A comparative study and application of revetment design methods

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SYNOPSIS

A Comparative Study and Application of Revetment Design Methods
by
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This study was carried out under the auspices of the International Institute of Hydraulic and Environmental Engineering, Delft as a partial fulfilment for the award of the M.Sc degree of the Institute.

The study embodies a critical assessment and evaluation of different revetment types as solution to bank problems. The efficacy and usefulness of these revetment types and the associated empirical design methods (incorporating hydraulic boundary conditions and load parameters) were checked with application to a particular case study in the delta region of Nigeria. The use of modelling approach as a design tool for the design of placed concrete block revetment was also partially verified in the study.

The contents of the study can be summarily viewed from three perspectives for quick appraisal. Chapter 1 which describes the stated objectives and scope of the study can be referred to as the first part. The subsequent chapters 2 and 3 (may be referred to as second part) are essentially theoretical description of the fundamentals of stability of a bank, failure modes, the predominant boundary conditions and load parameters, design criteria and the existing empirical/deterministic and modelling design methods to derive the structural parameters of both permeable and impermeable revetment types. All these are discussed. The remaining part which contains chapters 4, 5, 6, 7 and Appendix I involves the technical analyses and empirical designs for three revetment types namely Rip-rap, Open stone asphalt and Placed block revetments, using the theory and guidelines of chapters 2 and 3, were applied to the particular case study. The main loading conditions are predominantly hydraulic. An optimum design condition based on load and cost variations was derived as a factor for selection and justification for a particular revetment type for the case study. The modelling design methods (Steenzet/1 and Anamos) were also applied and compared with the empirical design methods. The conclusions are specifically highlighted in chapters 4, 5, 6 and 7.
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INTRODUCTION

1.1 General

The need for the basic requirements of human life which are partially provided by rivers has been the propelling force behind the study and understanding of river engineering by man since early civilization. River engineering can be defined as the design, construction, maintenance, operation and management of river and coastal defence systems. It involves river improvement works, the objectives of which are to aid navigation, to prevent flooding, to reclaim or protect land, or to provide watersupply for irrigation, hydro-power development, or domestic and industrial use. An important branch of river engineering is the river regulation work which involves the modification and adaptation of the original channel dimensions of a river, with an initial adoption of the river plan-form. This may be desired in order to provide sufficient depth and/or stabilize the river channel in a suitable form, and provide bank protection against wave action, particularly on constricted waterways.

1.2 Background

Generally, in different low-lying areas of the world, the protection against flooding and inland navigation is a major environmental concern. The use of hydraulic structures in form of sea coast dykes, canal and river embankment and other river training works is a common occurrence in such areas. The construction of these various works in some cases leads to channel region instability when adequate knowledge of the morphological process governing the regime channels are not understood. Of importance and relevance to this study, is the adverse natural bank erosion which is induced by these river engineering works. The understanding of the mechanism of bank erosion is an essential tool for the determination of the need for protective work and for the design of these works. Notably, bank erosion develops in different form such as gradual recession of the bank and a consequent loss of bank vegetation or cliff development (the frequent transition between the river bank and the groyne field beach). Occasionally, rapid erosion may occur during floods. Among related factors governing the erosion process are the frequency and duration of high current velocities caused by navigation and other flood. For the design of protection work, the river engineer needs to have an incisive knowledge in the interaction between river bank erosion, river flow, bed topography, erosive forces and the soil mechanical properties that characterise the ability of the river bank to resist erosion.
1.3 **Objective**

The objective of this study is to:

(a) identify the mechanism and failure modes of bank protection work,

(b) apply the existing semi-empirical / deterministic methods for the design of a flexible revetment structure for a particular case study; a coastal channel in the southern part of Nigeria,

(c) apply the STEENZET and ANAMOS computer models for a placed block revetment design and

(d) make a comparison between the deterministic and model design methods for a placed block revetment.

1.4 **Scope**

1.4.1 The scope of the study is limited to technical appraisal of fundamentals of failure mechanisms of a revetment associated with hydraulic and geotechnical boundary conditions. Hydraulic loads caused by wind/ship induced waves (PIANC, 1987a guidelines incorporated for ship wave analysis) and current are given relative importance during load consideration for the design of the revetment types for the case study. Rip-rap, Open stone asphalt and Placed block revetment types are considered.

1.4.2 The study did not attempt to evaluate the environmental integrity of the designed and selected revetment type. Selection of the preferred revetment type was based solely on technical and optimum cost analysis. Furthermore, construction, maintenance and management aspects were not considered due to limited information and data.

1.4.3 The scope also covers the application of recently created computer models for comparison with the deterministic design and notably as a future tool for actual design or checking the inherent structural, material and stability characteristics of Placed block revetment structures.
CHAPTER 2

2

FUNDAMENTALS OF STABILITY OF REVETMENTS

2.1 General

The cut-offs and bends in rivers usually originate from bank recession. River banks can be characterised according to the degree of exposure to erosive factors and these are the upper and lower banks (see fig. 2.1). The lower bank which is the part below water is highly susceptible to erosion which is usually caused by high turbulent flow velocity. The lower bank also acts as a foundation to support the upper bank.

![Fig 2.1: Definition sketch of a bank](image)

The upper bank can be described as the portion between low water and high water and it is subjected to much stronger parallel and perpendicular flow current and erosive surface run-off. Recession leading to total collapse of the upper bank is usually activated by the erosion of the lower bank particularly at the toe. The protection against such bank recession requires the use of bank protection methods in form of natural bank protection, vertical bank and sloping bank revetments. The revetment structure as used in this text is defined as a layered system of cladding or covering which is constructed on a sloping soil bank to protect and stabilize its surface against erosion by current and wave action.

2.2 Components of a revetment

A typical revetment consists of the armour layer and the underlayer as shown schematically in fig 2.2 below. The performance of the revetment structure depends on:
(a) functional requirements (i.e. the loads),
(b) the nature of the subsoil and
(c) the effectiveness of the crest, toe and edge construction of the sub-soil structure.

![Diagram of a typical revetment]

Fig 2.2: Components of a typical revetment.

2.2.1 Armour Layer

The armour layer (sometimes referred to as cover layer) provides protection against the direct erosive forces of current, wave action or other external loads. It also confines and shields the underlayer materials partially or fully (depending on its permeability) from the hydraulic load created by current or waves.

2.2.2 Underlayer

This is the layer between the armour layer and the subsoil formation. It usually comprises of granular materials or a geotextile, or a combination. The selection or type of materials in the underlayer is essentially based on functional requirements of the revetment structure as a whole.

2.3 Types of revetments

Revetment types are selected or designed after careful consideration of the bank protection method to be employed and potential hydraulic load to be resisted. Selected type may be based on several factors amongst which are permeability, strength, durability, flexibility, environment, economic and construction techniques. Generally, various forms of revetments have been used for different protection works in areas such as sea-coast, lake, rivers, canals (shipping, irrigation, drainage e.t.c.) and other protection works (see Fig 2.3(a-f)). Apart from the several factors mentioned for the basis for selection of a revetment type, two main types of revetments can easily be distinguished namely permeable and impermeable.
2.3.1 Permeable Revetment

The permeable revetment usually has a partially or fully porous armour layer such that water flows into and out of the bank structure. The permeable types generally incorporates the use of one or more layers of filter materials to retain soil particles. During the movement of water into and out of the structure, the head loss the armour layer or underlayer is strongly dependent on the relative permeabilities of the components. Essentially, the use of a filter (synthetic or granular) is employed for the case of sand underlayer or bank. But in situations, where permeable revetment units are placed on clayey banks, the use of a filter can be eliminated or confined to a synthetic filter. Examples are rip-rap or rock armour, hand-pitched stones, gabion mattresses, artificial block revetment, grass-mat on a permeable sub-layer and open stone asphalt revetment. Fig 2.4 illustrates the effects of different combinations of high and low permeability in the armour layer, underlayer and subsoil for the condition of high groundwater level in the adjacent bank draining into the channel.

2.3.2 Impermeable revetments

These types are water-tight such that water is not allowed to move
into or out of the bank. The flow field on this type of revetment is largely external and the behaviour of the revetment is strongly influenced by the hydraulic pressure forces built up behind it. The armour layer is normally designed to resist impact forces due to wave action and surface drag forces. Example of this type of revetment are grass-mat on an impermeable sub-layer, bitumen grouted stone, block or slab revetment and dense asphalt revetment.

![Diagram of hydraulic gradient in bank with steady groundwater](image)

Fig 2.4: Effect of relative permeability of revetment and subsoil on hydraulic gradient in bank with steady ground water (after van Zanten, 1986)

2.4 Processes and modes of bank failure

2.4.1 General

The determination or specification of any revetment type for a bank generally requires the understanding of the physical processes and the combined effect of active and reactive forces which are generated during the mutual interaction between the bank structure and the physical environment. An approach to the description of the various processes involved in the mechanics of bank protection is the soil-water-structure (SOWAS) concept (Kolkman et al, 1988) in which a schematised interaction diagram is drawn to represent the interaction of the structure with the geotechnical and the hydraulic environment. Refer to fig 2.5 which illustrates the system response (centre box) of the structure (i.e. the revetment) to the combined effect of the external processes (i.e. the soil and water) on one hand and the interaction of soil and water on the other hand. Although the interface between soil and water is imaginary, the interaction phenomena can be described with mathematical equations. Also, the interaction between the structure and the individual or combined effect of water and soil can be represented with use of equation involving parameters for determining loads and hydrodynamic reaction forces.
2.4.2 Causes of Bank Failure

2.4.2.1 General

Bank recession may be induced by natural forces due to hydraulic or geotechnical loads, or artificially by the activities of man. Such causes can be listed as:

(a) removal of soil particles from the bank surface by impact from raindrops
(b) sheet and rill erosion by surface runoff flowing down the bank slope
(c) removal of soil particles from the bank by wave forces
(d) erosion of the surface due to frictional force induced by tangential flow
(e) impingement eddy erosion due to curvilinear flow and eddy current respectively
(f) sliding of bank along failure planes due to increase of slope as a result of scouring
(g) undermining of the toe of the lower bank with a resultant collapse of upper bank due to lack of support
(h) pipping due to seepage of ground-water flow into the channel
(i) sliding of bank slope due to liquefaction caused by long duration flood
(j) collapse of bank slope under sudden draw-down condition
(k) erosion by mechanical action due to freeze-thaw, desiccation, boat impact/collision and destruction of vegetal cover by man and animals

A schematised view of these causes is shown in fig 2.6

Fig 2.6: Processes responsible for mass failure.

A further simplified approach to identification and quantification of the prominent failure mechanism is the use of the fault tree (Pilarczyk et al, 1988) which is shown in fig 2.7.

Fig 2.7: Simplified fault tree.
The fault tree approach is an important tool in decision making during research, design and execution of river bank projects. The macro-stability as indicated in fig 2.7 involves the global stability of the structure as a whole whereas the local stability indicates the stability of the bank under the influence of localised load. The appearance of the local stability may eventually lead to overall bank failure. The scope of this work is limited to the analysis of the stability of the revetment under the influence of local hydraulic loads such as flow current pressure uplift.

2.4.2.2 Simplified Failure Fundamentals

(a) Flow current: Influence of critical shear stress

The empirical criteria for the initiation of motion of bed materials in channels of non-cohesive and generally uniform materials developed by Shields (1936) which is shown below serves as a basis for the computation of the critical shear stress generated by flow current.

\[ \Psi = \frac{\tau_{cr}}{(\rho_s - \rho_w)gD} = \frac{U_{cr}^2}{\Delta gD} - f\left(\frac{U_{cr}D}{v}\right) = f(Re_c) \] .......(2.1)

where

\( \tau_{cr} \) = critical shear stress for particle motion
\( \rho_s \) = density of particle
\( g \) = acceleration due to gravity
\( D \) = soil particle size
\( \Delta = (\rho_s - \rho_w)/\rho_w \)
\( U_c \) = critical shear stress velocity
\( v \) = kinematic viscosity of water
\( Re = \) Reynold's number based on grain size and shear velocity

Refer to fig. 2.8a for the Shield's curve and fig 2.8b for the derived graph showing the relationship between the particle grain size and the critical shear velocity. Using a safe value for the Shield's parameter \( \Psi = 0.03 \), the following expression is deduced for stability of stone.

\[ \frac{\bar{u}_{cr}}{\sqrt{\Delta gD}} = 1.0 \log \frac{6h}{D} \] .......(2.2)
where

\[ u_{cr} = \text{critical mean flow velocity} \]

\[ h = \text{normal depth of flow} \]

Fig 2.8: Shield's diagram

Other empirical relationships developed by Isbash (1935), Goncharov, Levi and Maynord (1978) for stability of stone are shown in fig 2.9

Fig 2.9: Critical velocity for stone
Slope factor, $K$:

Generally the total bed shear stress is not fully mobilised on the bank slope and a slope correction factor, $K$ has to be applied. The value of $K$ for a channel bed consisting of non-cohesive materials can be determined by applying the tractive force theory. Consider Fig 2.10 in which a particle of weight, $W$ is lying on the side slope. We have shear forces $S_b$ and $S_s$ due to the current acting in the direction of the channel bed and slope respectively. On the slope

$$R = \sqrt{W^2 \sin^2 \alpha + S_s^2} \quad \ldots(2.3)$$

and for equilibrium condition

$$R \leq W \cos \alpha \tan \phi \quad \ldots(2.4)$$

Therefore substituting eq. (2.4) in eq. (2.3) and simplifying

$$S_s \leq W \cos \alpha \tan \phi \sqrt{1 - \frac{\tan^2 \alpha}{\tan^2 \phi}} \quad \ldots(2.5)$$

When $\alpha = 0$, $S_s = S_b$

Therefore

$$S_b = W \tan \phi \quad \ldots(2.6)$$

Fig 2.10: Forces acting on soil particle

The tractive force ratio or Slope factor, $K$ which defines the relationship between the critical shear force on the channel bed and bank is:

$$K = \frac{S_s}{S_b} \quad \ldots(2.7)$$

Substituting equations (2.5) and (2.6) in (2.7) then
The condition is if \( a > 0 \), then \( K < 1 \).

Figure 2.11 indicates the value of slope factor \( K \) as a function of the side slope \( a \) and the angle of repose \( \phi \).

![Slope factor K as a function of side slope a and angle of repose phi](image)

*Fig 2.11: Slope factor K as \( f(a, \phi) \)*

Baring other factors, it can be concluded that the amount of shear stress or force necessary to cause instability or lift a particle off the slope of a bank can be determined once the critical bed shear stress is determined.

### (b) Drop in water surface level: Influence of ground-water flow

Consider another cross-section of a channel as illustrated in fig 2.12a. It is obvious that the stability of the soil particles on bank and bed will be improved if water flows from the channel to the adjacent land whereas the stability will be endangered if the flow is vice-versa. In reality the occurrence of the later case is more probable and an instance is the drop of water level in a channel at the end of the wet season in which ground water flows towards the channel for a temporary period.

(i) Soil above water:

Consider soil particle \( X \) above the water surface in which the ground water flow is almost horizontal. This situation may occur under tidal circumstances or may be due to surface drop induced by navigation. The force due to the flow pressure is NOT dynamic but STATIC and it acts in the direction of flow.
Fig 2.12: Ground water flow towards a canal

It is equal to

$$\gamma_{wi} \text{ per unit volume of particle}$$

where

$$\gamma_{wi} = \text{hydraulic gradient per unit volume or the difference in pressure between the front and back of the unit volume (it can be estimated from flow net diagram)}$$

For equilibrium condition, consider component of forces as shown in fig 2.12(a-c)

Forces along the slope = $$(\gamma_s - \gamma_w) \sin \alpha + \gamma_w \cos \alpha \quad \ldots\ldots(2.9)$$

Forces normal to the slope = $$(\gamma_s - \gamma_w) \cos \alpha - \gamma_w \sin \alpha \quad \ldots\ldots(2.10)$$

The condition for no particle motion is:
but \( i = \tan \alpha \) since the phreatic line is on the boundary of soil-air. The expression can be simplified by assuming that

unit weight of soil, \( \gamma_s = 2 \)

unit weight of water, \( \gamma_w = 1 \)

Therefore eqn (2.11) becomes

\[
\tan \phi \geq \frac{(\gamma_s - \gamma_w) \sin \alpha + i \gamma_w \cos \alpha}{(\gamma_s - \gamma_w) \cos \alpha - i \gamma_w \sin \alpha} \quad \ldots \ldots (2.11)
\]

\[
de\alpha = \phi / 2 \quad \ldots \ldots (2.13)
\]

Equation (2.13) indicates the condition for the particle erosion not to occur above the water surface and this is

**ANGLE OF SLOPE \( \leq \) HALF THE ANGLE OF REPOSE**

For instance if \( \Phi = 30 \), \( \alpha = 15 \) (or 1:4 is the steepest slope allowable for no erosion)

(ii) Soil below water:
Consider soil particle \( Y \) below water surface. For this condition the direction of flow is normal to the bank and bed and it is obvious that the pressure gradient is largest where the ground water flow velocity is largest. Considering all forces normal and along the slope:

Forces along the slope = \((\gamma_s - \gamma_w) \sin \alpha \) \ldots (2.14)

Forces normal to the slope = \((\gamma_s - \gamma_w) \cos \alpha - \alpha_w i \) \ldots (2.15)

For equilibrium condition:

\[
\tan \phi \geq \frac{(\gamma_s - \gamma_w) \sin \alpha}{(\gamma_s - \gamma_w) \cos \alpha - \gamma_w i} \quad \ldots (2.16)
\]

and if \( \gamma_s = 2 \) and \( \gamma_w = 1 \), then equation (2.16) becomes

\[
\tan \phi \geq \frac{\sin \alpha}{\cos \alpha - i} \quad \ldots (2.17)
\]
Equation (2.17) indicates that the steepest possible slope \( \alpha \) for the bank structure under water can be determined if the hydraulic gradient is known. But this condition does not fully determine the stability of the soil particle under water since erosive process due to current parallel to the bank is involved under normal circumstances. The following relation is hereby deduced if the bank shear stress is considered:

\[
D_m \geq \frac{\beta}{\Delta} \frac{u^2}{2g} \left[ (\cos \alpha - \frac{i}{\Delta})^2 - \frac{\sin^2 \alpha}{\tan^2 \phi} \right]^{-0.5} \quad \text{...(2.18)}
\]

where

\[
\beta = \text{flow coefficient varying between 0.7 and 1.4 depending on degree of turbulence}
\]

\[
\Delta = \frac{(\rho_s - \rho_v)}{\rho_v}
\]

\[
\bar{u} = \text{mean flow velocity in channel}
\]

Equation (2.18) shows the relationship between a minimum particle size, \( D_m \) of a soil particle lying on a bank (for stable condition) and the steepest possible bank slope, \( \alpha \).

2.5 Load - Strength

2.5.1 Requirement

The behaviour of the revetment under loaded condition is highly complex and the failure modes and equation as described in paragraph 2.4.2 are not adequate enough to fulfil the design requirement of a revetment structure. Until recently the design of bank protection was based on experience, but unique circumstances, new materials and increased loading have resulted in a new approach towards design. The design of a revetment structure (armour layer and underlayer) must be able to recognise the characteristic nature of load with respect to size, variation and distribution in time and space and location, and at the same time established a strength parameter which takes account of the factors influencing the strength and variation of this strength along the structure. The load is generally equivalent to various hydraulic loads such as waves and currents while the strength may consist of the mass of the revetment and the frictional force. In essence and reality, the load strength parameters are not characterised by a single variable but are predominantly stochastic in nature. Presently, the method of design in which specific deterministic quantities are used is such that for the structure not to fail; the deterministic loads should be less than the deterministic strengths. The revetment is therefore "over designed" to function satisfactorily throughout its lifespan with preservation of the structural strength by means of regular control and maintenance procedure. In this regard the functional requirement to be satisfied by the design of the revetment structure are (according to PIANC, 1987):

(a) Stability --- the revetment must have enough resistance against
imposed loads (combination of wave and current attack and uplift pressure) and must have the necessary strength characteristic to resist displacement. And also it must retain and prevent the migration of underlying sub-soil particles. The whole revetment structure must be stable against sliding.

(b) Flexibility - the revetment must be capable of accommodating possible form changes (settlement and/or scour) of the bank slope without its structural bond being adversely affected.

(c) Durability - the revetment must not degenerate or suffer loss of function due to ageing and/or abrasive erosion due to materials being carried in the flows over it. Resistance to attack by chemicals, ultra-violet light, micro-organisms and vandalism are part of durability.

(d) Maintenance - the revetment design should put into consideration the need for quick and easy repair as a result of local damage and removal of degradable materials.

(e) Safety --- the design should eliminate potential hazards likely to be encountered by construction workers and at the same time incorporate safety feature to accommodate all activities (legal or illegal) that may take place on or about the bank by users.

(f) Social and environmental acceptability the blending of landscape and local ecological system coupled with social gains in form of employment (during construction and maintenance) must be ensured on a broader scale during the design.

(g) Cost - the design must satisfy all functional requirements within the desired life cycle.

The requirements (c)-(f) are special requirements which are not within the scope of this study. The characterisation of the requirements listed above with the boundary conditions as described below form the basis of design methods as described in the chapter 3.

2.5.2 Boundary Conditions

The technical decision as regards the design of a revetment structure usually incorporates the determination of boundary conditions since various processes are involved. The collection, interpretation and transformation of various data, and identification of design constraints are inherent factors leading to the establishment of the design parameters. A basic approach towards the establishment of the design parameters is the
estimation of all possible failure modes/risks with the use of fault tree (refer fig 2.13) as described as described at page 8. The fault tree can further be compressed into a basic scheme as shown in fig 2.13 below. The basic scheme in fig 2.13 describes concisely the expected boundary conditions/loads on the bank of an inland water-course.

![Fault Tree Diagram](image)

Fig 2.13: Basic scheme to assess bank protection response.

2.5.2.1 Environmental Condition

The environmental conditions are the natural hydraulic and geotechnical characteristics usually identified in front of the revetment structure or inside the sub-soil structure itself (i.e. dam, embankment or dike). These environmental conditions are not influenced by the structure. These environmental parameters may therefore be described as wave height, wave height distribution, wave period and length, wave breaking parameter, water-way geometry, ship type and speed, and current. Others are wind speed, fetch length, water depth, wind set-up and water levels. The knowledge and evaluation of these environmental conditions could easily lead to estimation of the hydraulic and geotechnical parameters.

2.5.2.2 Hydraulic Parameter

The stability design and dimensioning of a revetment is mostly governed by the variation and size of the hydraulic RESPONSE in front and inside the structure.

![Hydraulic Parameters Diagram](image)

Fig 2.14: Main hydraulic parameters
The major hydraulic responses are wave impact, wave run-up and run-down (short period, medium and long period waves) and flow current characterised by open-channel and ground-water flow. Refer to fig 2.14 for the main hydraulic parameters.

2.5.2.3 Geotechnical Parameter

These parameters are mainly related to overall bank failure resulting from liquefaction, dynamic gradients, excessive pore pressure, piping and settlement. Refer to fig 2.15. The full analysis of geo-technical failure modes is not within the scope of this study.

Fig 2.15: Type of mass failure of banks

2.2.5.4 Modelling approach

The determination of the hydraulic and geotechnical parameters needed for the establishment of the design criteria for a revetment involves a complex analysis because of the stochastic nature of the loads and it is still within the realm of research. It is sometimes difficult to interpret the result of the hydraulic phenomenon in a physical content especially when the solution is in the form of an infinite series or complex mathematical functions, or in the form of integrals of complicated function so that a computer solution is utilized. Therefore the need for modelling technique, to reproduce the behaviour of the complex hydraulic phenomenon on a different and more convenient scale can not be overemphasised.

Fig 2.16 illustrates a schematic representation for the development of a model for a revetment (Kontor, et al) in which Transfer Functions I, II and III are used to express mathematically the separate or combined interactions of the bank geometry with the hydraulic and geo-
technical phenomena. The solution of these transfer functions for the external and internal flow through the DOMAIN (bank protection) implies the determination of the RESPONSE, usually in form of head, hydraulic gradient and displacement distribution resulting from the EXCITATION which is in the form of boundary and initial condition acting upon the fully specified SYSTEM i.e. the investigated flow domain or field within the bank. Although the exact mathematical statements

![Flow Domain Diagram](image)

**Fig 2.16:** Schematic representation of a model development for a revetment.

for transfer function I, II and III of the flow problem may be set up, an analytical solution may sometimes be practically impossible because of the non-linearity of the problem. In such situation, relative satisfactory solution (for example maximum pressure difference and armour layer movement) may be obtained by dimensionally scaled models or analog in which active parameters like the leakage length are adjusted to have the same significance as in the actual problem investigated. Therefore the load-strength parameters, loading zones and boundary conditions for bank protection are still within the realms of physical and analytical research.

2.6 Conclusion

The various factors likely to affect the integrity of an unprotected earth bank had been presented in a concise form in this Section. The Section also gives the fundamental flow theories to assess and illustrate the significance of these destabilising factors during a preliminary analysis for a bank protection. Although, all failure modes, load-strength parameters and boundary conditions are not described completely, it is obvious that the mechanics-cum-design of a stable revetment requires an in-depth understanding by the designer. The subsequent chapters will highlight the actual load considerations and existing design methods.
3.0 LOAD PARAMETERS AND DESIGN METHODS

3.1 Introduction

The boundary conditions as defined by the different load characteristics (hydraulic and geotechnical) on a revetment generate a structural response which defines the stability of the revetment. The resistance offered by the revetment against the extreme load can therefore be represented by as the strength of the revetment structure needed to balance the acting extreme load. If the strength of the revetment is exceeded by the acting load, the revetment will thus fail. This procedure implies the Ultimate Limit State condition which in many instances is not real in nature because of the stochastic nature of the parameters involved. Presently, the established design methods are still subject to various investigation (physical and mathematical modelling) and clearly there are no rigid design criteria for a revetment, rather the design process is based on the variety of experience of the designer.

3.1.1 Design Approaches

The existing design philosophies oscillate between the Deterministic and Probabilistic methods and the ranges are as follows (PIANC, 1987a):

(a) Deterministic design method

This is the oldest and traditional design method and it involves the selection of load parameters that are assumed to be adequately high and thus safe. Often average values are selected for the strength properties with a factor of safety to cater for uncertainties. The choice of loading is mainly based on experience and some elements of quasi-probabilistic method is incorporated where practicable.

