TECHNICAL RECOMMENDATIONS FOR HIGHWAYS

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GUIDELINES FOR THE HYDRAULIC DESIGN AND MAINTENANCE OF RIVER CROSSINGS

VOLUME III : EMBANKMENT AND PROTECTION

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PREFACE

At the rate at which new information is generated and made available it is becoming increasingly difficult for the practising civil engineer to decide on the appropriate norms and analytical methods to be used in designs. Although there will always be cases necessitating a comprehensive independent literature study to ascertain the best suited norms and methods to achieve a sound solution, it is recognised that they tend to be the exception rather than the rule. The designer cannot be expected to undertake such detailed studies for each case as this would become impractical. Consequently the need for practical guidelines.

The main aims of these guidelines are to make recommendations on methods of calculation, design norms as well as legal and other issues which need to be taken into consideration in the pursuit of providing safe, economical and viable river crossings. The intention is not to stifle original thinking and new development, and thus designers are expected to deviate from the general recommendations where optimum solutions clearly fall outside the general applicable norms. The guidelines are furthermore intended to serve as a basis for governing bodies to formulate their policies on design standards with due consideration of legal and other risks.

These guidelines comprise seven volumes each dealing with a particular subject or related subjects.

SYNOPSIS

During the 1970s and 1980s a number of major floods caused serious damage to the road and rail infrastructure in southern Africa. These events prompted the re-evaluation of design norms to secure cost-effective and safer designs to withstand the expected imposed forces.

This volume deals with the protection of embankments, flow channels and the foundations of structures against water erosion, flood damage, wave action and scour as relevant.

Various types of protection applicable to southern African conditions are presented under the headings : Direct Protection, and Indirect Protection. The aspects covered are the various armouring systems and structures that can be utilized as protection against erosion, overtopping and other hydraulic actions. Recommendations, incorporating recent research and technology, are made regarding methods of calculation, appropriate coefficients and criteria to be adopted, as well as issues to be taken into consideration in the pursuit of finding viable solutions. In some cases suggestions are made with regard to economic considerations and environmental, and ecological acceptance of the protection methods.

Whilst this document should not be considered as comprehensive and all-inclusive, it does present the engineer with the basics from which most decisions can be made. Further reading, however, may well be necessary for complex applications.

Keywords : Embankments, bank protection, direct protection, indirect protection, maintenance, design, materials and construction, rock fill embankments.

CSRA Guidelines for the hydraulic design and maintenance of river crossings
Volume III, 1994, Committee of State Road Authorities, South Africa

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NOTATION

Most of the symbols used in this volume are recorded here for reference. Symbols not found here are defined where first mentioned.

		SI UNITS
A	Surface area	m^2
A_{n2}	Cross-sectional flow area below normal water level through the bridge opening	m^2
b	Width of bridge opening perpendicular to direction of flow	m
B_b	Minimum width of berm	m
B_c	Crest width	m
c	Factor	-
C	Coefficient of discharge for free flow	-
C	Shape factor i.r.o. packing density of armour units	-
C_s	Coefficient of discharge for submergence	-
d	Diameter of stone	-
d	Embankment height	m
d_b	Breaker depth in front of structure	m
d_s	Local scour depth for a free fall	m
D_{50}	Nominal 50% or median size stone i.e. 50% of the stones being lighter than this size (= $(M_{50}/\rho_s)^{1/3}$)	m
D	Flow depth	m
D_u	Nominal rock size	m
f	Factor for variation in scour patterns	-
g	Gravitational acceleration	m/s^2
h	Height of dolos	m
h	Tailwater depth	m

		SI UNITS
h_b	Tailwater depth relative to roadway crest	m
H	Upstream water depth relative to roadway crest or overtopping head	m
H	Wave height at structure	m
H_b	Depth limited breaker height	m
Hmo_x	Significant wave height with recurrence interval x	m
H_1	Total head above the embankment crest	m
H_t	Total drop in head, measured from upstream to downstream energy grade line	m
i	Embankment slope	m/m
K	Factor	-
K_D	Stability factor	-
L	Length of inundated roadway	m
L_e	Effective length of spur from the bank line	m
L_s	Length of guide bank measured along its top	m
M	Average mass of stone	kg
M_e	Equivalent mass of quarry stone	t
M_{50}	Median stone mass i.e. 50% of stone having a smaller mass	kg
n	Number of layers	-
N	Number of dolosse per unit area	-
p	Porosity of armour layer	%
q	Unit discharge	$m^3/s.m$
q_d	Design unit discharge	$m^3/s.m$
q_f	Failure unit discharge	$m^3/s.m$
q_t	Critical unit discharge	$m^3/s.m$

		SI UNITS
Q	Total discharge	m^3/s
Q_f	Design discharge over flood plain (one side) measured just upstream of bridge crossing	m^3/s
Q_{30}	Design discharge in 30 m of main channel measured adjacent to the bridge abutment	m^3/s
r	Waist to height ratio of dolos	-
R	Riprap stability coefficient	-
R	Wave run-up	m
S	Energy gradient	-
S	Spacing between spur toes	m
S_e	Embankment slope	m/m
t	Thickness of mattress lining	m
T	Thickness of armour cover	m
t_u	Thickness of underlayer	m
u	Critical flow velocity	m/s
u_o	Velocity at downstream roadway crest	m/s
U_A	Windstress factor	-
U_{10}	Safe max. hourly average wind speed with 10 year return period	m/s
V	Volume of armour unit	m^3
V	Depth average velocity	m/s
V_1	Average approach velocity	m/s
V_b	Residual velocity on bed	m/s
V_d	Average downstream velocity	m/s
V_{n2}	Average velocity through bridge opening	m/s
V_{ss}	Settling velocity of interlocking blocks or bed elements	m/s

		SI UNITS
V_{bend}	Maximum depth-average velocity in the bend	-
W	Average width of river channel top	m
W	Weight of dolos	N
W_{p1}	Travelled way width	m
W_r	Roadway or crest width	m
W_s	Average stone weight	kN
W_u	Median weight of stone	N
X	Measurement along X-axis for the ellipse equation	m
y_o	Flow depth above roadway crest	m
y_1	Flow depth over weir	m
Y	Measurement along Y-axis for the ellipse equation	m
Z_d	Height of river bed above datum	-
Z_1	Height of weir above datum	m
α	Factor	degrees
α	Embankment side slope with horizontal plane	degrees
α_d	Angle of downstream embankment side slope with horizontal plane	degrees
α_u	Angle of upstream embankment slope with horizontal plane	degrees
β	Expansion angle downstream of spur tips	degrees
γ_s	Specific weight of stone	kN/m^3
γ_w	Specific weight of water	kN/m^3
γ_{f3}^{1}	Reduced partial load factor	-
Δ	Relative density of stone or armour unit $(= (\rho_s - \rho_w)/\rho_w)$	-
δ	Packing stone density	-

		SI UNITS
ρ	Mass density of fluid	t/m^3 or kg/m^3
ρ_g	Mass density of container cell filler	t/m^3
ρ_s	Mass density of stone or concrete	t/m^3 or kg/m^3
ρ_s	Mass density of bed elements	t/m^3
ρ_w	Mass density of water	t/m^3 or kg/m^3
τ	Shear stress along bed	kN/m^2
τ_c	Resistant or critical stress when movement starts	kN/m^2
τ_m	Active shear stress on embankment	kN/m^2
τ_s	Resistant stress on embankment	kN/m^2
ϕ	Internal friction angle	degrees

TERMINOLOGY

Active shear stress

The stress developed by the flow of water across the revetment.

Anchorage block

A structure to which the armour layer on the embankment slope can be attached to prevent sliding.

Armour layer

The outer layer of a revetment which protects the underlying material against erosion by currents and waves.

Breakwater

A structure usually protecting a shore area, harbour, anchorage or basin from wave action.

Bridge waterway

The effective distance between the bridge abutments through which river flow can pass unimpeded.

Contraction

The reduction in the effective width of the bridge waterway caused by returning flow from the flood plain.

Critical height

The maximum depth that the sides of a scour hole can withstand before further collapse takes place during the process of localised scour.

Critical velocity

The maximum velocity at which the revetment will remain stable without movement of the stone fill.

Dead men

The buried anchors to which steel jacks are attached to provide additional stability to the system during floods.

Diversion lines

The row in a jetty field aligned according to the desired river bank.

Dolos

A concrete armour unit predominantly for coastal protection and closely resembling a ship anchor or 'H' with one vertical side perpendicular to the other.

Estuary

The part of a river that is affected by tides and where the fresh water of the river mixes with the salt water of the sea.

Flukes

The two dolos parts connected by the shank.

Head cutting

The progressive scouring at the upstream stepped end of an erosion channel.

Headers

Sand-cement bags laid with the longer side of the bag at right angles to the face of the embankment.

Interstitial velocity

The flow velocity through the voids in the riprap layer.

Limiting velocity

The velocity at which there is some deformation of the mattress due to movement of individual stones.

Local average velocity

The velocity upstream of a pier or other obstruction in the river channel beyond the influence of the obstruction.

Local scour

The generation of scour around piers and abutments caused by the deflection of flow and the subsequent development of vortex systems induced by these obstructions to the waterway flow.

Median stone size

The size of stone for which 50% is smaller by mass.

Nick points

The small beginnings of scour ruts.

Normal stage

The level the unobstructed natural flow in the river reaches during the design flood.

Resistant shear stress

The ability of the stones to resist movement.

Revetment

A cladding constructed on an embankment to protect and sometimes stabilise its surface against erosion by currents and wave action.

Runup

The extent to which water reaches up a structure or beach on the breaking of a wave or the vertical height above the still water level to which the rush of water reaches.

Stretchers

Sand-cement bags laid with the longer side of the bag parallel to face of the embankment.

Sward

Expanse of short grass.

Thalweg

The line of greatest depth along the river channel.

Threshold flow

The limiting flow below which no damage is to be expected.

Tussocky grass

Grass that grows as individual tufts.

Vertical closure method

Method of construction of the embankment fill in horizontal layers until complete.

Wing lines

Rows of steel jacks in a jetty field which extend from the banks of a river at an angle to the direction of flow.

1. INTRODUCTION

1.1 Background

Road embankment protection against erosion dates back to the early times of road construction. The methodology follows on from the knowledge of systems developed over the years between the establishment of riverside settlements in pre-history to the present times. Early methods of protection developed as an art rather than a science and use was made primarily of naturally occurring materials. These materials are used today alongside the new materials and new systems, which are continually being developed, to meet the increasing demands by responsible authorities.

The extent of the protection is to a large degree determined by the materials used in the construction of the embankment, exposure to damage and the severity of attack.

For the proper design of embankment protection considerable river engineering judgement and knowledge of the historic channel behaviour is required.

Although in most cases velocity is used as the common denominator in the design of armour as protection against erosion, it is in fact the least meaningful parameter in describing the erosive potential of the in situ material. To clarify this concept, Section 1.6 "Critical Erosion Conditions" is included as background information to understand the erosion action.

1.2 Scope

This volume covers the protection to :

- Road embankments contiguous to waterways against contact flow
- Embankments located within the tidal zone where wave action is the overriding consideration
- Banks of flow channels
- Foundations of structures

The methods used for protection can be subdivided as follows :

- Direct protection which includes sloping and vertical bank protection

- Indirect protection which comprises manmade controls in the waterway to regulate flow paths and velocities to minimize erosion forces

The following conditions are dealt with herein :

- Road embankments, where the toe of the road embankment runs parallel to or even protrudes into the waterway

- Road embankments encroaching into the flood plane or flow channel

- River banks, where erosion thereof may threaten a nearby road or structure

1.3 Planning

Planning involves the necessary site investigations together with preliminary design studies to identify those key factors specific to erosion and/or embankment failure that would dictate the protection measures to be adopted. In the study due cognizance is taken of the interim and final conditions affecting the design.

The following factors, although not all inclusive, requiring assessment are :

- Vegetation and soil conditions
- Channel course, shape, size and slopes
- Long term stability of the water course
- Adjoining development and features affecting the natural behaviour patterns of the water course
- Manmade structures within the channel
- Aesthetics
- Recreational activity e.g. boating etc.
- Hydraulic parameters viz. :
  - Hydrological data
  - Flow velocities
  - Flooding incidences
  - Scour potential
  - Potential wave action

Once the main factors dictating the potential for erosion and/or scour have been identified, a short list of preferred options may be identified in accordance with the consideration shown in Figure 1.1. Consideration can also be given to a composite approach comprising part vertical and sloping protection. The type and size of protection will be dictated by the severity of a attack and the level of service demanded. The selection of the optimal solution is an iterative process taking due cognizance of the influence of the above-mentioned factors and economic considerations.

1.4 Economic considerations

To assess the viability of a protection system, a cost analysis should be made taking into account the capital cost and expected concomitant maintenance/rehabilitation/replacement costs. When comparing the costs of different protection systems the full life cycle costs shall be used and assessed in terms of the available/ready funds, with due consideration be given to labour based factors.

Under maintenance considerations the ability to carry out the appropriate inspections and monitoring and the availability of expertise and skills enabling the undertaking of particular types of work should be clarified with the owner at the onset. Any shortcomings in the maintenance chain of events would critically affect the serviceable life of the service.

In the economic assessment due consideration should be given to the strategic importance of the service and the consequential loss to the user in the event of disruption thereto. Thus services of high economic or strategic importance warrant more durable and sophisticated protective measures than those for services of secondary importance.

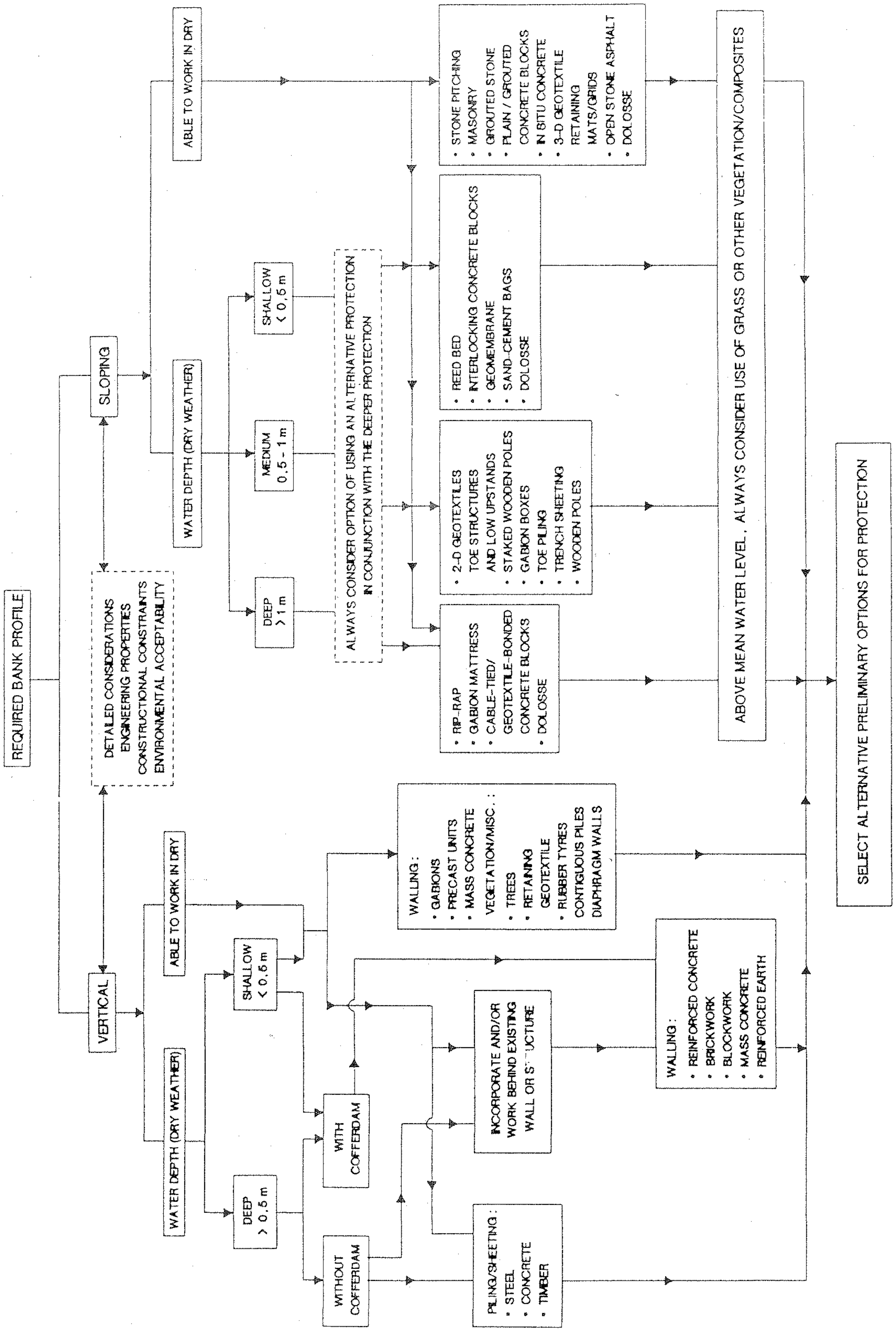

FIGURE 1.1 : CONSIDERATIONS FOR SELECTION OF PRELIMINARY OPTIONS

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Listed in Table 1.1 are a selection of bank protection systems subdivided into slope protection and vertical protection with generalized relative installation costs associated with them. The relative costs only refer to items within the same subdivision. The list also includes protection systems not normally used in this country, but are given for interest.

