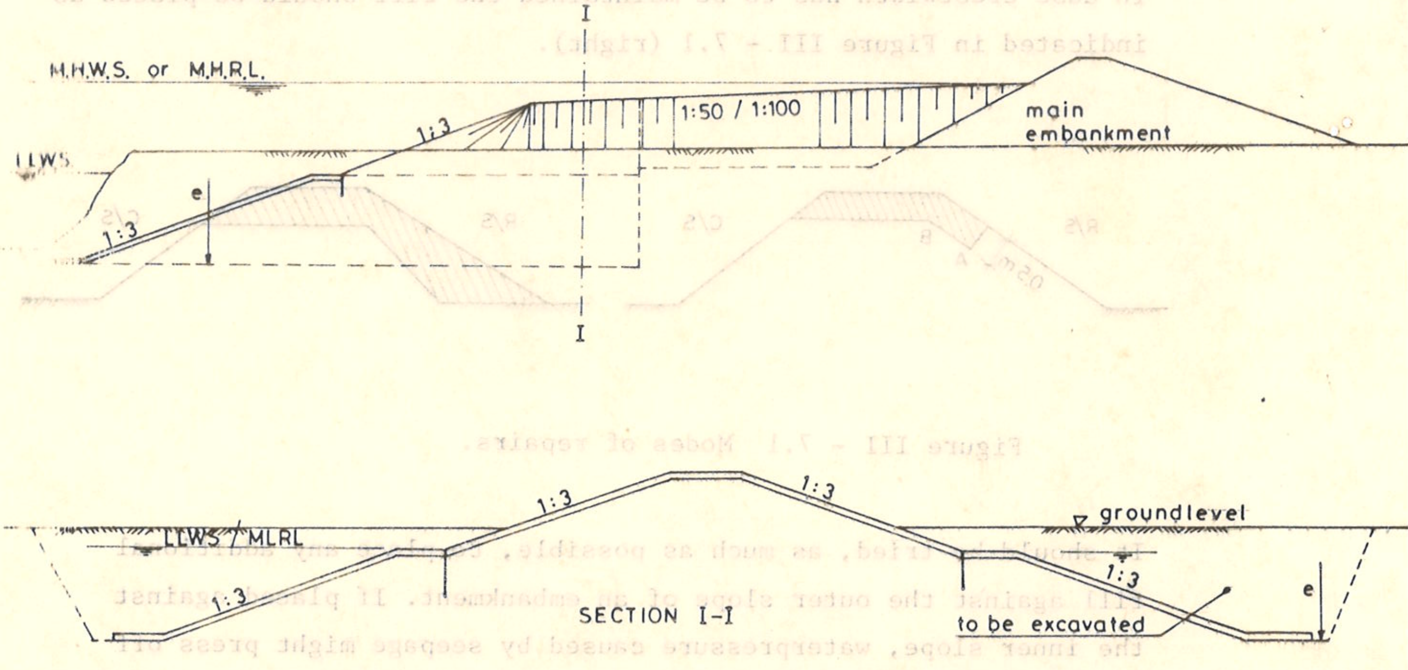


Figure III - 7.2 Groyne.

To prevent that the continued erosion of a riverbank will reach the toe of an embankment, groynes can be build perpendicular to the embankment. Although this is a rather expensive solution because the groynes and specially the groynes tips have to be revetted heavily and to a great depth, the method is effective in keeping the river's main flow in its main bed.

Figure III - 7.2 shows a schematic lay-out of a groyne.

Crest level should be slightly sloping down towards the main stream, in order to prevent flow-concentration during overtopping stages. The average crestlevel should be lower than the design flood level of the embankment.

In tidal areas it is taken at the average yearly high water springtide (M.H.W.S.) (=mean high water springtide).

Slopes on both upstream and downstream side are 1 into 3 (horizontal) and the crest should be revetted to resist the overtopping flow. The under water slope is to be constructed as indicated in paragraph 5.5.2 using mattresses if building in the dry is not possible.

90 35 25 50 15 15

Chapter 8. Closure dams in tidal rivers

8.1 Introduction

In principle there are two ways to construct a closure dam across a tidal channel, either building the dam in horizontal direction, extending the crossdam from one or both banks along the designed alignment, or building the dam in a vertical way, raising crest level gradually along the entire length of the alignment. In the latter case one can distinguish the gradually raising in horizontal layers, or the filling of layers following the bottom profile. In this case the least disturbance of current pattern and bottom configuration is obtained (see Figure III - 8.1).

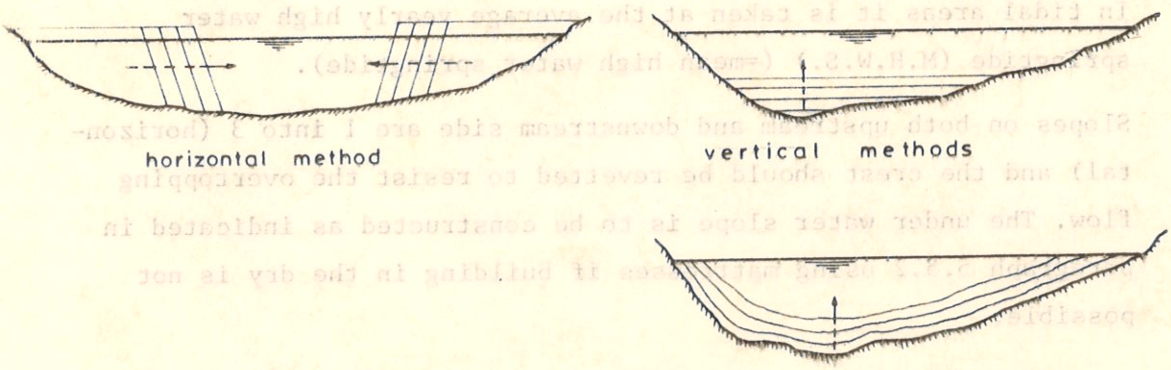


Figure III - 8.1 Closure methods.

Vertical building-up implies the use of floating equipment or temporary scaffoldings or bridge for the execution of the works. Horizontal build-out is less complicated with regard to execution and it is the most logical way to build a dam in stagnant water or in case currents through the final closure gap are very limited. A combination of both methods is also possible; the design problem lies then in the moment at which one has to change over from one method to the other. Which ever method is chosen depends on the conditions and circumstances at the particular closure site, the evaluation of costs, execution possibilities and capacities and the assessment of which method gives the best possibilities for successful completion of the works.

8.2 Site selection

The selection of a site for a closure dam is to be based on the following criteria.

- The future closure dam preferably should be build at a location that is not exposed to waves or winds that might strike on the construction site after a long fetch. Both features will make the operations more difficult.
- The closure site should be in a straight part of the river, where the bottom depth is uniform across the width. A triangular shaped bottom profile as is present in river-bends with waterdepths along one bank much bigger than at the opposite bank, is unfavourable for closing operations. The current will be concentrated in the deeper part, while for closing operations it is aimed to have currents evenly spread across the width of the river.
  The best location therefore can be found just in between two consecutive river bends.
- The availability of good earth in sufficient quantities is another prerequisite. For the core of a closure dam the best earth contains a fair amount of clay. If clay content is less than 25%, the material is less suitable to be used as fill for gunny bags since it will wash out rather quickly. For the earthwork of slopes and final profiling of the dam upto full design section, earth with a higher sand and/or silt content can be used.

8.3 Required data

In order to carry out the necessary hydraulic computations on which basically the choice of closure method and the details for design and construction are based, data has to be collected on :

- Water levels and tides during the period of the year in which the closure dam will be constructed. Preferably a continuous record of an auto-gauge during this period should be available. Daily recording of high and low-water levels and their timings should be available for at least a five years period.

- current velocities; both pattern and distribution over the width of the river and velocity profile from surface to bottom have to be known in detail for all stages of the tide.
- Discharge; tidal discharge measurements and tidal volume computations should be executed for spring tide and neap tide conditions during the time of the year that the closure dam will be constructed. The tidal discharge measurements will provide the information to compute the tidal storage area :

$$V = A \times \Delta h$$

V = tidal volume

A = tidal storage area

Δh = tidal difference.

- meteorological conditions; wind velocities, rainfall have to be known to assess wave action, wind setup, during construction and after completion and to assess disturbances of velocities and discharges due to local rain storms.
- Soil conditions; both of excavation pits and along the proposed alignment borings have to be carried out and samples should be tested. Bottom material of the river should be sampled to assess erodability.
- a semi-detailed hydrographic survey should be executed of the river to be closed, covering the river stretch upto a distance of 5 times the river width measured both upstream and downstream of the proposed closure site.

Especially the tidal discharge determination and the velocities are of vital importance.

From the tidal discharge information and the tide curve, the tidal storage area is calculated, which is an important value in the calculation of the velocities that occur when the tide is passing through a closure-gap or across a closure sill.

The physical conditions at site and the expected maximum velocities fully determine the method of closure and type of materials that can be used.

It will be clear, that the need for protection measures and their dimensions depends on the expected velocities. For this purpose, the expected velocities should be computed for each and every phase of the tide and each phase of the decreasing gap during the closure operations. The result of these computation determines the way of execution and the phasing of the work in relation with the daily increase or decrease of the tide and thus with the phase of the moon. Details of the computations are given in paragraph 8.6.

8.4 Design of closure dam cross-section

In principle the design of crest height, crest width, slopes, and revetments, if required, are the same as mentioned in the previous Chapters of this Volume.

For the under water slopes and the lower parts of the above water dam, some additional remarks are to be made. For loose dumped earth, carried by head load and dumped at the waterline or in the water, it is found that a natural slope of 1 into 4 to 1 into 5 will develop automatically, if the water level is constant.

If the tidal movement is present, this slope will be 1:5 to 1:7, depending on the gradation of the earth.

In designing a closure dam, these slopes should be adhered to. Furthermore a berm is to be provided of 10 m wide at the level of (H.W.S.) (= high water spring) at both sides of the crest. The function of this berm is to provide extra stability of the dam and to serve as extra safety with respect to subsidence of the under water slope, which might occure during the settlement and shrinkage of the dam.

In case the dam is not revetted the slopes above berm level are generally taken 1:3 both for c/s and r/s.

In order to prevent overtopping of the closuredam, the crest height is to be raised 0.5 m above the adjacent embankments. Damage due to overtopping at embankments has less serious consequences and is easier to repair than repairing a breach in a closuredam. Figure III - 8.2 gives a section a closuredam.

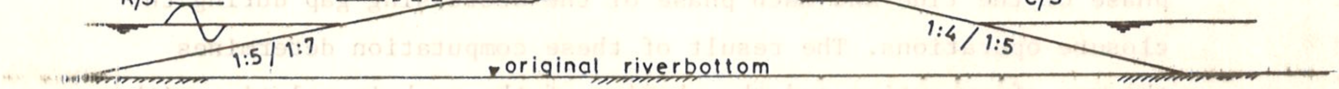


Figure III - 8.2 Typical section of a closure dam.

8.5 Closure methods

8.5.1 Principle of horizontal closure method

In the horizontal closure method, the river will be closed by building out spur-dam(s) from the bank(s). The velocities in the remaining gap will increase gradually as the wet area of the gap is decreasing and will be maximum just before the final closing. (see Figure III - 8.3).

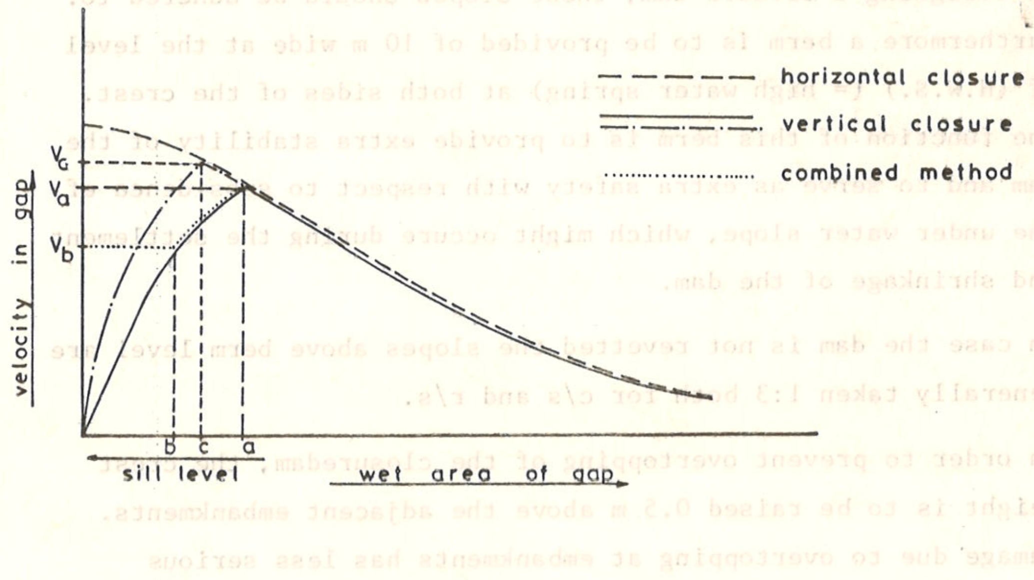


Figure III - 8.3 Velocity characteristics during closures.

The works start with the construction of a bottom protection at the spot where the final gap will be closed, in order to prevent scouring as a result of the heavy current in the remaining gap. Next, construction starts from both banks of the river by building up the dam in the direction of the final gap.

During construction the gap becomes smaller and consequently velocities are increasing, a head difference is introduced, discharge capacity will be reduced and the tide in the area to be closed off will fall behind in phase and amplitude in comparison with the tide outside.

From a certain moment onwards the velocities have reached such values that heavier building materials must be chosen for continued dam construction in order to prevent a high loss of material. With these steadily increasing velocities, also the bottom will be scoured, if no protection has been applied before. It should be computed in advance, what value these extreme velocities will reach in order to determine the size and type of bottom protection required.

The horizontal closing method is characterized by the fact, that the extreme velocities in the closure gap have no limit : they keep increasing until the final gap has been closed.

However, the available construction materials can only withstand a certain maximum velocity and thus this velocity is determining the size of the final closure gap. This remaining final gap must then be closed during short period around slack tide when velocities are small.

On the other hand, this final closure gap, which has to be closed at once during a short period at the turn of the tide, is determined by the maximum production capacity of filling the gap during this limited period with acceptable low current velocities.

8.5.2 Principle of vertical closure method

In a vertical closure, a dam is raised gradually over its full length.

The heightening of the dam can be done using sand, sandbags, clay, stones, mattresses, etc., depending on the current velocities. At the start of the works, the watermovement over the submerged dam-crest (the sill) will be a sub-critical flow with moderate velocities.

In proportion with the rising of the dam, the velocities will increase (see Figure III - 8.3) and consequently the choice of materials will be limited to heavier ones. If the heightening of the sill is continued, from a certain moment onwards critical flow conditions will appear, at first during maximum ebb current, later (higher sill level) during maximum flood current too.

When the sill has reached the level which causes critical flow during both ebb and flood current, the maximum velocities will not increase any more. Further heightening of the sill will cause gradual decrease of the maximum velocities. In case the closure would continue by maintaining this sill level but by narrowing the profile, the maximum velocities would remain constant (V_a). The same happens if after heightening the sill up to a level b above its critical value, the rest of the gap is closed horizontally.

In that case during the final stage, the maximum velocities remain constant (V_b) at a value below the critical flow velocity. (see Figure III - 8.3).

The main hydraulic characteristic of vertical closures is the fact, that the highest maximum velocities do occur at transition from sub-critical flow to critical flow.

8.5.3 Combination of horizontal and vertical closure methods

In case of a vertical closure the required sill depth and the required sill length are determined by the tidal volume and the tidal difference in water level during the closure works. The amount of water passing over the sill during the tidal period determines the velocities that will occur.

For wider rivers with a comparatively low tidal volume, computations might indicate a sill which does not necessarily have to cover the whole river width from bank to bank. If the difference in riverwidth and minimum required sill-length is appreciable, it can be considered to apply a combination of horizontal and vertical closure method.

For instance, if the closure has started according to the horizontal closure method, at a certain stage the gap between the two spur-dams cannot be narrowed any further without heavy material losses due to the increasing currents. If at this stage, the gap is still too wide to be closed in one slack tide with the available production capacity, then the closure could be continued with the vertically filling up of the gap upto a sill-level where maximum allowable velocity will occur. Further closure can then continue either using horizontal or vertical method.

In both examples of Figure III - 8.4 a combination of horizontal and vertical closure is applied.

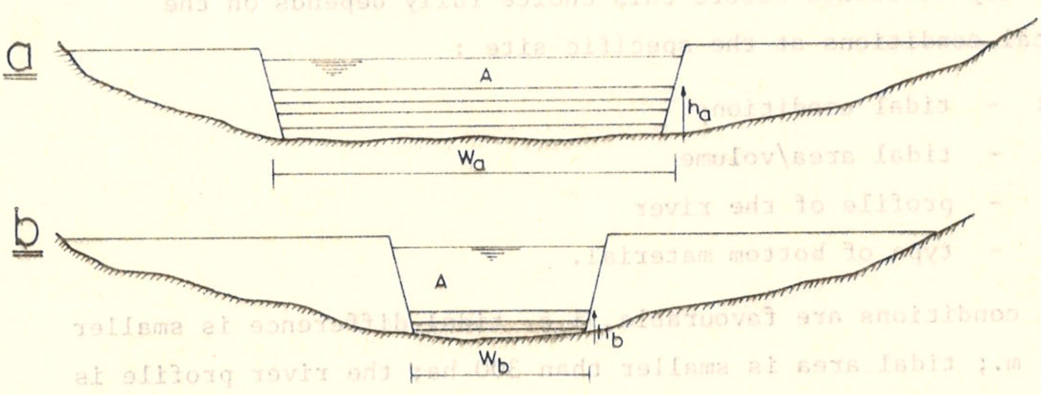


Figure III - 8.4 Combination of horizontal and vertical closure methods.

In Figure III - 8.4(a) the vertical closure starts at a width W_a. When the sill reaches level h_a (remaining wetted area A), critical flow conditions appear, so that further heightening of the dam will reduce the velocities to values lower than V_a. (see Figure III - 8.3).

In Figure III - 8.4(b) with a smaller width of the sill, the profile has a wetted area A before critical flow conditions are reached. In case b the highest occuring maximum velocity V_c is higher than the highest maximum velocity in case a.

The critical flow conditions in a narrow opening appear at a smaller wetted area, thus resulting in higher maximum velocities, if compared with a shallow wide opening.

8.5.4 Choice of closure method

One of the first questions the design engineer has to answer is what closure method has to be adopted for a specific closure.

As already mentioned before this choice fully depends on the physical conditions at the specific site :

i.e. :
- tidal conditions
- tidal area/volume
- profile of the river
- type of bottom material.

If all conditions are favourable, i.e. tidal difference is smaller than 2 m.; tidal area is smaller than 300 ha; the river profile is of a U - type (uniform depth across the width), the width is not too wide (less then 150 m) and the bottom consists of material that can resist a certain increase in flow velocity, then a horizontal closure can be designed.

The area of the final closure gap should be protected against scour by applying bottom mattresses as highlighted in paragraph 8.8.

In case the physical conditions at site are less favourable and especially the tidal volume exceeds $10^7 m^3$ and/or the tidal difference at neap tides is in excess of 1.5 m, and the riverwidth is over 500 m then a final closure according to the vertical method is suggested.

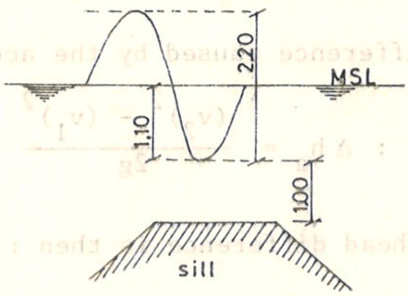
Whether in this case the initial closure works can be according to the horizontal method depends on the calculation of the required sill-width for the final vertical closure.

If it is adopted that the minimum waterdepth on top of the sill should be 0,5-1.00 m, then this sill-width can be calculated from the graph presented as Figure III - 8.13.