(b) Quasi-probabilistic design method

This method assumes a safe and characteristic values for all basic variables (load and strength). The characteristic load implies the load which has a 5% probability of being exceeded during the lifetime of the structure. Likewise, the characteristic strength properties are chosen values which have 5% probability of falling below the lower limit. Refer to fig. 3.1 for the sketch of the probability density. Partial safety coefficients are specified for the characteristic strength and load to account for possible uncertainties.
(c) Probabilistic method

This method involves the use of statistical approach to represent all basic variables in form of probability density functions. The probability of failure is then estimated from a reliability function derived from the probability density functions of the load and strength parameters. According to (CUR, 1989), the following points are essential for probabilistic design of a block revetment structure:

(i) the probability distribution of the high water level
(ii) the relation between wave height and high water level
(iii) the probability distribution of wave steepness
(iv) the wave-breaking criterion for wave breaking in front of the bank
(v) the model for describing the stability of the placed revetment
(vi) the probability distributions of the various parameters which determine the strength of the revetment, e.g. the slope angle, the block thickness and the resisting friction between the blocks.

In the remaining paragraphs, the current literature on design criteria for different types of revetments will be examined.

3.2 Design criteria

The selection of design criteria for a revetment structure involves a lot of preliminary studies and iterative processes. The establishment of the technical parameters is not enough and therefore must be considered along with other key factors such as functional, constructional, environmental and economic bases which may deviate or contradict the objective of the technical requirements. There are times that the selection of a
preferred system of revetment may be constrained by social acceptance, financial, management and maintenance consideration. Other factors such as legal, planning and execution may be obstacles. These factors are ignored and in the following paragraphs under this chapter, attention is mainly focused on technical requirements basically hinging on hydraulic load parameters and boundary conditions. Emphasis is placed on transfer functions from wave or current attack to hydraulic load and the corresponding structural strength derived from the revetments structures also encompassing the filter requirements. Although geotechnical requirement (already mentioned in chapter 2) is as essential as hydraulic requirement, the scope of this study is limited; and as such mention will only be made where necessary or if the geotechnical parameter is paramount or has an overriding consideration.

3.2.1. Loads

The hydraulic loads usually generated on coastal structure or revetment placed on dike are mainly due to:

(i) Water level variation: caused by tide, wind, flood or the combined effect

(ii) Wave attack: usually generated by wind or induced by ships

(iii) Current which can erode the bottom in front of a bank protection work and subsequently undermine the toe.

3.2.1.1 Waves

Waves are characterised by the wave height $H$, wave length $L$, period $T$ and speed of propagation $c$. Refer to figure 3.2.

```
<table>
<thead>
<tr>
<th>Description</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H$</td>
<td>vertical distance between trough and successive crest</td>
</tr>
<tr>
<td>$L$</td>
<td>horizontal distance between two successive wave crest</td>
</tr>
</tbody>
</table>
```

Fig 3.2: Definition sketch of a wave profile
\[ c = \frac{L}{T} \]

\[ T = \text{time between successive upward crossings of the still water level} \]

The profile shown in figure 3.2 is for a regular wave where \( H, L \) and \( T \) do not vary with time. But generally, sea waves are irregular and erratic. They are difficult to express in mathematical terms because of the non-linearities, three-dimensional characteristic and the stochastic nature. A wave statistical model can be accomplished:

(i) if the statistical parameters needed to characterise a set of a wave representing an interval of constant sea condition can be determined and

(ii) if the frequency of occurrence of the above condition can be determined.

The common statistical parameter which is convenient is the significant wave height, \( H_s \) and is defined as the average value of the highest one-third of the observed waves. It represents the wave heights of natural irregular waves and is established with the aid of the Rayleigh Distribution (theoretical model). Refer to figure 3.3. Rayleigh Distribution as shown in figure 3.3 is described by

\[ P(H_{\text{max}}) = e^{-\frac{H_{\text{max}}^2}{2H_s^2}} \quad ...3.1 \]

where

\[ P(H_{\text{max}}) = \text{the probability of exceedance of wave, } H_{\text{max}} \]

\[ H_s = \text{the significant wave height of record} \]

The chance \( P(1) \) that the wave height, \( H_{\text{max}} \) is exceeded at least once for a set of \( N \) waves, characterised by \( H_s \) is

\[ P(1) = 1 - \{1 - P(H_{\text{max}})\}^N \]

According to Pilarczyk, the problem of proper definition of the design wave height in design formulas is more urgent for structures which are much more susceptible to immediate or complete failure such as block revetments on permeable underlayer (filter) and that model test results have shown that the displacement of a block can result from the action of a single wave. Therefore revetments should be designed for the expected highest wave. Generally, the most probable value of \( H_{\text{max}} \) (in the Rayleigh Distribution) depends on length of record (storm duration) and the set of waves, \( N \) in which the revetment is exposed during the storm duration. A relationship between \( H_{\text{max}} / H_s \) and the number of waves \( N \) is expressed below
Fig. 3.3: Rayleigh Distribution

<table>
<thead>
<tr>
<th>(N)</th>
<th>(\frac{H_{32}}{H_s})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 : 10</td>
<td>1.07</td>
</tr>
<tr>
<td>1 : 100</td>
<td>1.52</td>
</tr>
<tr>
<td>1 : 1000</td>
<td>1.86</td>
</tr>
<tr>
<td>1 : 10000</td>
<td>2.15</td>
</tr>
</tbody>
</table>

Table 3.1: Relation of number of waves, \(N\) with ratio \(\frac{H_{32}}{H_s}\)

The use of table 3.1 and figure 3.2 indicates that the specification of the
design wave height is still ambiguous and needs to be properly defined. The Delft Hydraulics Laboratory investigation for irregular waves (with a narrow spectrum) concluded that design wave height $H$ should be taken as $1.3H_i$ (for $N = 3000$ waves).

3.2.1.1 Conclusion

Although there is no correlation for the destructive effect between a regular wave train, the design wave height used for a prototype design will need some modification if the wave height is derived from a test result of uniform wave.

3.2.1.2 Wind-generated waves

Wave types can be classified by the ratio of water depth, $h$ and deep-water wave length, $L_i$ (Airy - Laplace wave theory)

$$\frac{h}{L_i} \leq 0.05 \text{ for shallow water waves}$$

$$0.04 < \frac{h}{L_i} < 0.25 \text{ for intermediate waves}$$

$$\frac{h}{L_i} > 0.25 \text{ for deep water waves}$$

and

$$L_i = (gh)^{0.5}T \text{ for shallow water wavelength} \ldots (3.2)$$

$$L = g\frac{\tau^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right) \text{ for intermediate wavelength} \ldots (3.3)$$

$$L_o = g\frac{\tau^2}{2\pi} \text{ for deep water wavelength} \ldots (3.2)$$

where

$L_o$ = deep water wavelength

$h$ = still water depth

In deep water, the wave height is limited by the wind speed, wind-duration and the fetch length. In shallow water the effects from friction of the bed of the water-course usually restrict the wind energy thereby limiting the height. The values of $H_i$ and $T$ for wind-generated waves depend on the velocity, $U$ of the wind, its duration, and the fetch $F$ which is the distance that the wind blows over open water. Most methods of predicting wave characteristics from values
of \( U \), \( F \) and duration have been developed from open-sea conditions and therefore needed to be extrapolated down to the shorter fetches which typically apply for rivers and waterways. For inland waterways, wave height are limited by the fetch rather than by the duration period. According to Hemphill and Bramley, 1989 the equations for estimating \( H_s \) and \( T \) for such conditions are the simplified Sverdrup - Munk - Bretschneider (SMB) equations (Bretschneider, 1952; Owen, 1987):

\[
H_s = 0.00354 \left( \frac{U_{10}^2}{g} \right)^{0.58} F^{0.42} \quad \text{......(3.5)}
\]

\[
T = 0.581 \left( \frac{FU_{10}^2}{g^3} \right)^{0.25} \quad \text{......(3.6)}
\]

Where

\( U_{10} \) = the wind speed at a height 10m above stillwater level

\( F \) = fetch of the wind

\( g \) = acceleration due to gravity

Values of \( U_{10} \) is estimated from local observation and typical U.K values are:

\( U_{10} = 19 \) m/s for sheltered area

\( U_{10} = 23 \) m/s for average area

\( U_{10} = 26 \) m/s for exposed area

\( F = 1000 \) m

---

Fig 3.4: Wind-generated wave attack, fetch and obliqueness.

F is further categorised as \( F_e \), effective fetch length to account for the plan
shape of water area.

\[ F_z = B \quad \text{(if the wind blows across the channel reach)} \quad (3.7) \]

\[ F_z = 2.5B \quad \text{(Saville method if the wind blows at 45° to the reach and if } L > 20B \text{)} \quad (3.8) \]

\[ F_z = (3L_t + 67B) \quad \text{(if the wind blows along channel reach)} \quad (3.9) \]

40

The conclusion is that the designed wind waves should be determined by identifying the largest value of \( F_z \) occurring in the range of the incident angle ±45°.

3.2.1.3 Ship-induced waves

The water motion produced by ship depends upon (PIANC, 1987a)

(a) size and the geometry of the waterway

(b) ship type

(c) ship speed

(d) sailing course of ship

and this water motion can be classified as

(a) waves

(b) current

(c) change in water level

Refer to Fig. 3.5 for illustration of the ship-induced motion in a navigable water course.

3.2.1.3.1 Water level depression and Return current

The velocity head of the water flowing past a moving vessel in a restricted waterway causes the water level around the vessel to fall in order to maintain a constant hydraulic head (Bernoulli theorem). The difference between the lowered water level and the still water level is called the Water Level Depression. The water motion velocity is referred to as the Return Current and is opposite in direction of the moving vessel (refer to fig. 3.5). The average values of the water level depression, \( \Delta H \) and the return current velocity, \( u \), can be calculated from the energy approach (after Jansen and Schijf, 1953) Refer to Appendix II for equation of motion describing water level depression calculation. The calculation of \( \Delta H \) and \( u \) is simplified by the
use of the Schijf's chart (refer to figure 3.6). The induced ship-waves can be classified as primary waves and secondary waves.

![Diagram of water motion associated with boat in navigable waterway](image)

**Fig. 3.5: Water motion associated with boat in navigable waterway (after PIANC, 1987a)**

### 3.2.1.3.2 Primary waves (Front and Transversal waves)

The primary waves consist of the front and transversal stern waves which are associated with the general flow of water around the boat which produces changes in water level within the constricted cross-section of the channel. The front and transversal stern waves are basically induced by the return current flow in the opposite direction of the boat. The front wave is the transition between the undisturbed water level in front of the vessel and the water level depression; whereas the transversal stern wave is the transition between water level depression and the normal water level behind the ship. The transversal stern wave sometimes take the form of a breaking wave, depending upon the vessel speed and the channel depth. The height of the transversal stern wave can be taken as

\[
Z_{\text{mix}} = \Delta h
\]

The maximum value of the gradient of this wave is limited to between 0.1 and 0.15. The height of the front wave can be calculated from

\[
\Delta h_f = 0.1\Delta h + \Delta h
\]

and the gradient can be estimated as

\[
i_f = 0.03\Delta h_f
\]
The front wave and its gradient depend on $A_h$, and the sailing eccentricity; and are important in determining the prevailing pressure gradient in the subsoil.

3.2.1.3.3 Secondary waves and Interference peaks

These waves are water surface disturbances originating mainly at the bow and stern of the boat, and are propagated obliquely outwards as transverse and diverging waves which travel towards the bank. And if combined together, they form interference peaks. These waves generally depend on the boat motion and are similar to wind waves, but not random since they travel in a coherent group. Fast unloaded vessels and tugs produce significant secondary waves. The wave height of the interference peak at the bank can be estimated as

$$H_i = a_1 h \left(\frac{h}{s}\right)^{0.33} F_h^{4} \quad .......(3.13)$$

where

- $H_i$ = wave height of interference peak
- $h$ = water depth in channel
- $s$ = distance between the bank and the boat's side
- $F$ = $V_g / \sqrt{gh}$ (Froude number)
- $V_g$ = vessel speed
- $a = \text{coefficient depending on type of vessel}$
  - 1.0 (loaded push-tow unit)
  - 0.50 (unloaded push-tow unit and tug boat)
  - 0.35 (conventional inland motor vessel)

The wavelength can be taken as

$$L_{wi} = 0.67 \times 2\pi V_g^2 / g \quad .......(3.14)$$

and the condition for this equation is

$$F_h < 0.7$$

$$6.5 < h < 8.5$$

Another quantity that influences the magnitude of the ship-generated waves and the water motion generally is the relative blockage factor, $k$ where

$$k = \frac{\text{midship cross-sectional area}}{\text{waterway cross-sectional area}} = \frac{A_1}{A_c}$$

and the condition is such that (according to Hemphill & Bramley, 1989)
\[ k > 0.1 \text{ (if primary waves > secondary waves)} \]
\[ 0.05 < k < 0.1 \text{ (if primary waves = secondary waves in magnitude)} \]
\[ k < 0.05 \text{ (secondary waves are dominant)} \]

3.2.1.3.4 Vessel speed and screw race

The vessel speed is an important factor in ship-induced water motion in unrestricted waterway and it depends only on the ship geometry and the method of ship propulsion. According to Schijf (1953), a maximum velocity or Limit speed \( V_1 \) exist for every in a restricted waterway irrespective of the amount of propeller power available. Although it is possible to calculate the actual vessel speed of a vessel if the engine power and the type of propeller are known, but for design purposes it is appropriate to assume the actual vessel speed, \( V_s \) as

\[ V_s = 0.9 V_1 \]

where \( V_1 = \text{speed limit} \) .......(3.15)

\( V_1 \) can be found from equations in Appendix II derived from the energy equation (Jansen & Schijf, 1953). Recent investigation has shown that a single moving vessel along a fairway can produce critical hydraulic load.

The screw race is a high velocity jet of water produced behind the propeller and this may impinge on the waterway bed or bank. Serious scour usually results from this action when a ship is stationary or while in motion. Refer to Appendix II velocity equation behind the propeller.

3.2.1.4 Other Wave Characteristics

The knowledge of the wave characteristics and boundary condition is important for the dimensioning of the revetment. Because the flow pattern caused by wave action on a revetment is so complicated, the loading zone on a bank or a bank is usually approximated between the zone which is permanently submerged and the zone above the design level where only wave run-up occurs. It is evident in principle that a bank slope revetment will not function differently when it is loaded under normal circumstances from when it is loaded under extreme condition. The emphasis in this regard is on the persistent character of the wave attack rather than on the magnitude. Sometimes, the degree of damage (exceedance of serviceability limit state) may be high in relative normal condition (and before the occurrence of the extreme condition) such that the revetment is no longer sufficient.
Fig. 3.6: Schijf's chart for estimating return current and water level depression

Factor of safety $\alpha_1 = 1.1$
$$b_w = \text{waterline width of waterway}$$

$$b_b = \text{bottom width of waterway}$$

$$h = \text{normal water depth}$$

$$a = \text{slope angle}$$

$$A_c = \text{wetted area of channel cross section}$$

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{waterway_geometry}
\caption{Waterway geometry}
\end{figure}

$$L_s = \text{length of ship at waterline}$$

$$B_s = \text{beam width}$$

$$T_s = \text{draught}$$

$$A_m = \text{wetted area of midship section of ship}$$

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{ship_geometry}
\caption{Ship geometry}
\end{figure}

\begin{tabular}{|c|c|c|c|c|c|}
\hline
Class & Carrying capacity (tonnes) & Beam (m) & Length (m) & Height unloaded (m) & Draught (m) \\
\hline
I & 300 & 5.00 & 38.5 & 3.55 & 2.20 \\
II & 600 & 6.60 & 50.0 & 4.20 & 2.50 \\
IIA & 1,000 & 8.20 & 67.0 & 3.95 & 2.50 \\
III & 1,350 & 9.50 & 80.0 & 4.40 & 2.50 \\
IV & 2,000 & 11.50 & 95.0 & 6.70 & 2.70 \\
V & 10,000 & & & & \\
\hline
\end{tabular}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{ecmt_classification}
\caption{ECMT Classification}
\end{figure}

Fig. 3.7(a-c): Waterway geometry and ship geometry and classification.
3.2.1.4.1 Wave breaking criteria

The wave theory expresses practical limitation for the height of a wave approaching a coast or a shallow water in form of:

(i) Steepness limit, $H/L$

The maximum steepness, $H/L$ of a non-breaking wave is 0.142 where

$H = \text{waveheight}$
$L = \text{wavelength}$

Kinsman (1965) pointed out the this criterion is applicable only in deep water.

(ii) Wave height / water depth limit, $H/h$

A fixed ratio between the depth, $h$ where waves break and the height of the breaking wave, $H$ is given by (Shore Protection Manual)

$$H/h = 0.78$$

This is invalid for a solitary wave. For irregular wave

$$\frac{H_{s\text{mat}}}{h_s} = 0.5$$

where

$H_{s\text{mat}} = \text{maximum significant wave height after breaking}$
$h_s = \text{the depth at breaking}$.

The value 0.5 is still subject to discussion because it depends on slope angle $\alpha$. Refer to fig. 3.8 for the sketch of a breaking wave as it approaches a slope.

![Diagram of wave breaking on dike slope](image)

Fig. 3.8: Wave breaking on dike slope
3.2.1.4.2 Breaker type on slope

The breaking of wave on dike slope is characterised by the wave steepness, the slope gradient and the internal structure of the bank. The characteristic parameter used for identifying different breaker types is the wave-breaking parameter, $\xi$ (Battjes, 1974).

$$\xi = \frac{\tan \alpha}{\sqrt{\frac{H}{L_0}}} \quad \ldots \ldots (3.16)$$

where

- $\alpha$ = slope angle
- $H$ = wave height
- $L_0$ = deep water wavelength = $g T^2 / 2\pi$
- $g$ = acceleration due to gravity
- $T$ = wave period

According to Patrick and Wiegel (1955), the following types of breaker can be distinguished:

(a) Spilling breaker:

These types are found on very flat bottom gradients (refer to fig. 3.10). The wave usually breaks at a far distance and the breaker height decreases uniformly as the wave approaches the slope.
(b) Plunging breaker:

This breaker type overturns in a short period of time and occurs on steeper bottom gradient. The wave crest development in shallow water is not appreciably disturbed by some effects such as wind, crossing waves, current, irregularities of the bottom e.t.c. The wave energy is usually dissipated in turbulence and little is reflected back (refer to fig. 3.10).

Fig. 3.11: Plunging breaker type

0.5 < $\xi$ < 3

(c) Collapsing breaker:

This breaker type occurs on a steep slope and a vertical wave crest exists and collapses before any overturning takes place. Refer to fig. 3.12.

Fig. 3.12: Collapsing breaker type

$\xi$ ≈ 3.2

(d) Surging breaker:

The surging breaker occurs along extremely steep slope. The breaker zone is usually narrow and much of the wave energy is reflected. This breaker type actually lies in between the state of plunging breaker and non-breaking wave where all energy are reflected back.

Fig. 3.13: Surging breaker type

$\xi$ > 3.5
For the design of revetment structures, the surging and plunging breaker types are important. The classification of the breaking parameter shown above is only representative of the regular wave train on a flat and smooth slope. The wave breaking and impact phenomenon is more complicated in the real natural condition. Theoretical and physical researches are still being carried out to present a picture of the mechanisms involved. A loaded dike slope consists of wave run-up and run-down with periodically changing water load; and the breaker push on the revetment is referred to as the wave impact (a load of a very short duration). The amplitude of the wave impact depends on the kinematics of the wave, the geometry and stiffness of the dike, and its influence is mostly stochastic because of the existence of the air enclosed between the breaker and dike, and the air content within the breaker itself. Fuhrboter (1985) gave for a prototype dike with slope 1:4 and a wave height \( H = 1.25 \text{m} \), a maximum amplitude of \( 3.5 \times 10^2 \text{ Pa} \) which is approximately 2.5 times the wave height.

3.2.1.5. Wave Run-up (Zone of wave attack)

Wave breaking against a slope will run up to an elevation higher than the still water level depending on slope gradient and roughness of the bank protection (refer to figs. 3.8 & 3.9). Smooth surfaces generally experience higher run-up than rough slopes. The effective run-up, \( R \) up an inclined revetment is given by

\[
R = R_n r_R r_B r_D 
\]

where

\[
R_n = \text{run-up on a smooth plane slope and is equivalent to vertical height above still water level. (n = probability of exceedance index)} \\
r_R = \text{reduction factor due to slope roughness and permeability} \\
r_B = \text{reduction factor due to berm} \\
r_D = \text{reduction factor for the obliqueness of the wave attack.}
\]

(a) For regular waves, \( R_n \) can be expressed by Hunt's formula

\[
\frac{R_n}{H_s} = \xi \quad (\xi < 3) \quad \text{ ....(3.18)}
\]
For irregular waves (modified Hunt's formula)

\[ \frac{R_u}{H_s} = C_u \sqrt{2\pi \xi} \quad \text{for } \xi - \xi_p < 2.5 \]

\[ T - T_p \]

where

\[ C_u = \text{coefficient depending on the type of wave spectrum and probability of exceedance} \]

\[ T_p = \text{peak period (the period given largest wave impact)} \]

\[ \xi_p = \text{wave breaking parameter based on } T_p \]

Run-up exceeded by 2 waves that is \( C_u = C_{ui} \) are estimated from field measurements such that \( C_{ui} \) is

\[ C_{ui} = 0.55 \quad \text{(small spectrum)} \]

\[ C_{ui} = 0.70 \quad \text{(wide spectrum)} \]

Run-up due to wind waves:

\[ C_u = 0.70 \]

\[ R_u + H_s = 1.75 \xi, \quad (\xi < 2.0 - 2.5) \]

\[ R_u + H_s = 3.5, \quad (\xi \geq 2.5) \]

Run-up due to ship waves:

\[ C_u = 0.6 \]

\[ R_u + H_s = 1.5 \xi \]

where

\[ \xi = \tan \alpha / (H_s + L_s) \]

\[ H_s = \text{height of interference peaks} \]

\[ L_s = \text{wavelength of interference peak} \]

(b) The reduction factor for surface roughness and permeability, \( r_f \) for different revetment types is given in table 3.2 below.
<table>
<thead>
<tr>
<th>Armour layer</th>
<th>( r_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt, smooth concrete</td>
<td>1.0</td>
</tr>
<tr>
<td>Concrete blocks</td>
<td>0.95</td>
</tr>
<tr>
<td>Pitched stone</td>
<td>0.90</td>
</tr>
<tr>
<td>Rough and permeable</td>
<td>0.80</td>
</tr>
<tr>
<td>Gravel and gabion mats</td>
<td>0.70</td>
</tr>
<tr>
<td>Rip - rap (min thickness = 2D_{pp})</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Table 3.2: Reduction factor for surface roughness

(c) The reduction factor, \( r_s \) for slopes with a berm width, \( B \) and \( h_b < 0.5H_s \) and \( H_s/L_0 > 0.03 \) (optimum condition)

<table>
<thead>
<tr>
<th>Slope</th>
<th>( r_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:5 to 1:7</td>
<td>0.75 - 0.80</td>
</tr>
<tr>
<td>1:4</td>
<td>0.60 - 0.70</td>
</tr>
<tr>
<td>1:3</td>
<td>0.50 - 0.60</td>
</tr>
</tbody>
</table>

Table 3.3: Reduction factor for berms

(d) The reduction factor for oblique wave attack, \( r_j \) is

\[
r_j = \cos(\beta - 10)
\]

where

\[
\beta = \text{the angle of the direction of the wave propagation normal to the bank}
\]

3.2.1.5 Water level variation

Influences other than wind and ship induced waves can induced water motion in waterway and rivers. Water level variations due to tidal fluctuation and the discharge regime may take place over a period of hours or even days and may subsequent affect the revetment (refer to Fig. 3.9).

3.2.2 Strength Determination And Dimensioning

3.2.2.1 General

The hydrodynamic forces on the armour layer due to wave and current action, evolves from the interaction of the elements of the armour layer with the hydraulic flow field which surrounds them. The induced pore
pressure under the revetment layer and in the sub-soil constitute the internal load on the bank protective structure. If the armour layer is relatively impermeable, the strength of the armour layer will have to be designed against wave impact forces and uplifting. But this situation may lead to loss of internal stability of the under layer and sub-soil. And if the armour layer is relatively permeable and results in pressure dissipation, this situation may lead to geotechnical instability of the sub-soil. Therefore the relationship between external and internal loads play an essential role when considering the strength of the bank protection and the stability of the revetment. The following conditions are hereby considered.

3.2.2.1.1 Uplifting

Lifting of a revetment will occur if the uplift pressure difference at the moment of wave run-down is greater than the normal weight component of the revetment. Generally large hydraulic uplift pressure develops in sealed or water-tight impermeable revetments. This situation may also be expected after a storm surge in which water level outside a bank falls at a relatively faster rate than the water level inside the bank. The Electrical Analog and van der Veer formula has used to estimate uplift pressure under an impermeable revetment. The uplift criterion according to van der Veer

\[ h \geq \frac{\sigma_{wu}}{\rho_w g \cos \alpha} \]  

where

- \( h \) = thickness of revetment
- \( \sigma_{wu} \) = maximum uplift pressure = \( \rho_w g (p + h \cos \alpha) \)
- \( \alpha \) = slope angle
- \( \rho_f \) = bulk density of impermeable revetment
- \( \rho_v \) = density of water
- \( g \) = acceleration due to gravity

and

\[ p = c \cdot \Phi \]  

when

\( \frac{v}{(a + v)} < 0.8 \) to \( 0.85 \) (refer to fig. 3.14)

and

\( c = \) a coefficient depending degree of stationarity of the phreatic line.

\[ = \left\{ \begin{array}{ll} 1 + \left[ \frac{v}{(a + v)} \right]^{1/\theta} \right\}^{1/2} & \text{stationary flow} \\
1/\pi \cos^{-1}\left[ 2\left(\frac{v}{(a+v)}\right)^{1/\theta} - 1 \right] & \text{non-stationary flow} \\
\end{array} \]

and

\( \theta = \tan^{-1} (n) + \pi/2 \)
n = cota

Fig. 3.14: Schematization for revetment under hydraulic loading
(a) van der Veer formula

3.2.2.1.2 Sliding

The armour layer of a revetment without any toe protection or anchoring may slide due to the loading induced by wave attack or drop in water level. Stability of such revetment has to be guaranteed by the frictional force between the revetment and the sub-soil. The sliding criterion for impermeable revetment according to van der Veer is

\[ h \geq \frac{f \sigma_{w0}}{\rho g (f \cos \alpha - \sin \alpha)} \] ....(3.21)

where

- \( f \) = coefficient of friction = \( \tan \phi \) if \( \phi > \theta \) otherwise \( f = \tan \theta \)
- \( \phi \) = angle of internal friction of the sub-soil
- \( \theta \) = angle of friction between the revetment and the sub-soil

Further analytical solution of stability criterion for placed block has also been developed by Bakker and Meijer (1988) based on the solution of maximum pressure difference by Wolsink.
3.2.2.1.3 Equilibrium stability

The equilibrium stability of a revetment must consider the ability of the revetment to resist the combined external forces on the armour layer and the induced internal forces between the armour layer and the sub-soil. This resistance depends on some or all of the following factors (Hemphill and Bramley, 1989)

(a) Weight and/or dimensions of elements of the armour layer; or the weight/unit area of a continuous armour
(b) Support provided by the underlayer or sub-soil
(c) Friction between adjacent elements of the armour layer, and between the armour layer and the underlayer or the sub-soil formation
(d) Revetment slope
(e) Compressive forces in the plane of the revetment
(f) Interlock, grouting or cabling between elements of the armour layer
(g) Anchorage and mechanical shear restraint between elements of the armour layer and the sub-soil.