Method of Bank Protection	Comparative installation costs
Slope protection	
Grass	Low
Riprap	Low
Interlocking concrete blocks	Medium
Gabion mattresses	Medium
Geotextile mats and fabrics	Low
Stone pitching	Low
Sand-cement bags	Medium
Concrete blocks, grouted	Medium
Concrete blocks, cable-tied	High
In situ concrete slab	High
Dolosse	High
Vertical protection	
Steel sheet piling	High
Timber, piles/waling	Medium
Gabion boxes	Medium
In situ concrete wall	High
Brickwork wall	High
Reinforced earth	High

TABLE 1.1 : COMPARATIVE INSTALLATION COSTS FOR PROTECTION SYSTEMS

1.5 Environmental and ecological acceptability

The impact of the protection system on the environment must be considered from both the natural and the human environmental aspects. In certain high profile projects environmental impact studies by environmental specialists might well be justified.

1.5.1 Natural environment

Before design on the protection system is commenced with, communication shall be established with organizations responsible for the protection of the natural environment, to determine the extent of the modifications and preserve of features, so as not to impact negativity on the natural habitats. This may inter alia involve the Department of Environmental Affairs and/or the Department of Water Affairs and Forestry.

1.5.2 Aesthetics

Aesthetics of the protection system and the harmonious integration thereof with the immediate environment is essential. At worst it should not offend the beholder and at best it should enhance the surroundings. "Hard concrete" solutions may be better suited to urban situations, whilst natural solutions are desirable in rural locations. It is essential that attention be paid to both the appearance immediately after completion of the protection system and finally in the long term.

Consideration should also be given to the active human environment, as water facilities are increasingly used for recreation.

1.6 Critical erosion conditions

When the sediment transporting capacity of a stream becomes sufficient to entrain erodible material on the stream bed it is said that the critical condition has been reached.

Various parameters have been used to represent critical conditions. These parameters include average velocity (Hjulstrom-diagram) shear stress (Shields-diagram) and shear velocity (Liu-diagram). Of these parameters velocity is the least meaningful in describing erosive potential. The only way in which the critical transporting capacity of a stream can be linked uniquely to that required for entraining the bed material is by comparing the power being applied along the bed to the power that is required to lift the (cohesionless) fine bed material into suspension or the power that is required to initiate rolling/sliding in the case of coarser bed material.

If flow is one-dimensional, then the critical relationship derives from :

$$\left(\tau \frac{dv}{dy}\right) c = \alpha (\rho_s - \rho) g V_{ss} \qquad (1.1)$$

= power being applied per unit volume along the bed by the stream

$(\rho_s - \rho) g V_{ss}$ = power required to suspend bed elements (t/ms^3)

$\frac{dv}{dy}$ = vertical flow velocity gradients (s^{-1})

where

τ = shear stress along bed (kN/m^2)

ρ_s = mass density of bed elements (t/m^3)

ρ = mass density of fluid (t/m^3)

g = acceleration due to gravity (m/s^2)

V_{ss} = settling velocity of bed elements (m/s)

α & c = factors

In the case of one-dimensional rough turbulent flow over a bed that is even, this relationship becomes :

$$\frac{\sqrt{gD.S}}{V_{ss}} = 0,12 \tag{1.2}$$

where

D = depth of flow (m)
S = energy gradient in flow direction (m/m)

This relationship is valid for sediment particles > 2 mm in diameter, i.e. virtually all cases where protective elements are used to prevent bed scour.

For stones with density of 2 650 kg/m^3 and diameter d, this equation approximately becomes equivalent to :

$$d = 11\ D.S \tag{1.3}$$

the well-known criterion originally derived from the Shields diagram.

This equation cannot be applied to elements with shapes which are distinctly non-spherical, e.g. specially shaped concrete protection blocks.

It is thus possible in the case of one-dimensional turbulent flows to calculate quite accurately by means of Equation 1.2 the required settling velocity of elements that can not be transported by a stream. Equation 1.3 provides a less accurate definition of bed material stability in terms of required particle diameter and it is strictly valid only for round elements of density 2 650 kg/m^3.

In terms of Equation 1.1, and because $\tau \alpha \left(\frac{dv}{dy} \right)^2$, the transporting or erosive capacity of a stream can be expressed in terms of $\left(\frac{dv}{dy} \right)^3$. This means that the erosive capacity of a stream is highly dependent on the velocity gradient just above the bed.

Average flow velocity is often an unreliable indicator of erosive potential even in one dimensional flows. Where established critical flow velocities are indicated, they are only valid in cases where velocity gradients are equivalent.

Furthermore, with two- and three-dimensional flow conditions, the erosive potential can be linked to $\left(\frac{dv}{dz} \right)^3$ with dv being the maximum change in velocity over a distance dz in any direction. Erosive potential therefore needs to be evaluated in terms of rate of change in velocity in all directions. High velocities as such do not necessarily represent high erosive potential, unless these high velocities occur close to stream beds or banks and velocity gradients, therefore, need to be high. Velocities which vary greatly in magnitude and/or direction over short distances are representative of highly erosive conditions. The function $\frac{dv}{dz}$ not only represents velocity gradient in a z-direction, but also local angular velocity.

Speed of rotation rather than translatory speed in a stream, therefore, is significant in determining the erosive potential of a stream at any point.

Equations 1.2 and 1.3 can be used to determine whether or not cohesionless elements will be stable with regard to entrainment on a stream bed.

Figure 2.12 (Rooseboom et al. 1988), on the other hand, indicates critical velocities for cohesive bed materials at different flow depths, thus allowing for differing velocity gradients, and are valid even when the bed is deformed.

2. DIRECT PROTECTION

2.1 Slope bank protection

2.1.1 Introduction

River banks that are steep and liable to collapse shall be protected by providing vertical protection in accordance with Section 2.2, or by flattening the slopes to at least the natural angle of repose of the material.

Man-made embankments are invariably constructed to stable side slopes, but this may not prevent erosion due to water flow or wave action. To overcome erosion of embankments, a revetment is provided to give protection against the direct erosive forces of currents, wave action or other external effects caused by man and animal. This revetment usually also has to accommodate surface drainage water and ground-water movement or subsoil drainage in the underlying bank. Revetment is not intended or designed to improve the stability of the underlying bank or embankment, but purely as a protective cladding against erosion.

The component parts of a typical revetment are illustrated schematically in Figure 2.1. A typical revetment comprises the armour layer and an underlayer. The armour layer provides the protection against the direct external erosive forces from currents, wave action, etc., while the underlayer, comprising all materials between the armour layer and subsoil formation, serves a number of functions which are not dealt with in this document. The underlayer primarily consists of a layer of granular material or geotextile membrane or combination thereof.

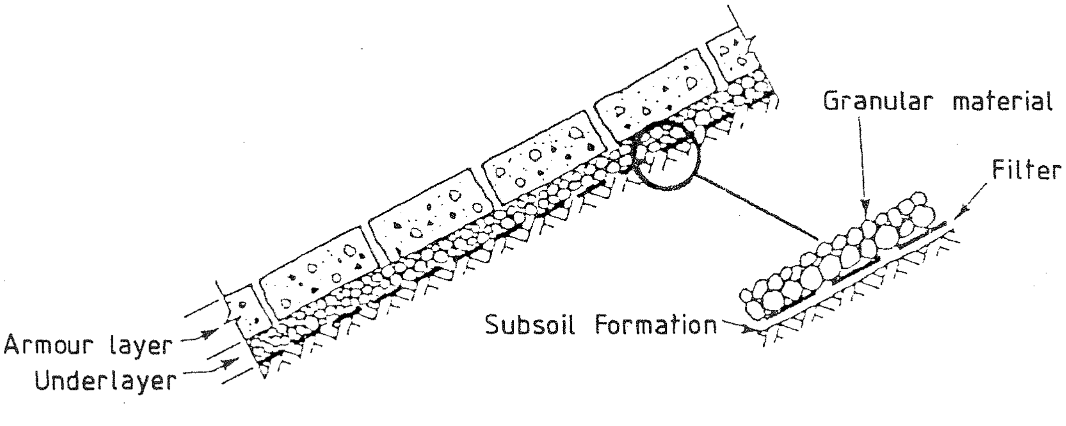

FIGURE 2.1 : COMPONENTS OF A TYPICAL REVETMENT

The performance of the revetment is dictated by the design and construction thereof. The design criteria shall be compatible with the characteristics of the subsoil and is dependent on a number of factors, of which the velocity of flow is only one of them. Refer to Section 1.6. Of equal importance is the effectiveness of the crest, toe and edge construction.

Only those revetment types listed below, which are considered effective under South African conditions, are described in these guidelines. This does not preclude the use of other types, such as rolled concrete, sprayed coatings, motor vehicle tyres, in situ and precast concrete or timber.

- Stone pitching
- Riprap
- Gabion mattresses
- Precast concrete blocks
- Sand-cement bags
- Vegetation
- Container cells
- Dolosse

2.1.2 Stone pitching

There are primarily three methods of stone pitching, each with its specific application, viz. :

- Plain stone pitching
- Grouted stone pitching
- Wired-and-grouted stone pitching

These methods are fully described in Section 5100 "Pitching, Stonework and Protection against Erosion" of the CSRA Standard Specifications for Road and Bridge Works 1987.

Stone pitching consists of a single rock course placed on prepared embankment slopes or stream banks and beds to form a relatively smooth surface. This form of revetment becomes economical wherever stones of the desired size and quality are readily available. It is usually labour intensive in that it is entirely laid by hand and requires a moderate level of expertise. It still remains one of the most widely used methods of scour protection and is one of the few types of revetment of which the material can be re-used.

Although stone pitching can be used for the protection of the entire section, it is generally used to protect only the toe of the bank or embankment below the design flood level, with vegetational protection at higher levels (Dobbie 1980). It does not offer any protection against scour, unless a rock toe is provided. However, due to its flexibility, it retains a degree of effectiveness under limited subsidence.

Stone pitching cannot be installed under water. If taken down to bed level, it should be constructed in a cofferdam, or when the channel is dry (Hempill and Bramley 1989).

Plain stone pitching systems will eventually permit some growth of vegetation when exposed to fresh water. To promote growth, it is desirable to cover the stone with top soil. The vegetation has a binding effect and tends to restore the natural roughness of the channel. It may, however, give an untidy appearance.

(i) Design

In Table 2.1 (Dobbie 1980) an indication only is given of the minimum mass of the individual stones as a function of the flow velocities adjacent to the bank. The values bear no relation to flow depth.

Surface flow velocity in m/s	**Min. mass of stone pitching in kg**			
	Bank slope			
	1 : 1	1 : 1½	1 : 2	1 : 3
1	0,09	0,03	0,02	0,01
2	5,7	2,0	1,3	0,89
3	64	23	15	10
4	360	130	84	57
5	1400	500	320	220

TABLE 2.1 : STONE SIZE FOR VARIOUS SURFACE VELOCITIES

Notwithstanding the above, it is recommended that this method is not used under conditions where the velocity exceeds 4 m/s or on slopes of 1 : 1 and steeper.

For stability it is recommended that cut-offs be considered along the perimeter of the area to be stone pitched.

Due to the lower Manning values for stone pitching, which ranges between n = 0,020 to 0,025, its hydraulic performance surpasses that of riprap.

Plain stone pitching itself is permeable and fine material will therefore tend to leach out. To counter this, if necessary, a natural or artificial filter can be included under the pitching. Cement grout is sometimes used to prevent leaching, but it also leads to the need for weep holes to release hydrostatic pressures within the bank.

(ii) Materials and construction

These aspects are well described in Section 5100 "Pitching, Stonework and Protection against Erosion" of the CSRA Standard Specifications for Road and Bridge Works 1987, and should be consulted in this regard.

As an alternative to choking the space between the pitching stones with spalls of the same rock, soil can be rammed into the spaces, whereafter the pitched surface is covered with topsoil and planted with grass.

(iii) Maintenance

Stone pitching requires routine inspection as well as repairs, to counter progressive failure occasioned by dislodged stones.

2.1.3 Riprap

Riprap is the term given to loose rock armour. The advantages of riprap protection are the following :

- General ease of construction including the placing with proper control under water
- Flexibility
- High hydraulic roughness which contributes to the attenuation of currents and waves
- Low demand on maintenance and ease of repair
- High degree of durability

Riprap is a widely used system for the protection of erodible river banks, road embankments and pier foundations. The system is not recommended at abutments where highly erosive flow conditions tend to be induced.

(i) Design

There is no definite grading for riprap stones. Stevens (1984) advocates a continuous grading as shown in Figure 2.2.

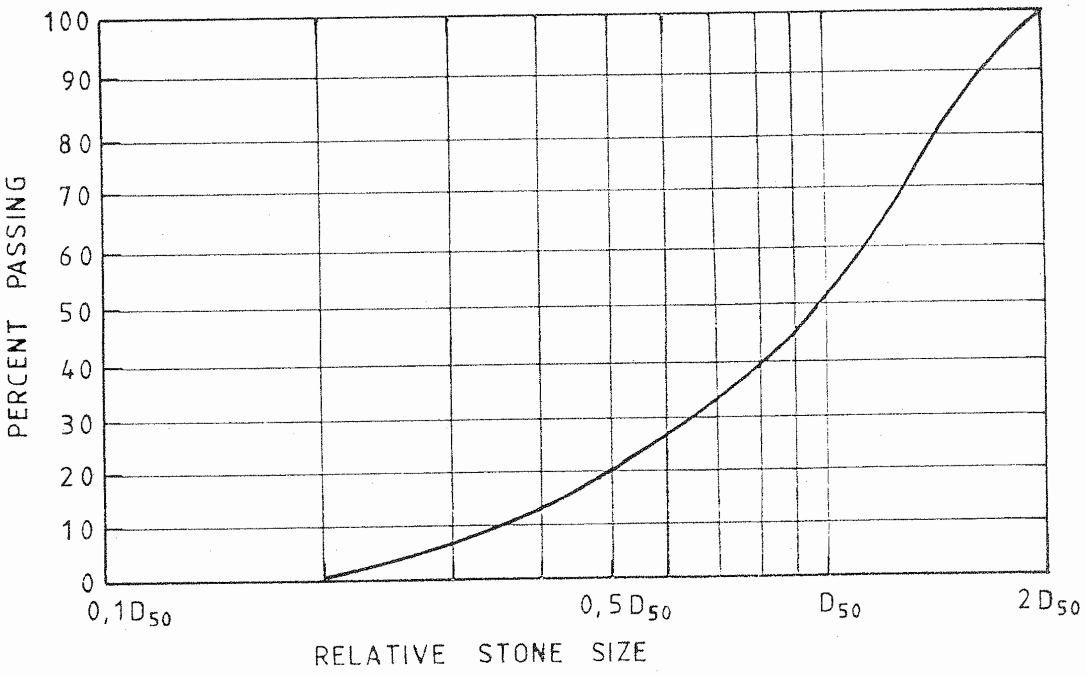

FIGURE 2.2 : RECOMMENDED GRADING FOR RIPRAP

The minimum thickness of riprap shall be sufficient to accommodate the largest stone used in the construction and shall be increased up to 50% depending on the severity of the expected wave action. To prevent fluidizing, the size of stone located under water shall be increased by at least 50% of the D_{50} size. Unless the subsoil comprises a well graded material forming a natural filter, a specially designed gravel and/or geotextile filter should be provided under the stones.