Example :

tidal difference at springtide : h = 2.20 m.

assume depth of silt 1 m below L.W.S. : a = 2.10.



in graph : $\frac{a}{h} = 0{,}95$

assume: maximum allowable velocity on top of sill during springtide : $V_{max} = 3.3$ m/s.

$\frac{V_{max}}{h} = 2.22.$

from the graph it follows that during floodtide the maximum value of factor $\gamma = 6$.

$$\gamma = 10^{-3} \times \frac{44700}{46000} \times \frac{5 \times 10^6}{b \times 2.2} = 6 \longrightarrow b = 545 \text{ m.}$$

This means that if the original riverwidth is 500 m, the closure should be carryed out as a vertical one.

8.6 Tidal computations

8.6.1 General physical phenomenas

For both horizontal and vertical closure methods the hydraulic situation at the beginning of the final closure operations, a state of equilibrium is assumed to exist : neither erosion, nor siltation. The gap is considered to be of constant size and the changes of the water level in the gap and the storage basin mainly depend on the changes of the tidal movements in the boundary conditions (Neap Tide, Spring Tide).

During the flood period, water is flowing inward, causing a rise of the water level in the polder section of the river. However the water outside is rising quicker than the water level inside can follow. A head difference will be created over the gap. This head difference consist of :

- head difference caused by resistance: $\Delta h_r = \dfrac{(v_2)^2 \cdot L}{C^2 . h}$

- head difference caused by the acceleration of the flow through the gap : $\Delta h_a = \dfrac{(v_2)^2 - (v_1)^2}{2g}$ (so called Bernoulli head)

The total head difference is then : $\Delta h = \dfrac{(v_2)^2 \cdot L}{C^2 . h} + \dfrac{(v_2)^2 - (v_1)^2}{2g}$

in which :

Δh = head difference (in m) in a certain stretch
v = average velocity in that stretch in m/s
L = length of stretch in m,
C = coefficient of Chezy in $m^{\frac{1}{2}}/sec$,
h = average depth in m in considered stretch,
v_2 = highest velocity in the gap in m/s,
v_1 = velocity upstream of the gap in m/s,
g = gravity acceleration in m/s^2.

In most cases, the profile of the gap is much smaller than the adjacent profiles of the river, so that v_1 is much smaller than v_2. In these cases $(v_1)^2$ can be neglected if compared with $(v_2)^2$. The Bernoulli head can now be computed as $(v_2)^2/2g$, in which v_2 is the maximum velocity in the gap. Also the head difference due to resistance is negligible in comparison with the Bernoulli head.

Due to the head difference during the entering tide, the water level behind the gap will be lower than at the seaward side (see Figure III - 8.5).

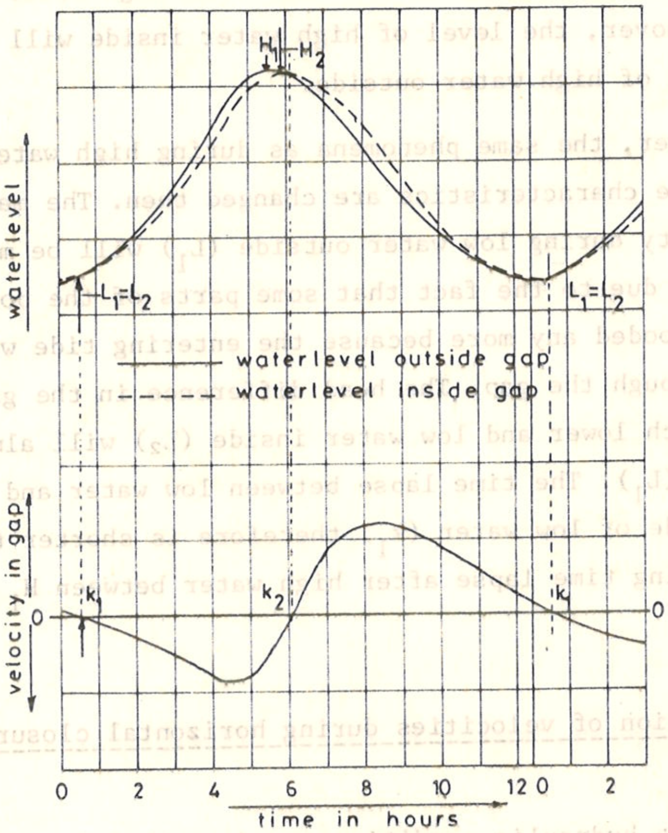


Figure III - 8.5 Water levels and velocities in the closure gap.

The vertical tide (= water level) inside is retarted with respect to the outside tide.

This means that at the moment of high water at the seaward side of the gap (point H_1 in Figure III - 8.5) the flood current is still going inward. Also the water level inside is still rising, because there is still a head difference between the storage basin and the outside tide. The water level outside starts falling while inside the level is still rising until both levels meet.

At that moment (point k_2 in Figure III - 8.5) the turn of the tide will take place and the water level inside will start to fall as well.

High water inside (H_2) will occur later than high water outside (H_1), and moreover, the level of high water inside will be lower than the level of high water outside.

Around low water, the same phenomena as during high water will occur, but some characteristics are changed then. The maximum current velocity during low water outside (L_1) will be much lower. This is due to the fact that some parts of the polder will not be flooded any more because the entering tide was restricted through the gap. The head difference in the gap at low tide is much lower and low water inside (L_2) will almost coincide with (L_1). The time lapse between low water and the turn of the tide of low water (k_1) therefore is shorter than the corresponding time lapse after high water between H_1 and k_2.

8.6.2 Computation of velocities during horizontal closure method

In this case the hydraulic conditions as described in paragraph 8.6.1 are applicable. The current through the gap flows in sub-critical conditions but velocities are increasing as the gap is narrowed and the head difference increases.

Because of the reduced discharge capacity of the gap, the tidal volume is reduced, and during the tidal period the storage basin cannot be filled up to the same level as before. Consequently, the average head differences over the gap increase in proportion with the reduction of the profile. The average velocities increase, the amplitude of the vertical tide in the basin decreases, and inside the gap the moment of turning of the tide will be retarded. Unlike the case with increasing sill levels, the extreme velocities are not restricted by critical flow conditions, so that these extreme velocities can reach higher values (see Figure III - 8.6).

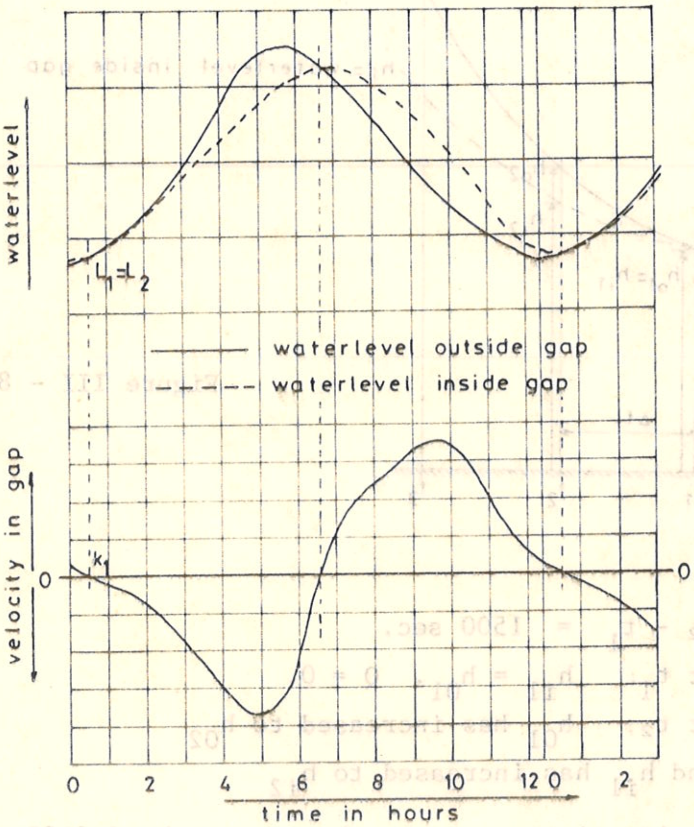


Figure III - 8.6 Water levels and velocities during advanced stage of horizontal closure.

For the calculation of the velocities in the closure gap flood routing calculation has to be made.

The start time of this routing is chosen at the moment of low tide, when water level inside (L_2) and outside (L_1) are equal. The time lapse between the moment of lowtide ($L_1 = L_2$) and the moment of the change in velocity (k_1) is assumed to be negligable.

The tidal period is now devided into equal time step of 1500 sec. and for each step the quantity of water flowing into the basin is calculated, assuming a head difference over the gap. (see Figure III - 8.7).

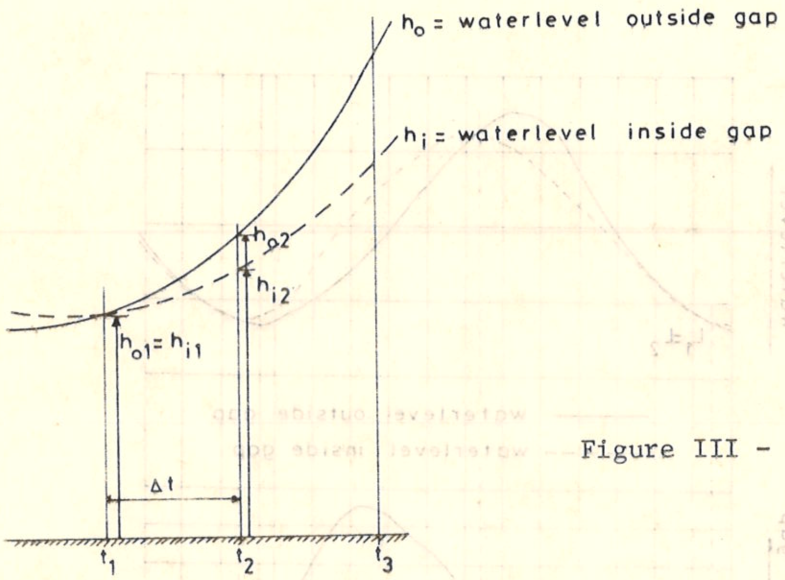


Figure III - 8.7 Definition sketch.

assume : $t_2 - t_1 = 1500$ sec.

at t_1: $h_{i1} = h_{01}$, $Q = 0$.

at t_2: h_{01} has increased to h_{02}

and h_{i1} has increased to h_{i2}

During time step Δt, the average head difference over the gap has been :

$$\Delta\bar{h}_{12} = \frac{h_{01} + h_{02}}{2} - \frac{h_{i1} + h_{i2}}{2}$$

The volume of water passed is then per unit width of the gap :

$$v = \sqrt{2.g.\ \bar{h}_{12}}$$

$$q_{12} = C \times v \times \left(\frac{h_{i2} + h_{i1}}{2}\right) = C\,\bar{h}_i \sqrt{2g \cdot \Delta\bar{h}_{12}}$$

in which : C is a contraction coefficient taken as 0.85 for gaps in horizontal closure - works.

$\bar{h}$ is average waterdepth on sill.

The total volume of water flowing in timestep Δt_{12} is then :

$$Q_{12} = C.A.\sqrt{2g\ \Delta\bar{h}_{12}}\ .\ 1500\ \ m^3.$$

in which A = wet area of the gap.

This volume of water should correspond with the increase in storage level in the area, i.e. the tidal area times the increase in water level inside ($h_{i2} - h_{i1}$). The tidal area is computed from the discharge measurement results. The total tidal volume entering/leaving the basin, is devided by the tidal difference yielding the tidal area. If it is assumed that this tidal area is constant (i.e. does not increase or decrease with changing water levels), then h_{i2} can easily be calculated.

example:

tidal area : A_T = 300 ha

tide curve : $h_{01} = h_{i1} = 6$ m.

: h_{02} = 6,60 m.

discharge during time step Δt_{12} : $Q_{12} = C.A. \sqrt{2g\ \Delta \bar{h}_{12}}\ \times\ 1500\ m^3$

in which: C = 0,9 (contraction coefficient)

A = 250 m² (wet area of the gap).

$$Q = 0.9 \times 250 \times \sqrt{2 \times 9,8 \times (\frac{6,6 - h_{i2}}{2})} \times 1500$$

Storage increase: $h_{i2} - h_{i1} = \frac{Q}{A_T}$

$$h_{i2} - 6 = \frac{0.9 \times 250 \times \sqrt{2.9,8.(\frac{6,6 - h_{i2}}{2})}}{300 \times 10^4} \times 1500$$

$\longrightarrow h_{i2} = 6,22$ m

The velocity in the gap can in the mean time be calculated in the same process.

Average maximum velocity during step Δt_{12} is calculated as follows :

$$v = \sqrt{2g\ \Delta \bar{h}_{12}}$$

$$v = \sqrt{2 \times 9,8 \times \frac{0.38}{2}} = 1.93 \text{ m/s}$$

The computation of the next h_i can now start using h_{i2} as a starting value.

When the whole tidal period has been routed in this way, a velocity curve as shown in the lower part of Figure III - 8.6 can be constructed, and maximum occurring velocities during this stage of the closure are known.

The calculation process is then repeated for a next phase in the closure operations with a more reduced gap-area.

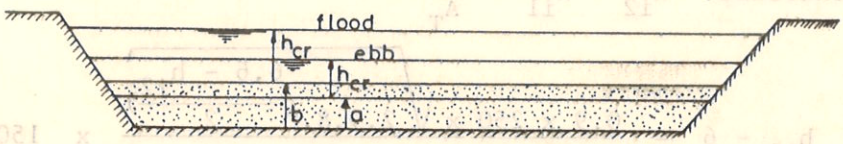
8.6.3 Computation of velocities during vertical closure method

In the vertical closure method, the dam will be built by gradually heightening the sill over its total length.

Three hydraulic stages can be considered :

a. The profile of the gap is such that sub-critical flow occurs during the entire tidal period. In that case the nature of the tidal movement is the same as described in paragraph 8.6.1 and presented on Figure III - 8.5.

b. The sill has reaches such level, that critical flow conditions do appear during ebb (level a in Figure III - 8.8).



a= sill level at start of critical flow during ebb
b= sill level when critical will also occur during flood.

Figure III - 8.8 Critical depth of sill.

c. The sill has been heightened to such level, that during both ebb and flood periods, critical flowconditions do appear (level b in Figure III - 8.8).

The essential difference with the case discussed in paragraph 8.6.2 is the appearance of the critical flow. An explanation of this phenomenom will be given here with the help of Figure III - 8.9.

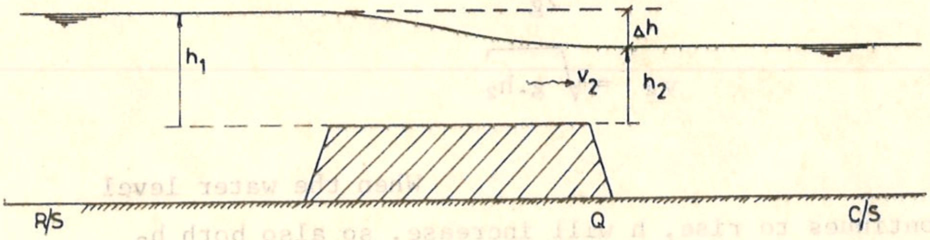


Figure III - 8.9 Definition sketch.

First the flood period will be looked at.

Outside, the water starts to rise because of the tide. The rise is independent of the situation behind the gap. The floodcurrent will start entering through the gap, causing the water level inside to rise. Because of the limited discharge capacity of the gap the water level inside cannot rise as quickly as outside. This retardation depends on :

a. cross-sectional area of gap
b. storage area of drainage basin

At first the head difference over the gap is :

$$\Delta h = \frac{(v_2)^2}{2g} \quad \text{(see also page III - 52)}$$

in which v_2 = maximum velocity through the gap.

and thus : $v_2 = \sqrt{2g.\ \Delta h}$ (see Figure III - 8.9).

This situation will continue until the waterdepth h_2 on the inside edge of the sill (near Q) is equal to $^2/_3$ of the waterdepth h_1 at the riverside of the closure gap.

Even if the water level outside rises quicker than inside, the head difference Δh will not exceed $1/3\ h_1$, because of the critical flow restriction. Velocity head over the sill will therefore never exceed the value of $1/3\ h_1$ or $\frac{1}{2} h_2$ under critical flow condition .

This yields for the maximum critical flow velocity :

$$\Delta h = \tfrac{1}{2} h_2 = \frac{(v_2)^2}{2g}$$

$$v_2 = \sqrt{g . h_2}$$

When the water level outside continues to rise, h will increase, so also both h_2 ($= 2/3\ h_1$) and v_2 ($= \sqrt{gh_2} = \sqrt{\frac{2}{3} gh_1}$) have to increase. Hence the flood current must increase as well, forcing the water level inside to rise quicker.

After the tide has reached its peak, the water level outside starts to fall but inside the level is still increasing, because there is still a head difference between river and polderside. The moment will come that the depth inside has become $2/3\ h_1$ and then will exceed this value, thus finishing the conditions of critical flow.

During the rest of the "flood" therefore, the velocity in the gap will be $v_2 = \sqrt{2\ g\ \Delta h}$, which is smaller than the velocity $v_2 = \sqrt{2/3\ g.h_1}$ during the critical flow period. The extreme current velocity appears at the moment of transition from critical to sub-critical flow, provided this happens before the moment of high water outside. If this transition occurs after the moment of high water outside, the extreme velocity appears at the moment of high water.

During ebb, the same happens in the opposite direction. (see also Figure III - 8.10.)

Because h is much larger at the start of the ebb current than at the start of the flood, the period in which $h_2 > 2/3\ h$ is longer at the beginning of the ebb, and it will take more time before the critical flow commences. (see arrow a and b in Figure III - 8.10). After this transition, the water level inside is still falling, so both h and v_2 have to decrease. The maximum ebb velocity does occur at the moment of the transition. It will make no difference for the flow conditions whether the water level outside falls to a level lower than the top of the sill or not.

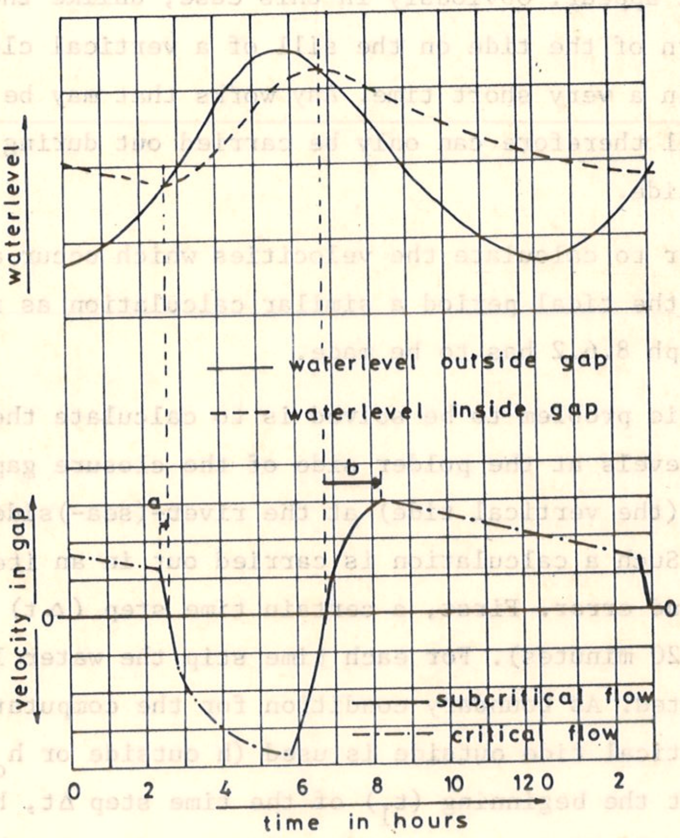


Figure III - 8.10 Water levels and velocities during advaced stage of vertical closure.

During critical flow, the current will go undisturbed over the downstream edge of the sill (Q), independent of the downstream level.

This means that the situation of critical flow can only come to an end when the water depth outside has exceeded 2/3 h above the sill, which will occur a considerable time after low water. At that moment, the level outside is already rising very quickly and, moreover, the value of h on the sill inside is moderate, so that a short time after the transition to subcritical flow, both water levels will reach the same value causing the turn of the tide in the gap.