Experimental data on many of these combinations are limited. Stability design analysis is mostly based on empirical relationships derived from laboratory studies or field experience since analytical models to determine the dimensions of the revetment have not been completely developed for general design purposes. The most prominent formulas for estimating the weight of armour layers are the formulas derived by

Irribaren

\[ G = \frac{K \gamma_s H^3}{\Delta^3 (f \cos \alpha + \sin \alpha)^3} \] .....(3.22)

and that of Hudson

\[ G = \frac{\gamma_s H^3}{K_D \Delta^3 \cot \alpha} \] .....(3.23)

where

- \( G \) = median weight of the armour unit
- \( H \) = design wave height
- \( \Delta \) = \( \rho_s - \rho_w / \rho_w \) = relative density of armour unit
- \( \rho_w \) = density of water
- \( \rho_s \) = density of armour unit
\[ \gamma_s = \text{specific weight of armour unit} \]
\[ a = \text{slope angle} \]
\[ K, K_n = \text{dimensionless stability coefficients} \]
\[ f = \text{friction / interlocking factor of the armour units.} \]

The stability coefficients \( K \) and \( K_n \) respectively in Iribarren and Hudson formulas are determined by laboratory tests carried out with a regular wave train. The height of the regular wave train is assumed to correspond to the characteristic wave height \( (H_s) \). Recent investigation (according to CERC) has proved that the influence of wave period, which is significant in the stability of rip-rap was not included in the Hudson formula. Ahren, et al used data from the investigation to show that the influence of the breaker parameter \( \xi \) on the stability parameter, \( N_{zd} \), called the zero-damage stability number and which is defined by

\[ N_{zd} = \frac{H_{zd}}{\Delta (G_{so}/\gamma_s)^{1/3}} - \frac{H_{zd}}{\Delta D_n} \]  

\( \text{where} \)

\[ H_{zd} = \text{zero-damage wave height} \]
\[ \Delta = \text{relative density} \]
\[ D_n = \text{nominal size of armour unit equivalent to the size of a cube} \]
\[ = \left( \frac{G_{so}}{\gamma_s} \right)^{1/3} \]

Combining equation (3.24) with Hudson formula gives

\[ K_D = \frac{N_{zd}^{-3}}{\cot a} \]

\( \text{where} \)

\[ H_s \leq \Psi u \Phi \frac{\cos \alpha}{\xi^b} \]; for \( \cot a \geq 2 \)

A further modified and elaborate formula developed by Pilarczyk (1989) in which the slope resistance was considered is

\[ \frac{H_s}{\Delta_m D} \leq \Psi u \Phi \frac{\cos \alpha}{\xi^b} \]; for \( \cot a \geq 2 \)

42
where

\[ \xi_t = \frac{\tan \alpha}{(H_s/L_0)^{0.5}} = 1.25T_s (H_s^{0.5} \tan \alpha) \]

\[ \psi = \text{empirical stability upgrading factor} \]

\[ (\psi = 1.0 \text{ for rip-rap as reference and } \psi \geq 1.0 \text{ for other revetment systems}) \]

\[ \phi = \text{stability function for incipient motion defined at } \xi_t = 1.0 \]

\[ H_s = \text{significant wave height} \]

\[ T_s = \text{average wave period} \]

\[ L_0 = \frac{gT_s^2}{2\pi} = \text{wave length} \]

\[ D = \text{specific size or thickness of protection} \]

\[ a = \text{slope angle} \]

\[ \Delta = \text{relative density of armour unit} \]

\[ b = \text{exponent related to the interaction process between waves and revetment characteristics (example roughness, porosity, permeability etc). (0.5 < b < 1).} \]

\[ = 0.5; \text{for rough and permeable revetments such as rip-rap} \]

\[ = 1.0; \text{for smooth and less permeable placed block revetment} \]

\[ = 2/3; \text{for other systems.} \]

\[ \Delta \text{ and } D \text{ as specified above is} \]

for rock; \( D = D_s = (\psi_{so} / \rho_s)^{1/2} \) and \( \Delta = \Delta = (\rho_s - \rho_v) / \rho_v \)

for blocks; \( D = \text{block thickness and } \Delta = (\rho_c - \rho_v) / \rho_v \)

\[ \phi = 6.2 (P_b)^{0.18} (S_b)^{2/3} / N \text{; for } \xi_t < 3 \]

\[ = 2.25; \text{for incipient motion of one to three stones} \]

\[ = 3.0; \text{approximate condition for maximum tolerable damage in a 2-layer revetment on granular filter where } S_b = 8 \text{ damage depth } \leq 2D_\Delta. \]

where

\[ P_b = 0.1; \text{ sand or clay core} \]

\[ N = 3000, \text{ number of waves} \]

The use of fig. 3.15 and Table 3.4 is recommended for effective design
Fig. 3.15: van der Meer’s formula for rip-rap stability for N = 3000 waves and an impermeable core ($P_b = 0.1$)

Table 3.4: Categories of protective systems.
3.2.2.2 Stability criteria for Rip-rap

A major advantage of rip-rap protection is that it is very flexible, so that damage tends to occur gradually and the manner in which the stones move relative to one another can be described to some extent as self healing. Useful engineering qualities of rip-rap include (Hemphill and Bramley, 1989):

(a) General ease of placing, can be placed under water
(b) Flexibility
(c) High hydraulic roughness to attenuate waves and current
(d) Low maintenance requirement and convenience of repair
(e) Durability

Sizing and grading:

There is no standard grading definition but specifications lie within the following upper and lower limits

\[
\begin{align*}
W_{100} / W_{50} & = 2 \text{ to } 5 \\
W_{85} / W_{50} & = 1.7 \text{ to } 3.3 \\
W_{15} / W_{50} & = 0.1 \text{ to } 0.4 \\
W_{85} / W_{15} & = 4 \text{ to } 12
\end{align*}
\]

Material smaller than \(W_{15}\) is volume of voids between larger rocks.

\[
D_{250} = (W_{50} / \gamma)^{1/2}
\]

Refer to figs. 3.16 & 3.17

Fig. 3.16: Typical rip-rap revetment
Fig. 3.17: Grading envelope for rip-rap

Sizing for wave attack:

(i) Ship transversal waves

\[ D_{n30} \geq \frac{Z_{\text{max}}}{1.5 (\cot a)^{1/3} \Delta_m} \]  

\[ \ldots (3.27a) \]

(ii) Ship secondary waves (interference peak)

\[ D_{n30} \geq \frac{H_i (\cos \beta)^{0.5}}{1.8 \Delta_m} \]  

\[ \ldots (3.27b) \]

where

\[ \beta = \text{angle of bow propagation normal to the bank} \approx 55^\circ \]

\[ H_i = \text{interference peak wave} \]

(iii) Wind waves

\[ D_{n30} \geq \frac{H_s \sqrt{\kappa}}{2.25 \Delta_m} \]  

\[ \ldots (3.28) \]

Sizing for current attack:

According to Pilarczyk (1989),

\[ D_{n30} \geq \phi_c \frac{K_T K_h}{K_i} \frac{u^2}{2g \Delta_m} \]  

\[ \ldots (3.29) \]
where

\[ \phi_c = \text{stability factor} \approx 0.75 \]
\[ K_T = \text{turbulence correction factor} \approx 1.0 \]
\[ K_v = \text{velocity correction factor} = \frac{628}{C} \]
\[ K_s = \text{bank correction factor} = \cos \alpha (1 - \tan \theta / \tan \theta) \]
\[ u = \text{velocity of flow in channel} \]

3.2.2.3 Stability criteria for placed block revetment

Placed block revetments used to be an economical substitute where rip-rap was not available but nowadays modern placement techniques and the development of the block mattresses have changed the situation. The various types of blocks are:

(i) Open-jointed or grouted blocks
(ii) Interlocking blocks
(ii) Cabled tied or geotextile-bonded blocks

Blocks are more stable if placed directly on an impermeable clay sub-soil which restrict the development of uplift pressure under the blocks during external wave attack or cyclic flow conditions. The wave attack is predominant in the sizing of pitched blocks, therefore usually for big channels, the effect of current attack is negligible.

Sizing for wave attack:

\[ D \geq \frac{H_s \sqrt{\xi}}{\phi_c \Delta \cos \alpha} \]

where

\[ D = \text{block thickness} \]
\[ \phi_c = \text{stability coefficient for regular pitched stones} \]
\[ (\text{varies between 3 and 3.5}) \]
\[ H_s = \text{significant wave height} \]

The use of fig.3.19 can be employed to determine the block thickness. It must however be stated that equation (3.30) does not consider the effect of:

(i) permeability ratio of armour layer to filter layer
(ii) surface dimension of block
(iii) filter layer thickness
Fig. 3.18: Typical block revetment

Fig. 3.19: Wave breaking parameter, $\xi$ as a function of the parameter $H_s/A_d$ for irregular waves and a permeable sub-soil.

These and all other factors are subject of mathematical and physical modelling. Information are now available on displacement pattern, deformation characteristics of blocks and the degree of uplift pressure which used to be unknown. Results of various large scale investigation carried out by Delft Hydraulics laboratory are shown in figs. 3.20 and 3.21.
3.2.2.4. Stability Criteria For Asphalt Revetment

Asphalt consists of mixture of bitumen and mineral aggregate. Open stone asphalt can provide a permeable armour layer while asphaltic concrete can provide an extremely durable and impermeable armour layer. Apart from the uplift pressure calculation mentioned for impermeable asphalt revetment in paragraph 3.2.2.1.1., the design of asphalt under wave attack considers the armour layer as a plate on an elastic foundation. Adequate plate thickness must be able to resist the induced bending moment from wave impact and this thickness (according to Rijkswaterstaat, 1985) is

\[ h = 0.75 \sqrt{\frac{27}{16} \frac{1}{(1-v^2)} \left( \frac{P}{\sigma_b} \right)^4 \left( \frac{S}{c} \right)} \]  

\[ \ldots \ldots (3.31) \]
where

\begin{align*}
  h &= \text{thickness of the asphalt armour (m)} \\
  \sigma_b &= \text{asphalt stress at failure (N/m}^2) \\
  P &= \text{wave impact (N/m)} \\
  S &= \text{Stiffness modulus of asphalt (N/m}^2) \\
  \nu &= \text{Poisson’s ratio for asphalt} \\
  c &= \text{modulus of sub-grade reaction (N/m}^2) \\
\end{align*}

Other considerations for asphalt design are the

(a) Tension induced in the revetment when sliding occurs as
    result of insufficient frictional resistance
(b) Erosion induced in joints due to current
(c) Settlement or deformation of subsoil.

Figures 3.22 - 3.24 show different types and composition of revetments and
resulting stability conditions.
Fig. 3.22: Revetment types

Fig. 3.23: Composition of revetment
3.2.3 Filter Requirement and Consideration

Recent studies (PIANC, 1987; Hewlett et al, 1987) have highlighted the fact that many failures of revetments result from inadequate performance of the underlayer (consisting of geotextile and/or granular materials) rather than direct failure of the armour layer. Depending on design conditions, flow in the underlayer may be in any of these directions:

(a) Up or down the slope of the revetment
(b) Perpendicular to the slope, into or out of the bank
(c) Along the revetment in the direction of channel alignment

The function of a filter is to prevent migration of the sub-soil material, and yet to permit movement of water across the filter-subsoil boundary without causing an unacceptable induced pressure. The different forms of filters are granular filters and geotextile filters. (Refer to Table 3.5 for features of geotextile and granular filter underlayers)
### Table 3.5: Features of geotextile and granular materials as underlayer.

<table>
<thead>
<tr>
<th>Geotextile</th>
<th>Granular material</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Advantages</strong></td>
<td><strong>Disadvantages</strong></td>
</tr>
<tr>
<td>Cost</td>
<td>Some uncertainty over long-term behaviour</td>
</tr>
<tr>
<td>In-plane tensile strength</td>
<td>Edges must be carefully protected</td>
</tr>
<tr>
<td>Limited thickness</td>
<td>Easy to damage; difficult to repair</td>
</tr>
<tr>
<td></td>
<td>Careful design and installation needed to accommodate settlement or uneven formation</td>
</tr>
<tr>
<td>Self-healing in some circumstances</td>
<td>Careful control needed to achieve specified grading and thickness</td>
</tr>
<tr>
<td>Generally very durable</td>
<td>Compaction difficult on steep side slopes</td>
</tr>
<tr>
<td>Deformable, retaining good surface contact above and below</td>
<td>Control of construction difficult underwater</td>
</tr>
<tr>
<td>Relatively easy to repair</td>
<td></td>
</tr>
</tbody>
</table>

3.2.3.1 **Granular Filter**

Granular filters are used for regulating energy dissipation and for secondary protection purposes. Granular filters are specified in terms of particle size and stability can be achieved by uniform composition of one or more layer. The granular filter requirements are defined by the so called filter rules for representative grain sizes of base or subsoil, $D_b$ and the filter $D_f$. The conditions are:

(i) Minimisation of segregation:

$$\frac{D_{50f}}{D_{50b}} < 20 - 25$$

(the two grading curves should not be far apart)

(ii) Permeability:

$$\frac{D_{15f}}{D_{15b}} > 5$$

(permeability of filter should be greater than that of sub-soil)

(iii) Piping and sand tightness:

$$\frac{D_{15f}}{D_{85b}} < 3 - 5$$

(for $U_b > 5$; well graded sub-soil)

$$\frac{D_{50f}}{D_{50b}} < 3 - 5$$

(for $U_b < 5$; uniformly graded sub-soil)

(iv) Minimum filter size:

$$D_{5f} > 0.075$$

(prevention of blockage)

(v) Stability against migration:

$$\frac{D_{60f}}{D_{10f}} \leq 10$$

(no migration)
3.2.3.2. Geotextile Filter

Geotextile properties are specified in terms of (i) fabric type, (ii) pore size and (iii) permeability. Geotextile filters are used for filtration, erosion control and separation functions. Geotextiles are in form of mesh fabrics, ribbon fabrics, mats, clothes and membranes. The characteristic opening size expressed as the pore size, \( D \), in which \( n \% \) of the pores are smaller than. The permeability of geotextile is defined in terms of permittivity, \( \psi \) which is equal to permeability divided by fabric thickness. The soil retention criteria are:

(i) \[ O_{90} < \lambda D_{90b} \quad (\lambda = 1 \text{ to } 2; \text{ for steady flow condition}) \]

(ii) \[ O_{98} < D_{95b} \quad (\text{for cyclic flow condition}) \]

(iii) \[ O_{98} < D_{95b} \quad (\text{for sand tightness}) \]

Minimum opening of \( O_{90} = \) 0.05 mm (according to PIANC, 1987)

Maximum opening of \( O_{98} = \) 0.3 - 0.5 mm (Ingold, 1984)

\[ \psi > 5 \times 10^4 k_b \quad (\text{in the absence of specific site values according to Hemphill and Bramley}) \]

Sometimes the permeability of geotextile, \( k \), is multiplied by a reduction factor, \( \eta \) (refer figure 3.25 for permeability reduction factor for woven fabric).

![Figure 3.25: Permeability reduction factor, woven fabric](image)
<table>
<thead>
<tr>
<th>SOILS NOT SUSCEPTIBLE TO DOWNSLOPE MIGRATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram of geotextile" /></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>SOILS SUSCEPTIBLE TO DOWNSLOPE MIGRATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image2" alt="Diagram of granular sublayer and geotextile" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(a) Attenuation of hydraulic gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image3" alt="Diagram of geotextile" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(b) Stabilization of soil surface</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image4" alt="Diagram of thick geotextile and coarse fibre layer" /></td>
</tr>
</tbody>
</table>

Figure 3.26: Recommended filter system
3.2.4 Transitional Details

A well designed revetment system may be susceptible to damage at points of transition from one revetment type to another and at terminal points of revetment. Attention should be given to details at crest and toe.

3.2.4.1 Toe Protection

Provision of cut-off at toe to withstand scour and increase of potential scour. Provision of armour skirt or apron can also prevent toe erosion.

3.2.4.2 Crest Detail

Upper level of bank protection must be designed to withstand overtopping in extreme flood. Attention must be given to geotextile or anchor.

Refer to figure 3.27 for various forms of toe protection.

Figure 3.27: Alternative toe protection design (PIANC 1987)
3.2.5 Modelling Technique

The various empirical design rules and criteria mentioned so far in this chapter were established as a result of cumulative research investigations. But physical and mathematical model studies on stability of placed blocks are still being carried out (Bezuijen et al, 1990; Fuhrboter and Sparboom, 1988) to provide a full understanding of the failure mechanism of the revetment structure so that a comprehensive and descriptive analytical model is achieved. As earlier pointed out in this report, the various hydraulic conditions, parameter and structural response are stochastic in nature. Presently, there is no calculation method to transform wave height and period to a time dependent pressure distribution on the revetments slope. But studies carried out by Banach (1987) and Klein Breteler (1988) have established empirical relations for wave pressure distribution based on static wave load. This paragraph will highlight the description of wave impact load and the pressure distribution derivation. Further mention necessary for understanding of the STEENZET and ANAMOS models will be made in chapter five.

3.2.5.1. Failure Mechanisms

Refer to fig. 3.28 for the schematic representation of a wave load on a placed block revetment.

Figure 3.28: Placed block revetment under wave attack.

Results of investigation carried out by Delft Hydraulics and Delft Geotechnic have identified possible 8 mechanisms of failure (Den Boer, 1983). Refer to fig. 3.29.
(a) The wave run-up has reached its maximum level, then flow returns under gravity influence. Pressures on the revetment is reduced during this return flow. Current forces, inertia forces and lift forces may be generated depending on the roughness of the revetment.

(b) Depending on permeabilities and bank geometry, the water which has caused a rise in phreatic line in the filter cannot flow out immediately, therefore uplift pressure is induced on the revetment. The effect is cumulative for a number of waves. It is noticeable that the wave withdraws to a level below the still-water line.

(c) The approach or arrival of the next wave on slope induces a further increase in pressure which is transmitted through the filter in front of the wave, therefore complementing the already existing uplift pressure under the revetment at this point. Influence of this pressure is limited to adjoining area in front of wave front.

(d) As a sequence to situation in c, velocity of the approaching wave reduces and changes direction. The subsequent effect of this is a lowered pressure above the revetment at this point.

(e) Depending on ξ, the wave breaks with an impact on slope, and simultaneously generating an instantaneous pressure rise of a short duration approximate to 0.05 to 0.25 second. Transmission of this instantaneous pressure into the filter induces a further short-term pressure increase under the revetment.

(f) The collapse of the wave produces a large mass of water on the slope leading further to high pressure transmission into the filter and lifting tendency of the revetment lying next (up the slope) to the point where the breaking of the slope takes place.

(g) Coupled with the event of f, a substantial pressure reduction occurs on the revetment (to certain extent, negative pressures) caused by oscillations of the entrained air in the breaking wave.

(h) The after-effect of wave breaking generates a wave run-up with increase in pressure on revetment. This stage presents no critical situation unless when the revetment is rough or if the blocks have been partially lifted out from the revetment. Current forces, inertia forces and lift force may then subsequently manifest.

Delft Hydraulics Laboratory and Delft Geotechnics concluded from various tests that the combination of failure mechanisms b :- "quasi-static pressure differences" and c :- "pressure due to the approaching wave front" are very important for the stability of a block revetment. Also, it should be noted that combinations of other failure mechanisms can occur together. Figure 3.30 illustrates the recording for a time dependent external and internal water pressures on the critical row in a revetment under wave attack.
Fig 3.29: Schematic representation of possible failure mechanisms

3.2.5.2 Boundary Condition

The external wave pressure distribution on a revetment was developed by Banach, 1987 with the use of two empirical parameters, $\phi_1$ and $\beta$ in which a static wave load on a slope at the time of maximum run-down is
taken to serve as the boundary conditions for computation. Figure 3.31 illustrates the representation of this static piezometric head on the revetment based on regular waves and constant slopes (1:2, 1:3, and 1:4) for wave steepness \(0.01 < \frac{H}{L_o} < 0.07\), the following relationships were derived:

\[
\frac{\phi_b}{H} = 0.36 \left( \frac{\tan \alpha}{H/L_o} \right)^{0.5} \quad \text{if} \quad \frac{\tan \alpha}{H/L_o} \leq 37 \quad \text{(3.32a)}
\]

\[
\frac{\phi_b}{H} = 2.2 \quad \text{if} \quad \frac{\tan \alpha}{H/L_o} > 37 \quad \text{(3.32b)}
\]

\[
\tan \beta = \frac{0.17}{\sqrt{H/L_o}} \quad \text{(3.33)}
\]

Figure 3.31: Piezometric head on revetment before wave impact
\[
\frac{d_b}{H} = -0.11 \left[ \frac{\tan \alpha}{h/L_o} \right]^{0.8} \quad \text{if} \quad \frac{\tan \alpha}{H/L_o} \leq 26 \quad (3.34a)
\]

\[
\frac{d_b}{H} = -1.5 \quad \text{if} \quad \frac{\tan \alpha}{H/L_o} > 26 \quad (3.34b)
\]

where

- \( H \) = the incoming regular wave height (m)
- \( L_o \) = wave length in deep water (m)
- \( T \) = wave period (s)
- \( g \) = acceleration due to gravity (m/s^2)
- \( a \) = slope angle
- \( \beta \) = angle of wave front
- \( d_o \) = depth of the wave front (m)
- \( \phi_0 \) = height of pressure wave front

These pressure relations are not valid for a composite slope or berm in the area of wave attack.

3.2.5.2.1 Pressure load under armour layer

The stability of the armour layer depends on the amount of the pressure difference (uplift) between the pressure on the slope and the pressure induced in the filter for the schematised boundary condition above. This pressure difference could be derived with the use of the ground water flow equations in semi-confined aquifers in which the flow in the filter layer is assumed parallel and quasi-static, and the flow in the armour layer is assumed to be perpendicular to the slope such that:

\[
\frac{d^2 \phi}{d^2 x} = \frac{\phi - \phi_a}{\Lambda^2} \quad ....(3.35)
\]
where

\[ \begin{align*}
\phi &= \text{piezometric head in the filter layer (m)} \\
\phi_a &= \text{piezometric head in the armour layer (m)} \\
x &= \text{length coordinate (m)} \\
\Lambda &= \text{leakage factor (m)} \\
&= \sqrt{\frac{kbD}{k'}} \\
k &= \text{permeability of filter layer (m/s)} \\
k' &= \text{permeability of cover layer (m/s)} \\
b &= \text{thickness of filter layer (m)} \\
D &= \text{thickness of the armour layer (m)}
\end{align*} \]

Using the vertical leakage factor, \( \lambda = \Delta \sin \alpha \), the solution of equation (3.35), for a revetment that is longer than the leakage factor was presented by Wolsink using the schematized boundary conditions of Banach:

\[
\phi_w - \left[ \frac{\lambda}{2 \tan \alpha \tan \beta} \left( 1 - e^{-\frac{\tan \alpha \tan \beta}{\lambda}} \right) + \frac{\lambda}{2} \right] \left[ 1 - e^{-\frac{2z_1}{\lambda}} \right] = 0 \tag{3.36}
\]

where

\[ \begin{align*}
\phi_w &= \text{maximum difference in piezometric head over the revetment} \\
\lambda &= \sin \alpha \sqrt{\frac{kbD}{k'}} = \text{vertical leakage factor} \\
\alpha &= \text{slope angle} \\
\beta &= \text{the angle of wave front} \\
z_1 &= \text{height of phreatic surface} \\
\phi_b &= \text{height of wave front}
\end{align*} \]

Refer to fig. 3.32 for the resulting piezometric head and fig.(3.34) for the result and graphical representation of equation (3.36).
It is obvious that the uplift pressure needed to lift the armour layer depends on the leakage factor \( \lambda \) and the angle of wave front \( \beta \). The application of numerical methods to this equation (3.36) leads to the development of the STEENZET/1 and ANAMOS models. The models evaluate the stability conditions of a placed block with varying other physical parameters. Full description and application of the models are given in chapter 5.
CHAPTER 4

DATA AND REVETMENT DESIGN FOR CASE STUDY
--- STRONG FACE CREEK CHANNEL, NIGERIA ---

4.1 General

The general blackbox (semi-empirical) procedures for the design of revetments highlighted in chapter 3 are hereby applied to a particular case study in Nigeria: The Strong Face Creek channel (refer to II.3). Three types of revetments are designed namely: Rip-rap, Open stone asphalt and placed concrete block. This semi-empirical approach for the revetment stability design presupposes a limit state design in which the load is just balanced by the resistance of the structure. Emphasis is placed on optimum solution based on three different wave conditions and corresponding allowable damage and life cycle cost of the structure. Furthermore various assumptions are taken and these assumptions will be highlighted where and when necessary.

4.2 Data

The provided data for the case study are given below. Although, the acquisition of detailed and complete data was not possible, assumptions are made where necessary.

4.2.1 Description and Project setting of Channel

The Strong Face Creek channel is located in the delta area of the southern part of Nigeria (see figs.II.1,II.2 and II.3). The project area, called Opobo town, is located at the confluence of this Strong Face Creek with another river called Imo river. The town is also located about 7km from the Atlantic Ocean. The Opobo town has a low density population (roughly estimated at 4000 which increases to about 60,000 during holiday periods). The mode of transportation of people and goods, to and from the town is water borne. The existing embankment bordering the town is being continuously subjected to severe erosion. The regime of the two rivers mentioned above is influenced by the tides and wave effect from the Atlantic Ocean, which means periods of high and low waters with corresponding landward and seaward current. The erosion of the banks at Opobo town is caused by these current, complemented by wind-generated and ship induced waves which attack both the above- and under- water slopes, thereby undermining adjacent land masses which become unstable and ultimately collapse. The erosion process is estimated at several metres per year. Presently, if adequate measures are not taken to prevent further erosion, the continuation of this erosion processes will on the long run destroy the major part of Opobo town. The need for shore protection work in
this respect is therefore important. This study is specifically restricted to about 860 m of part of the Strong Face Creek banks. The slopes of the river banks at this area are presently 1:5 and a low water beach consisting of mud is available at about 5 metres.