(a) Protection of river embankments

Maynord et al (1989) note that many engineers prefer design procedures based on velocity. The appropriate velocity to be used in the design of riprap can be determined by using relatively simple methods.

Average channel velocity can usually be determined, but this is not representative of conditions at the rock lining because of a wide diversity of embankment shapes.

Riprap stability, as defined by Maynord (1988), is represented in the equation:

$$\frac{D_{50}}{D} = R\left[\left(\frac{\gamma_w}{(\gamma_s - \gamma_w)\,gD}\right)^{0,5} V\right]^{2,5} \qquad (2.1)$$

where

D_{50} = nominal 50% or median size stone, i.e. 50% of the stones being lighter than this size (m)

D = flow depth (m)

γ_s = specific weight of stone (kN/m^3)

γ_w = specific weight of water (kN/m^3)

V = depth-averaged velocity (m/s)

g = gravitational acceleration (m/s^2)

R = riprap stability coefficient related to the degree of turbulence in the river channel

For practical purposes depth-averaged velocity can be considered to be approximately equal to average channel velocity, especially in the case of wide channels.

Determination of riprap stability coefficients is the subject of ongoing research. Until these are refined a coefficient $R = 0{,}75$ is suggested. For practical purposes the layer thickness is normally $1{,}5 \times D_{50}$.

It is recommended that a safety factor of 1,2 be applied to ensure the stability of the riprap (Maynord 1988). Introducing a safety factor, the design equation becomes :

$$\frac{D_{50}}{D} = SF \times R\left[\left(\frac{\gamma_w}{(\gamma_s - \gamma_w)\,gD}\right)^{0,5} V\right]^{2,5} \qquad (2.2)$$

Equation 2.2 is applicable to channel bottoms or side slopes with inclination of less than or equal to 1 : 2.

Riprap size is only one of the several important factors in the design. Toe protection, placement method and filter bedding are some of the other factors that must be considered.

Riprap placed in a bend should generally be designed for the location along the bend that has the highest velocity or the most severe attack. As a rule of thumb, it is recommended that :

$V_{bend} \quad \leq \quad 1{,}5\ V$ (Maynord et al 1989)

where
V_{bend} = maximum depth-averaged velocity in the bend

The designer is reminded to take cognizance of the superelevation of the water surface at bends and to make the necessary adjustments to the height of the riprap protection.

(b) Protection of road embankments for overtopping flow

Overtopping protection can be achieved by the placement of a rock layer on the downstream face of the embankment. However, undersizing of the riprap or layer thickness may result in a fluidizing of the protective layer, subjecting the embankment to severe erosive processes. Therefore, the objective in the design should be the prevention of stone movement and progressive riprap layer failure.

Specifications for riprap usually stipulate the use of angular-shaped stone. To determine the riprap layer stability for angular-shaped stones, the median stone size D_{50} designed to resist stone movement (Abt and Johnson 1991), is correlated to the unit discharge q_d and the embankment slope S_e in the equation:

$$D_{50} = 0{,}0523 S_e^{0{,}43}\ q_d^{0{,}56} \tag{2.3}$$

where
S_e = embankment slope (m/m)
q_d = design unit discharge (m^3/s.m)
D_{50} = nominal 50% stone size of angular-shape (m)

It must be noted that in the case of rounded riprap stone, oversizing of about 40% is advocated to provide an equivalent level of protection in comparison with angular riprap stone, although there is no evidence to support this contention at this stage.

(c) Protection of bridge piers

A leading cause of river bridge failures has proved to be the scouring of foundation material by floodwaters. Bridge piers obstruct flow and induce local secondary currents that have a much higher capacity for eroding bed material than unobstructed flow. Local scour holes form in unprotected alluvial streambeds surrounding bridge piers, often to the extent that the pier foundation is undermined. The undermining causes settlement and, in some cases, the collapse of supports and bridge spans.

One of the most common methods for protecting piers is the placement of a riprap apron to armour the streambed influenced by the secondary currents. However, the stone sizes required to protect the streambed tend to be large and the mass per stone can often be between 1 and 1,5 tons.

The equation, recommended by the Federal Highway Administration (FHWA) is well suited for determining stone sizes to protect varying shaped piers from scour. The method involves determining the local natural average velocity just upstream of the pier where the pier does not directly influence the flow. This velocity V is multiplied by a factor f which can range from 1,5 to 2,0 depending on the shape of the pier. This velocity is then used to calculate the stone size necessary for stability. The equation can be written as:

$$D_{50} = 0{,}347 \frac{(fV)^2}{g\,(\Delta - 1)} \tag{2.4}$$

where

D_{50} = nominal 50% size stone

Δ = relative density of the stone

V = depth-average velocity (m/s)

g = gravitational acceleration (m/s^2)

f = factor to account for variations in scour patterns (= 1,5 - 2,0)

Note that in this equation no allowance is made for variations in flow depth, which is normally taken into account.

It is recommended that in the equation :

f = 1,5 be used for *round-nosed piers that are :*

- Fairly well aligned with the flood flow
- Not subjected to debris accumulation
- Located in bridge openings where the general bed level is expected to remain above the rectangular base

f = 1,7 be used for piers :

- In which the rectangular base is expected to be above the general bed level
- That are founded on rectangular bases which are likely to be exposed by general scour throughout the bridge opening

For the rest engineering judgement should be applied. Parola and Jones (1991) suggest that a safety factor be applied to the stone sizes obtained. As research is not conclusive on the required layer thickness, it is recommended that the thickness be made three times D_{50}.

The riprap should preferably be placed in a preformed hole such that the top surface of the riprap is flush with the bed level. The extent of the riprap protection required is determined by the calculated size of scour hole, according to Section 4.2.2 in Volume I "Hydraulics, Hydrology and Ecology."

Research results clearly indicate that a riprap protection around a bridge pier can be constructed in a single homogeneous layer without filter layers (Blake and Millard 1993). The fine material in the river bed below the rock is leached out, allowing the riprap to settle to form a stable layer at a depth where velocities are reduced and no longer critical. The single layer protection will, in most cases, be cheaper than conventional riprap protections with filter layers.

(ii) Materials and construction

Riprap is normally placed by machine in the dry, although it can be hand laid or packed to improve the packing density or to key larger stones into the underlayer. The aim of machine placing should always be to dump the stone as close to its final position as possible, thereby ensuring a better grading. Spreading by bulldozer is considered to be a disadvantage and is best avoided, because it increases breakage, segregation and surface roughness.

The methods of construction and quality of material to be used are well documented in Section 5100 "Pitching, Stonework and Protection against Erosion" of the CSRA Standard Specifications for Road and Bridge Works 1987 and should be consulted in this regard.

(iii) Maintenance

A major advantage of riprap is its flexibility. Except in the case of extreme flooding, damage tends to occur gradually and, as the stones in the random pattern are inclined to move relative to one another, the riprap tends to be self healing. This tendency permits maintenance to be carried out on a routine basis to maintain its efficiency.

2.1.4 Gabion mattresses*

Gabions are an extension of the rockfill principle of providing a protective lining to river beds and banks and man-made encroachments in to the flood plain. They can provide effective protection against the action of moderate wave action. The main uses of such linings in this context are :

- Improvement of river bank stability
- Protection of approach embankment against scour
- Protection of embankment against erosion during overtopping
- Protection of abutment and pier foundations against scour

Gabion mattresses have many useful properties and advantages :

- Flexibility : The double twisted steel wire hexagonal mesh construction permits it to tolerate differential settlement without fracture. This is particularly important when a structure is on unstable ground or in an area where scour from waves or currents can undermine it.

- Strength : The strength and flexibility of the steel wire hexagonal mesh cage can withstand and absorb the forces generated by moving ground conditions or flowing water without destabilizing the gabion structure.

- Permeability : Hydrostatic heads do not develop behind or below gabion mattress structures because of their permeable nature. Their ability to combine drainage and retention functions make them ideal structures for slope stabilization. However, this permeability in turn permits excessive leaching of the retained material. Therefore, a graded stone filter or a geofabric lining is required

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* In some publications the mattresses are described by the trade name of "Reno mattresses".

behind or under the structure to prevent loss of fines from the subsoil, especially in river applications.

- Durability : Its efficiency increases with age since progressive consolidation takes place as silt and soil collect in the voids and vegetation establishes itself.

- Economy : Gabion mattresses tend to be more economical than rigid or semi-rigid structures, wherever suitable rock is readily available, because :

  - Maintenance is generally minimal
  - Construction is simple and does not require skilled labour, although it does require skilled supervision
  - Minimum foundation preparation is required

- Ecology : Gabion mattresses permit the growth of vegetation and to a large extent blends in with the existing environment.

The disadvantages of gabion mattresses are :

- Wire corrosion, especially when exposed to aggressive environments, certain polluted water and veld fires.

- The removal of the wire mesh through pilferage or vandalism causes a dramatic loss in its efficiency.

(i) Design

The most important factor in ensuring a stable protective layer is its total mass. The minimum mass per gabion should be at least 1,5 to 2 times greater than the mass of the minimum size individual stone that will not be transported by the flow condition. See also Section 2.3.3(i).

The stability of gabion mattress linings depends not only on the strength of the mesh, but also on the thickness of the lining and the grading of the stone fill. Once the average flow velocity is known, these parameters can be selected by using Table 2.2. These figures are considered accurate for river bed linings and reasonably accurate for bank revetments, wherever the Froude No. < 1 (subcritical flow), according to Agositini et al (1988).

The critical velocity is the velocity at which the revetment will just remain stable without movement of the stone fill. The limiting or threshold velocity is the velocity at which there is some deformation of the mattress due to movement of the individual stones. In practice the possible unsightliness of the deformed structure might well be the reason for adopting a lower limit state, i.e. the critical velocity in the design. In most cases it is more appropriate to design on the limiting velocity, thus leaving some reserve for inaccuracies in estimation of active shear stress. This is even more reason why good stone packing is so important.

Type	Thickness (m)	Rock fill thickness		Critical velocity (m/s)	Limiting velocity (m/s)
		Size (mm)	D_{50} (m)		
Gabion mattress	0,15 - 0,17	70 - 100	0,085	3,5	4,2
		70 - 150	0,110	4,2	4,5
	0,23 - 0,25	70 - 100	0,085	3,6	5,5
		70 - 150	0,120	4,5	6,1
	0,30	70 - 120	0,100	4,2	5,5
		100 - 150	0,125	5,0	6,4
Gabion	0,50	100 - 200	0,150	5,8	7,6
		120 - 250	0,190	6,4	8,0

TABLE 2.2 : INDICATIVE GABION AND GABION MATTRESS THICKNESSES IN TERMS OF FLOW VELOCITY

A comparison between riprap and gabion mattress revetments from the technical aspect, and not the economical aspect, is illustrated in Figure 2.3.

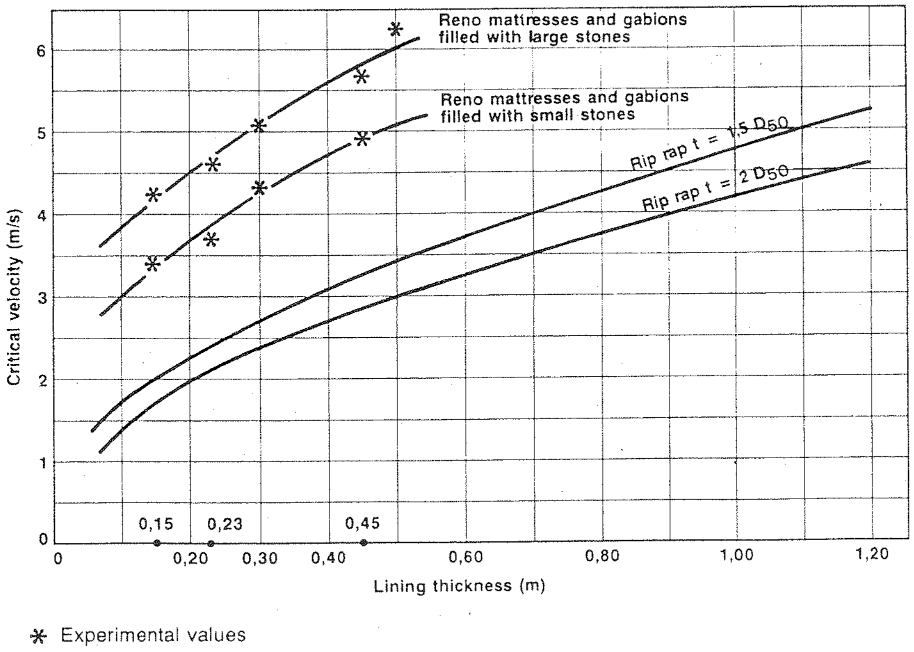


FIGURE 2.3 : CRITICAL VELOCITIES VERSUS LINING THICKNESS (AGOSTINI 1988)

Mattresses and other protective armour layers are capable of withstanding higher velocities where there are no changes in flow direction. Erosive potential is a function of the rates of change in velocity, i.e. vertically and horizontally.

A geofabric lining or graded stone filter against the embankment face substantially increases the stability of the armour by preventing the washing out of materials through turbulent interstitial flow fluctuations.

According to Hemphill and Bramley (1989), laboratory studies have indicated that sliding between the mattress and the underlying subsoil, due to wave down-rush and the water level downstream, is the predominent cause of failure on slopes steeper than 1 : 2. Failure due to uplift becomes critical on slopes flatter than 1 : 6.

The design for wave action is complex and specialist advice is recommended for the design of gabion mattresses to withstand waves with heights in excess of 0,5 m.

Figure 2.4 illustrates a schematic method for the design of gabion mattress linings, as suggested by Agostini et al (1988). The procedure to be followed in the design of gabion mattress linings is given in the publication "Channelling Works" from African Gabions.

(ii) Materials and construction

The materials to be used and the construction of gabions and gabion mattresses are well documented in Section 5200 "Gabions" of the CSRA Standard Specifications for Road and Bridge Works 1987.

The gabion baskets and mattresses comprise double-twisted hexagonal galvanised mild steel mesh selvedged sheets that are assembled by wiring together the edges. If gabions are exposed to aggressive conditions, the thickness of the galvanizing should be increased or the wire coated with a PVC coating, as appropriate.

To allow for settlement with time, the units should be slightly overfilled so that the lid is stretched when the unit is wired closed. For limited applications the gabion might be pre-filled and placed by crane, but generally this method is not recommended. However, for underwater applications where the water is deeper than 0,5 m (Hemphill and Bramley 1989), gabion mattresses may be preassembled and filled prior to placing by using semi-submersible barges to avoid deformation or damage. For this method the mattress must have sufficient flexibility.

Mattresses should be laid at right angles to the flow direction on embankments and in the direction of flow on the stream bed, as shown in Figure 2.5.

Mattresses are normally laid to a maximum slope of 1:1,5. For steeper slopes stacked gabions offer an alternative solution. If mattresses are used on slopes steeper than 1 : 1,75, it is recommended that stakes be driven through the mattress into the subsoil to resist the sliding and uplift suction, which may occur during flood flows.

To eliminate progressive scouring of the subsoil, the mattresses should be provided with gabion cut-off walls spaced at regular intervals.

Diaphragms are required where gabions are subject to continuous stress due to wave motion or high flow velocities, to restrict movement of the stone within the cage. The proposed spacing of the diaphragms is shown in Figure 2.6.