Shortly afterwards, the critical flow conditions for flood current appear. Obviously in this case, unlike the previous case, the turn of the tide on the sill of a vertical closure will take place in a very short time. Any works that may be necessary on the sill therefore can only be carried out during high water slack tide.

In order to calculate the velocities which occur across the sill during the tidal period a similar calculation as indicated in paragraph 8.6.2 has to be made.

The basic problem to be solved is to calculate the change in water levels at the polder side of the closure gap when the water levels (the vertical tide) at the river-(sea-)side of the gap are known. Such a calculation is carried out in an iterative way, by trial and error. First, a certain time step (Δt) is chosen (e.g. Δt is 20 minutes). For each time stip the water level inside is calculated. As boundary condition for the computation, normally the vertical tide outside is used (h outside or h_o). h_{o1} is the level at the beginning (t_1) of the time step Δt, h_{o2} at the end (t_2) of the time step Δt (see Figure III - 8.11). The water levels inside, (h_i) are the values to be calculated.

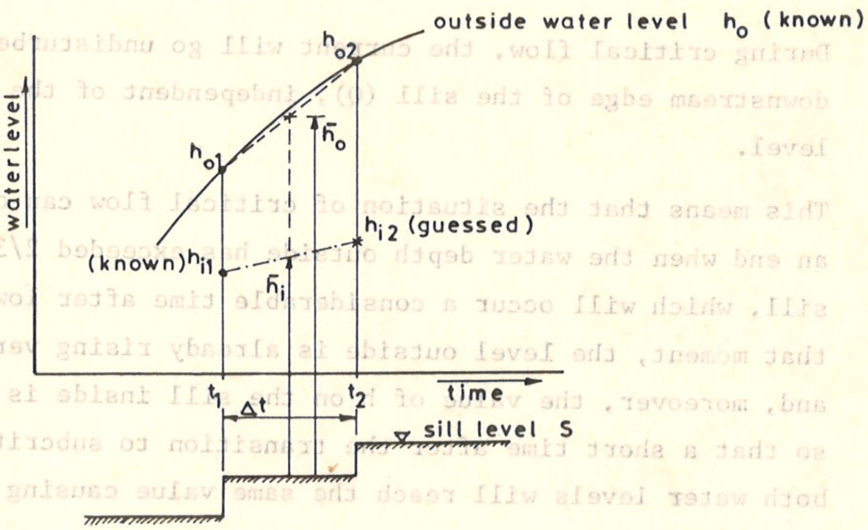


Figure III - 8.11 Symbol used in the tidal calculation.

a) The startvalue (h_{i1}) of each time step is known once the process has started and the value h_{i2} at the end of the timestep is guessed. With this guessed value the rest of the computation will be executed, and the results will be tested against the criterium of storage-volume. If they do not satisfy this criterium, a new value for h_{i2} must be introduced and the operation must be repeated until the guess appears to be accurate.
Then the water level inside is known for the end of the time step and the calculation can be continued for the next time-step, the start value for h_i now being known. If the start value for the first timestep is unknown, here also a guess has to be made. The introduced discrepancy will damp out in a few timesteps. However, after finishing the computations for the first tidal period, the entire period must be calculated a second time in order to know the influence of the introduced error. It is then easy to see where an equilibrium has been reached and the error disappeared.

b) Secondly, the dimensions of the gap must be introduced, the sill length W and the sill level S.

Now the volume of water which passes through the gap during the given timestep can be computed. For this, the average waterdepth above the sill level is calculated ($\bar{h}_o$ and $\bar{h}_i$, see Figure III - 8.11).

The average total head difference over the gap $\Delta h_{io} = \bar{h}_o - \bar{h}_i$ can be calculated, which during flood current has to be compared with the value of $\bar{h}_o$ (during ebb current with $\bar{h}_i$) in order to know whether the flow is critical or sub-critical.

In case of sub-critical flow, the discharge is :

$$Q = W \times h_i \times q$$

$$q = C\sqrt{2g.\Delta h_{io}} \qquad \text{(flood)}$$

contraction coefficient C is assumed 0.9 for vertical closures.

The total increase of water behind the closure dam during the time step is then :

$$Q.\Delta t = W. \bar{h}_i. C. \Delta t. \sqrt{2g. \Delta h_{io}} \quad \text{(flood)}$$

The decrease of this water volume during ebb stages is per timestep:

$$Q.\Delta t = W. \bar{h}_o. C. \Delta t. \sqrt{2g. \Delta h_{io}} \quad \text{(ebb)}$$

In case of critical flow during flood, the velocity will be :

$$v = C. \sqrt{2/3g. h_o}$$

and the increase in volume will be :

$$Q.\Delta t = W. 2/3\, h_o. C. \Delta t. \sqrt{2/3g. h_o} \quad \text{(flood)}$$

$$Q.\Delta t = W. 2/3\, h_i. C. \Delta t. \sqrt{2/3g. h_i} \quad \text{(ebb)}$$

This increase/decrease in water volume storage in the area behind the closure dam can also be calculated as follows :

Suppose that the water level in the storage are is horizontal and no head difference occurs when the basin is getting filled or flowing empty. The increase in volume (ΔV), will then be proportional with the rise of the water level inside Δh_i (= $h_{i2} - h_{i1}$) and the storage area B, so that $\Delta V = B. \Delta h_i$.
Now the continuity $\Delta V = B. \Delta h_i = Q. \Delta t$ can be checked. In case of discrepancy, a new guess of h_{i2} must be made and the entire operation is to be repeated.

If the system satisfies the continuity criterium, the guessed value of h_{i2} is approved and the computation can move to the new timestep.

The total procedure of the computation is given schematically in Figure III - 8.12.

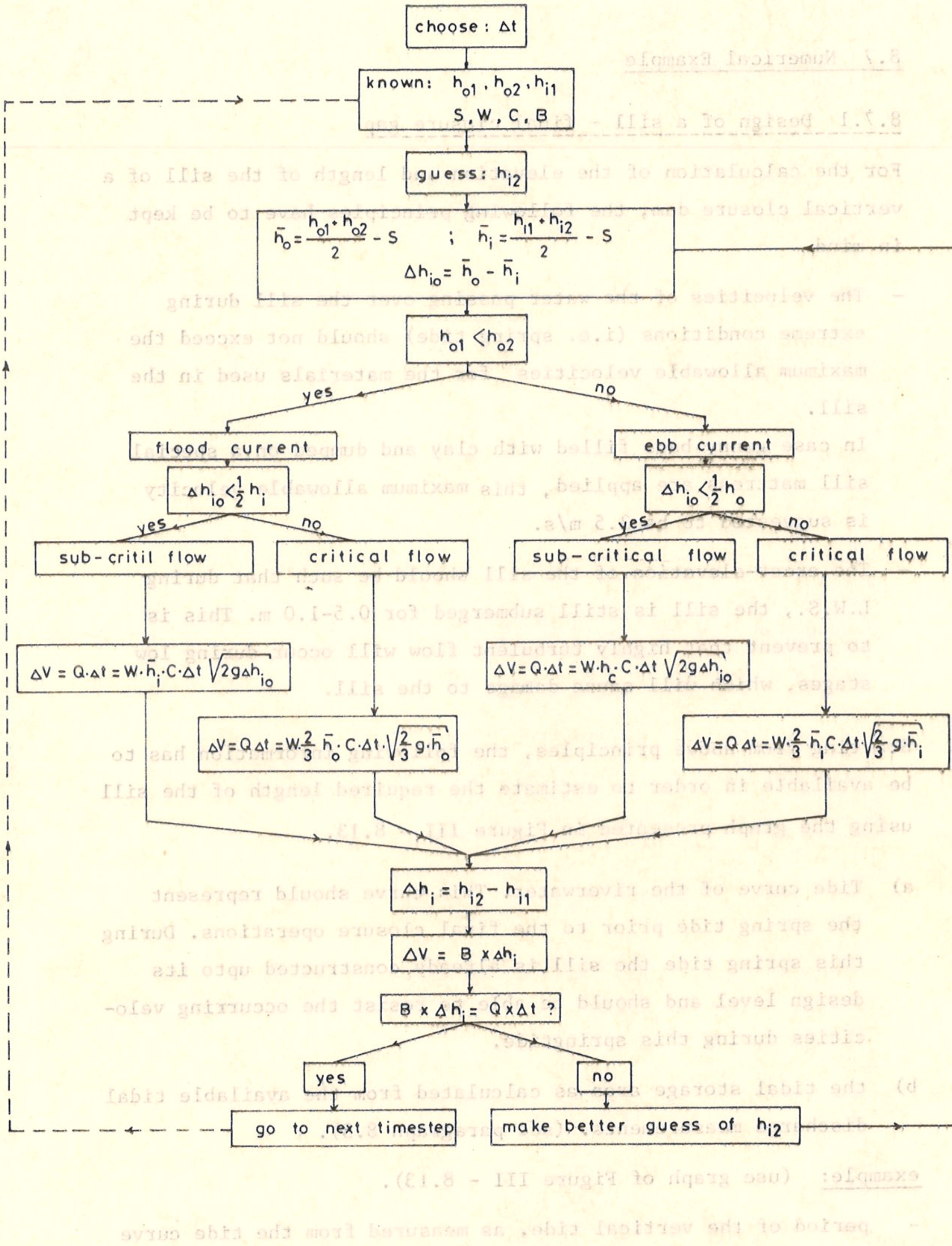


Figure III - 8.12 Flow schedule of tidal calculations for final closing operations.

8.7 Numerical Example

8.7.1 Design of a sill - final closure gap

For the calculation of the elevation and length of the sill of a vertical closure dam, the following principles have to be kept in mind.

- The velocities of the water passing over the sill during extreme conditions (i.e. spring tide) should not exceed the maximum allowable velocities for the materials used in the sill.
  In case gunny bags filled with clay and dumped on a special sill mattress are applied, this maximum allowable velocity is suggested to be 3.5 m/s.
- The crest-elevation of the sill should be such that during L.W.S., the sill is still submerged for 0.5-1.0 m. This is to prevent that highly turbulent flow will occur during low stages, which will cause damage to the sill.

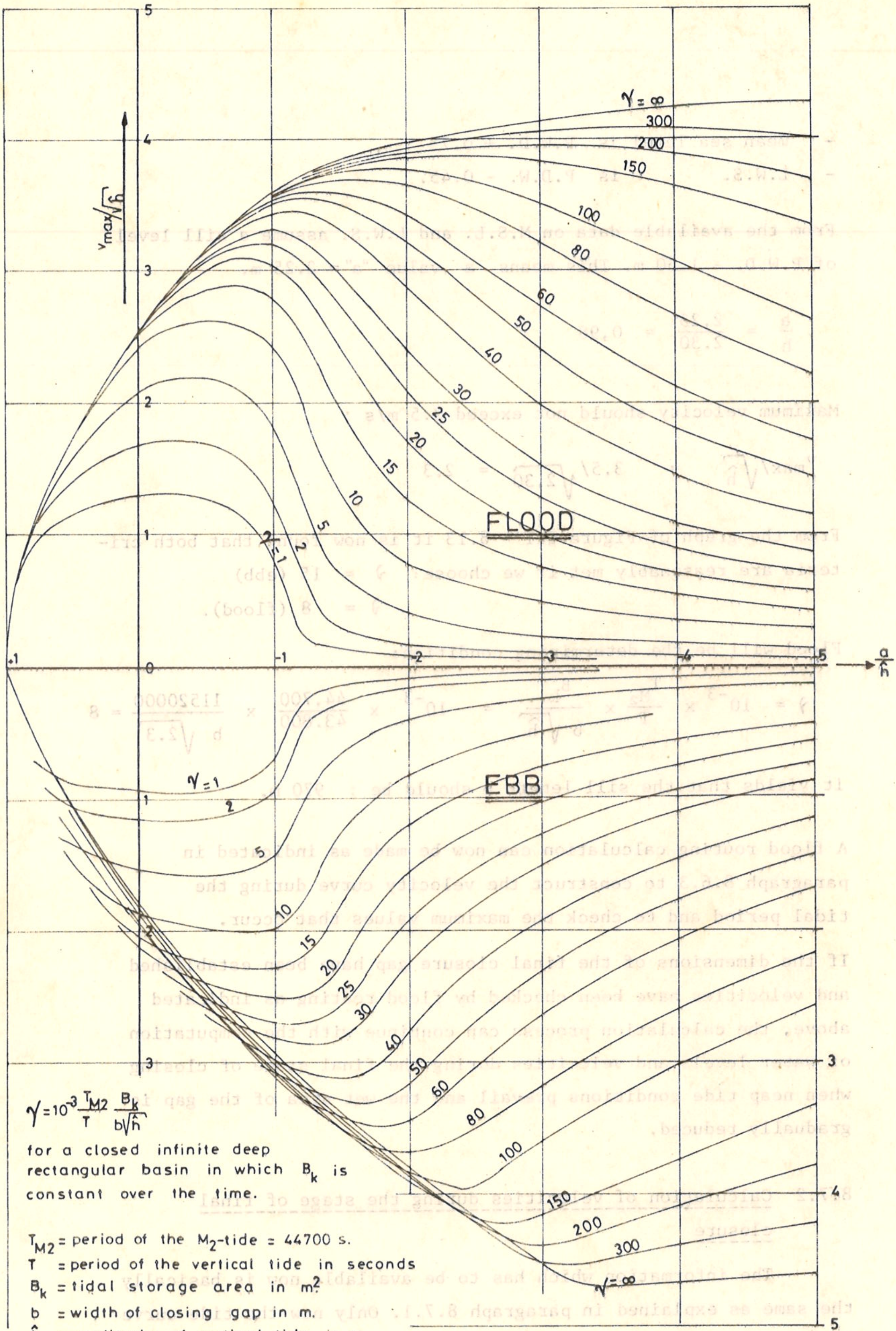
Starting from above principles, the following information has to be available in order to estimate the required length of the sill using the graph presented in Figure III - 8.13.

a) Tide curve of the riverwater. This curve should represent the spring tide prior to the final closure operations. During this spring tide the sill is already constructed upto its design level and should be able to resist the occurring velocities during this springtide.

b) the tidal storage area as calculated from the available tidal discharge measurements. (see paragraph 8.3).

example: (use graph of Figure III - 8.13).

- period of the vertical tide, as measured from the tide curve is 12 hrs 10 minutes = 43.800 sec.
- period of the M_2 - tide = 44.700 sec. (12 hrs, 25 min).
- tidal storage area is 1152 ha.
- tidal amplitude is h = 2.30 m.



$\gamma = 10^{-3} \frac{T_{M2}}{T} \frac{B_k}{b\sqrt{\hat{h}}}$

for a closed infinite deep rectangular basin in which B_k is constant over the time.

T_{M2} = period of the M_2-tide = 44700 s.
T = period of the vertical tide in seconds
B_k = tidal storage area in m^2.
b = width of closing gap in m.
$\hat{h}$ = amplitude of vertical tide in m.
a = sill level minus mean waterlevel in m.
$v_{max.}$ = maximum velocity above the sill in m/s.

Figure III - 8.13 Maximum velocities in closure gap.

- mean sea level is P.W.D. + 0.75
- L.W.S. is P.D.W. - 0.45.

From the available data on M.S.L. and L.W.S. assume a sill level of P.W.D. - 1.50 m. This means a value "a" = 2.25 m.

$$\frac{a}{h} = \frac{2.25}{2.30} = 0{,}98$$

Maximum velocity should not exceed 3.5 m/s :

$$v_{max}/\sqrt{\hat{h}} = 3.5/\sqrt{2.30} = 2.3$$

From the graph of Figure III - 8.13 it is now found that both criteria are reasonably met if we choose: γ = 15 (ebb)

γ = 8 (flood).

Flood will be the determining condition:

$$\gamma = 10^{-3} \times \frac{T_{M_2}}{T} \times \frac{B_k}{b\sqrt{\hat{h}}} = 10^{-3} \times \frac{44.700}{43.800} \times \frac{11520000}{b\sqrt{2.3}} = 8$$

it yields that the sill length b should be : 970 m.

A flood routing calculation can now be made as indicated in paragraph 8.6.3 to construct the velocity curve during the tidal period and to check the maximum values that occur.

If the dimensions of the final closure gap have been established and velocities have been checked by flood routing as indicated above, the calculation process can continue with the computation of water levels and velocities during the final stage of closing when neap tide conditions prevail and the wet area of the gap is gradually reduced.

8.7.2 Calculation of velocities during the stage of final closure

The information which has to be available now is basically the same as explained in paragraph 8.7.1. Only now the tide curve of the prevailing neap tide has to be known.

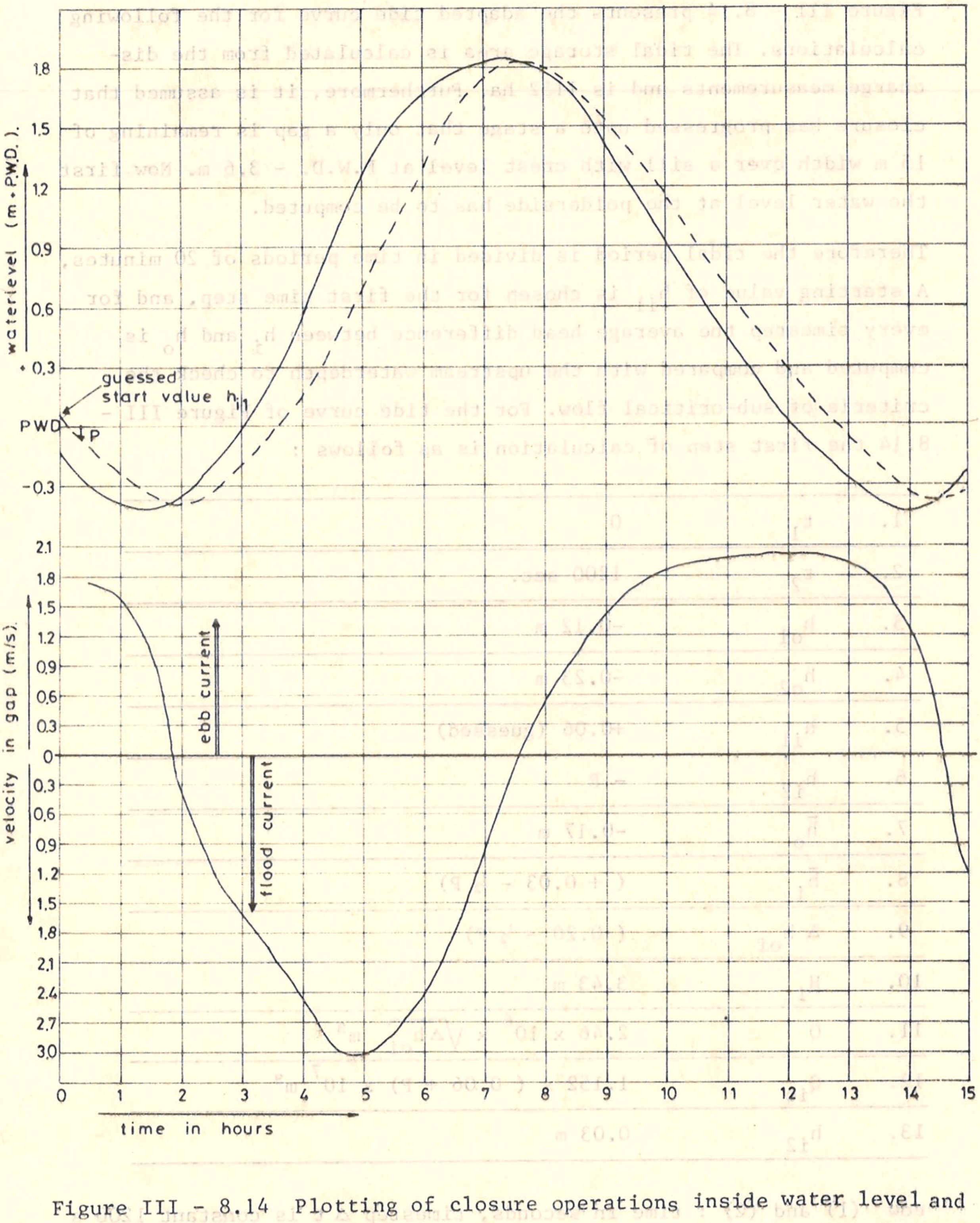


Figure III - 8.14 Plotting of closure operations inside water level and velocities.