4.2.2 Environmental Site Condition

The plan/map of the project is as shown in fig. II.3. In many cases the shoreline has reached the buildings and roads. The elevation of the bank is +1.5 M.S.L. and the maximum ground elevation recorded in Opobo town is +2.09 M.S.L. The major part of the embankment is vegetated with Raffia palms.

4.2.3. Channel Geometry and Profile.

The Strong Face Creek has a width of 500 m with maximum depths up to 9.0 m below M.S.L. which are found 70 m from the shoreline. The present bank slope is averagely 1:5 with depth up to 6.0 m found at 40 m from the shore. The river bed of Strong Face Creek at Opobo town corresponds with the bedform of a river-bend. The ground water level on the land side is +0.5 m above M.S.L.

4.2.4 Flow Observation

The established velocity in the channel profile is associated with tidal flow. A maximum tidal range of 2.28 m was recorded during a spring tide. And the maximum flood current recorded is 1.05 m/s and maximum ebb current is 1.06 m/s. A maximum water level at +2.0 m above M.S.L., has been predicted. A form of helicoidal flow pattern was noticed in which water particles tend to move outward at the surface and inward at the bed such that vertical velocity components result upwards at the inner bend and downwards at the outer bend.

4.2.5 Other Hydraulic Loads

4.2.5.1 Wind Waves

Wave measurements were recorded near the coast. The wave records were collected over interval period of 31 days spaced at duration of 1 hour. The significant wave height, \( H_s \) characterised by a set of 1000 waves were determined and fitted to a Weibull probability distribution (see fig. II.10). Since the creek is an exposed inland waterway, a wind speed of 26 m/s is assumed.

4.2.5.2 Ship-induced Waves

An ECMT classification was used in order to determine the ship type and classification. A commercial motor vessel (ship class IV) is
therefore assumed with a carrying capacity of 1350 tonnes. The beam width is taken as 9.50m. Also, the length of ship and draught are taken as 80m and 2.50m respectively.

4.2.6 Soil Profile and Properties

The sub-soil profile is indicated in Fig. II.6. The top layer consists of composite materials of silty peat, clay and sand, including some domestic debris. Below this layer are strata of sand and clay. Fine sand with occasional medium to cause sand were encountered at depths greater than 17m. The CPT value of the sand ranges from 4000 kN/m$^2$ to 8000 kN/m$^2$ at approximate depth of 3.0m. Over this depth the value ranges from 6000 kN/m$^2$ - 10,000 kN/m$^2$ at depths from 5m to 20m. The triaxial tests gave a cohesive value of 28 kN/m$^2$ to 45 kN/m$^2$ and zero angle of friction for the clay deposit. Specific unit weights of sand and clay were estimated at 25.7 kN/m$^2$ and 20 kN/m$^2$ respectively. The sieve analyses of the sub-soil are given in Figs. II.8 & II.9. Other load and material properties not mentioned are highlighted in the design sheets.

4.3 Design Approach

The procedure before any design specification is selected is that consideration must have been given to (i) identification of failure, (ii) site-investigation and (iii) ranking in order of preferred scheme (i.e. alternatives) through the use of economic assessment and comparison in form of life cycle cost, construction and maintenance techniques. All these are essential for a final design selection, but for the purpose of this study, detailed construction and maintenance philosophies are ignored due to insufficient data. The following assumptions are further taken:

(a) channel is in a state of regime flow (i.e. no permanent scouring of deposition)

(b) channel is an inland waterway

(c) environmental friendly solution is ignored

(d) the land-use pattern is ignored

(e) operation and management of the waterway is ignored
Fig. 4.1: Design methodology

The design methodology is outlined in Fig. 4.1. Some of the components of this methodology are left out, examples are acceptance, finance, management need and legal implication.

4.3.1 Design Procedure

The procedure follows the identification and computation of boundary conditions and load parameters. These are then followed by the calculations for three different load conditions for the three types of revetment choices taken in this study, which are:

(i) Rip-rap revetment
(ii) Open stone asphalt revetment
(iii) Placed block revetment

The choice of these revetment types is based on the fact that the existing sub-soil at project site consists mostly of permeable sandy layer which is liable to high fluctuation in induced pore pressure. The need and necessity to attenuate this high pressure build-up in the sub-soil is an overriding factor for the suitability of the chosen revetment type. An alternative solution or approach can be found in the use impermeable revetment type which provides a barrier to the development of high pressure build-up due to wave attack. But this solution will necessitate the use of a large thick layer of clay material on the sandy sub-soil before the impermeable revetment type is laid. But it is pertinent to mention that this solution may not satisfy underlayer- subsoil boundary soil migration or wash-out problem that may occur often or during reduction in water level or as an aftermath of a heavy down pour. Furthermore construction cost that may be
involved during the laying of such impermeable armour layer and the clay underlayer is another mitigating factor. The full design and analysis are given in Appendix I. But a summary of the design procedure and some key notes are highlighted below. The summary of the procedure are:

(i) Identification of design condition
(ii) Preliminary selection of revetment types
(iii) Assessment of geotechnical stability
(iv) Check on subsoil bearing capacity
(v) Design of armour layer
(vi) Design of underlayer
(vii) Detailing of crest, toe and edge

4.3.1.1 Boundary Conditions and Load Parameters

The two major boundary conditions analyzed are:

(i) Geotechnical
(ii) Hydraulic

4.3.1.1.1 Geotechnical

From the soil report and profile, it is noticeable that the existing soils at the project site possess enough shear strength and bearing capacity to withstand vertical load. Although the stability and deformation characteristics were not fully analyzed but the slope stability analysis and result give a good result. A maximum slope of 1:4 which is a pre-requisite for sliding of asphalt revetment was chosen. Other conditions for soil particle movement in the event of intense ground water flow were checked and found to satisfy this condition. The slope stability analysis result indicates a minimum factor of safety of 1.33 for the two cases analyzed are satisfactory. But this condition does not preclude some other form of failure such as horizontal movement or vertical deformation of the clay layer.

4.3.1.1.2 Hydraulic

The hydraulic loading consists of loads due to:

(i) ship-induced waves
(ii) wind-waves
(iii) current

4.3.1.1.2.1 Ship-induced waves:

A class IV (commercial motor vessel) type according to ECMT classification was assumed to be the type of vessel plying the Strong Face Creek. For this class of ship and the geometry of the channel, it was found that the blockage ratio, k is less than 0.05 which significantly indicates that secondary waves in form of interference peaks are predominant load on
The magnitude of the interference peak was 0.12m with a wave length of 5.47m. Values of the Front and Transversal stern waves determined are inconsequential but are also indicated in Appendix I.

(ii) Wind-generated waves:

The data given for wave characteristic are significant wave heights plotted to fit a Weibull Probability Distribution. Since the channel is somehow far from the coast and also an in-land waterway, it is obvious that the wave height will be fetch limited. The use of the simplified Sverdrup-Munk-Bretschneider (SMB) equation was employed to estimate the initial wave parameters. The wave height, $H_s$ was found to be 0.56m and wave period determined as 2.51 seconds. The deep water wave length, $L_o$ was also determined as 33.4m. The probability of exceedance of the wave height was also determined and the return period associated with this wave height was estimated to be 10 years.

(iii) Current:

The tidal current which is the dominant current in the waterway is used as the current load. The measure value of 1.06 m/s may seem inappropriate but this represents the maximum velocity.

4.3.1.1.3 Conclusion

Since the wind generated waves of $H_s = 0.56m$ is found to be greater than the interference peak, $H_i = 0.12m$ of ship induced wave, therefore wind waves are considered dominant and used as determinant load.

4.3.1.2. Dimensioning

The dimensioning involves the selection and determination of adequate armour units against wave and current attack. Furthermore, the underlayer (i.e. the filter layer), which represents a special part of the structure was also determined. Special attention was paid to:

(i) downslope migration
(ii) geotextile specification
(iii) granular specification, under which permeability, piping, segregation and internal stability were checked.

The filter distribution was also plotted. The new bank profiles to withstand wave run-up were also established. The whole procedure was repeated for wave heights of $H_s = 0.625m$ (return period of 30 years) and $H_s=0.65$ (return period of 50 years). These computation were done in order to establish the optimum design conditions which are necessary to establish the most economical balance between construction costs and damage (maintenance) costs such that the sum of the two costs is minimized. The whole procedure is further repeated for the three different types of chosen revetments. The following parameters established for the three different revetment types are highlighted below. (Full calculations are in Appendix I).
4.3.2 Computation for Rip-rap

The armour unit layer and underlay sizes determined for rip-rap under different wave situation are shown in fig. 4.2 below.

<table>
<thead>
<tr>
<th></th>
<th>Bank slope = 1 : 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H₁ = 0.56m</td>
</tr>
<tr>
<td></td>
<td>ξ₁ = 1.93</td>
</tr>
<tr>
<td></td>
<td>Return period = 10yrs</td>
</tr>
<tr>
<td>ARMOUR LAYER</td>
<td>W₅₀ = 32.24kg</td>
</tr>
<tr>
<td></td>
<td>Thickness = 415mm</td>
</tr>
<tr>
<td></td>
<td>H₂ = 0.625m</td>
</tr>
<tr>
<td></td>
<td>ξ₂ = 1.83</td>
</tr>
<tr>
<td></td>
<td>Return period = 30yrs</td>
</tr>
<tr>
<td>UNDERLAYER</td>
<td>D₅₀ = 1.20mm - 6.0mm</td>
</tr>
<tr>
<td></td>
<td>Thickness = 150mm</td>
</tr>
<tr>
<td></td>
<td>H₃ = 0.65m</td>
</tr>
<tr>
<td></td>
<td>ξ₃ = 1.79</td>
</tr>
<tr>
<td></td>
<td>Return period = 50yrs</td>
</tr>
<tr>
<td></td>
<td>Thickness =</td>
</tr>
<tr>
<td>GEOTEXTILE</td>
<td>O₂₀ = 1.50mm</td>
</tr>
<tr>
<td></td>
<td>Thickness = 9mm</td>
</tr>
<tr>
<td></td>
<td>RUN-UP ELEVATION</td>
</tr>
<tr>
<td></td>
<td>(above MSL)</td>
</tr>
<tr>
<td></td>
<td>CREST ELEVATION</td>
</tr>
<tr>
<td></td>
<td>(above MSL)</td>
</tr>
</tbody>
</table>

Fig. 4.2: Dimension characteristics determined for Rip-rap.

4.3.3. Computation for Open Stone Asphalt

The computation for Asphalt is also in Appendix I. The predominant load factor criterion for Open Stone Asphalt design is the dimensioning for thickness that will resist uplifting and sliding conditions simultaneously. Although the wave impact condition was also checked, but it is obvious that the derived asphalt layer thickness is substantial enough to resist any wave attack. Refer to Fig. 4.3 below for the dimensioned parameters for the Open Stone Asphalt condition.
### Table: Dimension Characteristics for Open Stone Asphalt

<table>
<thead>
<tr>
<th></th>
<th>Bank slope = 1 : 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( H_s = 0.56 \text{m} )</td>
</tr>
<tr>
<td>( \xi )</td>
<td>1.93</td>
</tr>
<tr>
<td>Return Period</td>
<td>10 yrs</td>
</tr>
<tr>
<td>ARMOUR LAYER</td>
<td></td>
</tr>
<tr>
<td>Thickness ( D_a )</td>
<td>0.50 m</td>
</tr>
<tr>
<td>UNDERLAYER</td>
<td></td>
</tr>
<tr>
<td>Thickness ( D_u )</td>
<td>1.20 mm - 6.0 mm</td>
</tr>
<tr>
<td>Thickness ( T )</td>
<td>150 mm</td>
</tr>
<tr>
<td>GEOTEXTILE</td>
<td></td>
</tr>
<tr>
<td>Thickness ( O_{G0} )</td>
<td>1.50 mm</td>
</tr>
<tr>
<td>Thickness ( T )</td>
<td>9 mm</td>
</tr>
<tr>
<td>RUN-UP ELEVATION</td>
<td></td>
</tr>
<tr>
<td>( \text{above MSL} )</td>
<td>+3.85 m</td>
</tr>
<tr>
<td>CREST ELEVATION</td>
<td></td>
</tr>
<tr>
<td>( \text{above MSL} )</td>
<td>+4.75 m</td>
</tr>
</tbody>
</table>

Fig. 4.3: Dimension characteristics determined for Open Stone Asphalt

4.3.4 Computation for Placed Block Revetment

The computation for placed block revetment assumes that the block are loose and not interlocked. Although a fill material is expected to be placed in any crevice that may develop during construction. Further specification of holes in the block is desirable in order to relieve the induce pressure load underneath the blocks. A secondary protective layer called the outfill layer, consisting of gravel is also specified. Apart from fulfilling the job of secondary protection, the outfill layer makes the placement of the blocks easier and furthermore it helps in attaining stability against sliding and control of settlement of the placed block due to any deformation that may be associated with the bank body. Refer to fig. 4.4. for the dimensioned parameters of the placed block revetment.
Bank slope = 1:4

<table>
<thead>
<tr>
<th></th>
<th>Return Period = 10yrs</th>
<th>Return Period = 30yrs</th>
<th>Return Period = 50yrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_s )</td>
<td>0.56m</td>
<td>0.615m</td>
<td>0.65</td>
</tr>
<tr>
<td>( \xi )</td>
<td>1.93</td>
<td>1.83</td>
<td>1.79</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ARMOUR LAYER</th>
<th>Thickness = 230mm</th>
<th>Thickness = 250mm</th>
<th>Thickness = 300mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>UNDERLAYER</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(i) Outfill</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( D_{50} )</td>
<td>20.0mm-24.0mm</td>
<td>20.0mm-24.0mm</td>
<td>20.0mm-24.0mm</td>
</tr>
<tr>
<td>Thickness</td>
<td>100mm</td>
<td>100mm</td>
<td>100mm</td>
</tr>
<tr>
<td>(ii) Filterlayer</td>
<td>1.20mm - 6.0mm</td>
<td>1.20mm - 6.0mm</td>
<td>1.20mm - 6.0mm</td>
</tr>
<tr>
<td>Thickness</td>
<td>150mm</td>
<td>150mm</td>
<td>150mm</td>
</tr>
<tr>
<td>GEOTEXTILE</td>
<td>1.50mm</td>
<td>1.50mm</td>
<td>1.50mm</td>
</tr>
<tr>
<td>Thickness</td>
<td>9mm</td>
<td>9mm</td>
<td>9mm</td>
</tr>
<tr>
<td>RUN-UP ELEVATION (above MSL)</td>
<td>+3.85m</td>
<td>+3.96m</td>
<td>+4.00m</td>
</tr>
<tr>
<td>CREST ELEVATION (above MSL)</td>
<td>+4.75m</td>
<td>+4.86m</td>
<td>+4.90m</td>
</tr>
</tbody>
</table>

Fig. 4.4: Dimension characteristics determined for Placed Block Revetment

### 4.4 Cost Analysis

The technical requirements have been considered and three different types of revetments have been designed. The next step towards the selection of the appropriate choice is the life cycle cost of such a choice. But it must be noted that apart from life cycle cost, the ease of construction may also be a decisive factor towards a final choice. Selection is also generally an iterative process involving many variables such as site location, labour, plant, materials, environmental acceptability and some other engineering properties already mentioned. For the purpose of the cost analysis in this study, the following variable parameters were used in the optimum design analysis:

(i) Location of project

(ii) Volume of bank filling

72
(iii) Revetment type
(iv) Wave condition
(v) Construction materials

4.4.1 Computation procedure

For the purpose of this analysis, unit costs of materials were chosen and based on the prevailing market prices of these materials. Other extraneous factors like transportation and labour were also assumed in the unit cost. The cost computation is based on computation of capital cost and damage cost for the three revetment types and evaluated repeatedly for the return period (or probability of exceedance) for 10 years, 30 years and 50 years. All cost calculations are shown in Appendix I.

4.4.1.1. Capital Cost

The Capital Cost is basically the initial investment or construction cost necessary to put the structure in place.

4.4.1.2 Damage (or Maintenance) Cost

Damage cost consists of both direct and indirect damage costs. Direct damage cost is basically the cost required for maintenance throughout the lifespan of the structure. The indirect damage cost is the cost associated with monitoring and operation of the revetment which certainly is beyond the scope of this study since the information on management or organisational set-up of the administering authority is not available.

Direct damage cost:

The computation of this cost is based on the loss of stability of some parts of the revetment under consideration. Estimate is based on the cost of repair arising from the most vulnerable part of the revetment type. For the rip-rap the most vulnerable part is the armour layer. For the open stone asphalt and placed block, the armour and the underlayer are the most vulnerable. The damage cost analyses further assume a maintenance philosophy in which repair is only carried out once in a year. Certain damage percentages are assumed in this respect to facilitate computation. Furthermore annual interest rate of 15% and lifespan of 50 years are assumed for the structures. The life cycle capitalized damage cost is calculated with the following formulae:

\[
\text{Capitalized damage cost} = \text{p.w.f} \times \text{Annual damage cost}
\]

where \( \text{p.w.f} = \text{present worth factor} \)
\[ i = \text{interest rate} = 15\% \]
\[ n = \text{life span} = 50 \text{ year} \]

4.4.1.3. Total Cost

\[ \text{Total Cost} = \text{Construction Cost} + \text{Capitalized Damage Cost} \]

4.4.2 Cost Summary

The cost summary for all the three types of the revetments are shown in the tables below. Refer also to figures 4.5 - 4.7 for the cost curves for the three revetment.

\[
\begin{array}{|c|c|c|c|}
\hline
\text{Design Wave Height} & H_s = 0.56m & H_s = 0.625m & H_s = 0.65m \\
\hline
\text{Item} & 4,116 & 1,292 & 853 \\
\text{1 Annual Damage Cost} & 27,415 & 8,605 & 5,681 \\
\text{2 Capitalized Damage Cost} & 21,661 & 22,312 & 23,637 \\
\text{3 Construction Cost} & 49,076 & 30,917 & 29,318 \\
\hline
\end{array}
\]

Table 4.1: Cost Summary for RIP-RAP per metre length

\[
\begin{array}{|c|c|c|c|}
\hline
\text{Design Wave Height} & H_s = 0.56m & H_s = 0.625m & H_s = 0.65m \\
\hline
\text{Item} & 682 & 175 & 88 \\
\text{1 Annual Damage Cost} & 4,542 & 1,166 & 586 \\
\text{2 Capitalized Damage Cost} & 40,979 & 41,495 & 41,679 \\
\text{3 Construction Cost} & 45,521 & 42,661 & 42,265 \\
\hline
\end{array}
\]

Table. 4.2: Cost Summary for OPEN STONE ASPHALT per metre length.
Table 4.3: Cost Summary for Placed Block per metre length.

From Fig. 4.5 - 4.7, the minimum points of the three total cost curves (which represent the optimum design) are shown below:

(a) For RIP-RAP, minimum point occurs at:

- Design wave height, \( H_s = 0.66 \text{m} \)
- Total Cost = 29,100 per metre length
- Construction Cost = 24,500 per metre length
- Maintenance Cost = 4,600 per metre length

(b) For OPEN STONE ASPHALT, minimum point occurs at:

- Design wave height, \( H_s = 0.665 \text{m} \)
- Total Cost = 42,100 per metre length
- Construction Cost = 41,800 per metre length
- Maintenance Cost = 300 per metre length

(c) For PLACED BLOCK, minimum point occurs at:

- Design wave height, \( H_s = 0.60 \text{m} \)
- Total Cost = 32,400 per metre length
- Construction Cost = 31,500 per metre length
- Maintenance Cost = 900 per metre length
FIG. 4.5: COST CURVES FOR RIP-RAP

- Cost per metre in thousand
- Design wave height in metres
- Total
- Construction
- Damage (or maintenance)
- $H_s = 0.66 \text{ m}$

The graph shows the cost curves for rip-rap construction, with the design wave height plotted on the x-axis and cost per metre on the y-axis. The points and curves indicate the relationship between the design wave height and the cost, with a specific point marked at $H_s = 0.66 \text{ m}$. The graph also includes cost components such as construction and damage (or maintenance) costs.
FIG. 4.6: COST CURVES FOR OPEN STONE ASPHALT

DESIGN WAVE HEIGHT IN METRES

$H_s = 0.665 \text{ m}$
FIG. 4.7: COST CURVES FOR PLACED BLOCK

DESIGN WAVE HEIGHT IN METRES

$H_s = 0.60m$
4.5 Conclusion

From the results presented above, the obvious choice for selection is the Rip-rap because it is by far the cheapest and also accommodates the second highest hydraulic load (wave condition) out of all the three alternatives. But there are other reasons which support the use of Rip-rap as the most preferable and these are:

4.5.1 Engineering properties

(a) Flexibility:

Rip-Rap represents the most flexible out of the three alternatives. It responds easily to slight deformation that may arise out of the sub-soil structure. Asphalt revetment is also flexible but problem of piping may easily arise out of the voids or joints.

(b) Durability:

The durability of Rip-rap is not in doubt. It can be easily repaired at convenient time and sometimes self healing. But little damage may result into substantial damage should Open stone and Placed block be left unrepaired immediately.

(c) Permeability:

Rip-rap is the most permeable and damage to the filter layer will be considerably reduced when compared to the other two revetment types. Underlayer erosion problem (soil-migration) is usually common with Open stone. Place block permeability is better than that Open stone.

(d) Hydraulic roughness:

The ability to reduce run-up or overtopping is another advantage for Rip-rap because the other two have more or less smooth surfaces.

4.5.2 Construction method

The construction method for the three alternatives is difficult but the relative ease of the mode of construction is a determinant factor for selection. It is preferable and cheaper that construction be carried out from the land side rather than from inside the channel. Rip-rap has the advantage that construction can easily be executed from the land. A special problem associated with both Open Stone and Place block is the construction under water. This requires special techniques and equipment. The cost arising from use these special requirements may soar the already existing cost.
4.5.3. Availability of Material

From the data report, Rip-rap can only be obtained at 25 km from the project site. Open stone asphalt can easily be prepared since the site location lies in an oil producing region. Cement for the concrete is produced some 15 km away. But despite all these facts, the Rip-rap cost of material is by far the cheapest.

4.5.4. Maintenance

Rip-rap is the most easy to repair out of the three alternatives. The quarry-stones can easily be replaced without further consequential damage to the overall structure. Repair can even be deferred within some relative short period if money is not available immediately. But maintenance of Open stone and Placed block can not be deferred indefinitely or else further incalculable damage may affect the integrity of the whole bank. And also scouring of the toe protection for the Open stone may be difficult to repair.

4.5.5. Environmental

The use of Rip-rap has an undue advantage of natural, human and visual acceptability. It has an aesthetic quality that rhymes with the environment.

4.5.6. Recommendation

From the various functional, technical and financial analyses carried out in this chapter it is my considered opinion that the selection or choice of Rip-rap is the most adequate or appropriate revetment to be used as bank protection work for the banks of Opobo town adjacent to the Strong Face Creek. Furthermore, various analyses can still be carried out to obtain other feasible solutions if the following variable parameters are considered:

(i) Other types of revetments (e.g. gabion mattress)
(ii) Crest elevation of bank
(iii) Slope angle
(iv) Composite revetment
(v) Interest rate
(vi) Life span of structure
(vii) Water levels
CHAPTER 5

APPLICATION AND VERIFICATION WITH STEENZET/1 AND ANAMOS MODELS

5.1 Introduction

The development of the two models STEENZET and ANAMOS is based on calibration with some physical data in which a fair similarity is obtained between the models' response and actual pressure distribution on the placed block revetment. The overall objective of the two models is to determine the response in the underlayer in form of uplift pressure and hydraulic gradient resulting from wave distribution on the bank slope and interaction between the soil, water and bank geometry. The transformation of the wave height into time and length dependent wave pressure distribution was achieved through model test (Bezuijen et al, 1988) and empirical relations derived for regular wave (Banach 1987; Klein Breteler 1988). The accuracy of the model test and wave linearization may be heightened in future through the use of accurate physical data and measuring equipment. The basic underlying principle of those two models is the presence of characteristic equations which are the principle of conservation and transport that govern physical phenomenon. The STEENZET/1 model considers mainly the external and internal hydraulic conditions of the revetment while the ANAMOS goes further to determine the actual response of the structure. Furthermore the two models are assumed valid for relatively closed blocks and where permeability of the filter layer is larger than that of the placed block.

5.2 STEENZET/1 model

5.2.1 Description

The STEENZET/1 program uses a numerical approach to compute the pressure distribution for various time steps and different geometries. The differential equation (3.35) of chapter 3 is replaced by an algebraic finite difference equation. An explicit finite difference analog (equation 5.1 below) is used to find the value of the dependent variable, $\phi$, (written in terms of independent variables) at a predetermined number of discrete points (nodes) along the revetment slope. The solution of the set of the difference equations is then solved numerically and simultaneously using the result of wave measurements (performed in a wave flume) as boundary conditions.

$$
\phi_i = \frac{1}{1 + 2 \frac{k_b D}{K L^2}} \left( \frac{k_b D}{K L^2} (\phi_{i-1} + \phi_{i+1}) + \phi_{i,1} \right)
$$

\ldots(5.1)
where

\[ \phi_i = \text{the piezometric head in the filter layer at joint } i \text{ (m)} \]

\[ \phi_{t,i} = \text{the piezometric head on the cover layer at joint } i \text{ (m)} \]

\[ b = \text{thickness of the filter layer (m)} \]

\[ D = \text{block thickness (m)} \]

\[ L = \text{block length dimension (m)} \]

\[ k = \text{the filter layer permeability (m/s)} \]

\[ k' = \text{the cover layer permeability (m/s)} \]

Fig. 5.1: Finite difference scheme for STEENZET/1

The formulas for flow in narrow joints were derived from special permeability test to determine the permeabilities of the placed block and filter. The Forchheimer relation was used to incorporate laminar flow as well as turbulent flow such that

\[ \Delta i = a_q q + b_q q^2 \]

where

\[ \Delta i = \text{hydraulic gradient} \]

\[ q = \text{specific discharge (m/s)} \]

\[ a_q = \text{Forchheimer's coefficient for laminar flow (m/s)} \]

\[ b_q = \text{Forchheimer's coefficient for turbulent flow (m/s)} \]

and the derived permeability, \( k \) from \( q = k_i \) is given below

82
\[ k = \frac{-a_f + \sqrt{a_f^2 + 4b_f i}}{2b_f i} \] ... (5.3)

The variation of the phreatic line in the bank structure is incorporated in the solution and \( \Phi_i \) is assumed equal to the vertical position of such a phreatic surface at the particular location where it exists in the filter layer. The program assumes that flow through the cover layer is homogenous as long as the length and width of the blocks are small compared to the leakage length \( \Delta \) and that the decisive parameter for calculation of the internal pressure is \( \Delta \), and not the position of the phreatic surface. The program brings out graphical output in form of

(i) Wave pressure (KPa) on block against time (sec)
(ii) Block movement (m) against time (sec)
(iii) Uplift pressure (KPa) on block against slope elevation (m)
(iv) Piezometric head (m) in block and in filter against slope elevation (m) at the time of run-down
(v) Pore pressure (KPa) in filter layer underneath placed block against slope elevation (m)
(vi) Hydraulic gradient against slope elevation (m)

5.2.2 Application

The Steenzet/1 model was applied to the empirically designed placed block of chapter 4 for wave height, \( H_s = 0.56 \text{m} \) and varied for five different conditions as described below:

(a) Case 1: Block with no hole, with block spacing, \( s = 0 \text{mm} \) and with allowance for flow and block movement (it was observed that the value of \( a_f \) and \( b_f \) for the block could not be calculated by the program and values of \( a_f = 0.01 \text{ s/m} \) and \( b_f = 0.01 \text{ (s/m)}^2 \) were assumed.