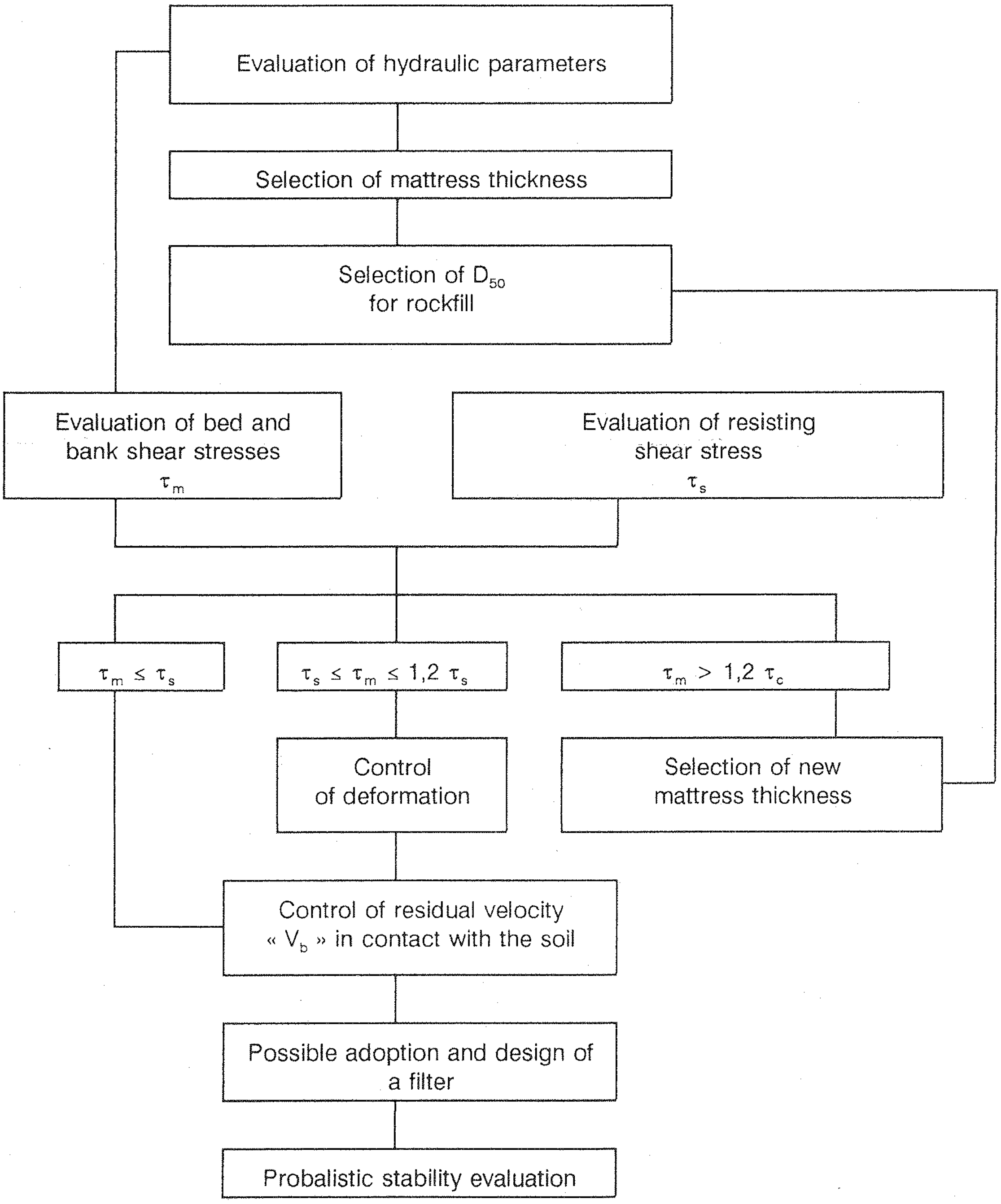


FIGURE 2.4 : FLOW CHART FOR THE DESIGN OF A GABION MATTRESS LINING

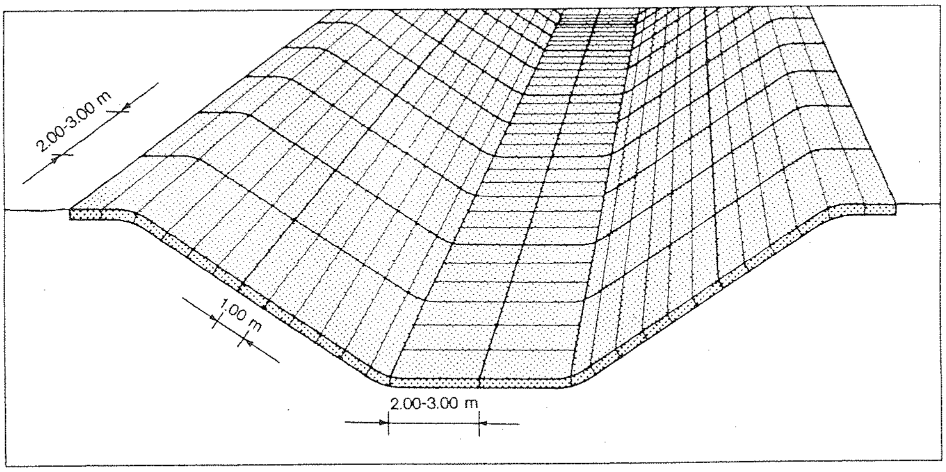


FIGURE 2.5 : PLACING ARRANGEMENT OF MATTRESSES

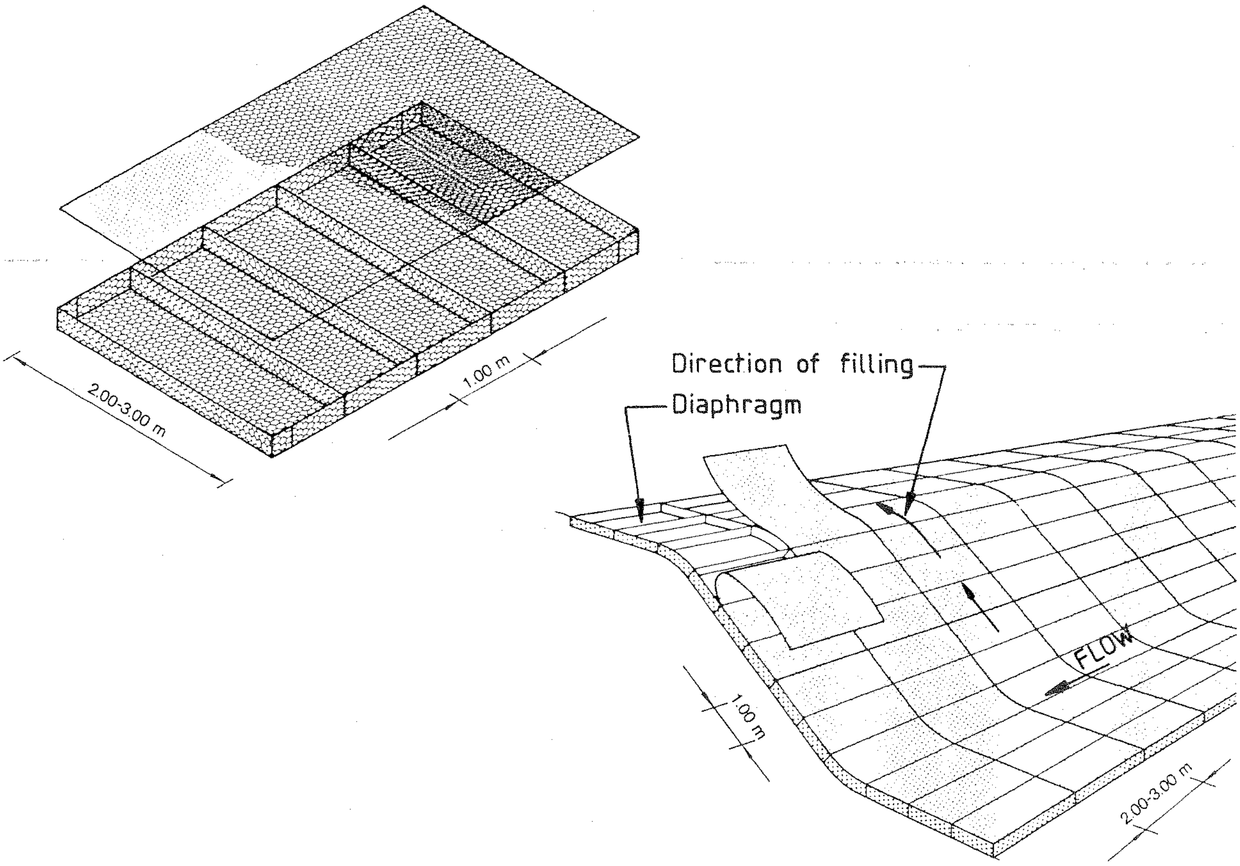


FIGURE 2.6 : DIAPHRAGM AND LID ARRANGEMENTS OF MATTRESSES

CSRA Guidelines for the hydraulic design and maintenance of river crossings
Volume III, 1994, Committee of State Road Authorities, South Africa.

2.1.5 Precast concrete blocks

Precast interlocking concrete blocks have the distinct advantage over cast in situ concrete slabs in that they permit the necessary flexibility in the channel armour, which is essential for long term scour protection. The small joints between the interlocks serve also to relieve pore water pressure without undue loss of fines. The following types of blocks are in use :

- Open-jointed or grouted blocks
- Interlocking blocks
- Cable-tied blocks

(i) Design

Notwithstanding the different types of blocks available on the market, this section deals solely with the design of the interlocking block that has a matrix of open cells for the establishment of natural vegetation.

The following design parameters are recommended by Hewlett et al (1987) for the hydrodynamic stability of interlocking blocks systems :

- Minimum superficial mass of 135 kg/m^2
- Minimum block mass of 15 kg
- Minimum block thickness of 85 mm
- Minimum block dimension in plan of not less than 3 x block thickness

Use of these blocks should be restricted to embankment slopes not steeper than 1 : 2. Flatter slopes are preferable where light wave action is to be encountered. According to Rooseboom (1983) a degree of uncertainty still exists regarding the limiting condition under which the interlocking blocks will be entrained and removed by flowing water.

It is suggested that the channel velocity should be limited to 3 m/s, provided the superficial mass of blocks is not less than 135 kg/m^2. Provided turbulent flow is not present (Hewlett et al 1987), the blocks will be hydro-dynamically stable in currents up to 4 m/s under normal flow conditions.

Individual blocks should be tapered to allow for the necessary flexibility. Whilst the permissible movement for interlocking blocks is small, it is usually sufficient to cater for the minor movements which occur in a generally stable bank. The use of a filter layer beneath the blocks is advocated for most situations where the loss of fines from the subsoil is likely. A geotextile filter fabric should be used with caution, as it prevents root penetration and can form a sliding plane between the blocks and the subsoil, if not anchored.

Wherever grouted blocks are used, weepholes should be provided at regular intervals to relieve the pore water pressure.

In the design of the system due consideration shall be given to restraining mechanism such as grass root action, mechanical stakes or by both against the hydraulic loading, vandalism, lack of maintenance, poor construction and drought.

(ii) Materials and construction

The precast concrete blocks should be of proprietary fabrication suited to the specific application. The blocks are available as individual blocks or tied mats.

The tied mats are available in various sizes and are formed by tying individual blocks together by means of galvanized steel or polyester rope to form an articulated mat. The mats are normally handled and placed with the aid of specially designed lifting rigs. The advantages of the mats over other systems are:

- Speed of construction
- Ease of underwater construction
- Less susceptible to progressive failure
- Greater flexibility

The blocks should be placed individually or in tied mats on a filter layer comprising either a graded granular filter or geotextile membrane that has been constructed on the pre-shaped area.

The interstices should be filled with a graded gravel, where the blocks are exposed to wave action or where they are located under water, and with topsoil above normal water level (Concor Technicrete 1990). The topsoiled areas should be seeded for the establishment of natural vegetation which would assist in binding the armour layer together.

Wyman et al (1983) suggest that the armour surface be supported by either a toe beam along the lower edge or a section embedded along the top edge, or by both as shown in Figure 2.7. Of these supports the latter is the more widely used method. Additional anchoring should be provided at intermediate positions to enhance stability under turbulent flow conditions. This anchoring can comprise cast in situ diaphragm walls or steel stakes placed at regular intervals. This is of special significance on the banks around bends.

To ensure stability of the interlocking block system under the design flow velocity the following conditions should be complied with :

- Downslope seepage flow below the armour layer is minimised by good contact between the blocks and the underlying subsoil.

- Surface irregularities, which can cause high localised uplift forces, are avoided.

- Good lateral restraint exists between adjacent blocks or panels.
- Good root or mechanical anchorage exists between the concrete armour and the subsoil.

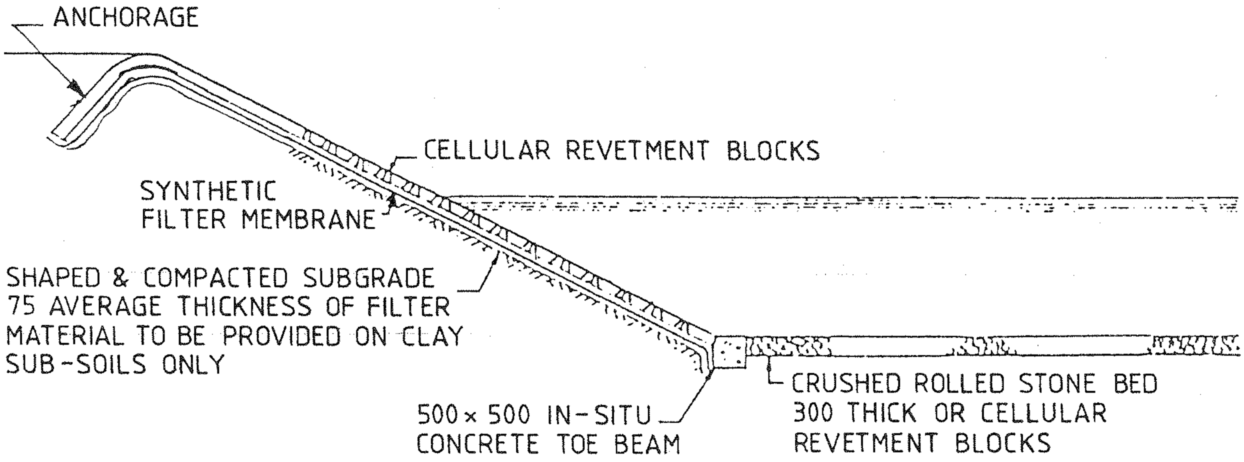


FIGURE 2.7 : TYPICAL USE OF PRECAST CONCRETE CELLULAR BLOCKS

2.1.6 Sand-cement bags

The sand-cement bag protection system is a relatively low cost method requiring semi-skilled labour for the construction thereof. It is especially suited to emergency embankment repairs and for temporary works, and can readily be placed under moving water.

(i) Design

For stability reasons sand-cement bags are normally not used as an armour layer against slopes that are steeper than the natural angle of repose of the embankment material. Therefore, its use if generally restricted to slopes not exceeding 1 : 1,5 while its use on slopes exceeding 1 : 1 is rare. In special applications and configurations sand-cement bags have been used successfully as temporary mass retaining walls.

When used as an armour layer, the protection should be high enough to prevent overtopping and flowing water from getting in behind the bag protection.

The protection should extend down to bedrock or at least to below the expected depth of possible scour. Cut-off diaphragms are recommended at the ends and at regular intervals on long stretches of protection. See Figure 2.8.