Figure III - 8.14 presents the adapted tide curve for the following calculations. The tidal storage area is calculated from the discharge measurements and is 1152 ha. Furthermore, it is assumed that closure has progressed upto a stage that only a gap is remaining of 15 m width over a sill with crest level at P.W.D. - 3.6 m. Now first the water level at the polderside has to be computed.

Therefore the tidal period is divided in time periods of 20 minutes, A starting value of h_{i1} is chosen for the first time step, and for every timestep the average head difference between h_i and h_o is computed and compared with the upstream waterdepth to check the criteria of sub-critical flow. For the tide curve of Figure III - 8.14 the first step of calculation is as follows :

1.	t_1	0
2.	t_2	1200 sec.
3.	h_{o1}	-0.12 m
4.	h_{o2}	-0.23 m
5.	h_{i1}	+0.06 (guessed)
6.	h_{i2}	- P
7.	$\bar{h}_o$	-0.17 m
8.	$\bar{h}_i$	(+ 0.03 - ½ P)
9.	Δh_{oi}	(0.20 - ½ P)
10.	H_i	3.43 m
11.	Q	$2.46 \times 10^6 \times \sqrt{\Delta h_{oi}}$ m³
12.	Q_{io}	$1.152 \times (0.06 + P) \times 10^7$ m³
13.	h_{i2}	0.03 m

row (1) and (2) : time in seconds, timestep Δt is constant 1200 s

row (3) is outside water level at t_1

row (4) is outside water level at t_2

row (7) is $\frac{(3) + (4)}{2}$

row (5) is inside water level at t_1 (guess for 1^{st} step)

row (6) is inside water level at t_2 (to be calculated for every step)

row (8) $= \frac{(5) + (6)}{2}$

row (9) $= \frac{(7) + (8)}{2}$

row (10) = waterdepth above sill, assumed to be sill-crest-elevation minus $\bar{h}_o$, both relative to P.W.D. datum.

row (11) $Q = W.\ H_i.\ C.\ \Delta t.\ \sqrt{2g.\ \Delta h_{oi}}$ for subcritical stage

$Q = W.\ 2/3\ \bar{h}_o.\ C.\Delta t.\ \sqrt{2/3\ g.\bar{h}_o}$ for critical stage

W = sill width = 15 m

C = contraction coefficient = 0.9

row (12) $Q_{io} = B \times (h_{i1} - h_{i2})$

B = tidal drainage area = 1152 ha.

At the start and end of each timestep (i.e. at t_1 and t_2) the water level outside h_o and the water level inside h_i and the average water levels $\bar{h}_o$ and $\bar{h}_i$ are taken from Figure III - 8.14. For the first time step h_{i1} and h_{i2} have to be guessed. For the next steps h_{i1} is then known and h_{i2} can be calculated. At the same time the velocities over the sill can be calculated, (see Figure III - 8.14) and thus it can be checked whether sill crest level and/or width have to be changed to yield allowable maximum velocities.

Table III - 8.1 Tidal calculation final closure stage

t_1	t_2	h_{o1}	h_{o2}	h_{i1}	h_{i2}	S	$\bar{h}_o$	$\bar{h}_i$	Δh_{io}	crit. of sub-crit.	Δh_i	Q. Δt	B. Δh_i	$\bar{v}_{gap}$
		m+PWD	m+PWD	m+PWD	m+PWD	m+PWD	m	m	m		m	m^3	m^3	m/s
14.00	14.10	-0.43	-0.42	-0.32	-0.35	-3.51	3.077	3.167	0.09	subcr.	0.03	34.5×10^3	35.11×10^3	+1.19
14.10	14.20	-0.42	-0.42	-0.35	-0.37	-3.20	2.775	2.836	0.06	"	0.02	25.51×10^3	24.58×10^3	+0.94
14.20	14.30	-0.42	-0.41	-0.37	-0.39	-2.90	2.48	2.513	0.033	"	0.015	16.37×10^3	17.56×10^3	+0.76
14.30	14.40	-0.41	-0.37	-0.39	-0.39	-2.59	2.205	2.202	-0.003	"	-0.003	-4.42×10^3	-3.51×10^3	-0.18
14.40	14.50	-0.37	-0.34	-0.39	-0.37	-2.29	1.935	1.905	-0.03	"	-0.012	-12.12×10^3	-14.05×10^3	-0.79
14.50	15.00	-0.34	-0.28	-0.37	-0.36	-1.98	1.673	1.612	-0.061	"	-0.012	-14.50×10^3	-14.05×10^3	-0.94
15.00	15.10	-0.28	-0.21	-0.36	-0.35	-1.68	1.429	1.321	-0.108	"	-0.015	-15.83×10^3	-17.56×10^3	-1.46
15.10	15.20	-0.21	-0.15	-0.35	-0.34	-1.37	1.189	1.030	-0.159	"	-0.012	-14.95×10^3	-14.05×10^3	-1.49
15.20	15.30	-0.15	-0.08	-0.34	-0.32	-1.07	0.953	0.738	-0.215	"	-0.012	-12.46×10^3	-14.05×10^3	-2.07
15.30	15.40	-0.08	0.00	-0.32	-0.32	-0.76	0.724	0.442	-0.282	crit.	-0.006	-8.66×10^3	7.02×10^3	-1.95
15.40	15.50	0.00	0.08^+	-0.32	-0.31	-0.46	0.498	0.142	-0.356	"	-0.003	-5.6×10^3	3.51×10^3	-1.62
15.50	16.00	0.08^+	0.17^+	-0.31	-0.35	-0.15	0.277	0.16	-0.437	"	-0.003	-1.98×10^3	3.51×10^3	-1.19
16.00	16.10	0.17^+	0.24^+	-0.31	-0.31	+0.15	0.053	0.463	-0.516	"	0	-17.36×10^1	0	-0.52

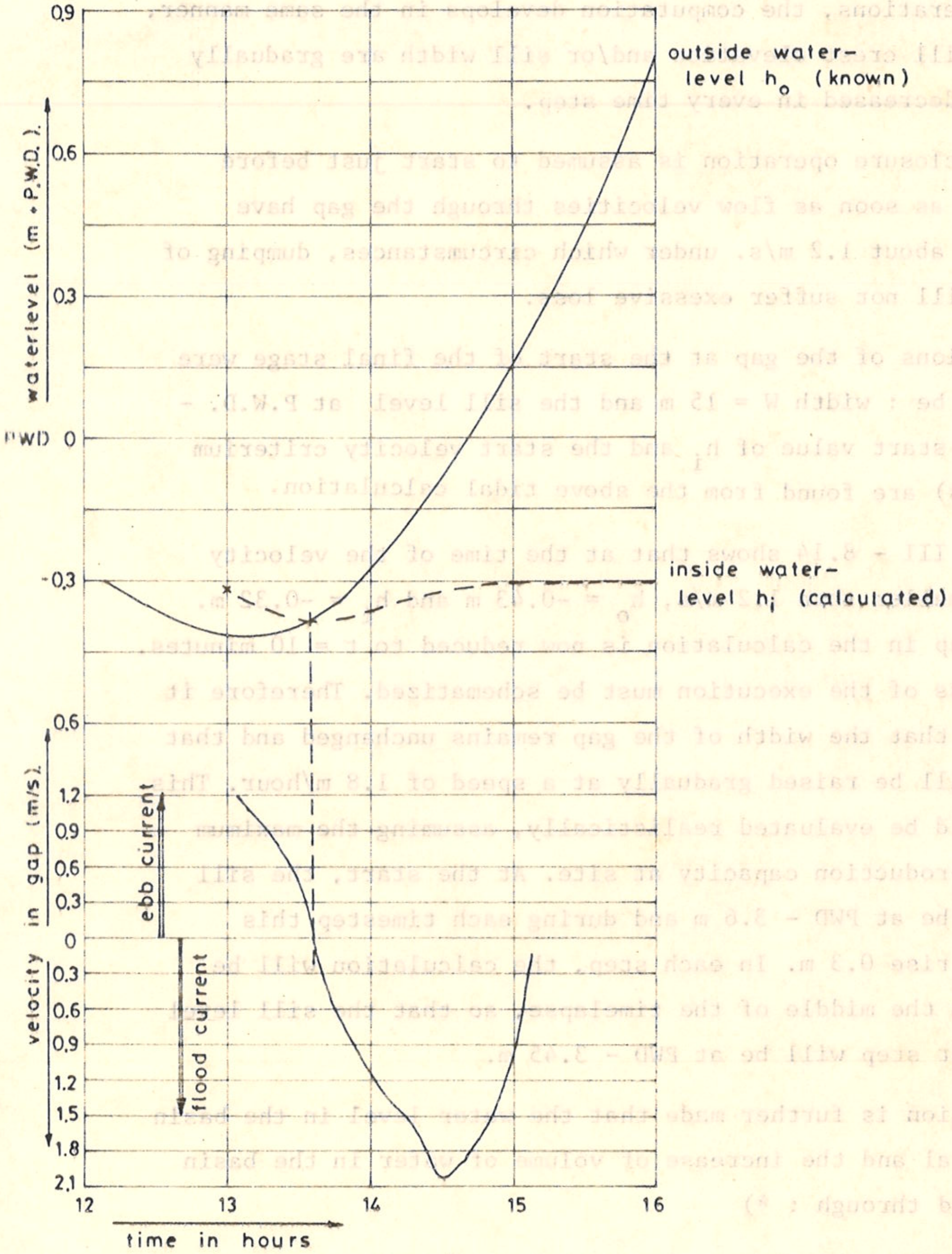


Figure III - 8.15 Plotting of calculation - results of final closure operations.

To check the velocities that will occur during the ultimate closure operations, the computation develops in the same manner, only the sill crest elevation and/or sill width are gradually increased/decreased in every time step.

The final closure operation is assumed to start just before low water, as soon as flow velocities through the gap have reduced to about 1.2 m/s. under which circumstances, dumping of material will not suffer exessive loss.

The dimensions of the gap at the start of the final stage were assumed to be : width W = 15 m and the sill level at P.W.D. - 3.6 m. The start value of h_i and the start velocity criterium ($v < 1.2$ m/s) are found from the above tidal calculation.

The Figure III - 8.14 shows that at the time of the velocity becoming smaller than 1.2 m/s, $h_o = -0.43$ m and $h_i = -0.32$ m. The timestep in the calculation is now reduced to t = 10 minutes. The progress of the execution must be schematized. Therefore it is assumed that the width of the gap remains unchanged and that the sill will be raised gradually at a speed of 1.8 m/hour. This speed should be evaluated realistically, assuming the maximum available production capacity at site. At the start, the sill level will be at PWD - 3.6 m and during each timestep this level will rise 0.3 m. In each step, the calculation will be executed in the middle of the timelapse, so that the sill level in the first step will be at PWD - 3.45 m.

The assumption is further made that the water level in the basin is horizontal and the increase of volume of water in the basin can be found through : *)

$\Delta V = B . \Delta h_i$ in which B is the tidal storage area (assumed to be 1152 ha).

---

*) Note : This of course should be checked for each particular case. In this check, the length of the basin should be compared to the length of the tidal wave L = c.T

T = period of the tidal wave = 44700 sec.

c = celerity of the tidal wave = $\sqrt{g.h}$

Assuming the average basin depth to be 10 m, the wave length of the tidal wave is

$$L = T. \sqrt{g.h.} = 44700. \sqrt{9.8 \times 10} = 443 \text{ km}$$

Most tidal basins in Bangladesh will have a length considerably less then 443 km and thus the water level in the basin can be considered to change as a horizontal plane.
However the inaccuracy introduced by assuming the basin to be of a constant size (not influenced by the stage of the water), and to be of infinite depth, will have to be accepted.

The results of the calculations are presented in the Table III - 8.1 and the resulting water levels and velocities are plotted in Figure III - 8.15.

8.8 Design of mattresses for bottom protection

8.8.1 The Mattress

The construction of a mattress is determined by the available construction materials, equipment and riverbed conditions. The size of a single mattress is governed by the tensile strength that has to be provided to allow towing of the mattress to the sink location and by the sink operation itself, in which the waterdepth is the determining factor. The minimum length should therefore be 2 times the water depth. The maximum length and width is only determined by the dimension of floating equipment that is used during the sinking operation. Usually the width of a mattress equals the effective length of the barge with ballast materials. The ratio length-width should however not exceed $\frac{L}{W} \leqslant 3$ to avoid overturning of the mattress along the longitudinal axis during ballasting and sinking.

A basic design for a bottom protection mattress is shown on Figure III - 8.16.

After manoeuvring the floating mattress to the right position, it is sunk by ballasting. As this ballasting is done manually, the weight of each individual ballast unit should not exceed 60 kg.

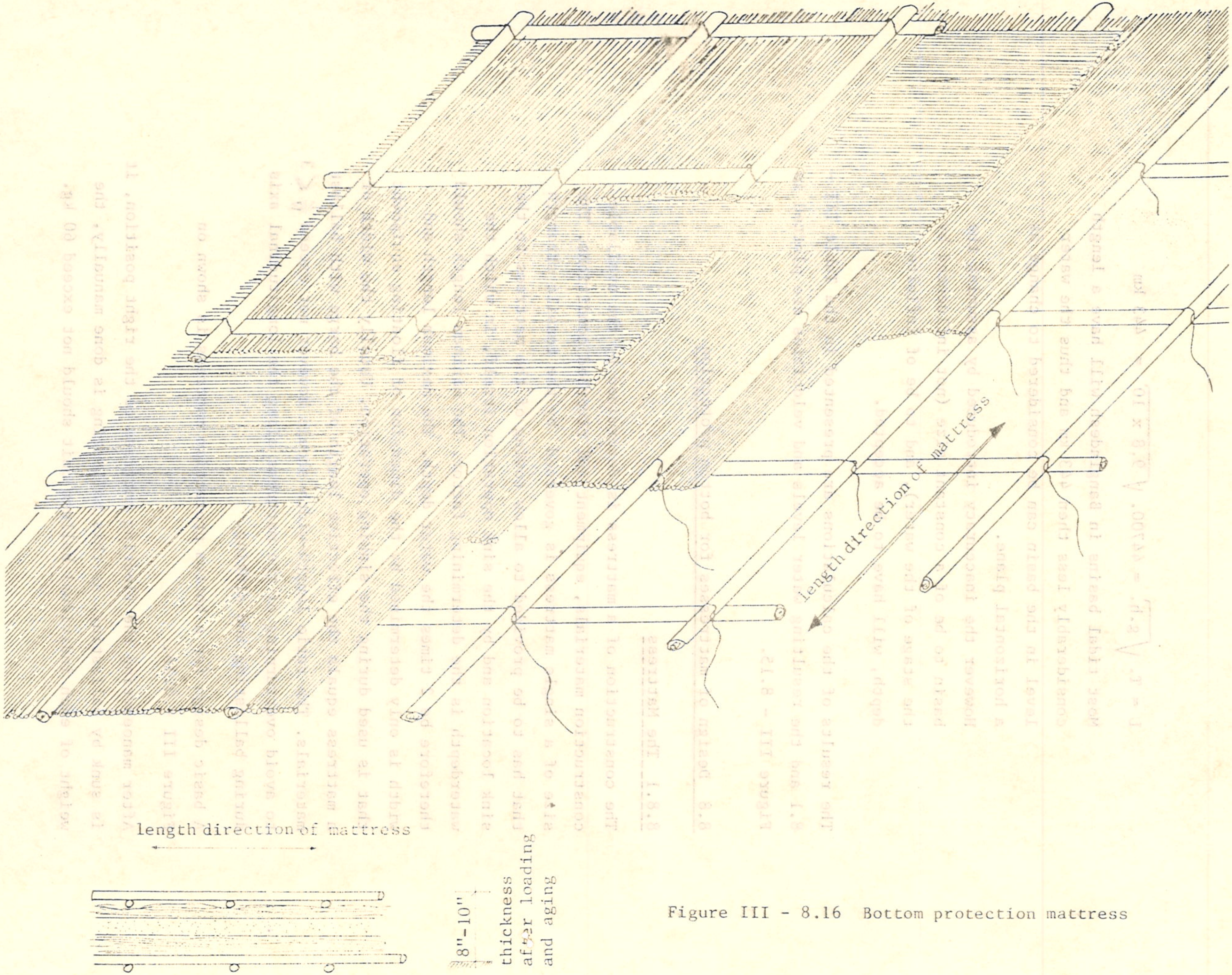


Figure III - 8.16 Bottom protection mattress

Depending on the floating capacity of the mattress, a ballast weight of 100-250 kg/m² consisting of unit weights from 10-40 kg. has to be applied. After it is sunk to the bottom, the mattress is ballasted with an extra weight, and the total load will be 300-500 kg/m² depending on the conditions. The unit weight of the ballast-units can be checked against current resistance with the help of Figure III - 5.7 and 5.8 (see Chapter 5.5 of Volume III).

Two mattresses sunk next to each other should have an overlap of at least 1.5 m, to avoid any bottom area to remain unprotected. Only in cases where waterdepth is limited and thus positioning and sinking of mattresses can be done very accurately, this overlap might be reduced or omitted.

As ballast material various materials can be used :

- gunny bags, size 0.75 x 0.45 m² (2½ x 1½ ft²), filled with 1 cft. earth (clay content of earth $\geq$ 30%).
- brick blocks - rip rap
- concrete blocks
- stones
- stone or brick gabions.

Experiences with gunny bags filled with clay show that during short periods currents of 2.5 - 3.0 m/s can be resisted if the bags have had time to settle. However some damage has to be accepted. If the mattress is built properly, and equipped with extra means to retain the ballast-units, the gunny bags on top can resist currents upto 4 m/s during short periods. The mattress is therefore equipped with rolls of reed which are tied to the bamboo frame and in between which the bags find additional support against washing away.

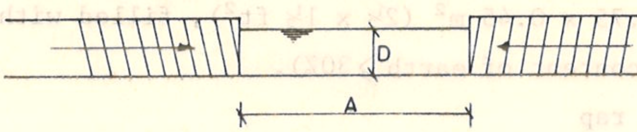
The dumping of all ballast materials should be executed during slack tides as soon as current velocities drop below 1 m/s.

For detailed descriptions of mattress construction and materials, sinking operations and cost estimates, reference is made to the Interim Reports I to V of the Early Implementation Projects dealing with closure works.

8.8.2 Area to be protected

In Figure III - 8.17 the bottom protection required for a horizontal closure is shown. The dimensions are indicative and should be chosen conveniently with respect to available machinery, equipment and materials.

For a vertical closure the bottom protection should extend 3 times the depth, beyond the toe line of the sill on both sides of the sill. Depending on the bearing capacity and characteristics of the bottom material it might be considered to omit the protection in the centre portion under the sill. (see Figure III - 8.18). Sill-mattresses have to cover the slopes and crest of the sill and should have an overlap of 10 metre at the toe at both sides of the sill to be able to provide a proper toe-ballast.