(b) Case 2: Block with no hole, with block spacing, \( s = 5 \text{mm} \) and with allowance for flow and block movement.

(c) Case 3: Block with no hole with block spacing, \( s = 5 \text{mm} \) and allowance for no flow and no block movement.

(d) Case 4: Block with hole equal to 1% of the block area, with \( s = 5 \text{mm} \) and with flow and movement.
Case 5: Block with the same condition as case 4 but with no flow and no block movement.

<table>
<thead>
<tr>
<th></th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>Case 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phreatic line</td>
<td>7.118m</td>
<td>6.844m</td>
<td>6.844m</td>
<td>6.135m</td>
<td>6.135m</td>
</tr>
<tr>
<td>Convergence of phreatic line</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Leak height</td>
<td>0.60m</td>
<td>0.60m</td>
<td>0.60m</td>
<td>4.20m</td>
<td>4.20m</td>
</tr>
<tr>
<td>Block displacement</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Maximum uplift pressure</td>
<td>2.2KPa</td>
<td>2.4KPa</td>
<td>2.38KPa</td>
<td>4.8KPa</td>
<td>4.8KPa</td>
</tr>
<tr>
<td>Pore pressure in filter at point of rundown</td>
<td>2.19KPa</td>
<td>2.3KPa</td>
<td>2.30KPa</td>
<td>3.68KPa</td>
<td>3.68KPa</td>
</tr>
<tr>
<td>Hydraulic gradient at the point of rundown</td>
<td>0.16</td>
<td>0.13</td>
<td>0.13</td>
<td>-0.02</td>
<td>-0.02</td>
</tr>
<tr>
<td>Top layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$a_f$</td>
<td>0.01s/m</td>
<td>21.4s/m</td>
<td>21.4s/m</td>
<td>153400</td>
<td>153400</td>
</tr>
<tr>
<td>$b_f$</td>
<td>$(s/m)^2$</td>
<td>$(s/m)^2$</td>
<td>$(s/m)^2$</td>
<td>$1.67E+7$</td>
<td>$1.67E+7$</td>
</tr>
<tr>
<td>Filter layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$a_f$</td>
<td>125s/m</td>
<td>125s/m</td>
<td>125s/m</td>
<td>125s/m</td>
<td>125s/m</td>
</tr>
<tr>
<td>$b_f$</td>
<td>$(s/m)^2$</td>
<td>$(s/m)^2$</td>
<td>$(s/m)^2$</td>
<td>$(s/m)^2$</td>
<td>$(s/m)^2$</td>
</tr>
</tbody>
</table>

Table 5.1: Result of the application of the Steenzet/1 model

Note: The Steenzet/1 model does not request for the parameters of the material in the block spacing. Refer to the print-out of the result in Appendix II
5.2.3 Observation and Result

Although the Steenzet/1 program does not specifically state whether a placed block is stable or not but there are calculated parameters that indicate stability conditions and examples of such are the leak height, hydraulic gradient and block displacement.

For cases 1, 2 and 3; the placed block may be considered stable because of the short leakage height associated with these cases. The no-hole condition of the block in these three cases may be viewed as preventing the influence of the wave attack on the block.

For cases 4 and 5; the leak height is excessively high and it can reasonably be concluded that the block is not stable. It can also be observed that the flow and movement input specification seemed not to have any appreciable effect on the construction.

The above conditions are also calculated with the Anamos model described below. Further sensitivity tests are carried out and described in chapter 6.

5.3 ANAMOS Model

5.3.1 Description

The Anamos model is a semi-analytic and semi-empirical method. The program uses the empirical wave load approximation at the time of run-down as described in paragraph 3.2.5.2 as boundary conditions (regression analysis of wave pressure registration) along the slope. An analytical solution of equation 3.35 is done to find the maximum pressure difference between the cover layer and filter layer. The model focuses on the stability of the most vulnerable part of the revetment, that is the block that rests only in that of the filter which is adjacent to the row below the current block. Loose blocks are usually found in this row. The model determines the strength of the placed block necessary to balance the internal hydraulic load resulting from the external wave load. During computation of the internal hydraulic load, the model assumes:

(i) that the block will move after certain critical value of resultant external load

(ii) that the consequent of this movement will induce a pressure reduction \( \Phi_{\text{red}} \), hence internal pressure load on block is \( S \) where

\[
S = S_0 - \Phi_{\text{red}} \quad \ldots\ldots(5.4)
\]

and

\[
\Gamma_b = \frac{S_0}{S} = \frac{(S + \Phi_{\text{red}})}{S} \quad \ldots\ldots(5.5)
\]

where

\( S_0 \) = maximum pressure difference across the top layer

\( \Gamma_b \) = pressure reduction multiplication factor
The strength (or the resistance), $R$ of the placed block to resist the maximum allowable upward pressure across the cover layer is computed and the following are assumed as part of the strength:

(iii) that friction between blocks are mobilized as part of the strength of a steady block as soon as the effective weight of the block is exceeded

(iv) that inertia forces are mobilized as part of the strength of a moving block as soon as the effective weight and the frictional resistance are resisted.

The multiplication factors for the contribution of friction and inertia forces are given as

$$R = R_0 \Gamma_s \quad \ldots(5.6)$$

$$\Gamma_s = \Gamma_{s1} + \Gamma_{s2} - 1 \quad \ldots(5.7)$$

where

$R_0 =$ effective weight of weight of block

$\Gamma_{s1} =$ multiplication factor for friction

$\Gamma_{s2} =$ multiplication factor for inertia

$\Gamma_s =$ combined multiplication factor of the effective weight to cater for the friction and inertia forces.

5.3.2 Application

The Anamos model was also applied to the empirically designed placed block of chapter 4 for wave height, $H_s = 0.56m$ and the following conditions were calculated:

(a) Case 1: Block with no hole and no spacing (actually $s = 0.5mm$ because of the limitation of the model)

(b) Case 2: Block with no hole, with block spacing, $s = 5mm$ and with spacing filling.

(c) Case 3: Block with hole, with block spacing, $s = 5mm$ and with filling.

(d) Case 4: Block with hole, with block spacing, $s = 5mm$ and without filling.

(e) Case 5: Block with hole, with spacing, $s = 5mm$, without filling and with reduced friction
## Case 1 Case 2 Case 3 Case 4 Case 5

<table>
<thead>
<tr>
<th>Leak height</th>
<th>0.98m</th>
<th>1.139m</th>
<th>0.998m</th>
<th>0.128m</th>
<th>0.128m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block displacement</td>
<td>0.007m</td>
<td>0.006m</td>
<td>0.006m</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Maximum uplift pressure</td>
<td>0.40m</td>
<td>0.405m</td>
<td>0.401m</td>
<td>0.223m</td>
<td></td>
</tr>
<tr>
<td>Top layer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a_f)</td>
<td>3528s/m</td>
<td>4662</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(b_f)</td>
<td>260E03</td>
<td>3027E03</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Permeability</td>
<td>0.0003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0152</td>
<td>0.152</td>
</tr>
<tr>
<td>REMARK</td>
<td>UNSTABLE</td>
<td>UNSTABLE</td>
<td>UNSTABLE</td>
<td>STABLE</td>
<td>STABLE</td>
</tr>
</tbody>
</table>

**Table 5.2: Result of application of Anamos model**

Filter layer: \(a_f = 153\) s/m \((s/m)\) (constant for the five cases)
\[b_f = 1638\] (s/m)\(^2\) (constant for the five cases)

### 5.3.3 Observation and Result

The Anamos program gives more insight into the stability of the placed block by stating whether a block is stable or not. It gives further help by stating whether stability condition can be achieved if the parameters are varied by 10%.

For cases 1 and 2; it was observed that the placed block was not stable due to the fact that a relatively impermeable condition was imposed. It is possible that elevation of the phreatic line in the filter layer was increased to the level of the wave front by the program despite the impermeable nature of these cases. Case 2 condition for both Anamos and Steenzet/1 are similar in terms of loading conditions. But we can notice large differences in their computation results. Case 3 of Anamos also shows that the placed block is unstable, but immediately the block space filling material in between the blocks is omitted as shown in cases 4 and 5; the placed block becomes very stable and with a high jump in permeability value of the block. This result portrays the presence of the block space filling material as negating against the permeability of the placed block and its removal will enhance the
stability of the block even if the frictional force is reduced. In practical situation, blocks may be with or without spacing and most probably with filling material.

5.4 Conclusion

5.4.1 There are discerning basic differences between the Steenzet/1 and Anamos in that Steenzet/1 focuses more on the applied and resultant load parameters on and underneath the placed block. It is a useful tool in examining the boundary between the top layer and underlayer during loading condition for various times. Steenzet/1 does not allow for variation of block space parameters and it must also be realised that the program does not allow for no block spacing. The program further assumes that friction is mobilized all the time. The Steenzet/1 program is suitable for individual block calculation.

5.4.2 The Anamos model on the other hand is a more useful tool than Steenzet/1 model. It provides for wider choices of load parameter input and strength output. The model may be very useful for design analysis if most of the limitations are modified in the future. The model may be adjusted to included computation for zero block spacing and no friction situation. The permeability formula of the cover layer may need to be studied further. A comparative table for Steenzet/1 and Anamos for similar conditions is shown below for case 2 of both Steenzet/1 and Anamos application:

<table>
<thead>
<tr>
<th></th>
<th>Steenzet/1</th>
<th>Anamos</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leak height</td>
<td>0.60m</td>
<td>1.139m</td>
</tr>
<tr>
<td>Block displacement</td>
<td>0</td>
<td>0.006m</td>
</tr>
<tr>
<td>Top layer a_t</td>
<td>21.4 s/m</td>
<td>4662 s/m</td>
</tr>
<tr>
<td></td>
<td>3.10E03 (s/m)^2</td>
<td>3027E03 (s/m)^2</td>
</tr>
<tr>
<td>Filter layer a_r</td>
<td>125 s/m</td>
<td>153 s/m</td>
</tr>
<tr>
<td></td>
<td>1360 (s/m)^2</td>
<td>1638 (s/m)^2</td>
</tr>
</tbody>
</table>

Table 5.3: Comparative table for Steenzet/1 and Anamos models

5.4.3 Theoretically, Steenzet/1 is more powerful, but its application requires an enormous lot of data input (especially the distribution of pressures of over time and place). Most of these data are stored in files of measured pressures, but there is a tendency for deficits in these records. Anamos is a very simplified model, but yet data about spacing, filling and filter materials are expected to be fed into the computer. Although, Anamos is much more user-friendly, it may be said that its reliability is not greater than that of Steenzet, provided Steenzet contains good pressure data. The two model
applications are definitely useful and comparable to the empirical formula for placed block. The models have the advantages of varying several parameters and characteristics of the revetment and base material at a relatively fast speed. But they are not yet exact in that there are several limitations still associated with them. The parameters in the non-linear equation for the permeability of the placed block may need further investigation and analysis. The computed Forchheimer's coefficients listed in the table above for the two models differ a lot for the top layer. It might be better to ignore friction forces because the mobilization of this force is a chance occurrence. Although for situation like inland waterway it may be desirable for economic reason but for a dike on an extremely active coast, it may become unreliable and hazardous. Also, it is important that the rise or fall of the phreatic line in the base soil during wave attack should be studied rather than the arbitrary assumption that it is on the same elevation as the wave front. It can reasonably be concluded that despite the inability of the semi-empirical method to transform wave height into wave pressures, the empirical formula in terms of wave height is still more useful and reliable for block dimensioning than the models. One may question the reliability of the empirical formula because the degree of permeability is not reflected but already built placed block revetment designed with empirical formula are abound to validate the empirical approach. The models require further modification before complete reliability can be guaranteed.
CHAPTER 6

SENSITIVITY TEST AND COMPARISON OF RESULT

6.1 Introduction

The use of the two models have been highlighted in the previous chapter, in which the models were applied to the semi-empirically designed placed block on sand, and with granular and geotextile underlayer. As discussed in chapter 5, the Steenzet/1 model is particularly useful for hydraulic response in the zone from top of the cover layer into the underlayer. The response is in form of wave pressure, uplift pressure, displacement, hydraulic gradient and leak height. It is difficult to distinguish an instability condition in the Steenzet/1 output, since for example, a slight displacement may not necessarily constitute an instability condition. The model is therefore particularly useful when a detailed specified hydraulic response in the underlayer is required. This chapter is therefore particularly devoted to sensitivity test based on Anamos model alone.

6.2 Test

The following condition for different revetment profiles were calculated with Anamos model and the result are presented below:

6.2.1 Test 1 (same placed block as in chapter 5 but with different variations in block hole, block spacing,e.t.c)

Fig. 6.1: Test 1 profile

\[ A_h = \text{Hole area,} \]

\[ s = \text{block spacing} \]

\[ 90 = \text{Block spacing filling material,} \] \( D_{50} = 1.2 \text{ mm} \]
<table>
<thead>
<tr>
<th>Condition 1</th>
<th>$A_h = \frac{h \times BL}{s}$</th>
<th>$s$ (mm)</th>
<th>Filling</th>
<th>No filling</th>
<th>RESULT</th>
</tr>
</thead>
<tbody>
<tr>
<td>nh-ns-nf</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>Yes</td>
<td>UNSTABLE block</td>
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</table>

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<th>Condition 2</th>
<th>1%-5%</th>
<th>0</th>
<th>-</th>
<th>Yes</th>
<th>STABLE block at $A_h \geq 5%$</th>
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</thead>
<tbody>
<tr>
<td>h-ns-nf</td>
<td></td>
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</tbody>
</table>

<table>
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<tr>
<th>Condition 3</th>
<th>0</th>
<th>0.5mm-30mm</th>
<th>Yes</th>
<th>-</th>
<th>UNSTABLE block for all conditions.</th>
</tr>
</thead>
<tbody>
<tr>
<td>nh-s-f</td>
<td></td>
<td></td>
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</table>

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<th>Condition 4</th>
<th>0</th>
<th>0.5mm-30mm</th>
<th>-</th>
<th>Yes</th>
<th>STABLE block at $s \geq 5\mm$</th>
</tr>
</thead>
<tbody>
<tr>
<td>nh-s-nf</td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Condition 5</th>
<th>1%-5.6%</th>
<th>0.5mm-30mm</th>
<th>Yes</th>
<th>-</th>
<th>UNSTABLE block for all conditions.</th>
</tr>
</thead>
<tbody>
<tr>
<td>h-s-f</td>
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</table>

<table>
<thead>
<tr>
<th>Condition 6</th>
<th>1%-5.6%</th>
<th>0.5mm-30mm</th>
<th>-</th>
<th>Yes</th>
<th>STABLE block at $s \geq 0.5\mm$ $A_h \geq 2%$</th>
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<tbody>
<tr>
<td>h-s-nh</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Note: $h = \text{hole}; s = \text{spacing}; f = \text{filling}$  
$nh = \text{no hole}; ns = \text{no spacing}; nf = \text{no filling}$

Table 6.1: Summary of sensitivity Test 1

The result of conditions 3, 4 and 5 are highlighted with the graphs plotted for leak height, $\lambda$ against block spacing, $s$ for conditions 3 & 4 (refer to Figs 6.2 & 6.3), and leak height, $\lambda$ against hole as percentage block area for condition 5 (refer to Fig. 6.4).

6.2.1.1 Conclusion

All the results indicate that

(a) the placed block will always be unstable insofar there is a filling material in the block spacing and no matter what value of block hole area you specify.
(b) the stability is assured for all blocks with spacing containing no filling material.

(c) the block hole area, if varied properly can help in achieving stability

(d) the friction factor, $\Gamma_S$, remained constant throughout the test indicating that the friction is permanently mobilized

(e) the inertia factor, $\Gamma_L$, is always greater than one for all unstable conditions, and remains equal to one for all stability conditions

(f) the pressure reduction factor, $\Gamma_p$, decreases with higher stability conditions.

6.2.2 Test 2 (Placed block with same condition in chapter 5 but the base material consisting of sand is replaced with clay)

![Diagram of Test 2 profile]

- Block thickness, $D = 0.230$ m
- Outfill (gravel), $D_{50} = 24$ mm
  (thickness = 0.10 m)
- Filter (sand), $D_{50} = 1.2$ mm
- Geotextile, $t_b = 9$ mm
  $O_{90} = 1.50$ mm
- Base material (clay), $D_{50} = 0.01$ m

Fig. 6.5: Test 2 profile

$A_t = $ Hole area, $s = $ block spacing

6.2.2.1 Conclusion

The result of test 2 is the same as that of test 1 despite the use of clay as a base material. The only explanation for this may be the thickness of the underlayer which attenuates the larger part of the uplift pressure. One observation of this result is that the hydraulic gradient parallel to the bank slope increases as the stability of the block increases.
6.2.3 Test 3 (Placed block on clay base material without outfill layer and with a 50mm filter layer of sand)

Block spacing filling material, $D_{50} = 1.2 \, \text{mm}$

Block thickness, $D = 0.230 \, \text{m}$

Filter (sand) $D_{50} = 1.2 \, \text{mm}$

thickness $= 50 \, \text{mm}$

Geotextile, $t_g = 9 \, \text{mm}$

$O_{90} = 1.50 \, \text{mm}$

Base material (clay), $D_{50} = 0.1 \, \text{mm}$

Fig. 6.6: Test 3 profile $B = 0.6 \, \text{m}$ $L = 0.6 \, \text{m}$

6.2.3.1 Conclusion

All test results carried out for all conditions for Test 3 indicate stability of the placed block. This result of Test 3 indicates that a placed block is more stable on a clayey subsoil than on a sandy subsoil and that it may be advantageous to put the placed block directly on clay. The simple reason is that clay does not allow for the development of uplift pressure or large pressure variation because of its extremely low permeability.

6.3 Observation on the Limitations of the Anamos model

Several other computer runs could not be carried out as a result of some inherent limitations of the model. Some of these observed limitations are listed below

(a) Model does not allow for placed block to be placed directly on clay (that is no outfill or no filter layer condition is possible).

(b) The model does not allow for clay layer to be used as an underlayer.

(c) The model does not allow for complete zero spacing of blocks.

(d) All other material specification cannot be varied beyond the specification of the model manual.
Figure 6.2 Test 1: condition 3
Figure 6.3 Test 1: condition 4
Figure 6.4 Test 1: condition 5
CHAPTER 7

7 GENERAL CONCLUSION AND RECOMMENDATION

7.1 General

This study has explored different existing literature on various types of bank protection methods notably revetments. The study further investigated the various causes of failures and mechanisms of these failures particularly the failure associated with hydraulic boundary conditions. The load parameters, design criteria and various design methods for three different types of revetments, (namely rip-rap, open stone asphalt and placed block) were studied, and selectively designed and applied as solutions for a particular bank problem (case study) in a low-land coastal waterway in southern part of Nigeria. The study also delved into the usefulness of model design for placed block revetment. Practically, the objectives of the study have been realised in that many different applications of bank protection methods have been identified; and that it is primarily essential that river banks of eroding channels should be given adequate protection and proper attention through identification of erosion processes and other factors that are likely to affect the integrity of the bank. The study also noticed that the present available design and protection methods are comparatively and fairly adequate when used as solutions to bank problems. And that the choice of a solution to a particular bank problem is basically governed by several varying and conflicting factors such as:

(a) Engineering properties of the revetment type
(b) Constructional constraints
(c) Availability of revetment material
(d) Environmental acceptability
(e) Economic consideration

7.2 Failure Modes and Load Parameters

The study can reasonably conclude that bank failure usually arises out of

(a) geotechnical composition, instability and deformation of the bank material
(b) hydraulic loading on the bank in form of waves generated by ship and winds, current, induced pore pressure within the bank and scouring

(d) environmental damage due to act of man or plant or animal

From the study, it can also be further deduced that generally, banks with mild slope are less susceptible to geotechnical failure and to a larger extent the hydraulic loading on such banks are greatly reduced which evidently will require the use of lesser protection. It is also noted and could be inferred that presently the most outstanding problem during wave consideration for revetment design is the choice an appropriate design wave height and the transformation of this wave height into accurate pressure load. This is greatly influenced by the stochastic nature of wind waves. This is an aspect in which further research in form of physical and model studies is recommended.

7.3 Empirical Method

The following conclusion could be deduced for the empirical method using the case study (a third world country) as a barometer:

(a) It is reasonably cheaper to use rip-rap as revetment where quarry stones are available.

(b) Rip-rap fulfills a major part of engineering properties like strength, durability, flexibility e.t.c.

(c) The design formulas and requirement for rip-rap are not as tedious as the other revetments.

(d) Notably, the design wave height, \( H \) is a decisive factor for most revetment except asphalt.

(e) The empirical formulas for revetment design do not incorporate the degree of permeability of the armour layer. A further study is recommended in this line to see the differential advantages in performance of closely packed and loosely packed rip-rap.

7.4 Model Method For Placed Block

The major conclusions for Steenzet/1 and Anamos models have been made in chapters 5 and 6. The modelling approach although difficult (because of many unscalable parameters), it is necessary to reiterate that

(a) the permeability calculation of the toplayer needs to be modified because it is evident from all computations that
the permeability of the armour layer as a function of

(i) block hole size

(ii) spacing between blocks

(iii) the filling material and its characteristics in the block spacing

is the decisive parameter to establish the stability of the armour layer if all other parameters remain constant.

(b) the many assumptions leading to the development of Anamos model need to be revised further through the use of full scale model tests. Example of such assumptions are:

(i) the assumed parabolic motion of block during movement which results in the inertia multiplication factor, $\Gamma$, should be reviewed

(ii) the assumption that piezometric head on cover layer is zero in the region above the run-down elevation may also need to be reviewed.
APPENDIX I

DETERMINISTIC METHOD FOR THE DESIGN OF REVETMENT FOR CASE STUDY

--- STRONG FACE CREEK, NIGERIA ---

TYPES OF REVETMENTS
The design is carried out for three types of revetments namely

(i) Rip-rap
(ii) Open Stone Asphalt
(iii) Placed Concrete Block

1. NAME OF WATERWAY: Strong Face Creek

2. DATA
The data is as given in Appendix II and partially in chapter 4.

3. WATER LEVELS AND FLOW OBSERVATION
Tidal range = 1.50m (Normal)
             = 2.30m (Spring)
Spring high tide = 1.10m above M.S.L
Spring low tide = 1.20m below M.S.L
Maximum predicted waterlevel = 2.00m above M.S.L
Ground water level = 0.50m above M.S.L
Maximum flow velocity, v = 1.05m/s (Flood tide)
                         = 1.06m/s (Ebb tide)
Roughness coefficient, k = 2.5 \times 10^{-4}
Chezy's coefficient, c = 83 m^{1/2}/s

4. WATERWAY PLAN FORM AND PROFILE
Waterway is a navigable canal
Location is about 7km from the coastline
Length of waterway = 10km
Average top width = 500m
Length of protection = 860m (left bank only)
Existing bank slope = 1 : 5 (average)
Water depth (at left bank) = 6.0m
Maximum water depth = 9.0m (occurring at 70m from the left bank)
Average water depth = 5.5m (refer to fig. II.)

5. BANK SOIL PROPERTIES
Sand (coarse):
Specific weight, $\gamma_s$ = 25.70 KN/m$^3$
Porosity, $n$ = 0.37 (assumed)
Angle of internal friction, $\phi$ = 30$^0$
Bearing capacity, $P_u$ = 6000 - 10,000 KN/m$^2$

Clay (fine):
Specific weight, $\gamma_s$ = 20.00 KN/m$^3$
Porosity, $n$ = 0.35 (assumed)
Cohesion, $c$ = 30.00 KN/m$^2$
Angle of internal friction, $\phi$ = 0
Liquid limit, L.L = 0.56 - 0.89
Plastic limit, P.L = 0.26 - 0.39
Bearing capacity, $P_u$ = 4,000 KN/m$^2$
Surcharge on bank = 15.00 KN/m$^2$

Other soil properties:
Refer to chapter 4 and Appendix II

6. GEOTECHNICAL ANALYSIS

Fig. I.1: Schematic representation and soil profile of the left bank of Strong Face Creek.
6.1 Determination of Maximum Slope

6.1.1 The maximum slope condition for sliding of an asphalt revetment is

\[ \tan \alpha \leq \tan \phi \left( 1 - \frac{\rho_s}{\rho_s} \right) \]

\[ \therefore \tan \alpha \leq \tan 30^\circ \left( 1 - \frac{1000}{2570} \right) \]

\[ \therefore \alpha \leq 19.5^\circ \]

\[ \therefore \cot \alpha \geq 2.82 \quad \text{(i.e. max. slope of 1:2.82)} \]

\[ \therefore \text{Use } \cot \alpha = 4 \quad \text{(i.e. bank slope of 1:4)} \]

or \[ \alpha = 14.03^\circ \]

6.1.2 The maximum slope condition for stability of a cohesionless bank material where groundwater flows out freely are:

(i) For soil material below water:

\[ \tan \phi > \tan \alpha \left/ \left( 1 - \frac{\left[ \rho_s / (\rho_s - \rho_w) \right] \times [\text{closa}] }{\rho_s} \right) \right. \]

assuming \[ \tan \phi = \tan \alpha = 0.25 \quad (\text{potential hydraulic gradient along bank slope}) \]
substituting all parameter values,

\[ \therefore \tan \phi > 0.30 \]

but \[ \tan 30^\circ = 0.577 > 0.30 \quad \text{hence slope is O.K} \]

(ii) For soil material above water:

\[ \tan \phi > \frac{\tan \alpha}{1 - \frac{\rho_w}{\rho_s} \left( 1 - \tan^2 \alpha \right)} \quad \text{hence substituting necessary values} \]

\[ \therefore \tan \phi > 0.43 \]

but \[ \tan 30^\circ = 0.577 > 0.43 \quad \text{hence slope is O.K} \]
6.1.3 Slope Stability Analysis

The existing bank slope (averaging 1:5) consists of a 6.5m layer of sand lying on top of 12m layer of clay. The clay layer is further underlain by an infinite depth of sand. The stability of the newly designed slope of 1:4 was analysed with the aid of computer program STABIL 5.1. The analysis was carried out for two conditions which are considered most critical:

(i) **Condition I**: Water level in front of bank is at MSL

Refer to the attached slope stability computation sheets in Appendix II.