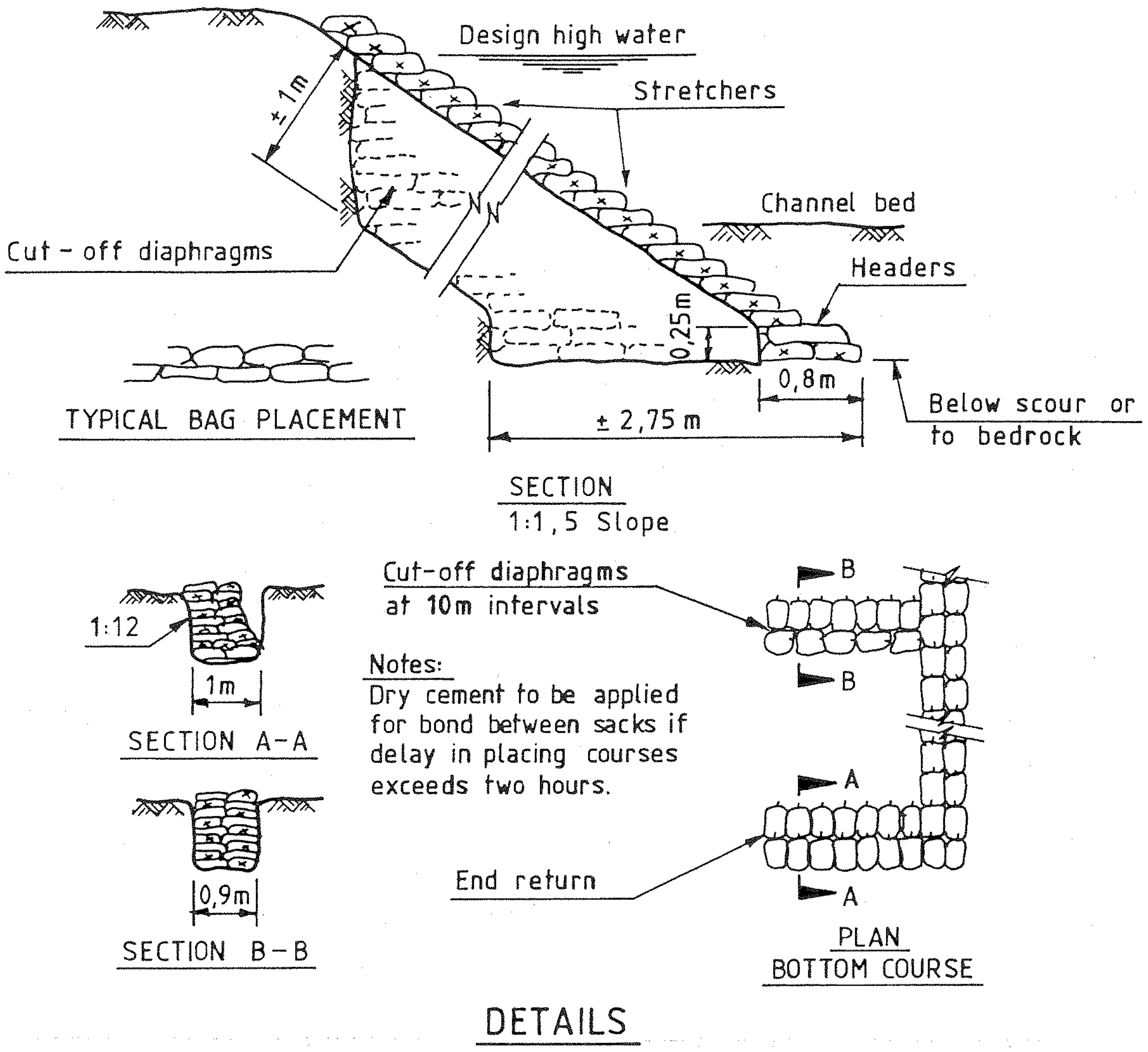


FIGURE 2.8 : DETAILS OF SAND-CEMENT BAG PROTECTION ON 1 : 1,5 SLOPE

The bag traditionally consists of a loosely woven fabric that allows the cementitious component to work through and provide bond between the bags. The size of the bag should be such that, when filled, two people would be capable of carrying and placing it.

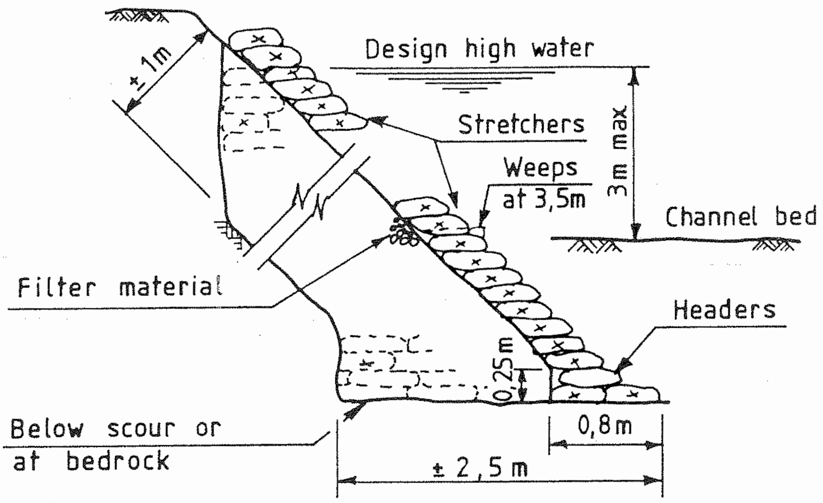
(ii) Materials and construction

The filler material should preferably consist of a sandy material, of which the PI should preferably not exceed the range of 6 to 8 with an upper limit of 10. The filler may contain a moderate content of gravel, with the maximum particle size not exceeding 38 mm.

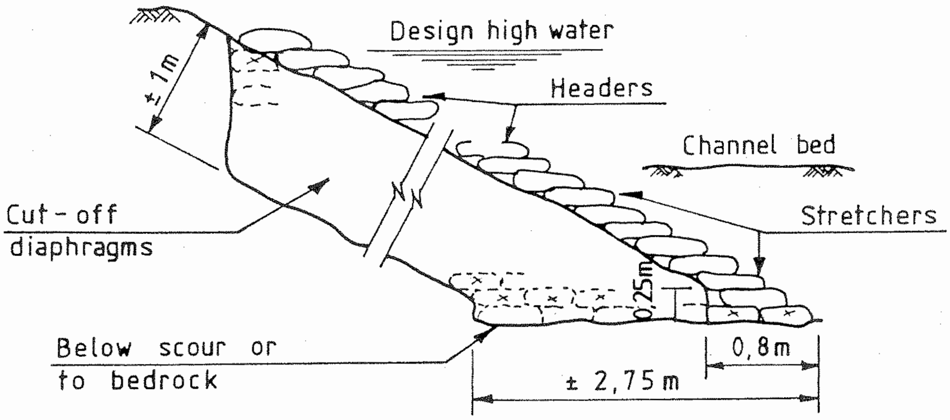
The cement/aggregate ratio is dependent on the application and, as a general guide, a mix with 6 to 8% cement by weight is recommended for most uses. Alternatively, plain clay or sand fill could be used, but are likely to be far less durable.

To ensure adequate contact between bags in successive rows, they should be somewhat underfilled, leaving sufficient material over to permit double folding to seal the bag.

On flat slopes the bond between individual bags would be limited and therefore the bags should be laid as headers rather than stretches on slopes flatter than 1 : 2. See Figure 2.9. It is not practical to endeavour to increase the area of bond by tilting the bags (Department of Public Works 1960).



SECTION
1·1 Slope



SECTION
1:2 Slope

FIGURE 2.9 : SAND-CEMENT BAG PROTECTION ON 1:1 AND 1:2 SLOPES

For appearance, preservation of shape and containment of the filler, the headers should be placed with the folds underneath and against the bank. When placed in stretcher bond, the folds shall be placed tightly against the previously placed bag. To ensure adequate contact between bags in successive rows, the bags should be somewhat underfilled.

The bags should be placed as soon as possible after mixing and filling.

To ensure horizontal bond between bags, successive layers should be staggered, as shown in Figure 2.8, with cement being added in the joints, if required.

2.1.7 Vegetation

The use of trees, shrubs and grass, whose root systems provide stabilization, are a recognised means of protection against erosion along river banks and road embankments. However, this section will be confined to the uses of grasses as the armour layer.

The use of deep rooted vegetal cover is well recognized as a low-cost and environmentally acceptable means of stabilising embankment slopes, which are subject to intermittent flow of water. According to Hewlett et al (1987), these can be effective with flows where the Froude No. < 1. In waterways with high velocity flow (Froude No. > 1), such as embankments subject to overtopping, the use of reinforcing materials can enhance the erosion resistance and protection provided by grass alone, while retaining much of its economic and environmental advantage over conventional engineering materials, such as concrete or rock. This form of armour can be improved substantially by the use of reinforcing materials which mostly consist of synthetic grids. However, for turbulent flows, when the Froude No > 1, vegetal cover in whichever form has limited value and then only for short duration flooding.

The planning and design of areas to be protected by vegetation against flowing water requires sound engineering judgement. These guidelines offer only a guide and the designers should call on expert advice in areas which are beyond their scope of expertise.

Possible disadvantages of this protection are that natural vegetation takes time to establish and become fully effective. Sometimes this can extend over several growing seasons.

(i) Design

The dominant factor affecting the type of vegetation growth for protective purposes is the water level regime in relation to the location of the plant. (Hemphill and Bramley 1989). The four zones identified in Figure 2.10 comprise :

- Aquatic plant zone, where at least the roots are permanently submerged
- Reedbank zone, where the plant grows along the water edge with roots below the water table. The plants can withstand occasional inundation.
- Damp, seasonal flood zone. This is the principal zone for bank protection, supporting the grasses and fast growing tree and shrub species.
- Dry, occasionally flooded zone. Vegetation here is relatively unaffected by the water level regime.

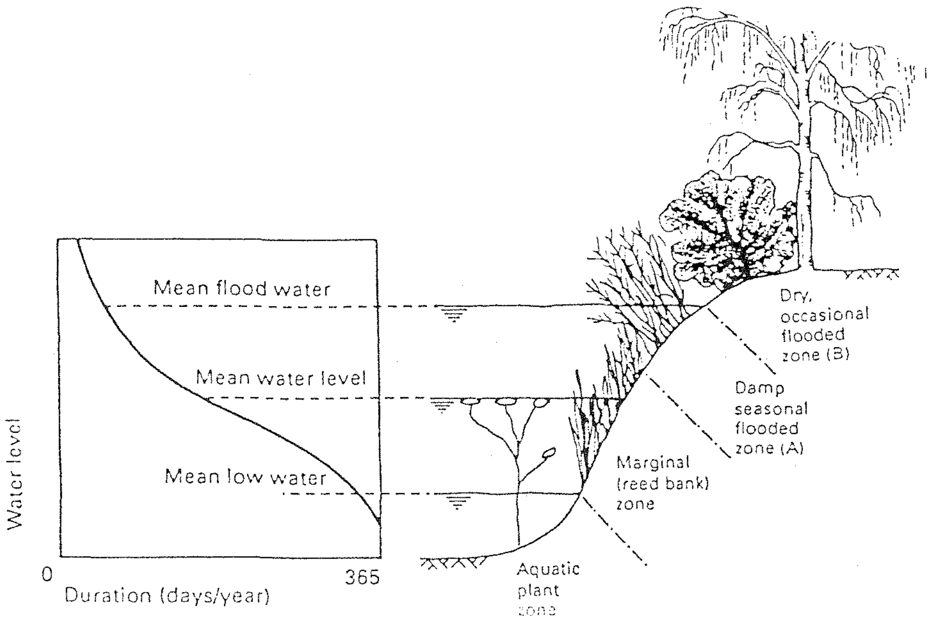


FIGURE 2.10 : VEGETATION HORIZONS ON A RIVER BANK

In South Africa with its seasonal rivers the predominant zones are the damp and dry zones, with the acquatic plant and reedbank zones allied to wetlands being the exceptions.

The applications of grass in this context relate to waterways, with uni-directional flow, having good grass growth and an underlying soil of relatively low permeability. Flow events should be of short duration, maximum of 2 days, upon which grass recovery must be possible.

Grass, plain or reinforced, is gaining popularity in use because of :

- Increasing availability of man-made reinforcing products.
- Increasing emphasis on environmental acceptability of the engineering solution.
- Low cost in application where the higher risk of damage or failure, compared to more expensive conventional forms of protection, is acceptable.

Reinforced grass protection can be used in situations where either :

- The flow velocity is sufficiently high to cause erosion which grass alone would not prevent, or
- The risks of not achieving adequate surface protection is high.

Dobbie and Partners (1980) state that in the design of grassed protection, the following basic requirements should be achieved:

- Full and intimate cover of the subsoil surface
- Discouragement of seepage flow down the embankment slope

- Good integration of the root mat with the underlying subsoil
- Avoidance of surface irregularities which will cause localised drag
- Correct soil conditions to enable the grass to achieve its intended growth potential. If this is in conflict with the geotechnical requirements for stability, grass should not be used

It should be noted that erosion is usually initiated by localised rather than general inadequacy of erosion protection.

In selecting a particular type of protection, care should be taken that the standards of construction and maintenance, implied by that type of protection, are achievable.

Two aspects of the hydraulic performance related to grass length are noted :

- When the sward is laid down during high flow, the ground cover is normally better than when the sward stands upright during low flow conditions. From a hydraulic design aspect, this can have the effect of reducing the Manning "n" value.

- The effect of the sward being laid down assists in reducing localised surface irregularities and thus reduces the drag forces during flow.

Thus, provided the botanical management requirements are satisfied, a uniformly long grass sward is beneficial from both an hydraulic viewpoint and as a suitable protection.

For plain grass the erosion resistance is assessed in terms of the velocity and duration of flow. The velocity/duration diagram, shown in Figure 2.11, is recommended as a guide for assessing the erosion resistance of plain grass protection with varying qualities of cover. Good grass cover is assumed to be a dense, tightly-knit turf established for at least two growing seasons. Poor grass cover, on the other hand, consists of uneven tussocky grass growth with bare ground exposed or with a significant proportion of non-grass weed species. Newly sown grass is likely to have poor cover for much of the first season.

To improve the erosion resistance of plain grass, a geotextile grid reinforcement system could be used to protect or stabilize the soil particles on the surface and to improve lateral continuity between the grass roots and tufts. The benefits from using mats and meshes are illustrated in Figure 2.11 (Hewlett et al 1987). Open mats have the advantage that good contact with the subsoil is achieved, as they can be filled with topsoil after installation ensuring full cover of all voids. The armour layer is, furthermore, likely to be less impermeable and more susceptible to relief of pressure.

Grassed slopes exposed to hydraulic flows should not be steeper than 1 : 3 and, for maintenance reasons, it is advised that the slope be restricted to 1 : 4.

Recommended design values for unprotected grass covers are given in Table 2.4 (Rooseboom et al 1983). The effectiveness of grass covers depends on both the grass species and the type of soil. It is firstly important to determine from Table 2.4 how well the particular grass species would establish, whereafter the appropriate system should be selected from Figure 2.12 to withstand the flow velocity. In no case may the flow velocity for a soil with plain grass cover exceed the flow velocity for unprotected soil by more than 30%.