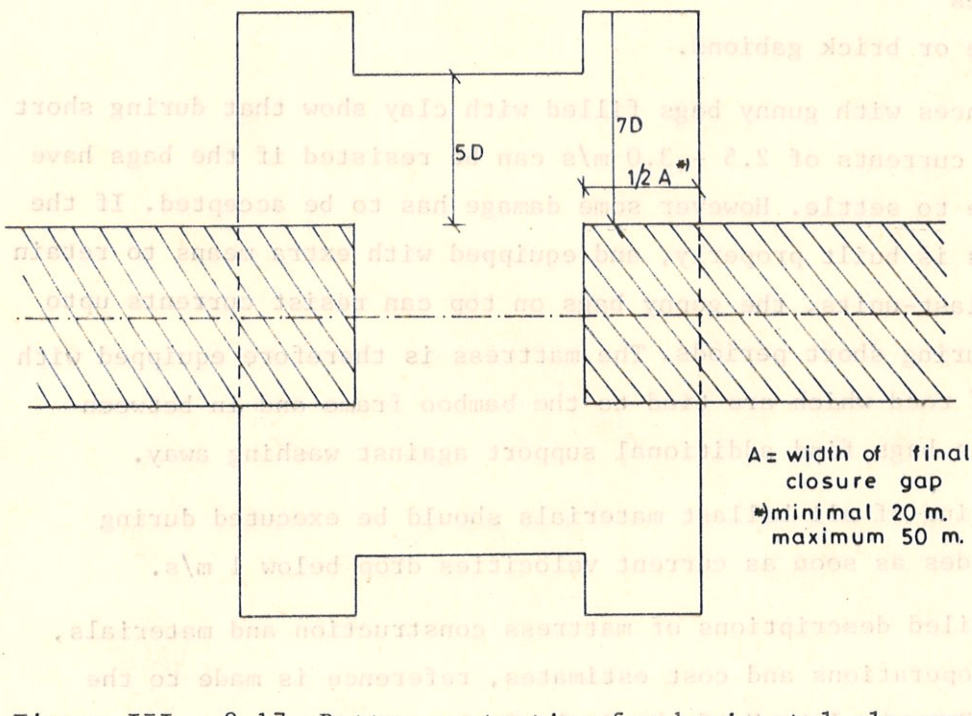


Figure III - 8.17 Bottom protection for horizontal closure.

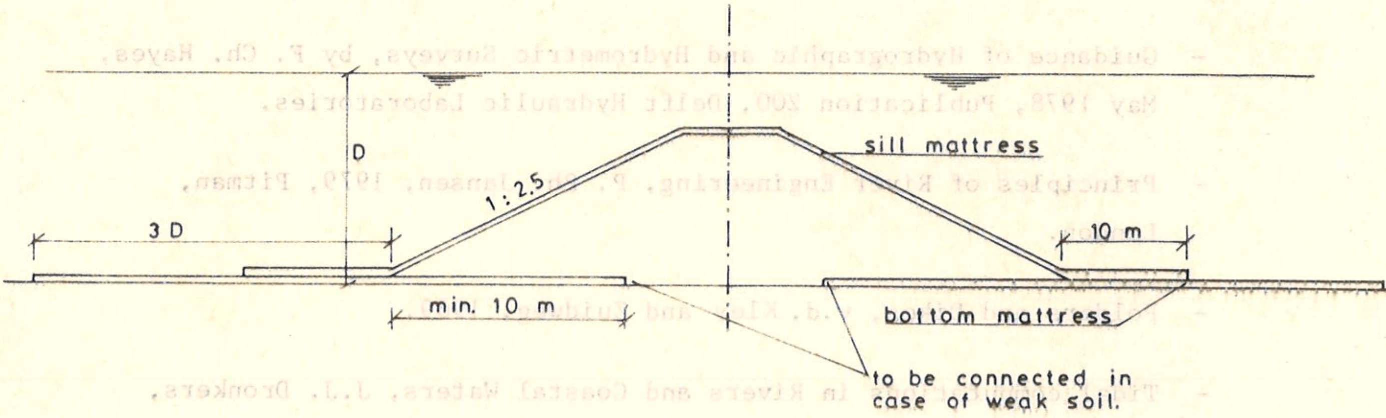


Figure III - 8.18 Bottom and sill protection for vertical closure.

References :

- Guidance of Hydrographic and Hydrometric Surveys, by F. Ch. Hayes, May 1978, Publication 200, Delft Hydraulic Laboratories.

- Principles of River Engineering, P. Ph. Jansen, 1979, Pitman, London.

- Polders and Dikes, v.d. Kley and Zuidweg, 1969.

- Tidal computations in Rivers and Coastal Waters, J.J. Dronkers, 1964.

- Hydraulic and Soil mechanical Aspects of Enclosures in Estuaries, J.J. Dronkers and W.A. Venis.

ANNEX III-1	R/S SLOPE				C/S SLOPE
	CURRENT (no- or insufficient foreland)		NO CURRENT (sufficient foreland.)		
TYPE OF EMBANKMENT	WAVES	NO WAVES	WAVES	NO WAVES	
RIVER	M.H.W. 1:2 L.R.L. 1:25 design revetment and gradient of slope	M.H.W. L.R.L. 1:25 1:2 brickpitching	W. moderate: close turfing W. serious: brick pitching from toe to ▽ M.H.W. ▽ 1:2	M.H.W. 1:2	1:3
DELTA	M.H.W.S. 1:3 L.L.W.S. 1:25 design revetment and gradient of slope	M.H.W.S. 1:3 L.L.W.S. 1:25 brickpitching	W. moderate: close turfing M.H.W.S. 1:3 W. serious: design revetment and gradient of slope	1:3	1:2
				INTERIOR 1:2	1:2
SEA	design revetment and gradient of slope	M.H.W.S. 1:25 1:7	design revetment and slope gradient	1:7	1:3

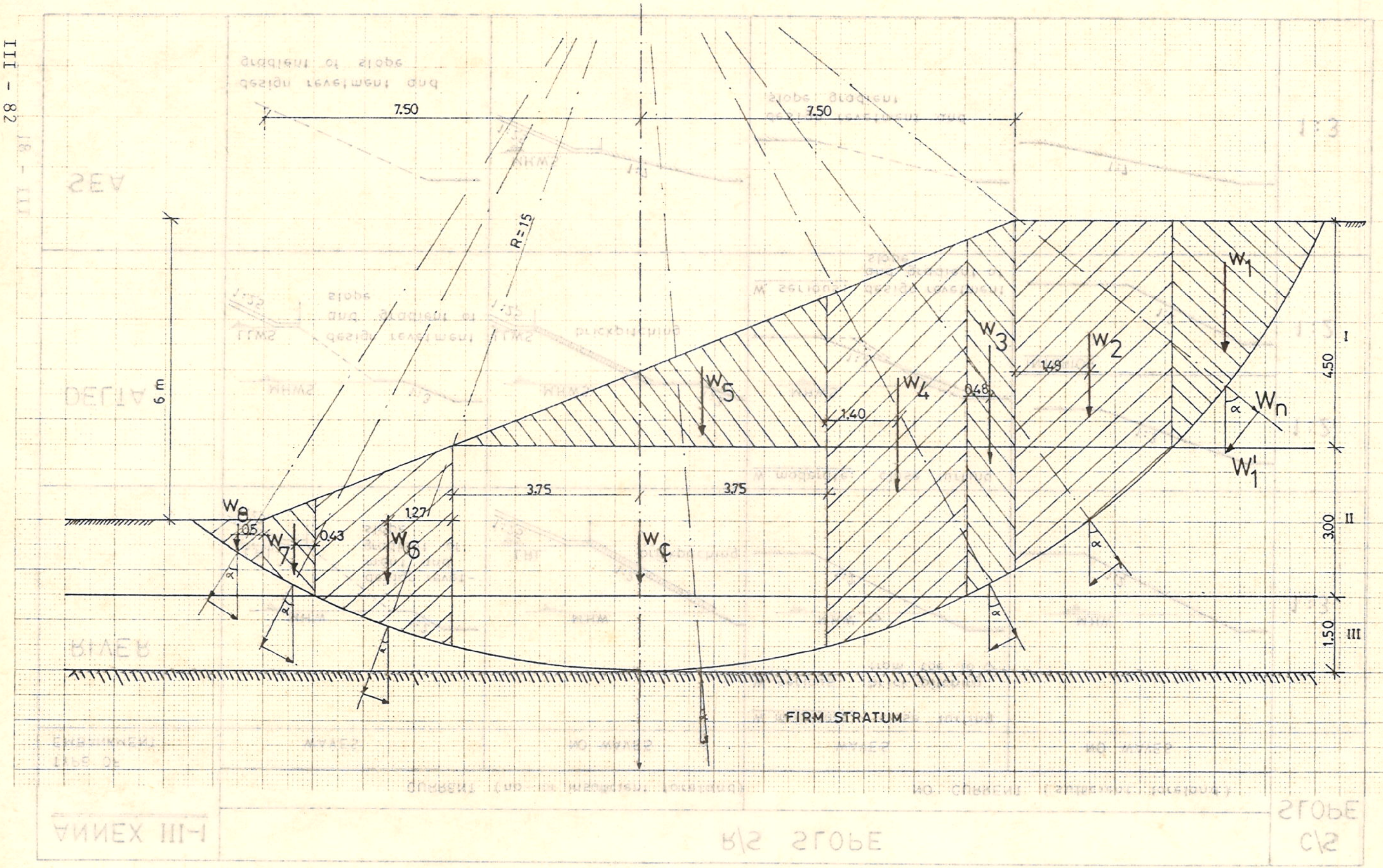


Figure A.III - 2.1 Slope stability calculation.

ANNEX III - 2 SLOPE STABILITY CALCULATION

(method of Fellenius)

	γ	c	φ
Layer I	18.85	0	35°
Layer II	18.07	9.58	18°
Layer III	18.07	19.15	0°
	KN/m^3	KN/m^2	

segment	vert. force	α	W'	W_N	φ	$W_N.\tan\varphi$	c	Δl	$c.\Delta l$
W_1	128.9	51.38	100.7	80.5	35	56.4	0		
W_2	344.5	36.82	206.5	275.8	18	89.6	9.58	4.08	39.1
W_3	125.7	27.90	58.8	111.1	18	36.1	9.58	1.09	10.4
W_4	364.1	20.08	125.0	342.0	0	-	19.15		
W_5	212.1	4.76	17.6	211.4	0	-	19.15	13.53	259.2
W_t	577.3	-		577.3	0	-	19.15		
W_6	199.2	19.55	- 66.7	187.7	0	-	19.15		
W_7	24.9	27.69	- 11.6	22.0	18	7.1	9.58	1.09	10.4
W_8	13.4	32.20	- 7.1	11.3	18	3.6	9.58	1.80	17.2
			423.2 KN			192.8			336.3

Moving force | resisting force

$W' = 423.2$ KN | $(W_N.\tan\varphi - c.\Delta l) = 529.1$

Safety against sliding $= \frac{529.1}{423.2} = 1.25$

ANNEX III - 3 DESIGN OF FILTER CONSTRUCTION

In case the river bottom or slopes of embankments or sluice outfalls have to be protected against erosive foces of current or waves, it has to be prevented that bottom material under the flow resistant protection layer is not washed out.

This washing out is prevented by applying a filter construction which should be sufficiently permeable for possible water pressure variations, but dense enough to prevent washing out of underlying layers.

If a filter construction is to be made of different layers of sand and gravel, the following gradation criteria should be noted.

nomenclature :

base material = lower layer of two consecutive layers in the filter construction

filter material = top layer of two consecutive layers in the filter construction

d_{15} = size of sieve in mm, through which 15% of the material is passing or: 85% of the material is bigger than this grain size (in mm).

Criteria :

I : anti wash out :

a) $\frac{d_{15} \text{ filtermaterial}}{d_{85} \text{ base material}} \leqslant 5$

Note : if base material is cohesive then d_{15} filter material can be bigger than 0,100 mm.

b] $$\frac{d_{50} \text{ filtermaterial}}{d_{50} \text{ base material}} = p$$

	p	q
for homogeneous round grains	5 - 10	5 - 10
for homogeneous sharp grains (Sylhet Sand)	10 - 30	6 - 20
for graded grains (sized khoa, stone chips)	12 - 60	12 - 40

II : Pro permeability :

a] $$\frac{d_{15} \text{ filtermaterial}}{d_{15} \text{ base material}} = q.$$

b] $d_{95} \geqslant 0.750$ mm.

III : The sieve curves of all layers should be almost parallel in the area of the smaller fractions.

IV : Minimum layer thickness :

Sand	0.10 m
Gravel	0.20 m
Stones	2 times stone diameter

Design :

The number of layers in the filter construction depends on the existing bottom-material and the mode of protection to be provided by the top layer.

Bottom samples should be collected and an average sieve curve should be determined which will form the basis of the further design.

Also the size and/or weight of the top layer is to be known from the current or waves to be resisted. The top-material is also to be plotted as a sieve curve.

In between these two curves the filter layers have to be determined.

example :

slope: 1:3 (α = 18,4°)

max. current velocity 2 m/s

Step 1 For the slope we have to apply a correction on the current velocity :

$$\sqrt[4]{1 - \frac{\sin^2\alpha}{\sin^2\varphi}} \times v_{slope} = v_{bottom}$$

for φ :

	round	sharp
Sand	30	34
Gravel	36	40
Stones	38	42

$$v_{slope} = \frac{2.00}{0.93} = 2.15 \text{ m/s}$$

Step 2

From Figure III - 5.7 it is found that the required diameter of the protection unit is 110 - 440 mm. In case brick blocks, or masonry rip-rap is used, the diameter of one unit will be roughly 0.36 m and the weight per unit ± 42 kg. (see Figure III - 5.8). This diameter is plotted as a straight line in the sieve curve diagram. (see Figure A . III - 3.1)

In the following example the characteristics of the top layer have been assumed: d_{50} = 200 mm and d_{15} = 15 mm.

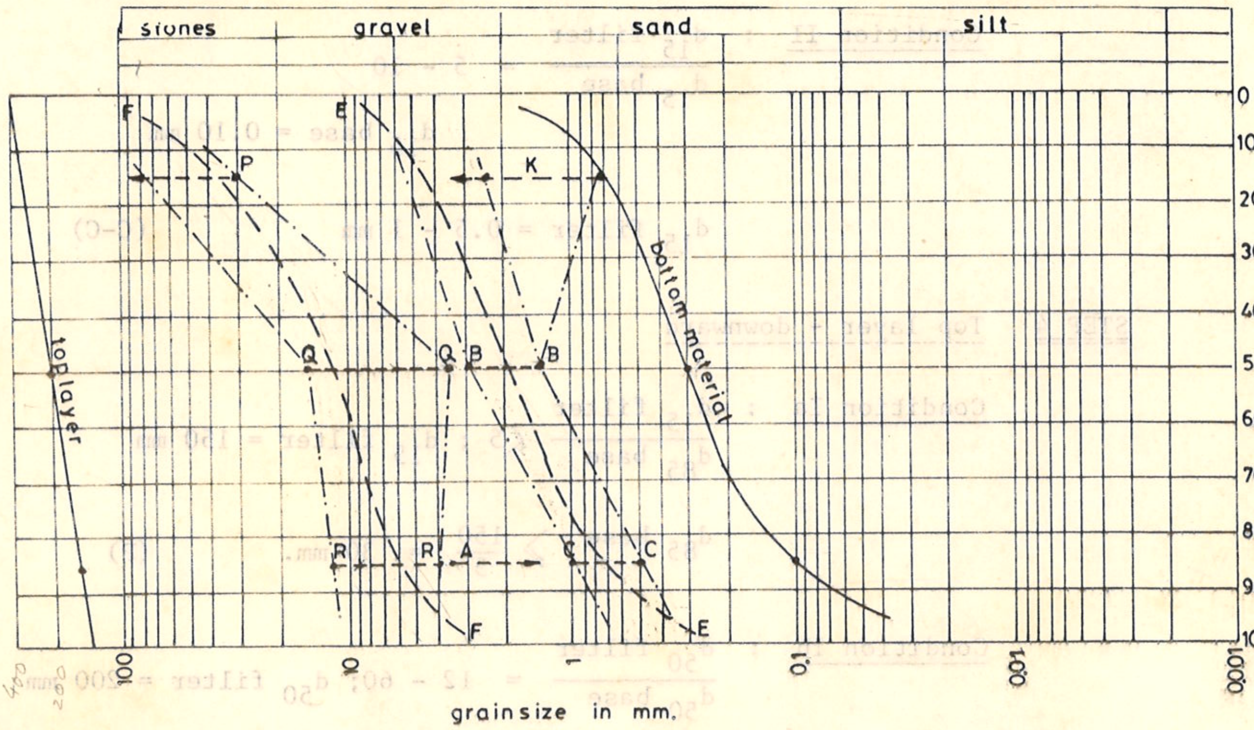


Figure A.III - 3.1 Sieve diagram for filterbed design.

From the sieve curve diagram, in which also the bottom material sieve curve is plotted, we proceed as follows.

STEP 3

Bottom layer - upward

Condition Ia. : $\dfrac{d_{15}\text{ filter}}{d_{85}\text{ base}} \leqslant 5$, d_{85} base = 0.700 mm

d_{15} filter $\leqslant 5 \times 0.700 = 3.5$ mm (A)

Condition Ib : $\dfrac{d_{50}\text{ filter}}{d_{50}\text{ base}} = 9.5 - 10.$

d_{50} base = 0.300 mm

d_{50} filter = 5 x 0,3 - 10 x 0.3

= 1.5 - 3 mm (B-B)

Condition II : $\frac{d_{15}\ \text{filter}}{d_{15}\ \text{base}} = 5 - 10$

d_{15} base = 0.10 mm

d_{15} filter = 0.5 - 1 mm (C-C)

STEP 4 Top layer - downward

Condition Ia : $\frac{d_{15}\ \text{filter}}{d_{85}\ \text{base}} \leqslant 5$; d_{15} filter = 150 mm

d_{85} base $\geqslant \frac{150}{5} = 30$ mm. (P)

Condition Ib : $\frac{d_{50}\ \text{filter}}{d_{50}\ \text{base}} = 12 - 60$; d_{50} filter = 200 mm

d_{50} base = 3.5 - 17 mm. (Q-Q)

Condition II : $\frac{d_{15}\ \text{filter}}{d_{15}\ \text{base}} = 14 - 40$; d_{15} filter = 150 mm

d_{15} base = 3.75 - 12½ mm (R-R)

STEP 5

from condition (R-R) follows :

$\frac{d_{15}\ \text{filter}}{d_{85}\ \text{base}} \leqslant 5$;

d_{15} filter = 3.75 - 12½ mm

d_{85} base $\geqslant$ 0.75 - 2.5 mm (K)

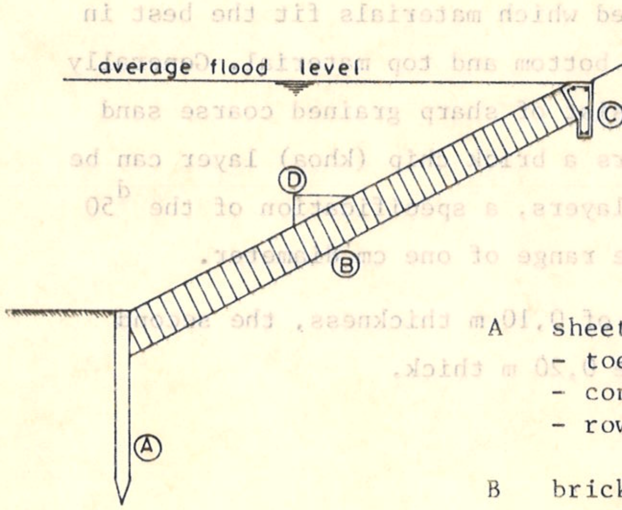
With the above information the shaded zones C-C/B-B/K and R-R/Q-Q/P can be constructed to which the characteristics of the filter materials have to be tested.

The two dotted lines E and F in the sieve curve diagram indicate an ideal solution for the given boundary conditions.

In practice the available materials (sand) are analysed and plotted in the diagram. Then it is checked which materials fit the best in the prevailing boundaries of the bottom and top material. Generally the first filter layer is to consist of sharp grained coarse sand (Sylhet) and for the second layers a brick chip (khoa) layer can be applied. Especially of the khoa layers, a specification of the d_{50} can be made, which will be in the range of one cm diameter.

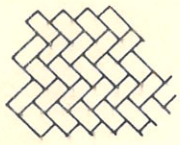
The first (sand) layer should be of 0,10 m thickness, the second (gravel or khoa) layers should be 0,20 m thick.

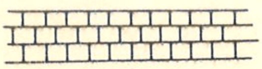
ANNEX III - 4.1 SLOPE PROTECTION IN CASE OF STRONG CURRENT AGAINST AN EMBANKMENT WITH SUFFICIENT FORELAND.