![Diagram of Condition I](image)

**Fig. 1.2**: Water level at M.S.L

Minimum factor of safety for this condition = 1.33

(ii) **Condition II**: Water level in front of bank is at low Spring Tide (-1.2m below M.S.L).

![Diagram of Condition II](image)

**Fig. 1.3**: Water level at Low water level.

Minimum factor of safety for this condition = 1.33

6.1.4 Conclusion

The stability analysis was considered for various slip circles and centers for the two different conditions. The minimum stability factor of safety computed for the two conditions was found to be 1.33. The result highlights the relative importance of the high cohesive strength of the compressive clay layer. The ability of the clay layer to resist the overburden
load is also influenced by the relative mild slope of 1:4. It must however be stated that this stability analysis does not preclude some other form of failure like horizontal movement and vertical deformation of the clay layer.

7. HYDRAULIC LOADING
Refer to wave data in chapter 4 and Appendix II.

7.1 Computation of Load due to Ship-induced Waves
Assume a fairway profile shown below for the Strong Face Creek.

Fig. 1.4: Schematised fairway profile for navigation on Strong Face Creek.

Assumed navigation conditions:

- Type of fairway profile: normal
- Ship class: IV (ECMT)
- Class IV: Commercial motor vessel
- Carrying capacity: 1,350 Tonnes
- Beam width, $B_s$: 9.50 m
- Ship length, $L_s$: 80.0 m
- Unloaded ship height: 4.40 m
- Draught, $T_s$: 2.50 m
- Wetted area of ship, $A_w$: $T_s \times B_s = 23.75 \text{ m}^2$
- Assumed normal depth of channel, $h$: 6.00 m
- Assumed bottom width of waterway, $b_1$: 440 m
- cotagent $\alpha$: 4
- Top width of waterway, $b_1$: 488 m
- Cross-sectional area of waterway, $A_c$

$$A_c = \frac{h}{2} (b_1 + b_0) = 2784 \text{ m}^2$$

Blockage ratio, $k$

$$k = \frac{A_w}{A_c} = \frac{23.75}{2784} = 0.01$$
(a) Establishment of maximum sailing eccentricity:
Vessel is assumed sailing eccentrically

Fig. 1.5: Vessel sailing with eccentricity, y from centre line of waterway.

Eccentricity = y and assumed sailing distance from the bank = 100 m
\[ y = \frac{b_v}{2} - 100 = 144 \text{ m} \]
\[ y < \frac{b_v}{3}, \quad b_v / 3 = 162.5 \text{ m} \]

Load on the bank is increased due to eccentricity, \( y = 144 \text{ m} \), therefore cross-sectional area of channel needs to be adjusted as

\[ \frac{A_{c_i}}{A_c} = 1 - c_2 \left\{ \frac{y}{(b_v - h \cot \alpha)} \right\} \]

where

\[ A_{c_i} = \text{imaginary wetted cross-sectional area of waterway} \]
\[ c_2 = \text{coefficient depending on type of vessel} = 1.04 (\text{Table II.}) \]

substituting \( y = 144 \),
\[ A_{c_i} = 1885.44 \text{ m}^2 \]

checking for the value of blockage ratio, \( k \) again
\[ k = \frac{A_t}{A_{c_i}} = 0.013 \]
\[ k < 0.05 : \text{secondary waves are still dominant.} \]

(b) Computation of Limit speed, \( V_L \)
(c) Estimation of vessel speed, \( V_s \)
(d) Computation of (i) return current velocity, \( U_r \)  
(ii) average water level depression, \( \Delta h \)

SCHIJF's equation

\[ V_L = \left( \frac{2}{3} \right)^{\frac{3}{2}} \left[ 1 - \frac{A_m}{A_{cl}} + \frac{V_L^2}{2gh} \right] \]

is transformed into Schijf's Chart
imaginary water depth = $A_{ci}/b_{v} = 3.86$

Using Schijf's Chart, Fig. 3.6. and minimum value of $A_{i}/A_{ci} = 0.10$ and $V_{s} = 0.9V_{L}/\sqrt{gh'} = 0.084$, hence Average return current, $n_{r} = 0.52$ m/s

$V_{s}/\sqrt{gh'} = 0.58$, hence Vessel speed, $V_{s} = 3.57$ m/s

Maximum return current, $\Delta h = c_{j} n_{r} = 1.49 \times 0.52 = 0.78$ m/s

Maximum water level depression, $\Delta h = 0.29$ m

where $c_{j} = 1.2 + 5 \times 10^{-4} F_{h}/b_{v}/y_{t} \cdot (L_{s})^{2}/h^{2}/A_{m}$

$F_{h} = $ Froude no. = $V_{s}/\sqrt{gh}$

$y_{t} = 0.5(b_{v} - h \cot a) - y$

(e) Transversal stern wave, $Z_{max}$

$Z_{max} = \Delta h = 0.43m$

(f) Front wave, $\Delta h_{f}$

$\Delta h_{f} = 0.1 \Delta h + \Delta h = 0.1 \times 0.29 + 0.43 = 0.46m$

(g) Computation of Interference Peak

(i) Wave height, $H_{i}$

$H_{i} = h (s/h)^{-1/3} F_{h}^{4} = 0.12m$

where $s = $ distance from the bank to the side of ship = 100 - 9.5/2

= 95.25 m

(ii) Wave length, $L_{i}$

$L_{i} = 0.67 \times 2\pi V_{s}^{2}/g = 5.47m$

7.1.1 Conclusion

Since blockage ratio $k < 0.05$, the secondary waves are more pronounced on the bank than the primary waves, therefore designed ship-induced wave characteristics are

$H_{i} = 0.12 m$

$L_{i} = 5.47 m$

7.2 Computation of Load due to Wind-waves

Refer to data in Appendix II. The following design condition exists:

Wind speed at 10m above M.S.L, $U_{10} = 26$ m/s

Direction of wind : perpendicular to the waterway
Length of straight reach of waterway, \( L \) = 250 m

Wind duration (unknown).

Period of wave measurement = 31 days

Waterway is assumed exposed and located in part of the estuary which is 7 km from the Atlantic Ocean.

The wave data given is plotted to fit WEIBULL Cumulative Probability Distribution.

(a) Wind speed, \( U_{10} \) = 20.0 m/s

(b) Computation of effective Fetch length, \( F_e \)

Wind is assumed blowing across the reach of the channel:

\[ F_e = B \]

where

\[ B = \text{width of waterway} = 488.0 \text{ m} \]

(c) Computation of the significant wave height, \( H_s \)

Using the S-M-B equation adjusted for an inland waterway:

\[ H_s = 0.00354 \left( \frac{U_{10}^2}{g} \right)^{0.58} F_e^{0.42} = 0.56 \text{ m} \]

(d) Wave period, \( T_w \)

\[ T_w = 0.581 \left( \frac{U_{10}^2}{g^3} \right)^{0.25} = 2.51 \text{ seconds} \]

The obtained value of \( H_s \) is checked with Fig. II.13 in Appendix II, when \( F_e = 488 m \) and \( U = 26 m/s \) with \( h = 6.0 m \) (extrapolating)

\[ H_s = 0.39 m \]

\[ \therefore \text{Use the wave height, } H_s = 0.56 m \]

and wave period, \( T_w = 2.51 \) sec

(e) Computation of Return period, \( T_r \)

From the Weibull Probability Distribution

\[ P( H_s > 0.56 ) = 0.0085 \]

but \[ T_r = \frac{\tau}{P( H_s )} \]
where \( \tau \) = interval period of measurement in years = \( \frac{31}{365} \) year
\[ \therefore T_r = \frac{31}{365} \times 1/0.0085 = 9.99 \text{ years} \approx 10 \text{ years} \]

(f) Computation of wavelength, \( L \) in waterway

Number of records, \( N \) = 1000
Sum of peak periods = 4626.5 secs
\[ \therefore \text{Average deep water wave period, } T_0 = \frac{4626.5}{1000} = 4.626 \text{ secs} \]
Deep water wavelength, \( L_0 = \frac{gT_0^2}{2\pi} = 33.4 \text{ m} \)

Checking for \( \frac{d}{L_0} = \frac{6}{33.4} = 0.18 \)
Since \( 0.05 < \frac{d}{L_0} < 0.25 \) : waterway is in intermediate category,

\[ \text{hence } \quad L = \frac{gT_0^2}{2\pi} \tan\left(\frac{2\pi d}{L_0}\right) \]

Assume \( \frac{L}{L_0} = 0.1 \Rightarrow 2\pi d/L_0 = 1.129 \)
1st iteration using \( \frac{L}{L_0} = 0.1 \)

\[ \frac{L}{L_0} = \frac{\tanh[\frac{2\pi d/L_0}{L/L_0}] + \tanh[\frac{2\pi d/L_0}{\tanh(\frac{2\pi d/L_0}{L/L_0})}]}{2} = 0.905 \]

2nd iteration use \( L/L_0 = 0.905 \)
\[ \therefore L/L_0 = 0.859 \]
3rd iteration gives \( L/L_0 = 0.864 \)
4th iteration gives \( L/L_0 = 0.863 \)
5th iteration gives \( L/L_0 = 0.864 \)
\[ \therefore \text{use } L/L_0 = 0.86, \text{ hence} \]
\[ L = \frac{L}{L_0} \times \frac{gT_0^2}{2\pi} = 28.72 \text{ metres} \]

7.2.1 Conclusion
Since \( H_s = 0.56 \text{ m} > H_i = 0.12 \text{ m} \), therefore for wave load attack use
\[ H_s = 0.56 \text{ m} \]
\[ T_s = 2.51 \text{ sec} \]
Estimated return period = 10 years

8 DIMENSIONING

The dimensioning is carried out for three types of revetments namely Rip-rap, Open stone asphalt and Placed block revetments. Different wave conditions are also applied subsequently to determine minimum cost, selection and suitability of each type.

8.1 RIP - RAP ARMOUR LAYER

8.1.1 Computation of Armour layer size, $D_{50}$

$$D_{50} \geq \frac{H_s}{\Delta_m} \cdot \frac{\xi b}{\Psi \cdot 2.25 \cos \alpha}$$

$\Psi = 1.0$
$\Delta_m = (2650 - 1000) / 1000 = 1.650$
$b = 0.5$ (for rock)
$\xi = \tan \alpha / \sqrt{H/L_0} = 1.93$ (Plunging breaker type)
$H_s = 0.56\text{m}$

substituting:

$D \geq 0.228\ \text{m}$

Hence use $D_{50} = 0.230\ \text{m}$

Weight of this stone size, $W_{50} = (D_{50})^3 \times 2650 = 32.34\ \text{kg}$

Using Fig. 3.17, the weight gradation is designed as follows for a 2-layer system as shown below:

Thickness of the two layers = $1.8D_{50} = 0.415\ \text{m}$

$W_{100} = 4W_{50} = 129\ \text{kg}$ (Upper limit)
$W_{50} = 2W_{50} = 65\ \text{kg}$ (Lower limit)

$W_{50,\text{max}} = 1.5W_{50} = 48.4\ \text{kg}$

$W_{85}$ and $W_{15}$ are as such

$1.7W_{50} < W_{85} < 3.3W_{50}$ and $0.1W_{50} < W_{15} < 0.4W_{50}$

8.1.2 Checking for Current attack

$$D_{n50} \geq \phi_c K_t K_{h} K^{-1} \frac{0.035}{\Psi \cdot 2g\Delta_m}$$

106
where

$$\phi_c = 0.75, \quad K_s = 1.5, \quad K_b = \frac{628}{c^2} = \frac{628}{(83)^2} = 0.09$$

$$K_s = \text{bank factor} = \cos \alpha \left(1 - \frac{\tan^2 \alpha}{\tan^2 \theta}\right)^{1/2} = 0.93 \ (\theta = 40^\circ)$$

$$\psi_{cr} = 0.035 \text{ (for loose rock), } u = 1.06 \text{ m/s (} u > \text{return current, } u_r \text{)}$$

Substituting \( \vdash \):

\[
\begin{align*}
D_{50} & \geq 0.004 \text{ m, hence the designed } \\
D_{90} & = 0.230 \text{ m is } O.K, \text{ current attack is satisfied.}
\end{align*}
\]

8.1.3 Design of Underlayer (granular and geotextile)

Reference from Fig. II.8, in data. Subsoil or base material consists of sand down to 6.0 metres below M.S.L. This sand material will constitute the main body of bank. The gradation of this material (Sample OP/ES/02) is as follows:

<table>
<thead>
<tr>
<th>Dm</th>
<th>D5b</th>
<th>D20b</th>
<th>D5b</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>0.85</td>
<td>0.65</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Assumed porosity, \( n = 0.37 \)

8.1.3.1 Check for down slope migration: \( D_{50}/D_{10} > 15 \)

\[
D_{50}/D_{10} = 5.1 < 15
\]

\( \vdash \) use a geotextile with a thick fibre mat on top of the subsoil surface slope

8.1.3.2 Geotextile specification:

- For soil retention \( O_{30} < 1.8D_{90b} \)
  \( i.e. \quad O_{30} < 1.53 \text{mm} \)

\( \vdash \) use \( O_{30} = 1.5 \text{mm} \) and geotextile thickness, \( t_g = 9 \text{mm} \)

Assume that geotextile Permittivity, \( \psi = 5 \times 10^4 \text{ k}_b \) where

\( k_b = \text{permeability of base (sub-soil) material} = 10^{-5} \text{m/s (assumed)} \)

hence geotextile permeability, \( k_g = \psi t_g = 4500 \text{ mm/s} \)

8.1.3.3 Granular specification:

(a) For permeability

\[
\begin{align*}
& D_{15b}/D_{15b} > 3 - 5 \quad \text{hence} \\
& 3D_{15b} = 0.3 \text{mm while } 5D_{15b} = 0.5
\end{align*}
\]
(b) For piping
\[
\frac{D_{15}}{D_{85b}} < 4 - 5 \quad \text{hence}
\]
\[4D_{85b} = 2.60\text{mm} \quad \text{while} \quad 5D_{85b} = 3.25\text{mm}
\]
\[\therefore \]
Filter particle size \(D_{15f}\) should be such that 
\[0.5\text{mm} < D_{15f} < 2.60\text{mm}
\]

(c) To minimise segregation
\[
\frac{D_{50f}}{D_{50b}} < 20
\]
\[
D_{50} < 20 \times 0.3 = 6\text{mm}
\]
Refer to Fig. I.9 for the filter layer distribution
\[
D_{10f} = 0.4\text{mm} \quad \text{(min.)}
\]
\[
D_{15f} = 0.5\text{mm} - 2.60\text{mm}
\]
\[
D_{50f} = 1.2\text{mm} - 6.0\text{mm}
\]
\[
D_{80f} = 1.45\text{mm} \quad \text{(min.)}
\]
\[
D_{90f} = 2.90\text{mm} \quad \text{(min.)}
\]
\[
D_{90f} = 3.80\text{mm} \quad \text{(min.)}
\]
Use a filter thickness, \(b = 150\text{mm}\)

(d) Internal stability
\[
\frac{D_{80f}}{D_{10f}} < 8
\]
Checking \(1.45/0.4 = 3.6 < 8\), therefore O.K

8.1.4 Upper Limit of Protection

Fig. I.6: Design water level

From the result of the hydraulic investigation and data of the Strong Face Creek, the water level has been predicted to reach +2.0m above M.S.L., which means that the existing bank will be inundated, therefore a higher bank should be designed.
The new Crest Elevation of bank above MSL (say $h_b$) will be

$$ h_b = 2.0m + R + \text{Freeboard} $$

where

- $R = \text{Effective run-up} = R_h r_s r_d$
- Freeboard = $0.3m + 1/10 \times \text{(water depth)} = 0.9m$

but

$$ R_h = R_{R} = 1.75 \times H_b = 1.89m \text{ (run-up being exceeded by 2% of waves)} $$

$$ r_s = 0.60 \text{ (from Table II.)} $$

$$ r_d = \cos(\beta - 10) = 0.98 \quad (\beta = 0) $$

$$ R = 1.89 \times 0.60 \times 0.98 = 1.11m \text{ hence} $$

Crest elevation $h_b = 2 + 1.11 + 0.9 = 4.01m$ above MSL

Elevation of Rip-rap protection = $2 + R = 3.11m$ above MSL

### 8.1.5 Lower Limit of Protection

Using the maximum low water level (L.W.L) because of ship motion

$$ \frac{R_d}{H_b} = 0.85 + 0.5 \text{ hence} $$

$$ R_d = \text{vertical height below maximum LWL which protection is required} = 1.14m $$

In practical situation, the Rip-rap usually extends to the bed of the waterway for full protection against tidal current. Therefore this design assumes that the Rip-rap is extended to the bed level of channel.

### 8.1.6 Bank Geometry

Existing crest elevation of bank = +1.5m above MSL

Designed elevation of Rip-rap protection = +3.11m above MSL

Designed crest elevation = +4.01m above MSL

Lower limit of protection = -6.0m below MSL

Refer to Figs. 1.7 and 1.8 for the bank geometry and detail of Rip-rap armour layer and filter.

#### 8.2 OPEN STONE ASPHALT CONCRETE ARMOUR LAYER

The hydraulic loading on an Asphalt concrete armour layer is the same as that of Rip-rap but the design criteria are different. The filter layer design is also valid for the asphalt concrete. Asphaltic mixtures generally consist of a mix of mineral aggregate and bitumen. The mineral aggregate consists of sand or gravel, crushed stones and/or filler. The most important factor which influences the properties of the mixture is the bitumen content. Open stone asphalt is a gap graded mixture in which gaps are present in the
Fig. 1.8: Detail of Rip-rap and Filter layer
Fig. 1.9: Filter layer design for open stone asphalt.
mineral gradation between the sand and crushed stone fraction. The void ratio in the mixture is about 20% - 30%, therefore making the asphalt mix permeable to sand and water.

8.2.1 Maximum Uplift Pressure due to Groundwater

![Fig. 1.10: Uplift pressure consideration for asphalt](image)

\[ p = c \cdot \phi_v, \quad \text{valid for } v/(a+v) < 0.8 - 0.85 \]

where
\[ \phi_v = \text{difference between water level in front of bank and the groundwater level} \]
\[ = 0.5 \text{m} \]
\[ v = 0.5 \text{m} \quad \text{and } a = 6.0 \text{m} \]
\[ c = 0.93 \quad (\text{coefficient for a non-stationary flow}) \]

maximum pressure, \( p = 0.93 \times 0.5 = 0.47 \text{m} \) without asphalt layer.

Maximum uplift pressure due to a revetment thickness, \( h \) is \( \sigma_v \):
\[ \sigma = \rho_v \cdot g \cdot (p + h \cdot \cos \alpha) = 9.37 \text{ kN/m}^2 \]

8.2.2 Checking for Sliding criterion

\[ h \geq \frac{f \cdot \sigma_w_0}{\rho_v \cdot g \cdot (f \cdot \cos \alpha - \sin \alpha)} \]

where \( \rho_v = 2260 \text{ kg/m}^3 \) and \( f = \text{coefficient of friction} = \tan 35^\circ = 0.70 \)
\[ h \geq 0.333 \text{m} \quad \text{after substitution} \]

Hence use \( h = 0.50 \text{m} \) as open stone asphalt concrete thickness

8.2.3 Checking for Uplifting Criterion

If \( h = 0.50 \text{m} \) and \( \sigma_v = 9.37 \text{ kN/m}^2 \)
\[ h \geq \frac{\sigma_{wo}}{\rho_s g \cos \alpha} \]

\[ h \geq 0.44 \text{ m}, \quad \therefore \quad h = 0.50 \text{ m} \text{ is adequate.} \]

8.2.4 Checking for Wave Impact

\[ h \geq 0.75 \sqrt{\frac{27}{16}} \left( \frac{1}{1 - \nu^2} \right) \left( \frac{P}{\sigma_b} \right)^4 \left( \frac{S}{c} \right) \]

where

- \( \sigma_b \) = asphalt stress at failure = \( 5.5 \times 10^5 \) N/m\(^2\)
- \( S \) = stiffness modulus of asphalt = \( 7 \times 10^6 \) N/m\(^2\)
- \( \nu \) = Poisson ratio for asphalt = 0.45
- \( c \) = modulus of sub-grade reaction (sand) = \( 2 \times 10^6 \) N/m\(^2\)
- \( P \) = wave impact

\[ P = p \cdot b = \rho_v g q H_s \cdot 0.4H_s, \quad q = 2.3 \]
\[ = 2.83 \times 10^4 \text{ N/m}^2 \]

substituting, hence

\[ h = 0.106 \text{ m}, \quad \therefore \quad h = 0.50 \text{ m} \text{ is O.K} \]

8.2.5 Toe Protection

For uplift due to groundwater: the same condition as in paragraph 8.2.3 applies, therefore toe thickness, \( h = 0.50 \text{ m} \).

For uplift due to wave action: \( h \geq \rho_v / \rho_s \cdot H_s/2 \)
\[ \geq 0.124 \text{ m} \]

\[ \therefore \quad h = 0.5 \text{ m} \text{ is O.K} \]

8.2.6 Upper Limit of Protection

\[ R = R_h \cdot r_b \cdot r_f \]
\[ r_b = 1.0 \text{ and } r_f = 0.98 \]
\[ R = 1.85 \text{ m} \]

\[ \therefore \quad \text{Upper limit of protection} = 2 + 1.85 = 3.85 \text{ m above MSL} \]
\[ \text{Crest elevation of bank, } h_b = 2 + 1.85 + 0.9 = 4.75 \text{ m above MSL} \]
Fig. 1.11: Bank geometry for Open stone asphalt
8.2.7 Lower Extent of Protection
The protection continues up to the toe.

8.2.8 Bank Geometry
Designed elevation of asphalt (upper limit of protection) = +3.85m
Designed crest elevation of bank = +4.75m
Designed lower limit of protection = -6.00m
Refer to Figs. 1.11 and 1.9 for bank geometry and revetment detail.

8.3 PLACED BLOCK ARMOUR LAYER
8.3.1 Computation of Block Thickness, D

\[ D \geq \frac{H_s}{\Delta_m} \frac{\xi^b}{\Psi_\kappa 2.25 \cos\alpha} \geq 0.204m \]

\[ : \text{ Use } D = 0.230m \]

and Block length, \( L = 0.60 \) m

Block width, \( B = 0.60 \) m

Sketch of a block
Surface area of block = \( L \times B = 0.36 \) m²
Assume a permeable block with hole area, \( A_h \)

\[ A_h < 4.5% \ L \times B \]

\[ : \text{ Use } 3%L.B, \text{ hence } A_h = 0.0108m^2 \]

Also use a gap space of, \( s = 5.0 \) mm

8.3.2 Underlayer (Granular & Geotextile)
The granular and geotextile filter design for the Rip-rap is also valid for Placed block but only the thickness of the granular layer will be changed. Furthermore, an out-fill layer for the placement of the block and secondary protection will be designed.
Out-fill layer specification:

(a) For permeability

\[ \frac{D_{150}}{D_{15f}} > 3 - 5 \text{ hence } 3D_{15f} = 7.8\text{mm} \text{ while } 5D_{15f} = 13\text{mm} \]

(b) For piping

\[ \frac{D_{150}}{D_{15f}} < 4 - 5 \text{ hence } 4D_{15f} = 11.60\text{mm} \text{ while } 5D_{15f} = 14.50\text{mm} \]

Outfill layer particle \( D_{150} \) should be such that

\[ 13.0\text{mm} < D_{150} < 14.50\text{mm} \]

(a) Minimization of segregation

\[ \frac{D_{500}}{D_{50f}} < 20 \]

\[ D_{500} < 20 \times 1.2 = 24\text{mm} \]

Refer to Fig. I.12, for Out-fill layer distribution

\[ D_{100} = 11.0\text{mm} \text{ (min)} \]

\[ D_{150} = 13.0\text{mm} - 14.50\text{mm} \]

\[ D_{500} = 24.0\text{mm} \]

\[ D_{50} = 43.0\text{mm} \]

\[ D_{900} = 50.0\text{mm} \]

Use filter thickness, \( b_1 = 0.50\text{m} \)

Use outfill layer thickness, \( b_2 = 0.10\text{m} \)

8.3.3 Upper Limit of Protection

\[ R = 1.85\text{m} \text{ (same as open stone asphalt)} \]

Upper limit of protection = +3.85m MSL

New crest elevation of bank = +4.75m MSL

8.3.4 Lower Extent of Protection

\[ R_d = 1.14\text{m} \]

Depth of block protection below MSL = 1.14 + 1.20 = 2.34m

8.3.5 Bank Geometry

Existing crest elevation of bank = +1.5m MSL

Designed elevation of block protection = +3.85m MSL

New crest elevation of bank = +4.75m MSL

Lower limit of block protection + = -2.34m MSL
Fig. 1.12: Filter and Outfill layer design for Placed block revetment.
Fig. 1.13: Detail of Placed block and underlayer
Fig. 1.14: Bank geometry for placed block

- placed block protection
- toe protection
- use stone pitching
- geotextile
- new crest elevation
- original crest elevation
- bank filling
- M.S.L
9 COST ANALYSIS

9.1 Introduction

The cost analysis in this paragraph is an attempt to choose a suitable optimum design amongst the three alternatives. This optimum design is based on dimensioning of the structure such the minimization of the maintenance cost is used as a criterion. The maximization of the benefit-cost ratio is not analyzed since many other social, economic and financial boundary conditions are not available as input to such analysis. Specific application of the optimum technique used in this study is based on minimisation of the sum of the cost of construction and capitalized damage. Several parameters like unit rate price of material and interest rate are arbitrary assumed as the prevailing conditions at the project site.