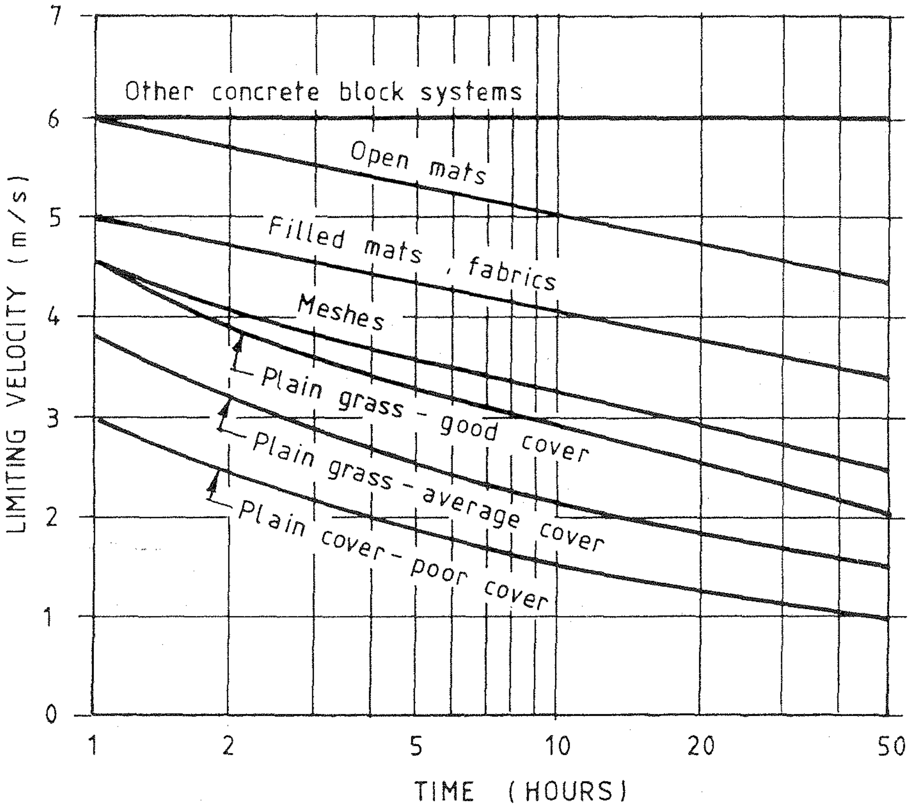


FIGURE 2.11 : RECOMMENDED LIMITING VALUES FOR EROSION RESISTANCE OF PLAIN AND REINFORCED GRASS TO TIME OF EXPOSURE

Annual rainfall (mm)	**<600**			**600 - 700**			**>700**		
Clay content of soil (%)									
Grass species	**>15**	**6-15**	**<6**	**>15**	**6-15**	**<6**	**>15**	**6-15**	**<6**
Kikuyu	-	-	-	1,8	1,5	0,8	2,5	2,0	1,2
NK 37	1,5	1,2	0,8	2,0	1,5	0,8	2,0	1,5	1,0
K11	-	-	-	1,5	0,8	0,6	2,0	1,5	1,0
Rhodes	-	-	-	1,2	0,8	0,6	1,5	1,0	0,8
*E Curcula	1,0	0,8	0,6	1,2	0,8	0,6	1,5	1,0	0,8
Blue Buffalo	1,0	0,8	0,6	1,2	0,8	0,6	-	-	-
Paspalum didatum	-	-	-	1,2	0,8	0,6	2,0	1,5	1,0

* Only on slopes of less than 3%

TABLE 2.4 : PERMISSIBLE VELOCITIES FOR GRASS COVERS

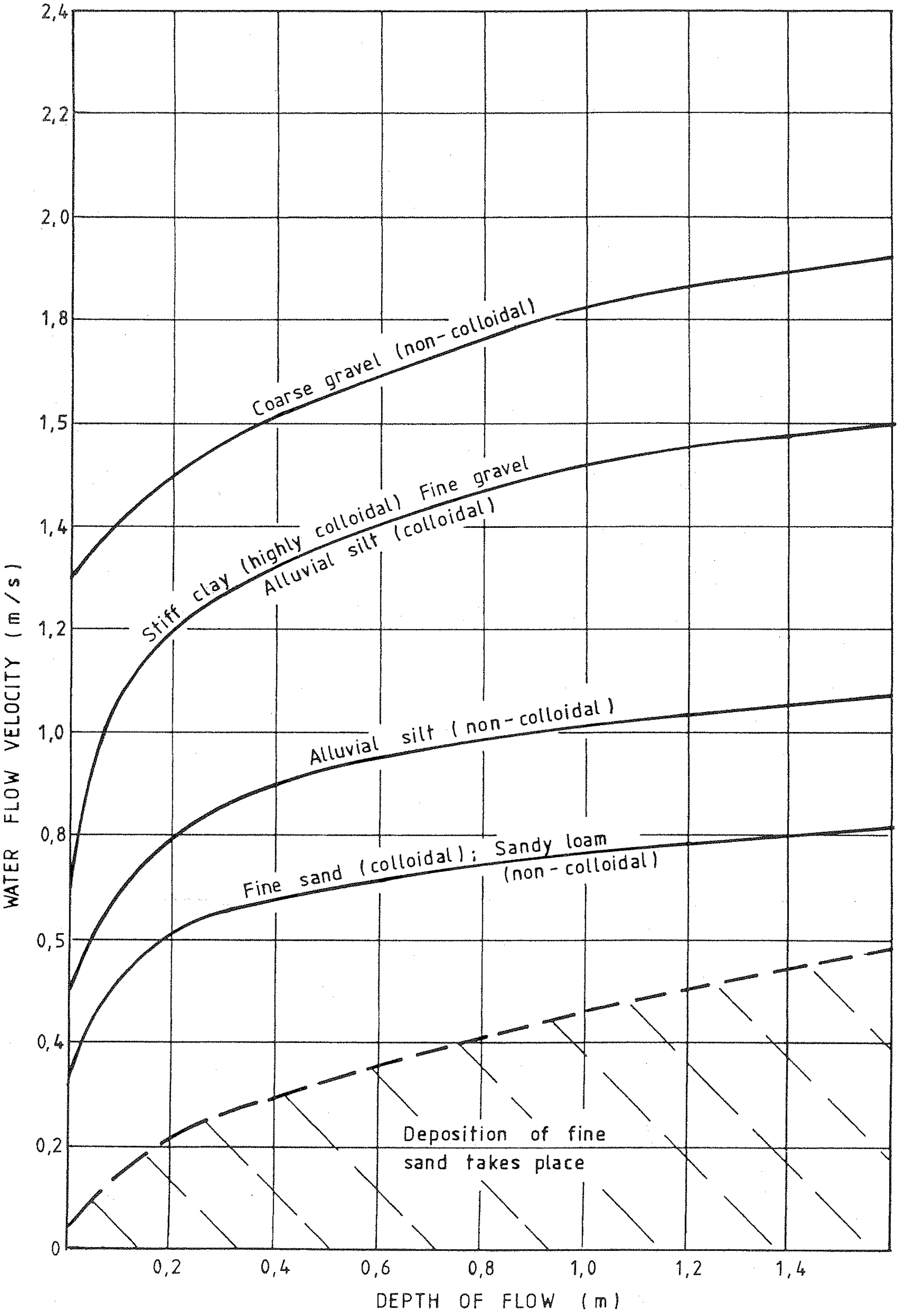


FIGURE 2.12 : PERMISSIBLE VELOCITIES, EROSION AND DEPOSITION (ROOSEBOOM ET AL 1983)

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For a grass waterway to perform satisfactorily a good grass cover is essential. This can be achieved by the correct selection of grass mixture, as well as the proper management of the grass.

The strength and health of the grass depends on the existence of a well-developed root system. Generally the root systems of most indigenous grass species are resistant to drought conditions once they are well established. Even the dried up roots will help to reinforce the soil structure to some extent. If the grass becomes damaged by fire, the roots would be unaffected with shoots beginning to grow again very quickly under favourable conditions.

(ii) Materials

The choice of a suitable seed mixture or quality of turf is most important for its performance and the selection must be based on the soil type, hydraulic and erosion suitability, and maintenance requirements. Designers are advised to liaise with the Research Institute for Reclamation Ecology at Potchefstroom University for expert advice on seed mixes and grass species for specific applications and locations. From an ecological viewpoint, it is recommended that standard seed mixes be avoided.

Reinforcement in a grass waterway can be provided by either concrete or geotextile. The use of concrete is covered in Section 2.1.5 "Precast Concrete Blocks". Geotextile materials for this purpose consists of meshes and mats. These are plastic grid-like structures with the three-dimensional products known as geocells. Although these products on their own reinforce the soil, their ability to resist the erosive forces is low, as can be seen from Figure 2.11. They should be manufactured from a synthetic polymer that will not be subject to degradation by the chemicals and organisms present in the soil. The sizes of the openings in the fabrics should be sufficiently large to permit growth of the grass through the geotextile.

(iii) Maintenance

When vegetation is used as the protective layer, management thereof is critical to maintain its long term protective roles. Once the grass has been established, the swards should be subjected to regular cutting or grazing. This repeated cutting promotes lateral growth, which eventually results in dense, evenly distributed grass surfaces.

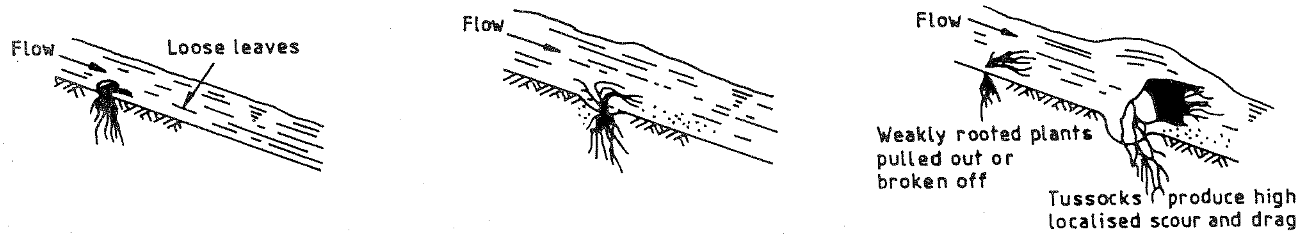
Hewlett et al (1987) found that the erosion of plain grass surfaces caused by flowing water occurs in different ways as illustrated in Figure 2.13.

2.1.8 Container cells

Container cells or geogrids, such as Hyson-Cells, are commonly used for the construction of channel linings and embankment protection against erosion.

The cells are manufactured from high density polyethylene sheeting to form a three-dimensional honeycomb grid of thin walled cells. The cell mattress is delivered in a compact collapsed roll which is expanded over the area to be lined and filled with the appropriate material.

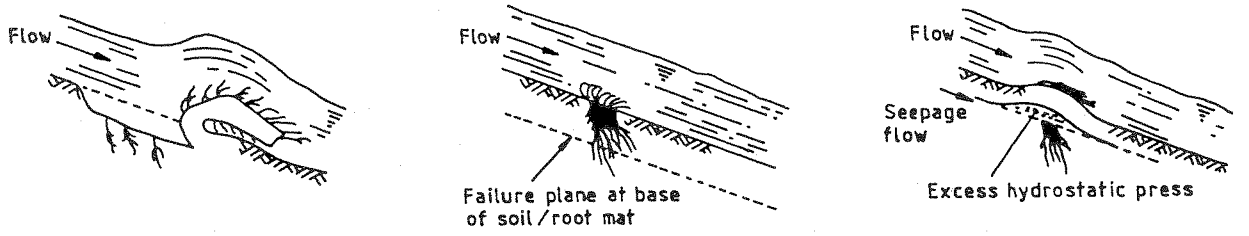
The cells are available in a wide range of sizes varying in height between 0,05 and 2 m.



(a) Removal of loose matter

(b) Scour of soil from roots

(c) Loss of individual plants



(d) 'Roll up' of soil/root mat

(e) Shallow slip

(f) Uplift from seepage flow

FIGURE 2.13 : EROSION MECHANICS FOR PLAIN GRASS WATERWAYS

The advantages of the system are :

- Flexibility : The mattresses are inherently flexible, they deform and bend without loss of strength and moulds readily to the contours of the surface to be covered.

- Permeability : Due to its open structure and subject to the type of fill material, it releases the build-up of hydrostatic pressure.

  They can be made reasonably impermeable by using an impervious fill or by grouting :

- Durability : Due to the flexibility of the cells its strength is not unduly affected by differential settlement or heaving.

- Versatility : Due to the range of fill materials, the system can effectively be applied in many different conditions. As the materials used in the construction are relatively light and easy to handle and use, the system lends itself to both machine and labour intensive construction.

An added advantage of the system is that it can effectively be used on fairly steep slopes i.e. up to slopes as steep as 1 : 1.

(i) Design

The usage and design procedures recommended by the suppliers should be followed to ensure reliable linings.

(ii) Materials and construction

The fill materials suitable for use in conjunction with the cells are topsoil, sand, mine tailings, soilcrete, ashcrete, plain or grouted stone, etc. depending on the intended application.

The procedures, methods and quality of work prescribed by the suppliers should be followed for the respective applications.

2.1.9 Dolosse

The dolos (plural dolosse) was invented by the then Harbour Engineer, East London, Mr E M Merrifield in 1963. It is one of the many concrete shapes developed by coastal engineers to replace large-sized natural rocks needed for the protection of harbour breakwaters and for coastal protection works. Due to the particular shape of these blocks, they are more stable than natural rock and the unit mass can, therefore, be considerably less, resulting in more economic designs. Extensive model testing by the CSIR and other overseas research laboratories showed the dolos to be more stable under wave attack than the other known units and, since the first dolos application on the East London breakwater, hundreds of thousands dolosse have been used in coastal works both in South Africa and in other countries worldwide.

Although the main application of dolosse is in harbour and beach projection works, dolosse have also found successful application in the protection against wave attack of embankments and banks built along the seashore, in estuaries and in large inland lakes where significant wave action can occur. Examples of such projects in South Africa include the Deal Party bank protection in Port Elizabeth and the protection of various bridge embankments on the Natal South Coast. Anyone of the available armour block shapes could be used for these works, but the high stability and thus the relatively small unit mass and the locally available dolos design and construction expertise make a dolos protection in South Africa an attractive and often most economical solution.

The information provided below will enable engineers to carry out a conceptual design of a dolos protection on which a realistic cost estimate can be based. For the final design, which may well require hydraulic model testing of the specific application, the designer should seek the advice of a coastal engineering expert. This is essential due to the many pitfalls in the design of a dolos structure and the potential misuse and resulting failure of such a structure as found with several major overseas projects (Zwamborn, 1979).

(i) Design

In order to design a dolos protection, the following basic design data are needed :

- Profiles of the bank/embankment above and below water

- Bottom contours in front of the structure (bathymetry) and possible bottom changes
- Sub-bottom information (foundation conditions)
- Water level and water level changes
- Wave data (direction, height and period)
- Wind data (direction, strength and duration)
- Current data (direction and strength)

In addition to the above, the function and the design life of the structure should be clearly defined, which affect :

- The recurrence interval of the design wave (normally between 25 and 100 years)

- Acceptable damage level and associated risk (0 to 5% storm damage is normally accepted depending on the ease of repair work)

- Degree of overtopping allowed, if at all. This is normally close to zero, if it is a strategic road, which effects the height of the protection.

Typical embankment and bank protection cross-sections are shown in Figure 2.14 and the basic dolos shape is given in Figure 2.15.

The design process covers the following aspects :

(a) Profiles

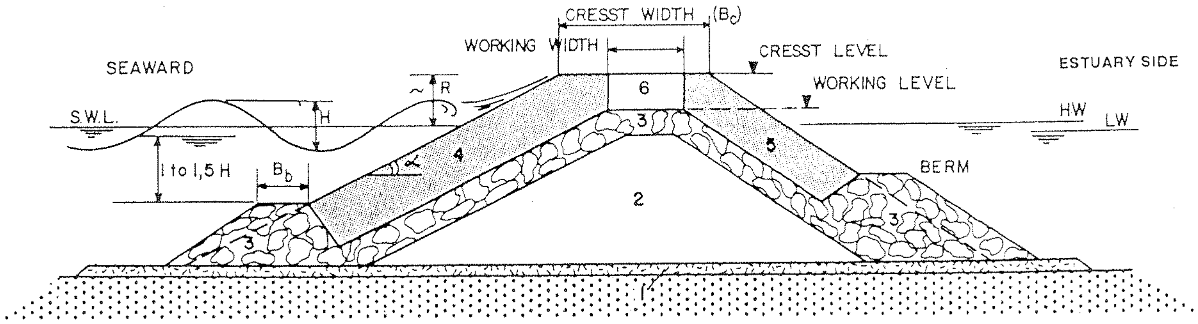
Above-water profiles can be obtained from conventional tacheometric measurements while underwater contours normally require an echo-sounding survey. The quality of the foundation material can be checked by coring (wash borings), which is of particular importance near the toe of the structure in the case of the breaking waves leading to possible scour.

(b) Design water levels and flow velocities

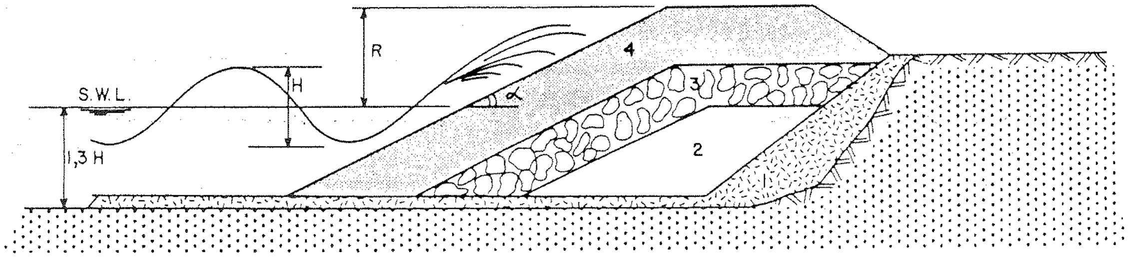
The stability of a dolos structure is dependent on the water depth directly in front of the structure.