A sheet piling of wood, concrete
- toeboards against pegs (A_1)
- concrete planks (A_2)
- row of pegs (A_3)

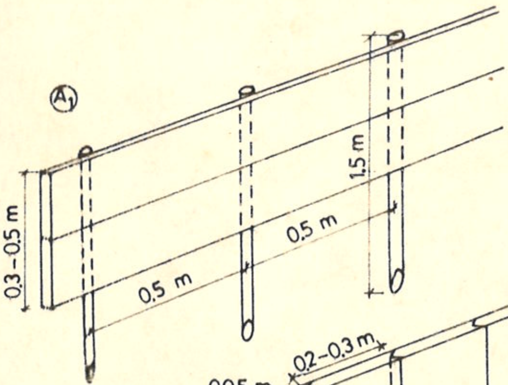
B brick pitchning (upright) direct on clay in a tight pattern. joints to be washed with clay.





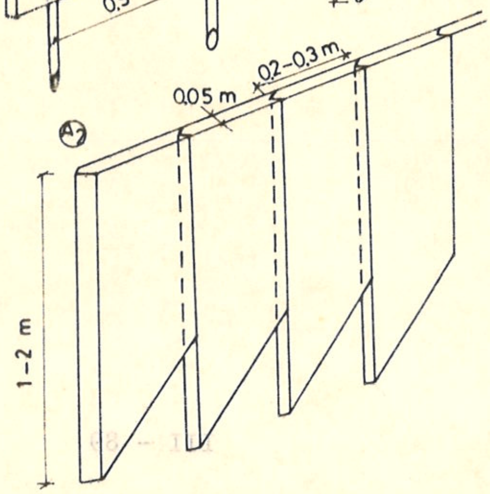
C concrete lock beam with light reinforcement.

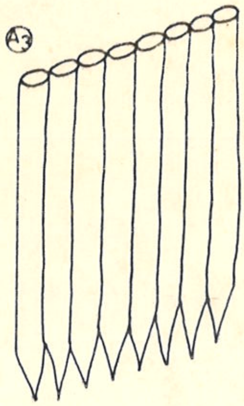
D not steeper than 1 : 1.5



height of boards depends on thickness of protection layer.

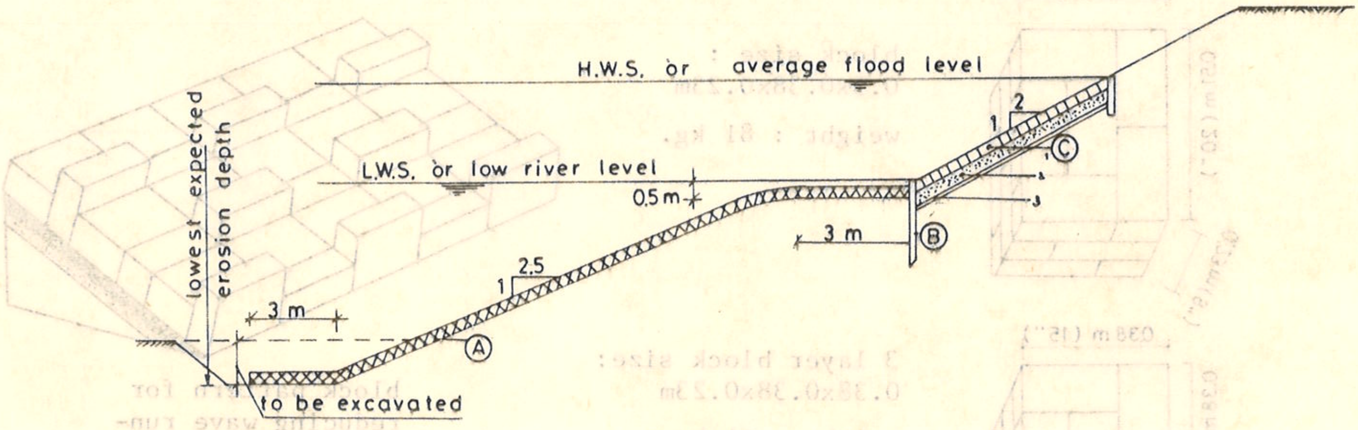
pegs Ø 0.10-0.15m.





hard wood pressure treated pegs, length 1.5-2.0m Ø 0.10-0.15m.

ANNEX III - 4.2 SLOPE PROTECTION IN CASE OF CURRENT AND WAVE ATTACK AGAINST EMBANKMENT WITH NO FORELAND.



A Mattress (bamboo/reed) with reed rolls or wattles to retain ballast. Ballast units 10-80 kg apiece, weight to be determined with Figure III - 5.7 and -5.8 of Volume III. Total required weight per m^2 is 300 kg.

For ballast units can be used:

- gunny bags filled with 1:8 sand/cement mixture,
- gunny bags filled with bricks/brickhalves,
- broken brickblocks,
- concrete units,
- natural stone (broken),

to be dumped from a barge.

B Anchor peg row plus toe-board or sheetpiling, (see Annex III - 4.1 (a).)

C Protection layer on filter construction,

C_1 : armour units
C_2 : 0.20 m khoa
C_3 : straw or fine bamboo mat, or 0.10 m coarse sand.

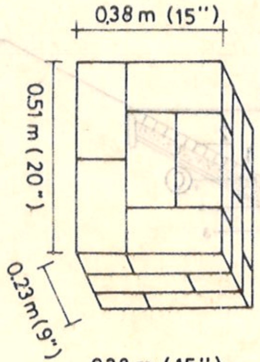
C_1 : armour units:

- brick blocks of various sizes
- concrete blocks,

both to be placed tightly together in a close pattern with minimum joints.

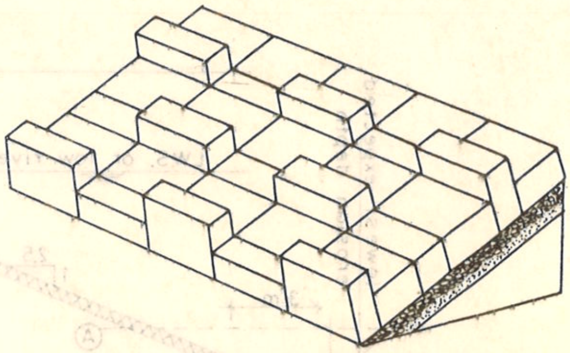
ANNEX III - 4.3 BRICK BLOCKS FOR SLOPE PROTECTION

Assumed size of one brick: (gross) 10"x5"x3"
Weight : 4.5 kg.

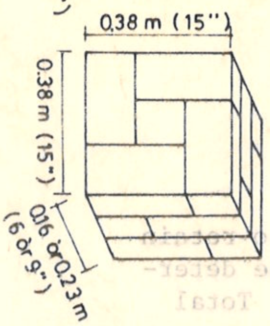


block size :
0.5x0.38x0.23m

weight : 81 kg.



block pattern for reducing wave run-up.

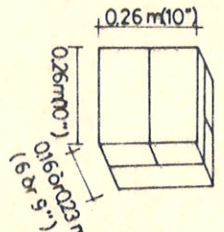


3 layer block size:
0.38x0.38x0.23m

weight : 61 kg.

2 layer block size:
0.38x0.38x0.16m

weight : 40 kg.

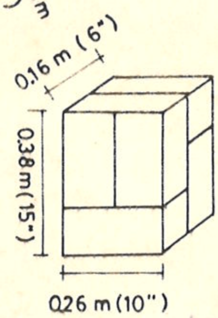


3 layer
block size :
0.26x0.26x0.23m

weight : 27 kg.

2 layer
block size :
0.26x0.26x0.16m

weight : 18 kg.

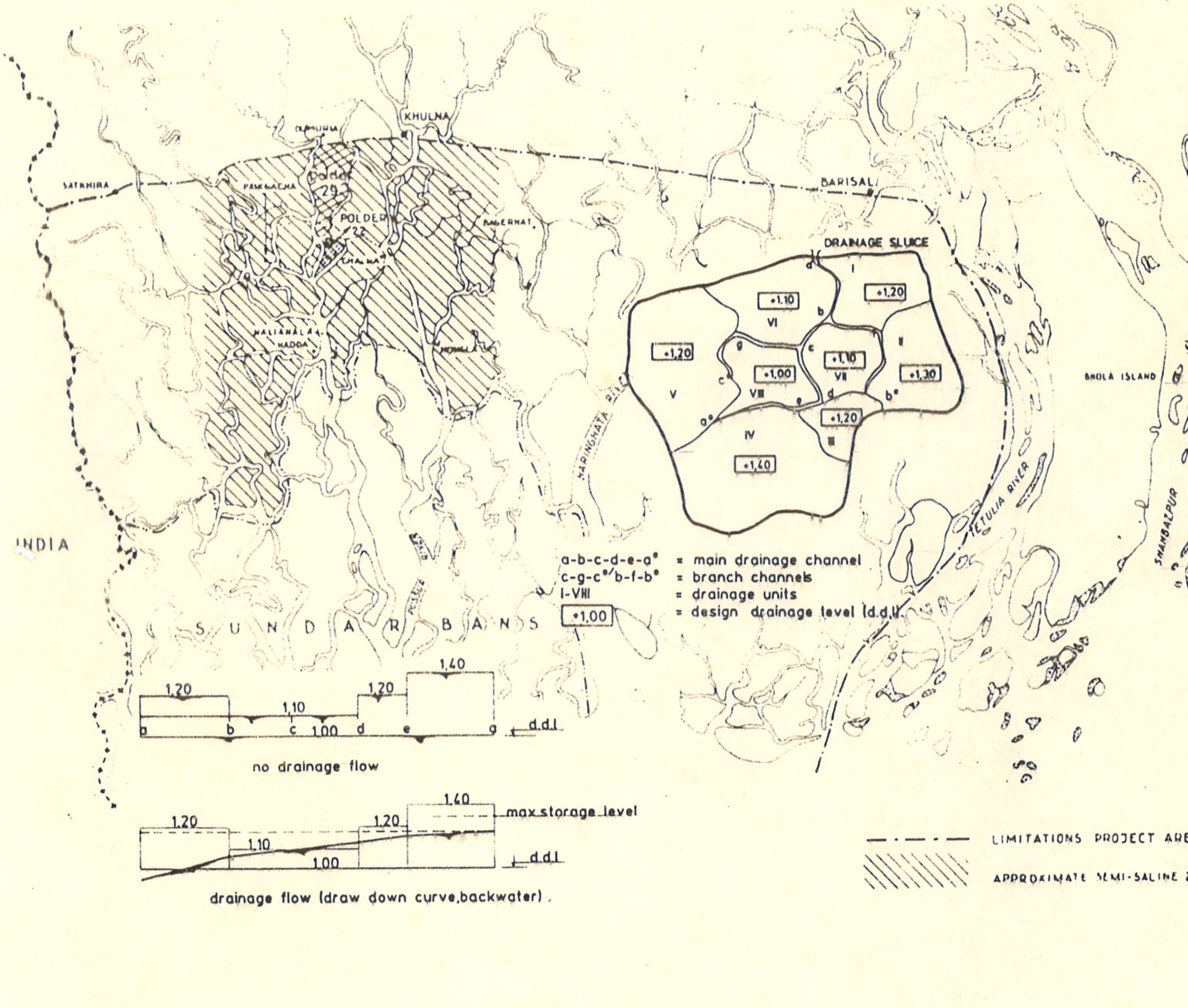


block size :
0.26x0.38x0.16m

weight : 27 kg.

DESIGN MANUAL

FOR POLDERS IN SOUTH-WEST BANGLADESH



VOL. IV

DELTA DEVELOPMENT PROJECT
BANGLADESH-NETHERLANDS JOINT PROGRAMME
UNDER BWDB

DHAKA
NOVEMBER 1985

VOLUME IV IRRIGATION AND DRAINAGE REQUIREMENTS - DESIGN CRITERIA

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Chapter 1. Introduction

1.0 General

Optimal growing crops consume large and predictable quantities of water. In case this water demand is not satisfied the crop responds with a yield reduction. During the dry season all water must be supplied by irrigation. During the rainy season most of the water requirements are satisfied by rainfall and only supplementary irrigation is called for.

In case of too much water the crop suffers as well, so a drainage system must be provided. In the case of dry season irrigation the drainage system should also assure groundwater control. During the rainy season salinity is no problem and only surface water control is required.

Because of the considerable costs of complete irrigation and drainage systems, their implementation is not justified in the case of supplementary irrigation. This is the case in such areas in the delta where the river water during the dry season is too saline for agricultural use. Because of the cost aspect irrigation and drainage in the delta must be by gravity by making use of the tides in the rivers.

1.1 The purpose of a drainage system

The purpose of a drainage system is the evacuation of excess water and to keep the water below a certain level.

In the low lying flat deltaic area during excess rainfall in the monsoon period, when rice is the predominant crop, the excess water will have to be evacuated by a surface drainage system. Surface drainage is the evacuation of excess water over the ground surface to an open drainage system.

In Volume II Chapter 5 it has been mentioned that it may be necessary to combine the shallow surface drainage system, which evacuates the excess rainfall, with additional drainage facilities to provide ground water control in order to reduce the salinization by capillary movement from the ground water. The water to be removed flows in this case through the soil into the open drains. The rather heavy soils with low hydraulic conductivity rates require a dense system of drains to provide an adequate subsoil flow into the drains that lowers the ground water table rapidly enough to prevent a capillary rise of the soil moisture. In general the implementation of such a dense system of drains is not feasible. In some cases, as for instance in Polder 22, a peat layer (with high hydraulic conductivity rates) exists at shallow depth and the distance between the drains could be increased but is still considered unfeasible in case of no irrigation possibilities in the dry period.

The case of irrigation throughout the year, which requires additional leaching of the salts which are added to the top soil during irrigation in the dry period, is not considered.

It should be mentioned that in the newly reclaimed areas in the coastal zone, temporary subsurface drains are installed in the initial phase for the first leaching of the fresh and highly saline deposits. These subsurface drains are made of natural local products such as bamboo, and deteriorate rather rapidly. They are not meant to serve as a permanent subsurface drainage system. This matter is outside the scope of this design manual.

1.2 The purpose of an irrigation system

The purpose of irrigation is to supply each field in the polder with the right amount of water at the proper time. Generally farmers are keenly aware of the consequences of having not enough water, however the consequences of over-irrigation are less well understood.

Proper watermanagement in the polder is necessary to avoid the situation of too much water in one part and not enough water in another part with consequently yield reductions over the full area.

The irrigation system can be :

- only for supplemental irrigation in the monsoon period (especially at the end of this season)
- for both supplemental irrigation in the monsoon period as well as full irrigation in the dry period in polders where suitable irrigation water is available around the year.

This manual discusses only the case of supplementary irrigation in the delta where river waters are only fresh during a certain period of the year.

1.3 Implementation problems

The fully cropped polders and the scarcity of land complicates the extensive construction of new canals, ditches and bunds. The compensation for the required land can not replace the property lost, no matter how high the compensation may be, because there is no other land available. The problem why certain people should suffer to improve their irrigation or drainage facilities for the benefit of the whole community or only for the benefit of other people,is difficult to solve.

The fragmentation of the land makes that many plots do not have a direct connection with a tertiary canal. The above mentioned problem makes it difficult to implement a system of farm ditches to connect each single plot with the tertiary system. However, in the areas with rice cultivation there is actually hardly any need for a quarternairy distribution system. The paddy fields themselves serve as conduits of water. It is only in the areas where other crops are cultivated that quarternairy ditches are required and problems may arise.

Experience has learned that for the relatively flat polders the topographical maps, how detailed they may be, can not be completely relied on to serve as the only basis for the lay-out of an irrigation and drainage system. Moreover, in the fully cropped polders there is already a farming system with an established lay-out and water management system and one should be very careful interfering with this system. It requires extensive field visits and interviews with the farmers to get a good picture of the shortcomings of the existing system and to get an idea of the actual requirements. This type of information is a pre-requisite for the implementation of improvements in the irrigation and drainage system; the importance cannot be over emphasized.

Chapter 2. Drainage Criteria

2.1 General

A good drainage system assures that the water level in the paddy fields does not rise above a prescribed water level and that the water level drops back to the original water level within a predetermined time period.

The permissible rise of the water level and its duration are dependant on the growth stage of the paddy crop as well as on the paddy varieties cultivated. These factors are discussed in paragraph 2.2.

Analysis of the rainfall frequency distribution is presented in paragraph 2.3.

In paragraph 2.4 both rainfall and allowable water level rise are combined to arrive at the drainage module to be used in the design of structures and watercourses.

2.2 Allowable water depth

A drainage system maintaining a constant water level under all conditions is not economically feasible. A certain rise in water level during a limited period should be allowed when designing the system. The allowable submergence depth and duration differ greatly according to the crop growing stage. In Table IV - 2.1 yield reductions are shown in relation to the flooding conditions.

Table IV - 2.1 Submergence of rice with clear water and the rate of yield decrease in % (Fukuda, Ref. 2).

Crop growth stage	Type of submergence	Flooding period (days) 1-2	3-4	5-7	7
20 days after transplanting	Completely	10	20	30	35
Young panicle formation	Partly[1)]	10	30	65	95-100
	Completely	25	45	80	80-100
Heading	Completely	15	25	30	70
Ripening	Completely	0	15	20	20

1) leave tips (9-15 cm long) remain above water surface

Experiments at BRRI (Ref. 12) with 7 days seedlings of many varieties (including local improved varieties as well as HYV's) exhibited a 80-100% survival rate after 12 days of submergence.

This is in general agreement with the data in Table IV - 2.1 where complete submergence of over 7 days at 20 days after transplanting results in a yield loss of 35%, while at panicle formation the same submergence duration results in a yield reduction of 100%.

The yield reductions at less extreme partial submergence in the paddy fields are less well documented. However there seems to be general agreement that the optimal water depth for high yielding varieties is 5 to 10 cm and that a 8 day rise of the water level of 10 cm is acceptable. For the traditional varieties the optimal water depth is equally 10 cm and a 8 day rise of 30 cm is acceptable.

As complete submergence is not acceptable the critical period is the first month after transplanting because then the plants are still small. In the delta this is July/August for the traditional varieties.

2.3 Rainfall duration frequency

To determine the drainage requirements the rainfall duration frequencies have to be determined.

The method as presented in the "Draft Technical Guideline Rainfall Analyses, East Pakistant 1971" (reference 13) is used with updated rainfall data (1962-1983) for Khulna, Barisal and Patuakhali. In ANNEX IV - 1 the rainfall - duration curves are presented for these stations for the months of May, June, July, August and September.

An example may clarify this part of the analyses. The highest rainfall on one day in May 1934 is, for example, 216 mm; the highest rainfall on one day in May 1935 is 160 mm and so on. For every year in the period of recording the highest value of daily rainfall is selected. This gives for a period of recording of 36 years, 36 values of highest daily rainfall in May. These values of highest daily rainfall are ordered from smallest to highest value and given a rank number and plotting position as described in Volume III, Chapter 3.2.2. The same procedure is followed in determining the highest rainfall on 2, 3, 5, 10 and 15 consecutive days in May. The highest rainfall on 2, 3, 5, 10 and 15 consecutive days is found out of the moving totals. This means that for a period of, say 5 consecutive days, the totals of rainfall from 1 - 5 May, 2 - 6 May, 3 - 7 May ... . 27 - 31 May are computed. In this way, 27 values of 5-days' total of rainfall in the month of May are obtained. Out of those 27 values the highest one is selected. This gives for a period of recording of 36 years, 36 years of highest rainfall on 5 consecutive days in May. These values are also ordered and given a plotting position and so on. The results of the analysis described above, have been presented as graphs on a double logarithmic scale. The lines in each graph represent so-called rainfall duration curves. (see Annex IV - 1.).

Example:

From the graph of Annex IV - 1 (Barisal, May) it follows that :

a) a rainfall of 117mm or more in one day occurs once in 10 years on the average
b) The rainfall in 10 consecutive days which occurs once in 5 years on the average is 205 mm or more.