9.2 Variable parameters

The variable parameters used in this optimum design analyses are:

(i) Location of project
(ii) Volume of bank filling
(iii) Revetment type
(iv) Wave condition
(v) Construction materials

9.3 Unit rates

The following unit cost rates of materials are taken for the location. The cost has already taken care of transportation and placement on site. Refer to Table I.1.

9.4 Computation procedure

The procedure in this analysis considers the computation of capital cost and damage cost for the three revetment types based on evaluation of return periods (or probability of exceedance of the chosen wave height) for 10 years, 30 years and 50 years. It must be noted that damage cost consists of both direct and indirect costs. The direct cost involves the maintenance cost of the revetments while the indirect cost involves the operation and monitoring of the revetments. This study only places emphasis on direct damage cost since indirect cost requires certain parameters that have to do with organisational set-up of the whole Delta management authority.
### Table I.1: Unit rate of material and execution

<table>
<thead>
<tr>
<th>Material</th>
<th>Uses</th>
<th>Unit</th>
<th>Price (Naira)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Quarry stone</td>
<td>(i) Armour layer for rip-rap</td>
<td>m³</td>
<td>300:00</td>
</tr>
<tr>
<td></td>
<td>(ii) Toe protection for all revetments types</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b) Gravel</td>
<td>Placed block outfit layer</td>
<td>m³</td>
<td>200:00</td>
</tr>
<tr>
<td>(c) Sand</td>
<td>(i) Filter layer for rip-rap, placed block and open stone asphalt</td>
<td>m³</td>
<td>65:00</td>
</tr>
<tr>
<td></td>
<td>(ii) Material for filling of the bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(d) Concrete</td>
<td>Placed block</td>
<td>m³</td>
<td>1000:00</td>
</tr>
<tr>
<td>(e) Geotextile</td>
<td>Primary filter and drainage protection for the bank</td>
<td>m²</td>
<td>60:00</td>
</tr>
<tr>
<td>(f) Open stone asphalt</td>
<td>Armour layer for the open stone asphalt revetment type</td>
<td>m²</td>
<td>450:00</td>
</tr>
</tbody>
</table>

9.4.1 Direct damage cost

This is based on the loss of stability of some parts of the revetment. The direct damage cost estimate is obtained from the cost of repair arising from the most vulnerable part of each revetment profile. For the rip-rap, the part that is most liable to lose its stability is the armour layer. For the Open stone asphalt, these parts are the armour layer and the filter layer. And for the placed block, the most vulnerable part are the armour layer and the filter layer. The following steps are carried out for the computation.

(i) Computation of three different profiles for return period of 10 years (already done), 30 years and 50 years for the three revetment types.

(ii) Computation of the capital cost, C for the three profiles for each revetment type.

(iii) Computation of probability of damage, P(damage) based on maintenance philosophy that maintenance is carried out once in a year such that:
year such that:

\[ P(D_{\text{Damage}}) = 1 - (1 - \Delta P)^M \]

where

\[ M = \text{number of storms per year} = 1000 \]
\[ \Delta P = \text{frequency of occurrence or the chance that a storm of the chosen wave height occurs during a given single storm interval} \]

(iv) Computation of damage coefficient ratio with corresponding damage percentages, D% (refer to Figs.1.18 & 1.19 for Damage coefficient ratio versus Damage percentages)

(v) Computation of Annual Damage cost

Annual Damage cost, \( D_c = \) D% \times \text{Cost of the most vulnerable part of revetment} \times P(D_{\text{Damage}}) \)

(vi) Computation of present worth factor, \( pwf \)

\[ pwf = \frac{(1+i)^n - 1}{i(1+i)^n} \]

where

\[ i = \text{interest rate per period in decimal} = 15\% = 0.15 \]
\[ n = \text{number of period (or assumed lifespan of structure} = 50 \) \]

(vii) Computation of Capitalized damage cost

Capitalized damage cost = \( pwf \times D_c \)

(viii) Computation of Total Cost, \( T \)

\[ T = C + pwf \times D_c \]

(ix) Plotting of Total cost, \( T \) against design wave height, \( H_i \). The minimum point of the resulting curve represent the optimum design condition.

(x) Selection of appropriate revetment.
9.5 Wave conditions

From the Weibull wave probability distribution, the wave heights and probability are established. The remaining part of the table below are the results of calculation:

<table>
<thead>
<tr>
<th>Probability of Exceedance</th>
<th>Design wave height</th>
<th>Return Period</th>
<th>Deep water wave length</th>
<th>Hs/L0</th>
<th>Breaker parameter ( \xi = \tan \phi (H_s/L_0) )</th>
<th>Breaker Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>P(Hs)</td>
<td>Hs (m)</td>
<td>Tp (years)</td>
<td>Lp (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0085</td>
<td>0.560</td>
<td>10</td>
<td>33.4</td>
<td>0.0168</td>
<td>1.93</td>
<td>plunging</td>
</tr>
<tr>
<td>0.0028</td>
<td>0.625</td>
<td>30</td>
<td>33.4</td>
<td>0.0187</td>
<td>1.83</td>
<td>plunging</td>
</tr>
<tr>
<td>0.0017</td>
<td>0.650</td>
<td>50</td>
<td>33.4</td>
<td>0.0195</td>
<td>1.79</td>
<td>plunging</td>
</tr>
</tbody>
</table>

Table 1.2: Established characteristic load parameters for the three different wave conditions.

9.5.1 Technical and cost analysis for Rip-rap

**CASE 1: \( H_s = 0.56 \text{m} \)**

For the condition \( H_s = 0.56 \text{ m} \), the following values have already been established:

(i) Thickness of armour layer = 0.415 m

(ii) Thickness of filter layer = 0.150 m

(iii) Elevation of Rip-rap protection = +3.11 MSL

(iv) New crest elevation of bank = +4.01 MSL

Quantities required per metre length:

(a) Armour layer quantities:

Length, \( L = \frac{1}{1} \left( 40.04^2 \right) + 10.01^2 \) = 41.27 m

Thickness, \( t = 0.415 \text{m} \)

Volume, \( V = 17.13 \text{ m}^3 \) (Quarry stone)

(b) Filter layer quantities:

\( L = 41.27 \text{ m} \)

\( t = 0.150 \text{ m} \)
\[ \begin{align*}
V &= 6.19 \text{ m}^3 \quad \text{(Sand)} \\
(c) \quad \text{Geotextile quantities:} & \\
L &= 6.18 + 41.27 = 47.45 \text{ m} \\
\text{Area, } A &= 47.45 \text{ m}^2 \\
(d) \quad \text{Toe protection:} & \\
V &= \frac{1}{2}(1.5 + 13.5) \times 1.5 = 11.25 \text{ m}^3 \quad \text{(Quarry stone)} \\
(e) \quad \text{Bank filling material:} & \\
V &= \frac{1}{2} \times 40.04 \times 2.51 + 2.51 \times 10 = 75.35 \text{ m}^3 \quad \text{(Sand)}
\end{align*} \]

**CASE 2: } H_s = 0.625\text{m}**

Repeating the same design procedure and quantities calculation, therefore we have

(i) Armour layer thickness = 0.450\text{m}  
(ii) Filter layer thickness = 0.150\text{m}  
(iii) New crest elevation of bank = +4.08 MSL

**Quantities required per metre length**

(a) Armour layer :- \( V = 18.70 \text{ m}^3 \) \quad \text{(Quarry stone)}  
(b) Filter layer :- \( V = 6.23 \text{ m}^3 \) \quad \text{(Sand)}  
(c) Geotextile :- \( A = 47.82 \text{ m}^2 \)  
(d) Toe protection :- \( V = 11.25 \text{ m}^3 \) \quad \text{(Quarry stone)}  
(e) Bank filling material:- \( V = 77.81 \text{ m}^3 \) \quad \text{(Sand)}

**CASE 3: } H_s = 0.650\text{m}**

Repeating the same design procedure and quantities calculation, therefore we have

(i) Armour layer thickness = 0.540\text{m}  
(ii) Filter layer thickness = 0.200\text{m}  
(iii) New crest elevation of bank = +4.10 MSL

**Quantities required per metre length**

(a) Armour layer :- \( V = 22.49 \text{ m}^3 \) \quad \text{(Quarry stone)}  
(b) Filter layer :- \( V = 8.33 \text{ m}^3 \) \quad \text{(Sand)}
(c) Geotextile  \[- A = 47.82 \text{ m}^2 \]
(d) Toe protection  \[- V = 11.25 \text{ m}^3 \text{ (Quarry stone)} \]
(e) Bank filling material:  \[- V = 78.52 \text{ m}^3 \text{ (Sand)} \]

Refer below for the table of construction cost as a function of the design wave heights considered above:

<table>
<thead>
<tr>
<th>TABLE I.3 : CONSTRUCTION COST AS A FUNCTION OF WAVE HEIGHT FOR RIP-RAP</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Item</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Quantity</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>1. Armour (Quarry stone)</td>
</tr>
<tr>
<td>2. Filter (Sand)</td>
</tr>
<tr>
<td>3. Geotextile</td>
</tr>
<tr>
<td>4. Toe protection (Quarry stone)</td>
</tr>
<tr>
<td>5. Bank material (Sand)</td>
</tr>
<tr>
<td>6. Other cost</td>
</tr>
<tr>
<td><strong>TOTAL Construction Cost, C</strong></td>
</tr>
</tbody>
</table>

Refer to the Damage Cost computation sheet for Rip-rap at the end of the chapter. The Damage cost is multiplied by the Present Net Worth factor calculated below:
\[ \text{pwf} - \frac{(1+i)^n - 1}{i(1+i)^n} \]

where \( i = 0.15 \) and \( n = 50 \) yrs, hence

\[ \text{pwf} = 6.6605 \]

From Tables I.3 & I.4, the Cost summary table for Rip-rap is therefore established as shown below:

**TABLE I.5: COST SUMMARY FOR RIP-RAP**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>Design Wave Height</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( H_s = 0.560m )</td>
</tr>
<tr>
<td>Annual Damage Cost, ( D_C )</td>
<td>4,116</td>
</tr>
<tr>
<td>Capitalized Damage Cost</td>
<td>27,415</td>
</tr>
<tr>
<td>Construction Cost, ( C )</td>
<td>21,661</td>
</tr>
<tr>
<td><strong>TOTAL COST, ( T = (2+3) )</strong></td>
<td><strong>49,076</strong></td>
</tr>
</tbody>
</table>

The Total cost, \( T \) is the plotted against the Design wave heights, \( H_s \) as shown in the Cost Curve for Rip-rap (refer to Fig.I.15). The whole procedure of paragraph 9.5.1 is repeated for Open stone asphalt and Placed block revetment types. Only the derived tables are highlighted.
9.5.2 Technical and Cost analysis for Open stone asphalt

The table below represents the Construction cost for Open stone asphalt

<table>
<thead>
<tr>
<th>Item</th>
<th>Design Wave Height</th>
<th>CASE 1</th>
<th>CASE 2</th>
<th>CASE 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H_s = 0.56m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Quantity Amount</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Armour (Open stone asphalt)</td>
<td>40.61 m^3</td>
<td>18275</td>
<td>41.07 m^3</td>
<td>18482</td>
</tr>
<tr>
<td>2. Filter (Sand)</td>
<td>10.15 m^3</td>
<td>660</td>
<td>10.27 m^3</td>
<td>668</td>
</tr>
<tr>
<td>3. Geotextile</td>
<td>49.32 m^3</td>
<td>2959</td>
<td>49.78 m^3</td>
<td>2987</td>
</tr>
<tr>
<td>4. Toe protection (Open stone asphalt)</td>
<td>5.00 m^3</td>
<td>2250</td>
<td>5.00 m^3</td>
<td>2250</td>
</tr>
<tr>
<td>5. Bank material (Sand)</td>
<td>105.16 m^3</td>
<td>6835</td>
<td>109.36 m^3</td>
<td>7108</td>
</tr>
<tr>
<td>6. Other cost</td>
<td>10000</td>
<td>10000</td>
<td>10000</td>
<td>10000</td>
</tr>
<tr>
<td>TOTAL</td>
<td>40,979</td>
<td>41,495</td>
<td>41,679</td>
<td></td>
</tr>
</tbody>
</table>
Refer to Damage Cost computation sheet for Open stone asphalt at the end of the Appendix.

From Tables 1.6 & 1.7, the Cost Summary is established for Open stone asphalt below.

<table>
<thead>
<tr>
<th>TABLE 1.8: COST SUMMARY FOR OPEN STONE ASPHALT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ITEM</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>1. Annual Damage Cost, $D_c$</td>
</tr>
<tr>
<td>2. Capitalized Damage Cost</td>
</tr>
<tr>
<td>3. Construction Cost, $C$</td>
</tr>
<tr>
<td>4. TOTAL COST, $T = (2+3)$</td>
</tr>
</tbody>
</table>

Refer to Fig.1.16 for the Cost curve for Open stone asphalt.

9.5.3 Technical and Cost Analysis for Placed block

The same procedural analysis is repeated and the construction cost as a function of wave height is established in the table below:
TABLE 1.9: CONSTRUCTION COST AS A FUNCTION OF WAVE HEIGHT FOR PLACED CONCRETE BLOCK

<table>
<thead>
<tr>
<th>Item</th>
<th>Design Wave Height</th>
<th>CASE 1</th>
<th>CASE 2</th>
<th>CASE 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H_s = 0.56m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H_s = 0.625m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H_s = 0.650m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Quantity</td>
<td>Amount</td>
<td>Quantity</td>
<td>Amount</td>
</tr>
<tr>
<td>1. Armour (Concrete)</td>
<td>10.76 m^3</td>
<td>10760</td>
<td>11.81 m^3</td>
<td>11810</td>
</tr>
<tr>
<td>2. Outfill layer (Gravel)</td>
<td>4.68 m^3</td>
<td>936</td>
<td>4.73 m^3</td>
<td>946</td>
</tr>
<tr>
<td>3. Filter (Sand)</td>
<td>23.40 m^3</td>
<td>1521</td>
<td>23.63 m^3</td>
<td>1536</td>
</tr>
<tr>
<td>4. Geotextile</td>
<td>43.79 m^3</td>
<td>2807</td>
<td>47.25 m^3</td>
<td>2835</td>
</tr>
<tr>
<td>5. Toe protection (Quarry stone)</td>
<td>11.25 m^3</td>
<td>3375</td>
<td>11.25 m^3</td>
<td>3375</td>
</tr>
<tr>
<td>6. Bank material (Sand)</td>
<td>105.45 m^3</td>
<td>6854</td>
<td>108.56 m^3</td>
<td>7056</td>
</tr>
<tr>
<td>7. Other cost</td>
<td>5000</td>
<td>5000</td>
<td>5000</td>
<td>5000</td>
</tr>
<tr>
<td>TOTAL Construction Cost, C</td>
<td>31,253</td>
<td>32,558</td>
<td>35,060</td>
<td>35,060</td>
</tr>
</tbody>
</table>

Refer to Damage Cost computation sheet for the case of Placed block at the end of the Appendix
From Tables I.9 & I.10, the Cost Summary is established for Placed block below.

<table>
<thead>
<tr>
<th>ITEM</th>
<th>Design Wave Height</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_s = 0.560m$</td>
</tr>
<tr>
<td>1. Annual Damage Cost, $D_c$</td>
<td>367</td>
</tr>
<tr>
<td>2. Capitalized Damage Cost</td>
<td>2,444</td>
</tr>
<tr>
<td>3. Construction Cost, $C$</td>
<td>31,253</td>
</tr>
<tr>
<td>4. TOTAL COST, $T = (2+3)$</td>
<td>33,697</td>
</tr>
</tbody>
</table>

Refer to Fig. I.17 for the Cost curve for Placed concrete block.

9.6 Optimum Design Wave height and Cost

From the three Cost curves (Figs. I.15, I.16 & I.17), the following optimum conditions are established.

**FOR RIP-RAP REVETMENT TYPE**

Optimum Design: Wave height = $0.66m$
Total Cost = 29,100 per metre length
Construction Cost = 24,500 per metre length
Maintenance Cost = 4,600 per metre length

**FOR OPEN STONE ASPHALT REVETMENT TYPE**

Optimum Design: Wave height = $0.665m$
Total Cost = 42,100 per metre length
Construction Cost = 41,800 per metre length
Maintenance Cost = 300 per metre length

**FOR PLACED BLOCK REVETMENT TYPE**

Optimum Design: Wave height = $0.60m$
Total Cost = 32,400 per metre length
Construction Cost = 31,500 per metre length
Maintenance Cost = 900 per metre length

The established conditions above are not the absolute parameters for the optimization of a revetment design or project, it must however be realised that optimum design conditions could still be analysed further if the following parameters named below are varied:

(i) Slope angle of bank
(ii) Composite revetment types
(iii) Interest rate
(iv) Life span of the revetment structure
(v) Water level
(vi) Crest elevation of the bank
FIG. I.15: COST CURVES FOR RIP-RAP

DESIGN WAVE HEIGHT IN METRES
FIG. 1.17: COST CURVES FOR PLACED BLOCK

DESIGN WAVE HEIGHT IN METRES

125c
## Concrete Block

<table>
<thead>
<tr>
<th>Damage (%)</th>
<th>Damage Coefficient $K_n$</th>
<th>Damage Coefficient Ratio $K'/K_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3.5</td>
<td>1.0</td>
</tr>
<tr>
<td>1</td>
<td>7.0</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>8.0</td>
<td>2.29</td>
</tr>
<tr>
<td>5</td>
<td>$\approx$ 14</td>
<td>4.00</td>
</tr>
</tbody>
</table>

Table I.12: Damage coefficient and ratio for different damage percentages for Placed block (and assumed for Open stone asphalt)

---

## Quarry Stone (smooth)

<table>
<thead>
<tr>
<th>Damage (%)</th>
<th>Damage Coefficient $K_n$</th>
<th>Damage Coefficient Ratio $K'/K_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td>2.4</td>
<td>1</td>
</tr>
<tr>
<td>5 - 10</td>
<td>3.0</td>
<td>1.25</td>
</tr>
<tr>
<td>10 - 15</td>
<td>3.6</td>
<td>1.50</td>
</tr>
<tr>
<td>15 - 20</td>
<td>4.1</td>
<td>1.70</td>
</tr>
<tr>
<td>20 - 30</td>
<td>5.1</td>
<td>2.13</td>
</tr>
<tr>
<td>30 - 40</td>
<td>6.7</td>
<td>2.79</td>
</tr>
<tr>
<td>40 - 50</td>
<td>8.7</td>
<td>3.63</td>
</tr>
</tbody>
</table>

Table I.13: Damage coefficient and ratio for different damage percentages for Rip-rap.
### TABLE I.4: RIP-RAP DAMAGE COMPUTATION SHEET

<table>
<thead>
<tr>
<th>Depth</th>
<th>%</th>
<th>Diam. [m]</th>
<th>d</th>
<th>(\frac{d}{D})</th>
<th>(\pi d^2)</th>
<th>Damage percent</th>
<th>Damage Cost</th>
<th>Annual Damage Cost</th>
<th>%</th>
<th>Damage percent</th>
<th>Damage Cost</th>
<th>Annual Damage Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.61</td>
<td>0.58</td>
<td>0.991</td>
<td>0.963</td>
<td>0.963</td>
<td>1.11</td>
<td>5.1</td>
<td>4.1</td>
<td>0.34</td>
<td>5.4</td>
<td>44</td>
<td>0.62</td>
</tr>
<tr>
<td>0.45</td>
<td>0.107</td>
<td>0.63</td>
<td>0.963</td>
<td>0.963</td>
<td>0.963</td>
<td>1.45</td>
<td>10.9</td>
<td>11.8</td>
<td>1.02</td>
<td>6.4</td>
<td>44</td>
<td>0.62</td>
</tr>
<tr>
<td>0.68</td>
<td>0.003</td>
<td>0.65</td>
<td>0.954</td>
<td>0.954</td>
<td>0.954</td>
<td>1.69</td>
<td>17.4</td>
<td>17.0</td>
<td>1.19</td>
<td>9.0</td>
<td>100</td>
<td>0.79</td>
</tr>
<tr>
<td>0.95</td>
<td>0.0003</td>
<td>0.70</td>
<td>0.928</td>
<td>0.928</td>
<td>0.928</td>
<td>1.74</td>
<td>19.8</td>
<td>19.2</td>
<td>1.74</td>
<td>5.2</td>
<td>228</td>
<td>0.96</td>
</tr>
<tr>
<td>0.75</td>
<td>0.000081</td>
<td>0.75</td>
<td>0.808</td>
<td>0.808</td>
<td>0.808</td>
<td>2.17</td>
<td>22.5</td>
<td>22.0</td>
<td>1.56</td>
<td>16.0</td>
<td>262</td>
<td>0.68</td>
</tr>
<tr>
<td>0.90</td>
<td>0.00015</td>
<td>0.825</td>
<td>0.825</td>
<td>0.825</td>
<td>0.825</td>
<td>3.30</td>
<td>30.5</td>
<td>29.7</td>
<td>2.30</td>
<td>23.9</td>
<td>250</td>
<td>0.78</td>
</tr>
<tr>
<td>0.90</td>
<td>0.000023</td>
<td>0.875</td>
<td>0.688</td>
<td>0.688</td>
<td>0.688</td>
<td>3.81</td>
<td>36.3</td>
<td>35.8</td>
<td>2.44</td>
<td>24.8</td>
<td>247</td>
<td>0.69</td>
</tr>
<tr>
<td>0.90</td>
<td>0.000023</td>
<td>0.875</td>
<td>0.688</td>
<td>0.688</td>
<td>0.688</td>
<td>3.81</td>
<td>36.3</td>
<td>35.8</td>
<td>2.44</td>
<td>24.8</td>
<td>247</td>
<td>0.69</td>
</tr>
</tbody>
</table>

**COST USED IN DAMAGE COMPUTATION**

Armour layer

Armour and filter layer

For damage between 0% - 2%, damage cost = \(2 \times \text{Armour layer cost}\)

For damage between 20% - 40%, damage cost = \(1.5 \times (\text{Armour + Filter layer})\)

For damage above 40%, damage cost = Total construction cost

125g
### Table 1.7: Open Stone Asphalt Damage Computation Sheet

<table>
<thead>
<tr>
<th>Design Base Height</th>
<th>H = 0.31</th>
<th>H = 0.425</th>
<th>H = 0.625</th>
<th>H = 0.83</th>
<th>H = 0.91</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage percent (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Damage Cost ($)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Annual Damage Cost (per acre)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Base Height</th>
<th>ANAAL</th>
<th>M_2</th>
<th>D_2</th>
<th>D_2, (per acre)</th>
<th>D_2</th>
<th>D_2, (per acre)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Base Height</th>
<th>ANAAL</th>
<th>M_1</th>
<th>D_1</th>
<th>D_1, (per acre)</th>
<th>D_1</th>
<th>D_1, (per acre)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Cost Used in Damage Computation**

**Armour layer + Toe**

\[= 20,525\]

**Armour and Filter layer + Toe**

\[= 21,185\]

For damage between 0% - 2%, damage cost = 2 x (Armour layer cost + Toe)

For damage between 2% - 4%, damage cost = 1.5 x (Armour + Filter layer) + Toe

For damage above 4%, damage cost = Total construction cost

125h
<table>
<thead>
<tr>
<th>Wt</th>
<th>Damage</th>
<th>Cost</th>
<th>Cost</th>
<th>Damage</th>
<th>Cost</th>
<th>Damage</th>
<th>Cost</th>
<th>Damage</th>
<th>Cost</th>
<th>Damage</th>
<th>Cost</th>
<th>Damage</th>
<th>Cost</th>
<th>Damage</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$F_{Wt}$</td>
<td>$G_{Wt}$</td>
<td>$\delta F_{Wt}$</td>
<td>$G_{Wt}$</td>
<td>$\delta G_{Wt}$</td>
<td>$H_{Wt}$</td>
<td>$\delta H_{Wt}$</td>
<td>$I_{Wt}$</td>
<td>$\delta I_{Wt}$</td>
<td>$J_{Wt}$</td>
<td>$\delta J_{Wt}$</td>
<td>$K_{Wt}$</td>
<td>$\delta K_{Wt}$</td>
<td>$L_{Wt}$</td>
<td>$\delta L_{Wt}$</td>
</tr>
<tr>
<td>0.55</td>
<td>0.56</td>
<td>0.005</td>
<td>0.993</td>
<td>0.11</td>
<td>0.10</td>
<td>0.22</td>
<td>0.22</td>
<td>0.62</td>
<td>0.56</td>
<td>0.036</td>
<td>0.79</td>
<td>0.62</td>
<td>0.036</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>0.61</td>
<td>0.62</td>
<td>0.01</td>
<td>0.993</td>
<td>0.11</td>
<td>0.10</td>
<td>0.22</td>
<td>0.22</td>
<td>0.62</td>
<td>0.56</td>
<td>0.036</td>
<td>0.79</td>
<td>0.62</td>
<td>0.036</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>0.65</td>
<td>0.66</td>
<td>0.005</td>
<td>0.993</td>
<td>0.11</td>
<td>0.10</td>
<td>0.22</td>
<td>0.22</td>
<td>0.62</td>
<td>0.56</td>
<td>0.036</td>
<td>0.79</td>
<td>0.62</td>
<td>0.036</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>0.70</td>
<td>0.71</td>
<td>0.005</td>
<td>0.993</td>
<td>0.11</td>
<td>0.10</td>
<td>0.22</td>
<td>0.22</td>
<td>0.62</td>
<td>0.56</td>
<td>0.036</td>
<td>0.79</td>
<td>0.62</td>
<td>0.036</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>0.75</td>
<td>0.76</td>
<td>0.005</td>
<td>0.993</td>
<td>0.11</td>
<td>0.10</td>
<td>0.22</td>
<td>0.22</td>
<td>0.62</td>
<td>0.56</td>
<td>0.036</td>
<td>0.79</td>
<td>0.62</td>
<td>0.036</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>0.80</td>
<td>0.81</td>
<td>0.005</td>
<td>0.993</td>
<td>0.11</td>
<td>0.10</td>
<td>0.22</td>
<td>0.22</td>
<td>0.62</td>
<td>0.56</td>
<td>0.036</td>
<td>0.79</td>
<td>0.62</td>
<td>0.036</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

**COST USED IN DAMAGE COMPUTATION**

Armour layer = 10,760

Armour, filter layer and outfill layer = 13,217

Outfill layer = 14,220

For damage between 0% - 2%, damage cost = 2 x Armour layer cost

For damage between 2% - 4%, damage cost = 1.5 x (Armour + Filter layer + Outfill layer)