The range of astronomical tides in the open sea around South Africa is about 2,0 m for spring tide, thus high spring tide is about 1 m above mean sea level (MSL). To this should be added 0,5 m for storm set-up to determine the maximum water depth in front of the structure. In estuaries and inland lakes the water levels will also be affected by flood discharges, the effects of which on water levels must be taken into account.

Tidal and river flow may cause high velocities along the bridge approach embankments, particularly around the ends at the abutments. These can be determined using the guidelines given in Sections 3.4, 7.1 and 7.2 of Volume I.



a. EMBANKMENT IN DEEP WATER, MODERATE OVERTOPPING



b. BANK PROTECTION WITH BREAKING WAVES

FIGURE 2.14 : TYPICAL CROSS-SECTIONS OF DOLOS EMBANKMENT AND BANK PROTECTION

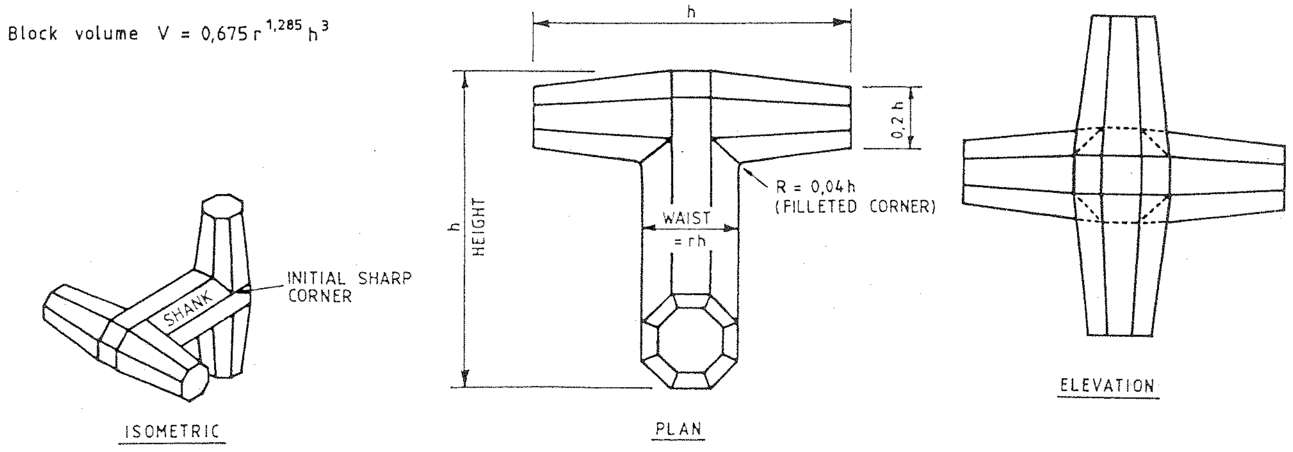

Block volume $V = 0{,}675\, r^{1{,}285} h^3$

FIGURE 2.15 : BASIC DOLOS SHAPE

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(c) Design waves

Since the stability of dolosse is related to the third power of the wave height, the determination of the correct design wave condition at the structure is extremely important.

Wave conditions along the South-African shore have been summarised in CSIR Report T/Sea 8401 (Rossouw, 1984) and through the CSIR's Waverider Network, are being continuously updated. The following offshore significant design wave heights, Hmo, given in Table 2.5, apply for the South African coastline (Rossouw, 1989):

Location	**Significant wave heights (m)**		
	Hmo_1^*	**Hmo_{10}^***	**Hmo_{100}^***
Oranjemund	6,2	7,6	9,0
Port Nolloth	6,3	7,7	9,2
Saldanha	7,0	8,7	10,3
Koeberg	6,7	8,4	10,0
Gouritsmond	7,0	8,5	10,1
Cape Recife	7,0	8,5	10,0
East London	4,9	6,0	7,0
Richard's Bay	≈ 6,0	≈ 7,0	≈ 7,8

* Significant wave heights with recurrence intervals of 1,10 or 100 years

TABLE 2.5 : SIGNIFICANT WAVE HEIGHTS FOR THE SOUTH AFRICAN COASTLINE

The above wave heights apply to relatively deep water of 20 to 100 m. Before using them in the design, they must be converted to nearshore wave heights in the area concerned. This wave transformation in shallow water is caused mainly by refraction and diffraction around headlands, generally resulting in a significant reduction in the wave heights reaching the structure. The designer, who is normally a coastal engineer, is referred to the well-known handbook CERC (1984) for details of the conversion of significant wave heights to nearshore wave heights. This process is also very dependent on the direction of wave approach. Approximate wave direction distributions are shown in Figure 2.16.

However, in the case of bridge approach embankments or bank protection, the design waves to be used will virtually always be depth limited, that is the depth of water causing wave breaking away from the structure, thereby limiting the wave height reaching the dolos armouring. As a first approximation the depth limited breaker height, H_b, can be found from :

$$H_b = 0,8\ d_b \qquad (2.5)$$

where

d_b = breaker depth in front of the structure (m)

Thus, in the case of depth limiting conditions, H_b becomes the design wave height. For the detailed design, the CERC method should be used.

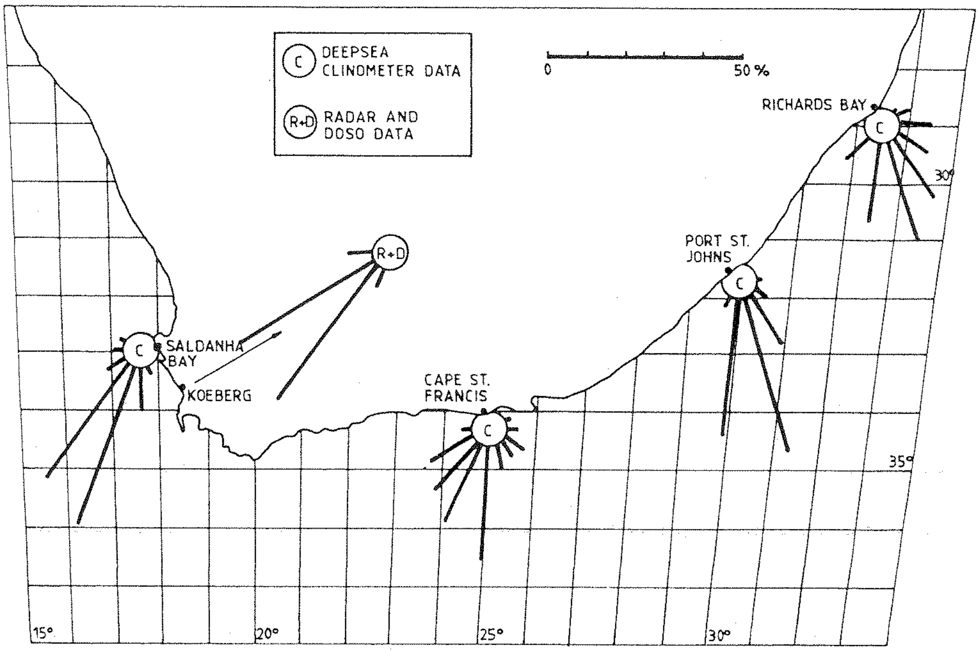


FIGURE 2.16 : DEEP SEA WAVE CLINOMETER WAVE DIRECTION DISTRIBUTIONS

(d) Design wind waves

In the case of estuaries or inland lakes, the design wave should be based on local wind data, which could be obtained from the Weather Bureau (1975) or the CSIR (1989). The latter data, collected in the South African ports, indicate that a safe maximum hourly average wind speed, with a 10 year return period, $U_{10} = 25$ m/s. Converting the accepted design wind speed U_{10} to the windstress factor U_A :

$$U_A = 0,71\ U_{10}^{1,23} \qquad (2.6)$$

Using the forecasting curves given in Section 7.1.3 of Volume I the wind generated design wave height can be found (CERC, 1984).

(e) Protection geometry

A protection with dolos armouring consists of the following main elements shown in Figure 2.14. The numbers shown below refer to the numbers used in Figure 2.14.

- 1. A filter layer 0,5 to 1,0 m thick consisting of suitable filter material such as a quarry run, quarry spalls, gravel and reinforced fabric. If scour at the toe of the structure is expected, allowance must be made for this in the filter design.

- **2.** Core material, normally a continuous graded quarry run. For example, if a 0 - 500 kg grading is to be used, the maximum percentage of fines must be specified. Alternatively, a 1 - 1 000 kg grading contains no fines.

- **3.** Well graded underlayer stone with a median weight, W_u

$$W_u \approx W/5 \tag{2.7}$$

where

W = weight of dolos (N)

The thickness of the underlayer, t_u is related to the nominal rocksize D_u

$$t_u = nCD_u \tag{2.8}$$
$$D_u = (W_u/\rho g)^{1/3}$$

where

n = number of layers i.e. $n = 2$ for a double layer
C = shape factor $\approx$ 1,0 (refer also Section 7.1.3 of Volume I)
ρ = mass density of rock (kg/m^3)

The berms can be made with the same underlayer rock and should have a minimum width B_b to allow for at least two rocks to be placed side by side :

$$B_b = 3\ C\ D_u\ (m) \tag{2.9}$$

- **4.** The design of the main armouring is given in Subsection (f) below. Its slope, α depends on the slope to be protected, which for dolosse is usually between 1:1.3 and 1:2 (vertical to horizontal). The main armouring should preferably reach from about H (wave height) below to H above the still water level (SWL) to allow for the wave run-up R. For breaking waves and no overtopping, a crest level of $R = 1{,}5\ H$ or more may be required, but this should be confirmed by model tests.

- **5.** The need for the lee-side armouring depends on the degree of overtopping and/or the expected lee-side wave attack, for instance due to wind generated waves and waves diffracting around the end of an embankment.

- **6.** The crest design will vary from case to case, depending on specific requirements. If maintenance is required, the design should make allowance for easy access. The crest width B_c should not be less than:

$$B_c \leq 3\ C\ D_u\ (m) \tag{2.10}$$

(f) Dolos armouring

The basic dolos shape is shown in Figure 2.15. All dimensions are expressed in relation to the height of the dolos h. Due to the potential for breakage of unreinforced large dolosse, the waist to height ratio r is increased for larger dolosse (Zwamborn and Beute, 1972) :

$$r = 0{,}34 \left[\frac{W}{20}\right]^{1/6} \qquad (2.11)$$

For the smaller units usually used for embankment/bank protection, i.e. with a mass equal to or smaller than 20 tons, $r = 0{,}34$ or even 0,33 is acceptable. Figure 2.15 shows filleted corners which were found to significantly reduce stress concentrations found in sharp corners of the original dolos design and these should be included in the design to reduce the potential for breakages. The concrete volume V of the dolos is determined from the equation :

$$V = 0{,}675 r^{1{,}285} h^3 = \frac{W}{\rho_s g} \quad \text{(Zwamborn, 1980)} \qquad (2.12)$$

where

V = volume of dolos (m^3)
r = waist to height ratio
h = height of dolos (m)
W = weight of dolos (N)
ρ_s = density of concrete (2,4 kg/m^3)

To determine the unit weight of the dolos armouring, the well-known Hudson formula can be used for the initial design (CERC, 1984) :

$$W = \frac{\rho_s g H^3}{K_D \Delta^3 \cot\alpha} \qquad (2.13)$$

where

W = weight of dolos (N)
ρ_w = mass density of water (kg/m^3)
ρ_s = mass density of concrete (kg/m^3)
Δ = relative density of dolos = $(\rho_s - \rho_w)/\rho_w$
H = wave height (m)
K_D = stability coefficient
α = embankment/bank slope with horizontal plane (degrees)

The Hudson formula greatly oversimplifies the dolos stability and the effects of many known and unknown factors are included in the stability coefficient K_D which must be determined experimentally. The following minimum K_D values (Table 2.6), which are based on extensive model tests by the CSIR (Holtzhausen and Zwamborn, 1990) and taking into account other published data (CERC, 1984), can be used for the conceptual design:

Dolos waist ratio (r)	K_D for expected damage	
	< 2% Displacement	< 2% Displacement plus rocking (total damage)
0,33	13	6
0,36	8	5
0,38	6	4

TABLE 2.6 : SUGGESTED K_D VALUES FOR USE IN DETERMINING ARMOUR UNIT WEIGHT

Normally 2% displacement (movement over a distance ≥ h or complete removal off the slope) can be accepted, but if maintenance is difficult or there is doubt about the quality of the dolosse, the design should rather be based on the 2% total damage (displacement plus rocking, causing possible breakage) criterion. The above K_D values are probably conservative and a more optimum design should be possible by consulting experts and employing project specific model tests.

The thickness t of the dolos armour layer is given by the equation :

$$t = n\,C\,V^{1/3}\ (\mathrm{m}) \qquad (2.14)$$

where

n = number of dolos layers (normally n = 2)
C = dolos shape factor which is related to the packing density
V = dolos volume (m^3)

The total number of dolosse N to cover an area A is found from :

$$N = AnC\left(1 - \frac{p}{100}\right)V^{-2/3} \qquad (2.15)$$

where

p = porosity of dolos armour layer (%)

The following values for C and p (Table 2.7) apply for dolosse with different r values (Holtzhausen and Zwamborn, 1990) :

Waist ratio (r)	0,32	0,36	0,38
C	1,18	1,09	1,05
p (%)	54,9	52,7	50,5

TABLE 2.7 : DOLOS DESIGN FACTORS

(g) Hydraulic model tests

Due to the many interacting factors, the detailed design of a dolos protection should be checked by experts and, where applicable, optimised using a hydraulic model. Model tests can account for the effects of different wave conditions (direction and energy spectra), water levels and current effects. Savings may be realized by optimizing the dolos size and the dolos packing densities, while the required height of the structure can also best be checked in a model.

The above K_D values basically apply to a straight embankment/bank protection and in the case of corners and the ends of embankments, model testing would be advisable to ensure an effective, yet economical, design.

(ii) Materials and construction

Dolosse derive their high stability from the specific anchor-type shape which results in a high degree of interlocking. However, unreinforced dolosse can break easily when dropped over a relative small height of 0,1 to 0,2 m on a solid base (Burcharth, 1980) or over 0,5 to 1,5 m in the breakwater situation (Zwamborn and Phelp, 1989). A broken dolos reverts to two (small) pieces of ineffective rubble. It is, therefore, essential to use a high quality concrete with a 28-day cube crushing strength of at least 40 MPa but, more important, with a flexural strength of at least 5 MPa and, preferably, a low elasticity (Youngs) modulus, to avoid breakages. The former should be regularly checked on site during construction with the beam tests.

The correct placing of dolosse is also of vital importance to achieve maximum stability. The number of dolosse N must be evenly spread over the surface to be covered according to a predetermined placing grid. The dolos units should be placed in a sequence to achieve maximum interlocking.

(iii) Maintenance

Even if a low-maintenance design approach is followed with a smaller than 2% total damage, maintenance may be necessary at some time in the future. However, to establish whether maintenance or repair is necessary, the condition of the structure must be surveyed, which can be done using one or more of the standard monitoring techniques (Kluger, 1988). These techniques include visual underwater inspections, photographic surveys by boat, crane or helicopter, tacheometric surveys and crane-and-ball surveys (underwater) as illustrated in Figure 2.17.

Although, in the past, regular maintenance was done on the breakwaters in several South African ports, it has been found to be more economical, both locally (Zwamborn et all, 1990) and overseas to allow a reasonable high degree of damage to develop, which is closely monitored and continuously evaluated in terms of risk of failure, before repair work is undertaken. This is in view of the special equipment required for these works, and thus the high mobilisation cost and the difficulties experienced in carrying out effective small repair works.