2.4 Drainage module

To estimate the design discharge or drainage module of the drainage basin we make use of the method*) as described in publication 16 of the Institute of Land Reclamation and Improvements (Ref. 1). In Figure IV - 2.1 the frequency distributions of 1, 2, 3, 5 and 10 day rainfalls with return periods of 2, 5, 10 and 25 year (the rainfall depth-duration-frequency curves) are plotted for Khulna from the data of ANNEX IV - 1. In paragraph 2.2 it is assumed that the maximum allowable water depth is about 200 mm during 8 days for short straw varieties. If we assume a normal water depth in the field of 100 mm for rainfed conditions, then the amount of water that can be temporarily stored without harming the rice plants is 100 mm for

*) for limitations and assumptions see Page IV-50

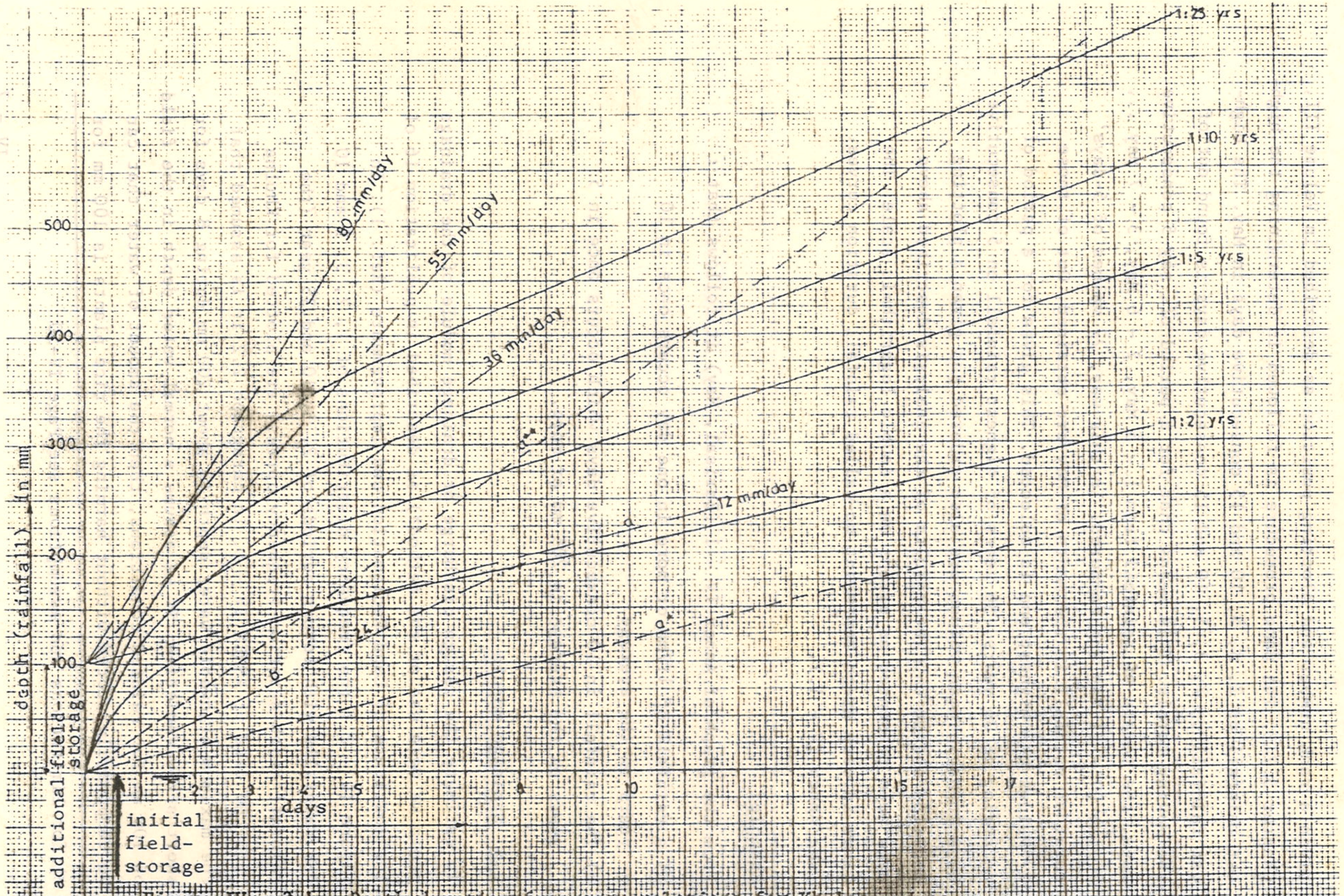

Figure IV - 2.1. Depth duration frequency relations for Khulna - June as derived from ANNEX IV - 1 and determination of the drainage modulus.

short-straw varieties. In areas where irrigation water is available in the first weeks after transplanting, this more reliable water source permits to maintain a more optimal normal water layer of only 50 mm. This results in an increase of the amount of water that can be temporarily stored to 150 mm.

To find the required drainage module for short-straw varieties at Khulna in relation to the return period, we draw tangent lines from the 100 mm point on the rainfall axis to the various duration curves. The slope of the tangent lines now represents the design discharge. Line 'a' shown for the 5 year rainfall in Figure IV - 2.1. With a 2-year return period this value is 12 mm/day; for a 5-year return period it is 36 mm/day; for a 10-years return period it is 55 mm/day and for a 25-year return period 80 mm/day.

To find the time required for the water level to return to normal the line paralel to the targent line and passing through the origin is drawn (a** in Figure IV - 2.1) where this line intersects the cumulative rainfall line the water level is back to its original depth of 100 mm.

With the drainage module of 36 mm/day this is nearly 8 days for the 5 year rainfall. This is in agreement with the 8 day criterium, and thus the 5 year rainfall with a drainage module of 36 mm/day is evacuated in 8 days. For a 2 year rainfall the line a* corresponding with a drainage module of 12 mm/day is intersecting the respective duration-frequency curve far beyond the 8 days. Therefore a new line b is to be drawn from the origin to the point where the duration-frequency curve intersects the 8 days line, yielding a drainage module of 24 mm/day.

For the drainage module of 36 mm/day, the 2 year rainfall is evacuated in about 4 days, the 5 year rainfall in about 8 days, the 10 year rainfall in about 11½ days and the 25 year rainfall in about 17½ days.

It should be remembered that all the water level rises as mentioned above are over the normal water depth which has been assumed as 100 mm. The rainfall depth-duration-frequency curves to be used for the determination of the drainage module should cover the critical month after transplanting. This month can be selected from the cropping calendar of the area under consideration. For a transplanted Aus crop this will be June; for a transplanted Aman this will be July or August.

It is recommended to use the 5-year return period curve.

For the short-straw varieties it has been determined that the amount of rainfall that can be temporarily stored is approximately 100 mm for rainfed conditions and 150 mm for irrigated areas. For the long-straw varieties with a maximum allowable water depth of 400 mm the amount that can be temporarily stored will be 300 mm for rainfed conditions, assuming a normal water depth of 100 mm. For the irrigated areas this amount will be 350 mm.

Table IV - 2.2 Estimate of drainage modules (mm/day) for Khulna, Barisal and Patuakhali for a critical period in June

	Storage (mm)	Return period 5 years	Return period 10 years
Khulna: short-straw varieties	100	36	55
long-straw varieties 1)	300	-	-
Barisal: short-straw varieties	100	37 1)	60
long-straw varieties 1)	300	9	18
Patuakhali : short-straw var.	100	71 1)	96
long-straw var. 1)	300	20	31

1) only if the 8 day condition is neglected.

Part of the tangent points for the estimates of the long straw varieties fall outside the figure. For the once in 5 year rainfall at Khulna the maximum rise will even stay below the maximum allowable depth for long straw varieties. If double cropping is practiced by growing a transplanted Aus followed by a transplanted Aman, the drainage requirement may be determined differently. The Aus crop has to be harvested during the monsoon period. It may be necessary to remove all rainfall within a few days to make harvesting possible within the limited period allowed for these operations. In this case it should be checked if the drainage module, as determined, in the previous section for the young rice can cope with the removal of the rainfall during the period fixed for the harvesting operations in the cropping calender.

2.5 Continuous and tidal drainage

To determine the drainage discharge of a channel section, the drainage module expressed in mm/day has to be transformed into a discharge rate expressed in l/sec.

The volume of excess water of 1 mm for an area of 1 ha is 0.001 x $10^4 m^3 = 10\ m^3 = 10000$ l. When continuous drainage is possible the value of 1 mm/ha = 10^4 l/ha can be transformed straight forward.

The evacuation time for continuous discharge per day = 24 hrs. = 86400 seconds. The drainage discharge of

$$1\ \text{mm/day} = \frac{10^4}{86400} = 0.116\ \text{l/s/ha}.$$

In case of tidal drainage when continuous drainage is not possible the drainage flow is blocked for some hours per day. For a drainage sluice the volume of water to be evacuated per day is :

drainage module (in mm/day) x area (in ha) x 10 m^3.

The average discharge during drainage in m^3/sec at the sluice will be:

$$\text{drainage module (mm/day)} * \text{area (ha)} * 1.16 * 10^{-4} * \frac{T_o}{T}$$

where To = tidal period in hours (12 hrs. 25 min.)

T = hours that polder water level is above tide level.

example: drainage module : 30 mm/day

area :1500 ha

To (tidal period) : 12,4 hrs.

T (drainage period) : 7,3 hrs.

Then:

average discharge through sluice during drainage when tide level is below polder water level:

$$\bar{Q}_{av} = 30 \times 1500 \times 1.16 \times 10^{-4} \times \frac{12,4}{7,3} = 8,87\ m^3/s.$$

Chapter 3. Irrigation Water Requirements

3.1 Introduction

This Chapter deals with the calculations of the irrigation water requirements for design purposes in general viz. the dimensions of the irrigation facilities, as well as the irrigation water quality. Since it concerns only rather small irrigation units, sophisticated and complicated simulation computer programmes for rice irrigation will not be considered in this manual.

In the transitional zone of the delta area, (part of the year fresh water, part of the year saline), irrigation is only possible during part of the rainy season. In general there is an adundance of rainfall during the monsoon but instances of inadequate rainfall and periodic droughts may occur, especially at the end of this season, and it should be investigated to what extent supplemental irrigation facilities in this period are required. The duration and frequency of droughts periods should be assessed for the determination of the economic viability, rather than the amount of dependable rainfall.

Irrigation Supply Requirements (IRS) are calculated as:

$$ISR = (ETc + Prc - Peff)/(Ef \times Ec) \quad \text{mm/month}$$

$$ETc = Kc \times ET \quad \text{mm/month}$$

where: ETc = crop water requirement (mm/month)

Kc = crop factor (dimension less),see paragraph 3.2.3

ET = reference evapotranspiration (mm/month), ,, 3.2.1

Prc = percolation (mm/month), - - - - - 3.4.2

Peff= effective rain (mm/month), - - - - - 3.3

Ef = field efficiency (dimension less), - - - 3.4.3

Ec = conveyance efficient (demensionless), 3.5

Furthermore the following terminology will be used:

Crop water requirement (ETc) refers to the water need of the crop. It includes transpiration of the crop as well as direct evaporation,of the soil, or the water between the rice plants.

Field irrigation requirement (FR = ETc + Prc - Peff) refers to the quantity of water to be applied at the head of the irrigation unit in order to meet the net irrigation requirement, as well as the water that is needed for the leaching and land preparation and to compensate for field application losses due to surface runoff and deep percolation.

Irrigation supply requirement (ISR) refers to the amount of water needed at the intake of an irrigation unit. Irrigation water requirements depend on the level of the water distribution process. The IRS consists of field requirements + distribution or conveyance losses i.e., canal leakage losses and/or operation losses.

3.2 Crop water requirements

3.2.1 Reference evapotranspiration

In conformity with the normal BWDB practice the Blaney - Criddle method is used. The resulting monthly reference evapotranspiration values for Khulna, Barisal and Patuakhali are presented in Table IV-3.1 In Chapter 3.2.2 the different methods to determine the reference evapotranspiration are discussed and the choice of the Blaney - Criddle method is suggested.

Table IV - 3.1. Reference evapotranspirations for three stations.

Month	1	2	3	4	5	6	7	8	9	10	11	12
Khulna	118	113	177	201	183	138	140	127	117	121	133	127
Barisal						138	146	130	117	124		
Patuakhali						129	133	121	114	130		

3.2.2 Selection of calculation method

Many methods have been developed to calculate crop water requirements or evapotranspiration by means of climatic data. In a recent FAO publication (Ref. 10) the following approach is recommended

$$ET_c = k_c \times ET$$

i.e. crop water requirement = crop coefficient x reference evapotranspiration.

The reference evapotranspiration is related to climate; the crop coefficient is an empirical factor depending on the kind of crop and its growing stage.

In the FAO publication, four methods are presented viz. the Blaney-Criddle, Radiation, Penman and pan evaporation method. Minimum input data for each method are given in Table IV - 3.2.

Table IV - 3.2 Minimum input data for various methods for the calculation of evapotranspiration.

Method	Temperature	Humidity	Wind	Sunshine	Radiation	Evaporation	Environment
Blaney-Criddle	x	0	0	0			0
Radiation	x	0	0	x	(x)		0
Penman	x	x	x	x	(x)		0
Pan						x	

x = measured data

0 = estimated data

(x) = if available, but not essential

If a complete set of meteorological data is available the Penman method offers the best results. Depending on the location of the pan, the pan method may be graded next. The radiation method, in extreme conditions, involves a possible error of upto 20 percent in summer, the Blaney-Criddle method should only by applied for periods of one month or longer; in humid, windy, mid-latitude winter conditions errors of up to 25 percent have been noted. Bangladesh has traditionally used the Blaney-Criddle method. IECO (ref.4) has compared the modified Blaney-Criddle, the Radiation and modified Penman method for Jessore and Barisal and selected the Blaney-Criddle method as being the most applicable for their study.

In the Agro-Climatic Survey of Bangladesh (ref.9) the Papadakis method has been used. This method gives values which are higher from November-March and much lower in the period June-September in comparison with the other three methods.

In view of the often limited availability of meteorological data, the simplicity of the method and the fairly good agreement with the other methods, the Blaney-Criddle method will be used in this manual.

3.2.3 Blaney-Criddle method

Reference is made to the FAO Irrigation and Drainage Paper No. 24 (ref. 10).

The original Blaney-Criddle equation involves the calculation of the consumptive use factor (f) from mean temperature (T) and percentage (p) of total anual daylight hours occuring during the period being considered. An empirically determined consumptive use crop coefficient (K) is then applied to establish the consumptive water requirements (CU) or CU = k.f = K.(p.T/100) with T in ^{o}F. CU is defined as "the amount of water potentially required to meet the evapotranspiration needs of vegetative areas so that plant production is not limited by lack of water". The effect of climate on crop water requirements is, however, insufficiently defined by temperature and day length; crop water requirements will vary widely between climates having similar values of T and p. Consequently the consumptive use crop coefficient (K) will need to vary not only with the crop but also very much with climatic conditions.

For a better definition of the effect of climate on crop water requirements, but still employing the Blaney-Criddle temperature and day length related f factor, a method is presented to calculate reference crop evapotranspiration (ETo). Using measured temperature data as well as general levels of humidity, sunshine and wind, an improved prediction of the effect of climate on evapotranspiration should be obtainable. The further on presented crop coefficients are considered to be less dependent on climate. (see Table IV - 3.4.)

The use of crop coefficients (K) employed in the original Blaney-Criddle approach is rejected because, (i) the original crop coefficients are heavily dependent on climate, and the wide variety of K values reported in literature makes the selection of the correct value difficult; (ii) the relationship between p(0.46T + 8) values and ETo can be adequately described for a wide range of temperatures having only minor variation in RHmin, n/N and U; and (iii) once ETo has been determined the crop coefficient k_c presented herein can be used to determine ETc.

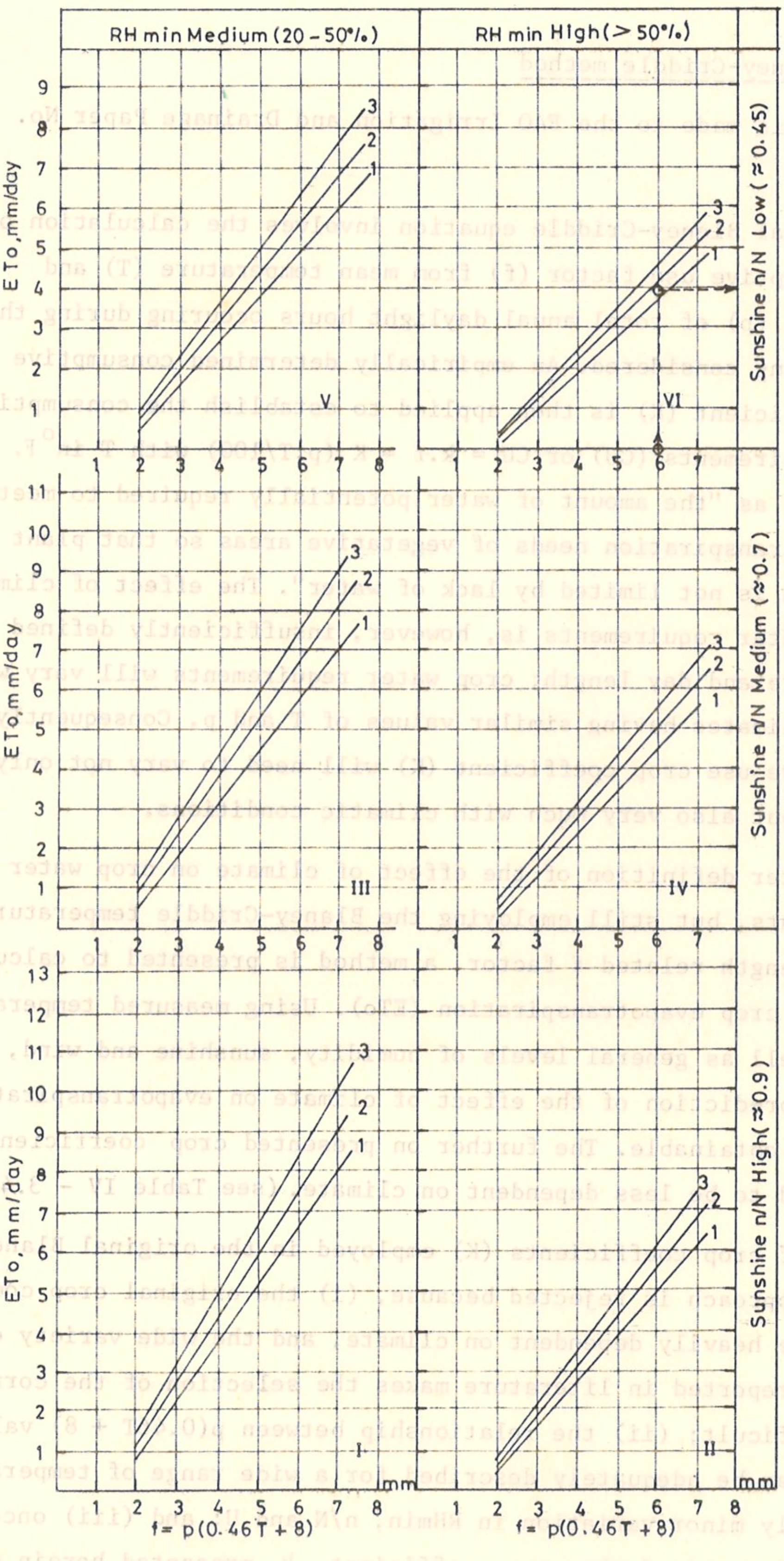

Figure IV-3.1. Prediction of ETo from Blaney-Criddle f factor for different conditions of minimum relative humidity, sunshine duration and day time wind.

3. U daytime = 5-8 m/sec (≈ 6.5)

2. U daytime = 2-5 m/sec (≈ 3.5)

1. U daytime = 0-2 m/sec (≈ 1.0)

The relationship recommended, representing mean value over the given month, is expressed as:

$$ETo = c \; [\; p.(0.46T + 8) \;] \text{ mm/day}$$

where: ETo = reference crop evapotranspiration in mm/day for the month considered

T = mean daily temperature in ^{o}C over the month considered

p = mean daily percentage of total annual daytime hours obtained from Table IV - 3.3 for a given month and latitude

c = adjustment factor which depends on minimum relative humidity, sunshine hours and daytime wind estimates.