For damage above 4%, damage cost = Total construction cost

125i
APPENDIX II

FORMULAS, DATA AND COMPUTER MODELS' OUTPUT

Equations of motion describing average water level depression, \( \Delta H \) and return current, \( \bar{u}_r \) are given below:

\[
\Delta H = \frac{v_r^2}{2g} \left[ a_1 \left( \frac{A_c}{A_w} \right)^2 - 1 \right] \quad \text{...(II.1)}
\]

\[
\bar{u}_r = v_s \left( \frac{A_c}{A_w} - 1 \right) \quad \text{...(II.2)}
\]

where:

- \( A_c \) = wetted cross-sectional area of canal
- \( a_1 \) = 1.4 - 0.4\{ \( \frac{V_s}{V_L} \) \} (safety factor \( \approx 1.1 \))
- \( A_w \) = \( b_b (h - \Delta h) + m(h - \Delta h)^2 - A_s \)

where:

- \( V_s \) = vessel speed
- \( V_L \) = Limit speed determined from Schijf's equation
- \( b_b \) = bottom width of waterway
- \( m \) = \( \cot \alpha \)
- \( A_s \) = wetted area of midship section

The limit speed, \( V_L \) for a particular vessel in a known canal cross-section can be determined from:

\[
\frac{V_L}{\sqrt{gh'}} = \left( \frac{2}{3} \right)^{\frac{2}{3}} \left[ 1 - \frac{A_m}{A_c} + \frac{V_L}{2gh'} \right]^{\frac{1}{3}} \quad \text{......(II.3)}
\]
where
\[ h' = \frac{A_c}{b_h} \]

The maximum value of \( \hat{A}h \) and \( \hat{u}_r \) depend on ship type:

For Push-tow (after Verhey and van der Wall, 1984):
\[
\begin{align*}
\hat{A}h &= c_j A_h \\
\hat{u}_r &= c_j D_r
\end{align*}
\]

where
\[
\begin{align*}
c_j &= 1.2 + \{5 \times 10^{-4} F \frac{b}{y} \frac{L_s^2}{L_h} \} / \{y_t h / A_h \} \\
y_t &= 0.5b - y - 0.5h \cot a \\
L_s &= \text{Length of vessel}
\end{align*}
\]

For Tugs (after Verhey and van der Wall, 1984):
\[
\begin{align*}
\hat{A}h &= 0.875 + 6.25 F \frac{b'}{y} \{ A_b/A_c \cdot b/y_{t1} \frac{L_s^2}{h/A_c} \}^{0.3} \geq 0.2 m \\
\hat{u}_r &= -1.33 + 7.86 F \frac{b'}{y} \{ A_b/A_c \cdot b/y_{t1} \frac{L_s^2}{h/A_c} \}^{0.17} \geq 0.4 m/s
\end{align*}
\]

For coaster and motor barge (after van der Knapp, 1986):
\[
\begin{align*}
\hat{A}h &= \hat{A}h \left( 3 - 4 \frac{A_c'/A_c}{A_c} \right) \quad \text{valid for } b'/L_s > 1.5 \\
\hat{u}_r &= \hat{u}_r \left( 2.5 - 3 \frac{A_c'/A_c}{A_c} \right) \quad \text{valid for } b'/L_s < 1.5
\end{align*}
\]

The velocity behind the vessel's propeller can be estimated as:
\[
\begin{align*}
u_b &= a_2 u_b D_p/z_b \\
\end{align*}
\]

where
\[
\begin{align*}
u_b &= \text{velocity at bed} \\
a_2 &= \text{coefficient depending on ship type} \\
&= \text{varies between 0.25 - 0.75} \\
u_0 &= \text{axial efflux velocity} = \frac{1.5(P_e/D_p)^{0.33}}{} \\
z_b &= \text{vertical distance from axis of propeller to fairway bed}
\end{align*}
\]
\( D_0 \) = initial diameter of water jet behind propeller
\( = D_p \) (propeller with a nozzle)
\( = 0.7D_p \) (propeller without a nozzle)

\( D_p \) = propeller diameter

\( P_p \) = installed engine power in KW
Fig. II.1: Map of Nigeria

MAP OF NIGERIA
Fig. II.3: Opobo town

ATLANTIC OCEAN
Fig. 11.4: General map location of measurements

LEGEND
- VELOCITY MEASUREMENTS
- BOTTOM SAMPLES
- DEEP BORINGS

ALL LEVELS IN METRES

ALL LEVELS ARE REDUCED TO A LEVEL AT 2 METRES BELOW A MARK ON THE CONCRETE ARMATURE OF THE JETTY OF OPOBO. MEAN SEA LEVEL IS APPROXIMATELY 2 METRES BELOW THIS MARK.
Fig. II.5: Discharge zone of Imo river and Strong Face Creek
Fig. 11.6: Typical bank profile of Strong Face Creek

CROSS-SECTION 1

LEVEL
CHAINAGE
SLOPE

LEVELLING
68 64 63 -24
43 15 15 58
17 14 1.2 1.4 1.9 1.7

CROSS-SECTION 2

LEVEL
CHAINAGE
SLOPE

LEVELLING
11 05 03 -25
55 37 08
13 16 1.2 1.5 1.4

HORIZONTAL SCALE 1:500
VERTICAL SCALE 1:200

OPOBO TOWN EROSION PROTECTION PROJECT

CROSS SECTIONS 1-2
Fig. II.7: Deep boring at Opobo town, location 1
Fig. II.8: Typical grain particle distribution 1
### Project: Erosion Protection - Beno and Oobo Towns

#### Grain Size Distribution

<table>
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<tr>
<th>Diameter of Grains in mm</th>
<th>Clay</th>
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- **Sample 1**: 100 m + to 0.40 m M.S.L.
- **Sample 2**: 0.40 m + to 1.10 m M.S.L.

#### Fig. II.9: Typical Grain Particle Distribution

Tests executed by Materials Testing Laboratory of Haskoning at Oobo Deep Boring 1.
Bonny LNG Wave
Channel I

Weibull Cumulative Probability Distribution

Period
August 1-31, 1989

Station 1 - Weibull distribution

\[ P(H_s > H_{sig}) = \exp\left(-\frac{(H-a)/b}{c}\right) \]

- \( a = 0.65 \)
- \( b = 0.275 \)
- \( c = 2.4 \)

Sheet No II
IFERT-RSUST

Fig. II.10: Weibull Cumulative Probability Distribution
Percentage Exceedance Diagram of Hs

Bonny LNG - channel 1 - Aug 1-31, 1989

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<table>
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Units in parts per thousand
based on 732 data points

Hs = 0.10 < No. points <= Hs
Tp = 0.50 < No. points <= Tp

Fig. II.12: Scatter diagram for Hs against Tp
Fig. II.13: Significant wave, $H_s$ against Fetch length, $F_e$ at speed, $U$
Fig. II.14: CURRENT VELOCITY MEASUREMENTS

OPOBOTOWN EROSION PROTECTION PROJECT
COMPUTED AND MEASURED CROSS-SECTION OF STRONG FACE CREEK NEAR ST. PAUL'S CHURCH

Fig. II.16: Computed and measured cross-section of Strong Face Creek

OPOBO TOWN, EROSION PROTECTION PROJECT
EROSION PROTECTION - IBENO AND OPONO TOWNS

COMPARISON BETWEEN DIFFERENT BED-LOAD FUNCTIONS

Fig. II.17
### GENERAL DATA

 mole = 1.16, 7.25, 2. 0.000, 0.000

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### STABILITY FACTORS

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## FIXED POINTS OF CIRCLES

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## STABILITY FACTORS

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<td>6</td>
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<td>9</td>
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</tr>
</tbody>
</table>
**Fig. II.19:** Output of Steenzet/1 model application

**STEEHIEI**

**DaM: 1991-03-26.17u571165**

---

**CASE 1**

**CHAPTER 5**

**CONTROLE VAN DE INVOER**

---

### GEOMETRIE

- Heiing van het talud (a): 14.04 grader
- Boventijd talud (Hmax): 0.85 m
- Onderzijd talud: 3.66 m
- Lengte van een steen (DX): 0.600 m
- Hoogte van het eventueel bewegende blok: 5.84 m
- Het nummer van dit blok: 28
- Aantal stenen in de zetting: 43

---

### GELVEN

- De stilwaterlijn (Hstil): 6.00 m
- De golfoogte: 0.56 m
- De golfperiode: 2.51 s
- De schalingsfactor voor de golfoogte: 2.00

---

### TOPLAAG

- Dikte van de toplaag (D): 0.23 m
- De coefficient a' voor de toplaag: 2.14E+01 s/m
- De coefficient b' voor de toplaag: 3.10E+03 (s/m)^2
- Relatieve doorlatendheid van de onderste spleet: 1.00

---

### FILTER

- Dikte van de filterlaag (B): 0.60 m
- De coefficient a voor het filter: 1.25E+02 s/m
- De coefficient b voor het filter: 1.36E+03 (s/m)^2
- Porositeit van de filterlaag (m): 0.57
MATERIAAL

Verhouding tussen de soortelijke massa van water en beton: 2.30
Wrijvingscoëfficiënt van beton op beton: 0.50
Wrijvingscoëfficiënt van beton op de ondergrond: 0.58

GEVARIEERD

Het bestand met de golven heet: C:\STEEN\BAHACH\RH415.C
Het bestand met gegevens van de opname heet: C:\STEEN\BAHACH\BAHIPUT.DAT

STURING

Het type van de berekening: met bewegend blok, met toestrooming

BEREKENINGEN

De lengte langs het talud is: 0.60 m

*** GEEN CONVERGENTIE VOOR H ***
H = 6.844
DH = -0.001
Verschildruk (kPa)

Spatie -> keuzemenu
P -> afdruk op printer
Sparie - kiekewend
P = afdruk op printer
### STEENZET 1
Datum: 1991-03-26, 18u 271

#### CONTROLE VAN DE INVOER

#### GEOMETRIE
- Helling van het talud (°): 14.04 graden
- Bovenzijde talud (Haax): 9.85 m
- Onderzijde talud: 3.66 m
- Lengte van een steen (Ol): 0.600 m
- Hoogte van het eventueel bewegende blok: 5.04 m
- Het aantal van dit blok: 28
- Aantal stenen in de zetting: 43

#### GOLVEN
- De stellwaterlijn (Hstil): 6.00 m
- De golfhoogte: 0.56 m
- De golfperiode: 2.51 s
- De schalingsfactor voor de golfhoogte: 2.00

#### TOPLAAG
- Dikte van de toplaag (θ): 0.23 m
- De coefficient a' voor de toplaag: 1.00E-02 s/m
- De coefficient b' voor de toplaag: 1.00E-02 (s/m)^2
- Relatieve doorlatendheid van de onderste spleet: 1.00

#### FILTER
- Dikte van de filterlaag (θ): 0.60 m
- De coefficient a voor het filter: 1.25E+02 s/m
- De coefficient b voor het filter: 1.36E+03 (s/m)^2
- Porositeit van de filterlaag (n): 0.37
**MATERIAAL**

Verhouding tussen de soortelijke massa van water en beton: 2.30
Wrijvingscoëfficiënt van beton op beton: 0.50
Wrijvingscoëfficiënt van beton op de ondergrond: 0.58

**GEVARIEERD**

Het bestand met de golven heet:
C:\STEEN\BAHACH\R415.C
Het bestand met gegevens van de opneemers heet:
C:\STEEN\BAHACH\BAHINPUT.DAT

**STURING**

Het type van de berekening:
Het bewegend blok met toestroming

**BEREKENINGEN**

De lek lengte langs het talud is: 0.60 m
**** GEEN CONVERGENTIE VOOR H ****
H = 7.113
DH = 0.000
Stijghoogte (kPa)

Tijd (s)

Spatie -> keuzemenu
P -> afdruk op printer
Verschuldruk (kPa)

Spatte -> kleuzewenu
R -> aldruk op printer

Hoeve (m)
Spatie -> keuzewenu
P -> afdruk op printer

Hoogte (m)

Stijghoogten (m)

3 4 5 6 7 8 9 10

5 6 7 8 9 10

164
Verscheidene bij beveging (kPa)

-0.5

0.0

0.5

1.0

Hoepte (m)

Spatie -> keuzemenu

P -> afdruk op printer
Spatie -> keuzemenu
P -> afdruk op printer
Verhangen bij beveging (-)

Spatie -> keuzemenu
P -> afdruk op printer

Hoeveelheid (m)
ANAMOS 2.00 PROJECT: no_hole_la
on. RWS

INVOERGEVEENS

GOLVEN
Significante golfhoogte : Hs = 0.560 m
Periode (van piek spectrum) : Tp = 2.510 s
Waterstand tov. de teen : h1 = 6.000 m

TALUD
Helling : \( \cot(\alpha) = 4.000 \) -
Wrijvingscoëff. toplaag/ondergr.: \( ft = 0.577 \) -
Nivo ondergrens zetting : h2 = 3.660 m
Nivo bovengrens zetting : h3 = 9.850 m

CONSTRUCTIETYPE

ingewassen dichte blokken
============================
uitvullaag
============================
filter
============================
geotextiel
============================
basis

INWASMATERIAAL
Karakteristieke korrelgrootte : D15 = 1.000 mm
Porositeit : n = 0.370 -

DICHTE BLOKKEN
Breedte (langs het talud) : B = 0.600 m
Lengte (evenwijdig dijkas) : L = 0.600 m
Dikte : D = 0.230 m
Spleetbreedte : s = 5.000 mm
Soortelijke massa : sm = 2300.0 kg/m³
Onderlinge wrijving : fwg = 0.500 -

UITVULLAAG
Laag dikte : b = 0.100 m
Karakteristieke korrelradius : D15 = 13.000 mm
Porositeit : n = 0.370 -

FILTER
Laag dikte : b = 0.500 m
Karakteristieke korrelradius : D15 = 1.000 mm
Porositeit : n = 0.370 -

GEOTEXTIEL ONDER FILTER
Dikte van het geotextiel : Tg = 9.000 mm
Karakteristieke openingengrootte : 090 = 1.000 mm

zie volgende bladzijde
VERVOG INVOERGEGEVENS

Weerstandscoefficienten van Forchheimer:

\[ \Phi = a \cdot q + b \cdot q^2 \]
\[ q = \text{specifiek debiet (m/s)} \]

In de relatie \( \Phi = a \cdot q + b \cdot q^2 \) is:

\[ a = 7.79 \times 10^{-1} \text{ s/m} \]
\[ b = 1 \times 10^{-1} \text{ s/m} \]

**BASIS**

Karakteristieke korrel diameter:

\[ D_{50} = 0.30 \text{ mm} \]
\[ D_{90} = 0.85 \text{ mm} \]

Porositeit:

\[ n_b = 0.37 \]
GEVOELIGHEID VAN DE RESULTATEN

DE WAARDE VAN DE INVOERPARAMETERS

In onderstaande tabel worden de eindresultaten gegeven behorende bij de
invoergegevens die alleen verschillen in de waarde van de parameter in de
eerste kolom. Deze parameter is in de nieuwe berekening 10% groter gekozen
dan in de oorspronkelijke invoer.

<table>
<thead>
<tr>
<th>TOPLAAG</th>
<th>R/S</th>
<th>Y/D</th>
<th>GRENSVLAK</th>
<th>icr/imax</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oorspronkelijke resultaten:</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>10% verhoogde invoerparameter:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Golfhoogte Hs</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Golfperiode Tp</td>
<td>1.00</td>
<td>.04</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Taludhelling cot(α)</td>
<td>1.00</td>
<td>.02</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Waterdiepte h1</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Blokdikte D</td>
<td>1.00</td>
<td>.02</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Blokoppervlak B*L</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Soleetbreedte s</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Uitvullaagdikte bu</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Uitvulkorrel Du15</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Filterlaagdikte bf</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Filterkorrel Df15</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
</tbody>
</table>

EEN AANWEZIGHEID VAN CONSTRUCTIEONDERDELEN

In onderstaande tabel worden de eindresultaten gegeven behorende bij de
invoergegevens die alleen verschillen in de aanwezigheid van het constructieonderdeel in de eerste kolom. Dit constructieonderdeel is in de nieuwe
berekening weggelaten ten opzichte van de oorspronkelijke.

<table>
<thead>
<tr>
<th>TOPLAAG</th>
<th>R/S</th>
<th>Y/D</th>
<th>GRENSVLAK</th>
<th>icr/imax</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oorspronkelijke resultaten:</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Weggelaten constructieonderdeel:</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Inwasmateriaal</td>
<td>1.32</td>
<td>.00</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Uitvullaag</td>
<td>1.00</td>
<td>.00</td>
<td>nvt</td>
<td></td>
</tr>
<tr>
<td>Geotextiel onder het filter</td>
<td>1.00</td>
<td>.03</td>
<td>nvt</td>
<td></td>
</tr>
</tbody>
</table>

Aanmerking: de parameter kan niet 10% groter gekozen worden, omdat anders
de maximum grens behorende bij die parameter overschreden wordt.
FINDRESULTATEN

STABILITEIT TOPLAAG
Belasting: $S = 0.307 \text{ m}$
Sterkte: $R = 0.307 \text{ m}$
Blokbeweging: $Y = 0.006 \text{ m}$
Conclusie: De constructie is INSTABIEL.
$H = H_s$ is maatgevend.

STABILITEIT TEGEN AFSCHUIVING
Stabiliteitsfactor: $\Gamma_a = 4.755 -$
Kracht op teen: $F_{\text{teem}} = 0.00 \text{ kN/m}^2$

STABILITEIT GRENSVLAK BASIS-FILTER
Het grensvlak is stabiel, want $D_{f15} < 5 \times Db90$. 
Tussenresultaten

Constructie

<table>
<thead>
<tr>
<th></th>
<th>Forchheimer coëfficiënten</th>
<th>doorlatendheid</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a (s/m)</td>
<td>b (s²/m²)</td>
</tr>
<tr>
<td>Toplaag</td>
<td>4662.562</td>
<td>3026.798 x 10^-3</td>
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<tr>
<td>Uitzuivlaag</td>
<td>.907</td>
<td>126.011</td>
</tr>
<tr>
<td>Inwasmateriaal</td>
<td>153.359</td>
<td>1638.137</td>
</tr>
<tr>
<td>Filter</td>
<td>153.359</td>
<td>1638.137</td>
</tr>
</tbody>
</table>

Geotextiel tussen basis en filter:
- k = .128 m/s
- m = .998 m

Leklengte
- LAMBDA = 4.697 m

Lekhoogte
- lambda = 1.139 m

Belaasting

Golfsteilheid
- Hs/Lo = .057

Brekerparameter
- ksi0 = 1.047

Belastingparameters
- Hs/delta D = 1.975

Voor de berekening van de blokbeweging wordt in dit geval gerekend met Hs. De bijbehorende belastingparameters zijn:

Hoogte stijghoogtefront: Φb = .422 m
Helling stijghoogtefront: tan(β) = .712
Diepte zwaarste golfaanval tov. SWL: ds = .201 m
Hoogte freatische lijn: zf = .221 m
Maximaal stijghoogteverschil toplaag: Φω = .405 m
Invloedsfactor wrijving: f₁ = 1.096
Invloedsfactor massatraagheid: f₂ = 1.019
Invloedsfactor toestroming: fb = 1.320
INVOERGEGEVENS

CASE A CHAPTER 5

**GOLVEN**

Significante golfhoogte : Hs = 0.560 m
Periode (van peik spectrum) : Tp = 2.510 s
Waterstand tov. de teen : h1 = 6.000 m

**TALUD**

Helling : cot(α) = 4.000
Wrijvingscoefficient toplaag/ondergr.: ft = 0.577
Nivo ondergrens zetting : h2 = 3.660 m
Nivo bovengrens zetting : h3 = 9.850 m

**CONSTRUCTIETYPE**

Niet ingewassen blokken met gaten

---

**UITVULLAAG**

---

**FILTER**

---

**BLOKKEN MET GATEN**

Blokafmetingen:

<table>
<thead>
<tr>
<th>Breedte (langs het talud)</th>
<th>B</th>
<th>0.600 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lengte (evenwijdig aan dijkas)</td>
<td>L</td>
<td>0.600 m</td>
</tr>
<tr>
<td>Dikte</td>
<td>D</td>
<td>0.230 m</td>
</tr>
<tr>
<td>Spieletbreedte</td>
<td>s</td>
<td>5.000 mm</td>
</tr>
<tr>
<td>Soortelijke massa</td>
<td>sm</td>
<td>2300.0 kg/m3</td>
</tr>
<tr>
<td>Onderlinge wrijving</td>
<td>fwg</td>
<td>0.500</td>
</tr>
</tbody>
</table>

Gatafmetingen:

<table>
<thead>
<tr>
<th>Gatgrootte (cm²)</th>
<th>Aantal per blok</th>
</tr>
</thead>
<tbody>
<tr>
<td>36.000</td>
<td>1</td>
</tr>
</tbody>
</table>

**UITVULLAAG**

Laag dikte : b = 0.100 m
Karakteristieke korreldiameter : D15 = 13.000 mm
Porositeit : n = 0.370

**FILTER**

Laag dikte : b = 0.500 m
Karakteristieke korreldiameter : D15 = 1.000 mm
Porositeit : n = 0.370

Zie volgende bladzijde
EOTEXTIEL ONDER FILTER

Dikte van het geotextiel : Tg = 9.000 mm
Karakteristieke openingengrootte : 090 = 1.000 mm

Weerstandscoefficienten van Forchheimer:

Φ = stijghoogteverschil over geotextiel (mm)
q = specifiek debiet (m/s)

In de relatie Φ = a*q + b*q^2 is :

: a = .7790E+01 s/m
: b = .1000E+01 s/m

ASIS

Karakteristieke korrel diameter : D50 = .300 mm
: D90 = .850 mm

Porositeit
: nb = .370 -
Tussenresultaten

Constructie

<table>
<thead>
<tr>
<th></th>
<th>Forchheimer coefficienten</th>
<th>Doorlatendheid</th>
</tr>
</thead>
<tbody>
<tr>
<td>a (s/m)</td>
<td>b (s^2/m^2)</td>
<td>k (m/s)</td>
</tr>
<tr>
<td>Toplaag</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uitvulllaag</td>
<td>.907</td>
<td>.0152</td>
</tr>
<tr>
<td>Filter</td>
<td>153.359</td>
<td>.1511</td>
</tr>
</tbody>
</table>

Geotextiel tussen basis en filter:
- k = 0.128 m/s
- m = 0.998

Leklengte: LAMBDA = 0.526 m
Lekhoogte: lambda = 0.128 m

Belasting

Golfsteilheid: Hs/Lo = 0.057
Brekerparameter: ksi-o = 1.047
Belastingsparameter: Hs/delta*D = 1.975

Voor de berekening van de blokbeweging wordt in dit geval gerekend met:
- Hs: de bijbehorende belastingparameters zijn:
- Hoogte stijghoogtefront: φb = 0.422 m
- Helling stijghoogtefront: tan(β) = 0.712
- Diepte zwaarste golfaanval tov. SWL: ds = 0.201 m
- Hoogte freatische lijn: zf = 0.221 m
- Maximaal stijghoogteverschil toplaag: φw = 0.223 m
- Invloedsfactor wrijving: Γs1 = 1.096
- Invloedsfactor massatraagheid: Γs2 = 1.000
- Invloedsfactor toestroming: Γb = 1.000
INDRESULTATEN

TABILITEIT TOPLAAG

| Belasting | $S$ | = 0.223 m |
| Sterkte   | $R$ | = 0.301 m |
| Blokbeweging | $Y$ | = 0.000 m |

Conclusie:
De constructie is stabiel.
$H = H_s$ is maatgevend.

TABILITEIT TEGEN AFSCHUIVING

| Stabiliteitsfactor | $\Gamma_a$ | = 23.961 |
| Kracht op teen    | $F_{teen}$ | = 0.00 kN/m² |

TABILITEIT GRENSVLAK BASIS-FILTER

Het grensvlak is stabiel, want $0.15 < 5 \times Db90$. 

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GEVOELIGHEID VAN DE RESULTATEN

DE WAARDE VAN DE INVOERPARAMETERS

In onderstaande tabel worden de eindresultaten gegeven behorende bij de invoergegevens die alleen verschillen in de waarde van de parameter in de eerste kolom. Deze parameter is in de nieuwe berekening 10% groter gekozen dan in de oorspronkelijke invoer.

<table>
<thead>
<tr>
<th>TOPLAAG</th>
<th>GRENSVLAK</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R/S</td>
</tr>
<tr>
<td>Oorspronkelijke resultaten:</td>
<td>1.35</td>
</tr>
<tr>
<td>10% verhoogde invoerparameter:</td>
<td></td>
</tr>
<tr>
<td>Golfhoogte Hs</td>
<td>1.31</td>
</tr>
<tr>
<td>Golfperiode Tp</td>
<td>1.31</td>
</tr>
<tr>
<td>Taludhelling cot(α)</td>
<td>1.38</td>
</tr>
<tr>
<td>Waterdiepte h1</td>
<td>1.35</td>
</tr>
<tr>
<td>Blokdikte D</td>
<td>1.48</td>
</tr>
<tr>
<td>Blokkopervlak B*L</td>
<td>-</td>
</tr>
<tr>
<td>Soletbreedte s</td>
<td>1.38</td>
</tr>
<tr>
<td>Uitvullaagdikte bu</td>
<td>1.33</td>
</tr>
<tr>
<td>Uitvulkorrel Du15</td>
<td>1.34</td>
</tr>
<tr>
<td>Filterlaagdikte bf</td>
<td>1.35</td>
</tr>
<tr>
<td>Filterkorrel Df15</td>
<td>1.34</td>
</tr>
</tbody>
</table>

DE AANWEZIGHEID VAN CONSTRUCTIEONDERDELEN

In onderstaande tabel worden de eindresultaten gegeven behorende bij de invoergegevens die alleen verschillen in de aanwezigheid van het constructieonderdeel in de eerste kolom. Dit constructieonderdeel is in de nieuwe berekening weggelaten ten opzichte van de oorspronkelijke.

<table>
<thead>
<tr>
<th>TOPLAAG</th>
<th>GRENSVLAK</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R/S</td>
</tr>
<tr>
<td>Oorspronkelijke resultaten:</td>
<td>1.35</td>
</tr>
<tr>
<td>Weggelaten constructieonderdeel:</td>
<td></td>
</tr>
<tr>
<td>Gaten in de blokken</td>
<td>1.32</td>
</tr>
<tr>
<td>Uitvullaag</td>
<td>1.46</td>
</tr>
<tr>
<td>Geotextiel onder het filter</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Betekenis '-': de parameter kan niet 10% groter gekozen worden, omdat anders de maximum grens behorende bij die parameter overschreden wordt.
REFERENCES