Figure IV - 3.1 can be used to estimate ETo graphically using calculated values of p(0.46T+8). The value of p(0.46T+8) is given on the x-axis and the value of ETo can be read directly from the Y-axis. Relationships are presented for (i) two levels of minimum humidity (RHmin); (ii) three levels of the ratio actual of maximum possible sunshine hours (n/N); and (iii) three ranges of daytime wind conditions at 2 m height (Uday). Note that air humidity refers here to minimun daytime humidity and that wind refers to daytime wind. If estimates of 24 hours mean wind are available, these need to be converted to daytime wind. Generally Uday/Unight = 2 and mean 24-hours wind data should be multiplied by 1.33 to obtain mean daytime wind. For areas with either predominantly night or daytime wind, the following factor can be used :

Uday/Unight ratio	1.0	1.5	2.0	2.5	3.0	3.5	4.0
correction on Uday	1.0	1.2	1.33	1.43	1.5	1.56	1.6

After determining ETo, ETc can be predicted using appropriate coefficient (k_c), or ETc = k_c . ETo (1.2)

Table IV - 3.4 the crop coefficient k_c is being presented as selected from previously mentioned Irrigation and Drainage paper no. 24 (ref. 10) and the South West Regional Plan (ref. 6).

Table IV - 3.4. Crop coefficient k_c.

Month	Jan 1	Feb 2	March 3	April 4	May 5	June 6	July 7	Aug 8	Sept 9	Oct 10	Nov 11	Dec 12
HYV T.AUS				1.10	1.10	1.10						
HYV B.AUS				1.10	1.10	1.85	0.95					
Local T.AUS				1.10	1.10	1.85	1.00					
HYV T.AMAN								1.10	1.10	1.05	0.95	
Local T.AMAN							1.10	1.10	1.10	1.05	0.95	
HYV BORO		1.10	1.10	1.25	1.25	1.00						
Jute			0.60	0.90	1.20	1.20	1.10					
Wheat	1.05	1.00	0.50									0.60
Sugar cane	0.55	0.80	0.90	1.00	1.05	1.05	1.05	1.05	1.05	1.05	0.80	0.60
Rabi (winter crops)	0.95	0.70								0.90	0.90	0.90
Winter vegetables	1.00	0.70								0.90	0.90	1.00
Summer vegetables				0.80	1.20	1.00	1.00	1.00				
Fruits	0.85	0.80	0.78	0.78	0.87	0.93	0.80	0.75	0.75	0.82	0.93	1.02

Table IV - 3.3 Mean Daily Percentage (p) of annual daytime hours for different latitudes

Latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
35^o	.23	.25	.27	.29	.31	.32	.32	.30	.28	.25	.23	.22
30^o	.24	.25	.27	.29	.31	.32	.31	.30	.28	.26	.24	.23
25^o	.24	.26	.27	.29	.30	.31	.31	.29	.28	.26	.25	.24
20^o	.25	.26	.27	.28	.29	.30	.30	.29	.28	.26	.25	.25

Example:

Khulna latitude 22.8^oN. September

T = 28.9^oC

n/N = 0.47; low // $RH_{min} > 50$ %

U_2 24 hrs. = 1.5 m/sec

U_2 daytime 1.33x1.5 = 2.0 m/sec (between curves 1 and 2)

p from Table IV- 3.3. = 0.28

p (0.46 T + 8) = 6.0

ETo in Figure IV - 3.1, - VI → 3.9 mm/day

3.3 Effective rainfall

The effective rainfall is that part of each shower that is subsequently used by the crop.

In conformity with the BWDB practice we propose to use the 90% dry rainfall, that is the monthly rainfall exceeded 9 out of 10 years. These values are presented in Table IV - 3.5. The theoretically correct method would be to establish the water balance of the paddy fields at different elevations in each polder. At an agreed water level each field starts overflowing into the lower fields until the outfall sluice is reached. The effective rainfall is the water depth that can be stored at the beginning of each shower. This calculation has to be performed with daily rainfall figures for the whole historical time series. The amount of work involved requires a computer simulation that would become injustifiably complex because of the tidal influence on the polder drainage outlet.

Table IV - 3.5 90% dry rainfall for three stations.

Month	1	2	3	4	5	6	7	8	9	10	11	12
Khulna					19	143	162	242	73	30	0	
Barisal					43	174	247	191	149	39	0	
Patuakhali					81	279	330	324	161	52	0	

3.4 Field irrigation requirement

3.4.1 Leaching requirement

As has been described in Volume II, Chapter 5 a severe accumulation of salts may occur in the dry season due to capillairy rise from the (often saline) groundwater. A certain amount of leaching is required before cultivation of a crop can start. No specific figures can be given for these leaching requirements since they are dependant on many factors like depth and quality of groundwater, soil type, crop, planting technique etc.

As a rough approximation (reference 17) one may estimate that for initial leaching of 1 cm of topsoil one needs 1 cm water depth. For transplanting, the first 20 cm of topsoil must be reasonably salt free, so 200 mm of leaching (rain) water should be required.

3.4.2 Land preparation

a) Rice crops

The water required to till and puddle the soil and to establish and maintain a layer of water on the field consists of :

- initial losses of water through cracks to the subsoil
- water to saturate and puddle the soil and to establish a waterlayer
- water to balance the losses by evaporation and percolation.

It can be shown (see reference 18) that the water requirements in this period become minimal if the polder is subdivided in rotation units : The available discharge is used alternately to presaturate and submerge new fields and alternately to replenish the evaporation and percolation losses in the already flooded fields, in each of these units.

The mathematical formulation is :

$$\frac{I}{M} = 1/n \times \frac{1}{\sqrt{1 - e^{\left\{-\frac{MT}{S} * \left(\frac{I/M}{n * \frac{I}{M} + \frac{n-1}{2}}\right)\right\}}}}$$

Where:

I = supply required during the presaturation period. (mm/day)

M = supply required to maintain the water layer after presaturation and flooding is completed. (mm/day)

T = total duration of presaturation period. (days)

S = water required for presaturation and flooding. (mm)

n = the number of rotation units in the polder. (dimensionless)

For ease of calculation a monogram is presented in Figure IV - 3.2.

Example:

- presaturation takes place in June, so T = 30 days
- the amount of water to saturate the topsoil and establish a water layer is estimated as 200 mm, so S = 200
- the water use on transplanted fields is : ETc + percolation - effective rainfall = 152 + 30 - 144 = 38 mm, so M = 1.27 (percolation estimated as 1 mm/day)
- the polder is subdivided in more then 5 units, so n > 5

So : $\frac{M \times T}{S} = 0.19$

From the nomogram : $\frac{I}{M} = 5$ So : I = 6.5 mm/day

= 195 mm/month

Normally the field losses during this period are neglected, but the conveyance losses must be taken into account, so the intake water requirements are 195/0.8 = 244 mm/month.

For the traditional varieties transplanted in July/August the effective rainfall exceeds the water requirements, so the value

Dimensionless graph of the formula: $\frac{I}{M} = \frac{1}{n} \cdot \frac{1}{\sqrt{1 - e^{-\frac{M.T}{S}\left(\frac{\frac{I}{M}}{n.\frac{I}{M} + \frac{n-1}{2}}\right)}}}$

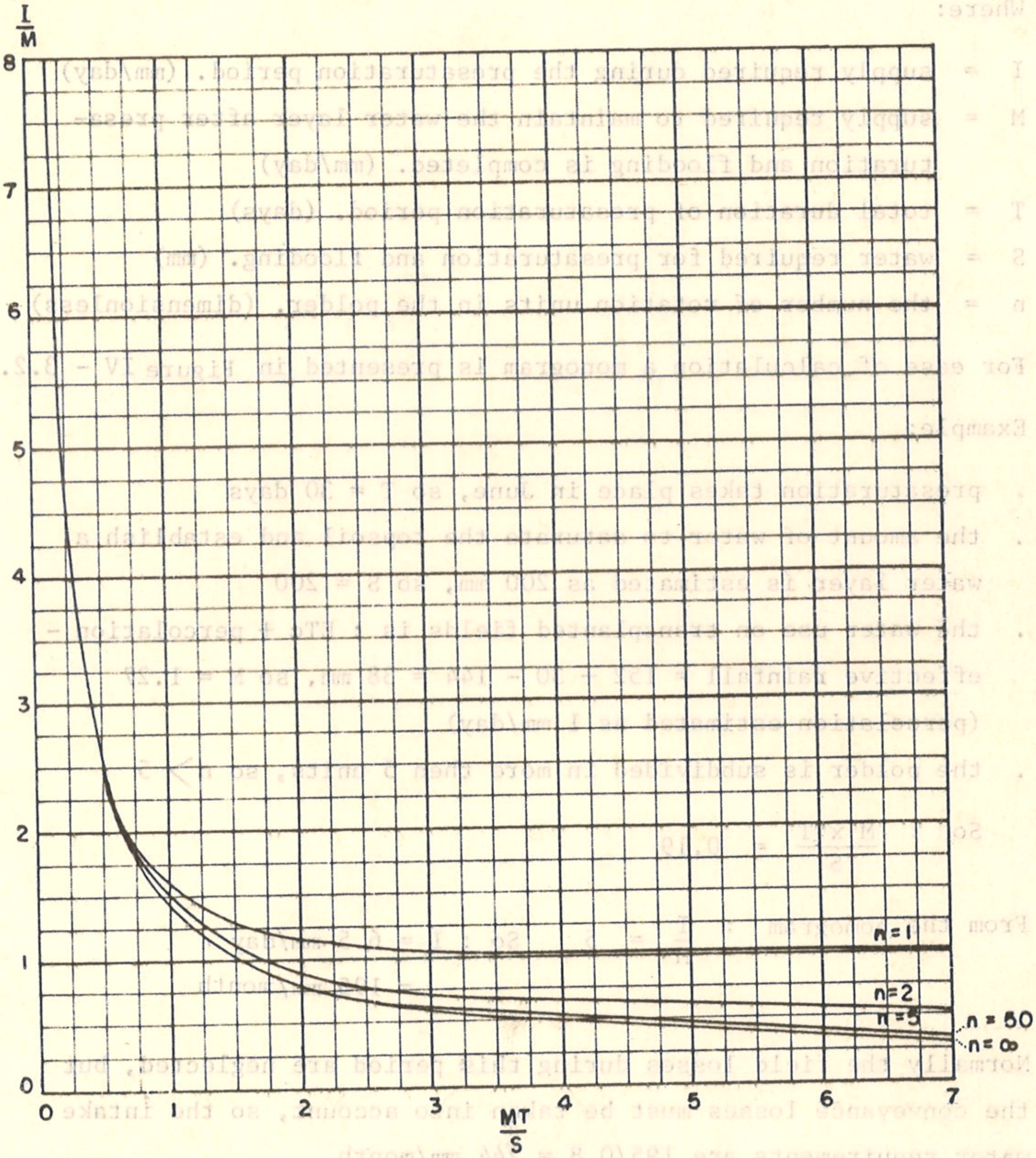

Figure IV - 3.2. Required supply of irrigation water for rice in case of rotating units.

M = 0 and the equation becomes invalid. The water requirements for land preparation now can be estimated as follows :

For Khulna, Local T. Aman:

month	ETc (Table IV-3.1) (Table IV-3.4) mm	perco-lation mm	saturation +flooding mm	rain 90 % dry year mm	water requirements[1)] mm
July	154	31	200	162	161
August	140	31	200	242	72

1) waterrequirements are estimated as:

$$\frac{2}{3}\ \text{ETc} + \frac{2}{3} * \text{Percolation} + 200 - \text{Rain}$$

The factor $\frac{2}{3}$ takes into account that only part of the area is flooded, and thus evaporating, during part of the month.

b) Other crops.

For land preparation a prewatering with about 75 mm is required.

3.4.3 Field application losses

a) Rice crops

It is assumed that in the low lying polders with high groundwater tables the subsequent percolation losses after the initial losses will be negligible. With high cropping intensities which is the case in the greater part of the project area, surface runoff is often not a loss because the water flows to a neighbouring plot. Therefore the field losses are estimated at only 20%; the field application efficiency is 0.8.

b) Other crops

Most dry-land crops will be grown in small basins and a rather high field efficiency may be expected. The field application losses are estimated at 30%; the field application efficiency will be 0.7.

3.5 Conveyance losses

The factors affecting the conveyance efficiency are, amongst others, the size of the irrigated area, the method and control of the operation and the type of soil and the depth of the groundwater table with respect to the seepage losses. The size of the irrigated areas

Table IV - 3.6 Example for the calculation of irrigation requirements, Polder 22

	J	F	M	A	M	J	J	A	S	O	N	D
Rainfall (mm) Khulna, mean	14.6	22.3	40.9	90.6	179	309	351	352	250	146	19	6.3
effective (90% probability)					19	143	162	242	73	30	0	
Evapotranspiration (B & C)	118	113	177	201	183	138	140	127	117	121	135	127
Trad. Varieties high land									110 days			
Trad. Varieties low land									127 days			
HYV's									110 days			
Trad. varieties												
average k_c							1.10	1.10	1.07	1.05	1.00	0.95
land use (%)							20	100	100	100	60	30
ET crop							31	140	125	127	81	36
land prep.						200						
storage								+75	+75		-150	
net irr. req.								-27	127	97		
Field irr. req.								-34	159	121		
Irr. supply req.								-42	198	152		
HYV												
average k_c							1.10	1.10	1.05	1.00	0.95	
landuse (%)							30	95	100	100	50	
ETcrop							46	133	123	121	64	
land preparation						200						
storage *)								+50	+50		-100	
net irr. req.								-59	100	91		
Field irr. req.								-74	125	114		
irr. supply req.								-92	156	142		

E crop = Evapotranspiration x k_c x landuse %

net irr. requirement = E crop - effective rainfall + storage

field irr. req. = net irr. req ÷ 0.8

irr. supply req. = field irr. req. ÷ 0.8

*) Storage = building up of water layer (positive) or end of season depletion (negative).

will, in general, be rather small (see Chapter 6.2). The conveyance losses are estimated at 20%; the conveyance efficiency will be 0.8.

3.6 Irrigation supply requirements

Once the cropping pattern and cropped area's have been selected, the net irrigation requirements have been calculated and the efficiencies of the irrigation system have been estimated, the monthly seasonal and yearly supply requirements for a given irrigation unit can be determined.

In Table IV - 3.6 an example is given for the calculation of irrigation requirements in Polder 22.

3.7 Irrigation water quality

3.7.1 Introduction

The amount of dissolved salts in water determines its suitability for irrigation purposes.

The characteristics which determine the classification of irrigation water are:

- total concentration of soluble salts
- relative proportion of sodium to other cations
- concentration of boron or other trace elements which may be toxic.

3.7.2 Evaluation of irrigation water

- Salinity

  The total salt content is often expressed as electrical conductivity (EC) in micromhos(= micro Siemens) per cm at $25^{o}C$.

  A general classification according to the electrical conductivity is presented in Figure IV - 3.2 (ref. 14).

  The effect of salinity on plant growth is directly related to the salt concentration of the soil solution in the root zone.

  Permissible salinity of the irrigation water very much depends on the irrigation frequency.

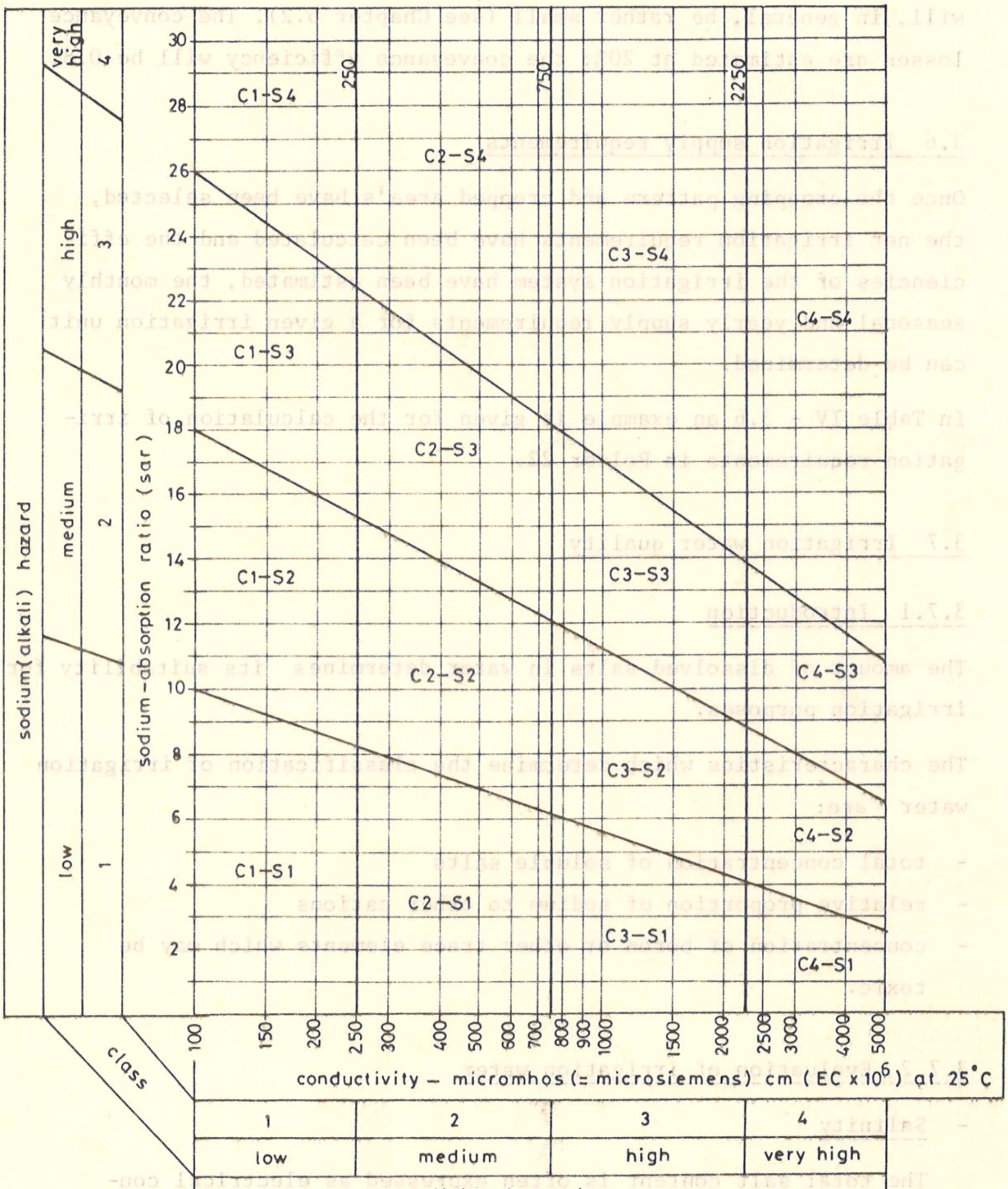

Figure Iv - 3.2. Salinity / sodium classification.

With intermittent irrigation of the non-rice crops the soil dries out in between two water applications, resulting in a reciprocal increase of the salt content of the applied irrigation water. Permanently flooding of paddy, however, results in a minimum of salt accumulation if sufficient water is used to permit return flows to the drainage system. The salt content of the soil solution under this condition would be about the same as the salt content of the applied water. The sensitivity of rice to soil salinity also varies upon the stage of growth. In Table IV - 3.7 the relatively paddy tolerance to soil salinity at various growth stages is indicated.

Table IV - 3.7 Salt level at which rice yields begin to be reduced (ref. 14)

Growth stage	% salt in soil 1) heavy clay 2)	% salt in soil 1) lighter soil 3)	EC saturation extract ($25^{o}C$)
At transplanting	0.10	0.05	2000 micromhos/cm
Tillering	0.25	0.13	5000
Booting stage before panicle emergence	0.75	0.35	15000 "
After panicle emergence	1.00	-	>15000 "

1) by weight
2) 75 volume % saturation
3) 37.5 volume % saturation.

Rice is tolerant during germination but young seedlings are sensitive. Damage to the rice plant during handling at transplanting increases its sensitivity to salinity and yields are reduced at this stage by application of water with salinity only one-tenth of that which would cause reduced yields after panicle emgergence; the paddy tolerance to salinity gradually increases as the plant matures.

In the seedling and transplanting stage water should not be admitted when the electrical conductivity of the river water exceeds 2000 micromhos. Only in case the crop shows effects of