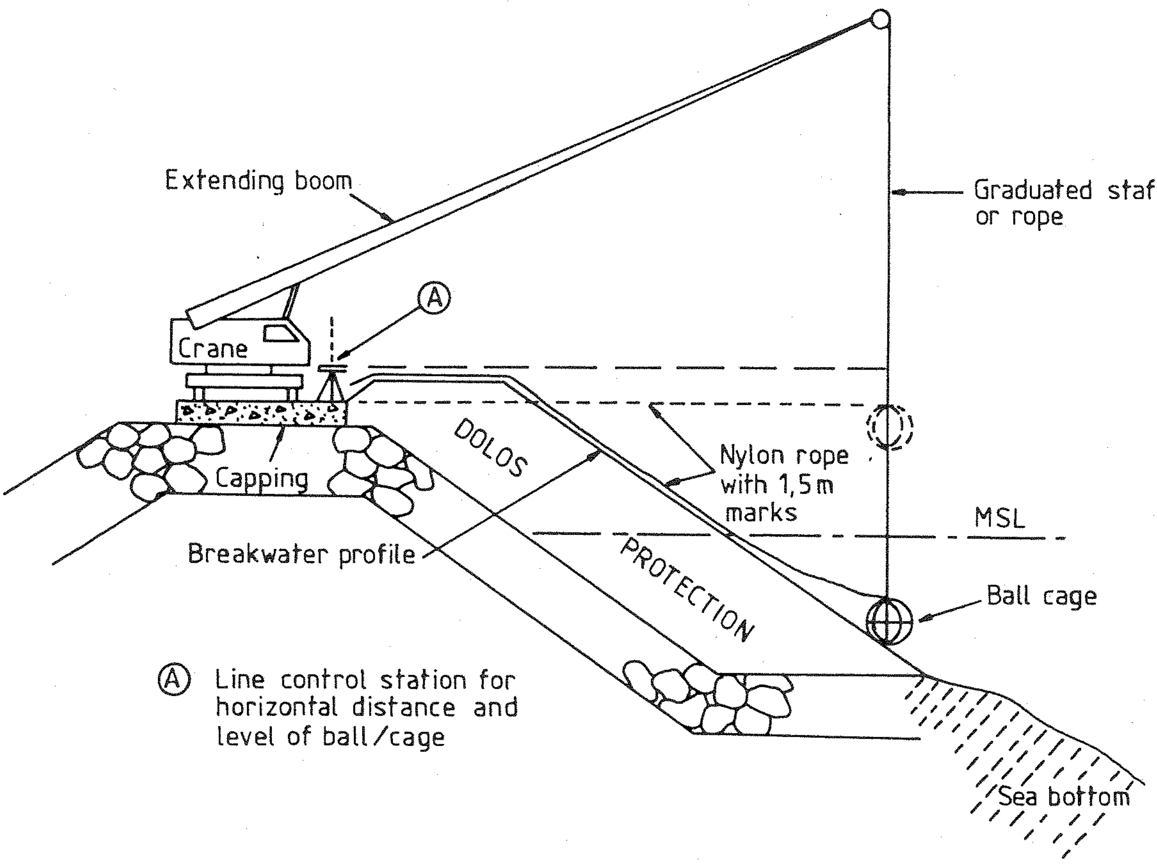


FIGURE 2.17 : METHOD OF CRANE-AND-BALL SURVEY

2.2 Overtopping protection

2.2.1 Introduction

This section deals with the protection of erodible embankments and banks for their safe overtopping under flood conditions using the systems described in Section 2.1. It is particularly relevant when dealing with low standard and low cost development on which minimum or no maintenance can be afforded.

The prerequisites of such systems are that they should be :

- Environmentally acceptable and attractive
- Substantially durable with low maintenance requirements
- Cost effective

Protection methods currently used and favoured are :

- Riprap
- Concrete block
- Natural vegetation (grass)
- Geotextiles
- Gabion mattresses
- Concrete, roller compacted concrete and soilcrete

2.2.2 Processes of erosion

It is necessary to understand the mechanics of erosion in order to evaluate the susceptibility of the unprotected road embankment to erosion, and to design protective measures accordingly. Failure through erosion of surface material by overtopping flow must be distinguished from failure by piping, by liquefaction or by mass sliding or slumping. They might, however, act in conjunction with the erosive forces created by high surface velocity in causing a failure. Failures due to piping and liquefaction require saturation of a relatively permeable soil. Mass sliding or slumping also require some significant degree of soil saturation and occurs particularly as flood flow recedes leaving the saturated slope in an unstable condition.

Various flow patterns have been observed as water flows over an embankment. These flow regimes are classified in Kindsvater (1964) as free-plunging flow, free surface flow, and submerged flow.

For the low-tailwater condition known as free flow, critical flow control occurs on the crest, and the discharge is determined by the upstream head. At higher tailwater levels, when the depth of flow over the crest is greater than the critical depth, the discharge is controlled by the tailwater as well as the headwater. Under conditions of tailwater control, the flow is said to be submerged. With a rising tailwater level, the change from free flow to submerged flow occurs rather abruptly. The flow pattern antecedent to the change is described as incipient submergence.

Free flow is subclassified into plunging flow and surface flow. Plunging flow occurs when the jet plunges under the tailwater surface, producing a submerged hydraulic jump on the downstream slope. Surface flow occurs when the jet separates from the crest surface at the downstream shoulder and "rides" over the tailwater surface. Whereas free flow can be either a plunging or a surface flow, submerged flow is always a surface flow.

Figure 2.18 shows the various flow regimes associated with road embankment overtopping.

The free-flow transition range is dictated by the range of tailwater levels within which a given discharge can produce either a plunging flow or a surface flow, depending on the antecedent conditions. With an initial low tailwater the plunging flow pattern will persist as the tailwater level rises until it reaches the upper limit of the transition range, whereupon the plunging flow changes abruptly to a surface flow. However, if the tailwater is initially high and the flow is a surface flow, this pattern persists as the tailwater drops until it reaches the lower limit of the transition range, whereupon the flow pattern changes abruptly to plunging flow.

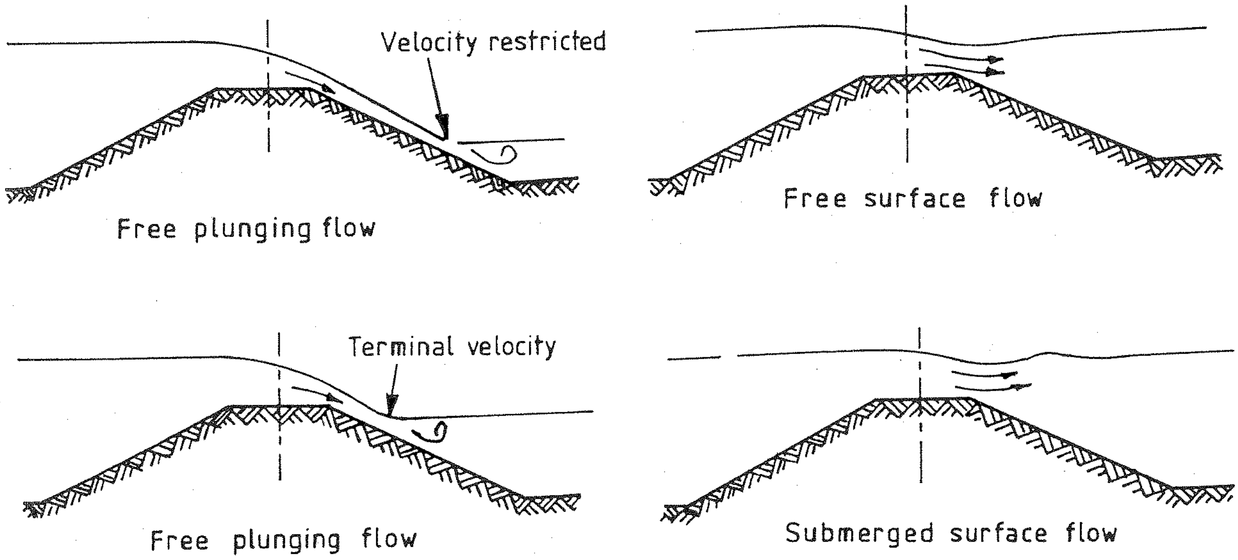


FIGURE 2.18 : DIFFERENT FLOW REGIMES

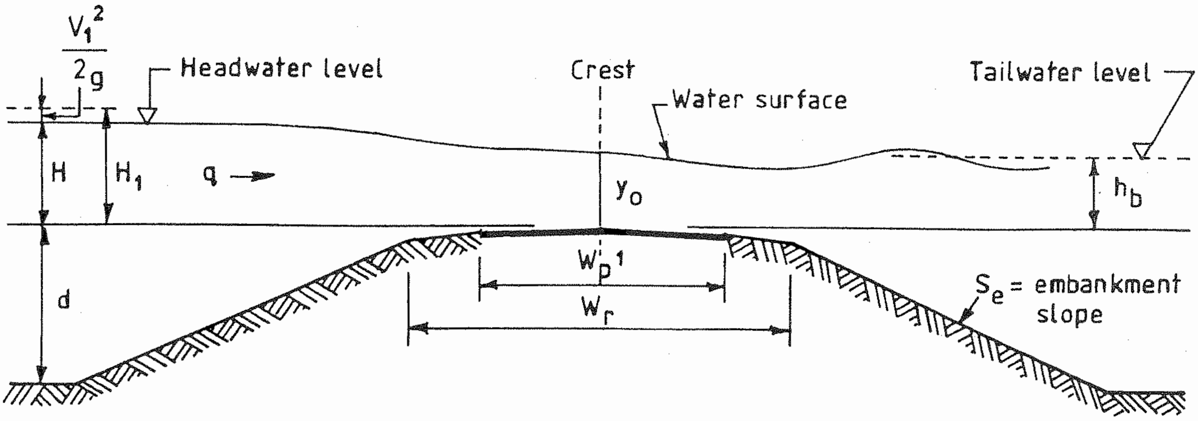
In order to determine the type and/or extent of embankment protection required for overtopping flow, it is important to identify the expected flow regime for a particular flow. This can be done by using Figure 2.19 which also gives the limits of the incipient submergence and free-flow transition ranges.

The maximum velocity attained on the downstream slope depends on the overtopping head, the hydraulic roughness of the downstream slope, the downstream slope and the head loss across the embankment. Experience has shown that surface flows are substantially less erosive than plunging flows.

A clear distinction is made between flow conditions over an embankment before and after the occurrence of localised surface discontinuities which signify the onset of failure due to high localised erosion.

Methods have been developed to estimate the rate of erosion of the embankment soil. The lowest erosion rates occur with highly cohesive soils. Low-cohesive soils show the highest erosion rates at low net shear stress, i.e. the difference between actual shear stress and critical shear stress. Non-cohesive soils, such as sand and gravel, share the highest erosion rates at high net shear stress.

It is considered undesirable to design high fills to be overtopped. The designs for overtopping are generally aimed at lower fill heights. The decision as to what constitutes a high or a low fill is left to the designer.



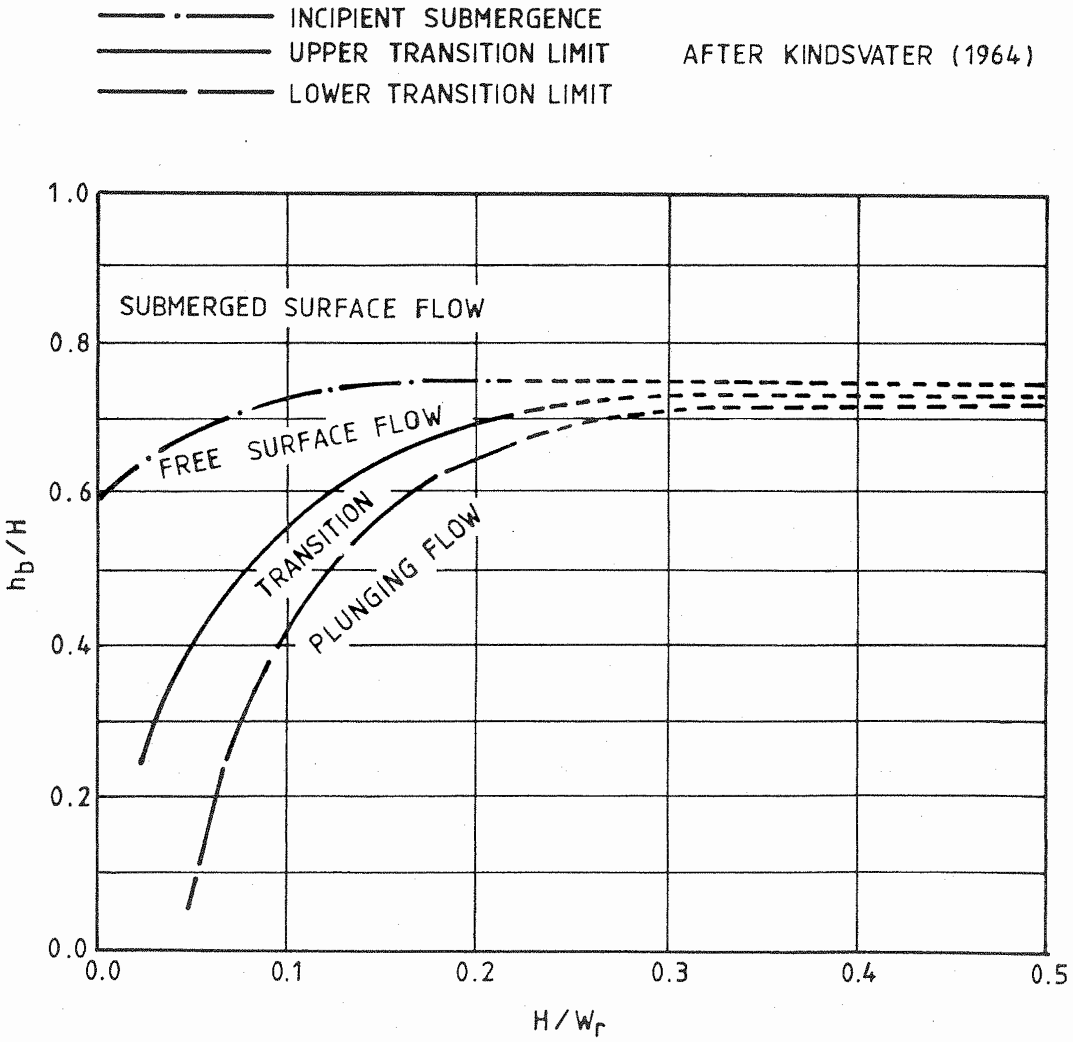


TABLE 2.19 : CLASSIFICATION OF FLOW PATTERNS

In the case of high embankments it is advisable to form a deliberate failure point in order to protect the drainage structure against damage during a major flood. This can be achieved by constructing a low point in the approach embankment using readily erodible fill material with the extremities of the failure zone usually protected with a rock fill. The relatively short length of wash out, so created, can be quickly reinstated with limited disruption to services.

2.2.3 Mechanisms of failure by erosion

(i) Cohesive soil embankments

At the onset of high localised erosion, a small overfall occurs at the point of a localised depression and a scour hole is initiated downstream of the overfall. The process of enlargement of the scour hole is dictated by the soil characteristics and flow pattern. The shape of the overfall increases the local flow concentration by capturing flow from the sides as well as upstream. Consequently, the scour hole is deepened forming a vertical face at the head until a critical height is reached, whereupon the vertical face collapses. Headcutting thus occurs due to the progressive upstream advance of the overfall through the embankment. Finally, breaching of the embankment occurs through loss of support due to erosion of the downstream face and not through direct erosion.

The following observations can be made on scour hole progression :

- The shear strength of the soil along the perimeter of the overfall determines the depth the overfall can attain before further head collapse occurs.
- The near half-rounded shape of the overfall provides vertical support to the soil mass through horizontal arching.
- Attaining critical height for head collapse depends on the removal of material from the scour hole by the flow. In this respect, submergence of the scour hole by a rising tailwater substantially reduces the rate of erosion.
- The rate of erosion can be accelerated by the presence in the embankment of any granular elements, such as toe drains or drainage blankets, which tend to undermine the more resistant cohesive material

(ii) Non-cohesive soil embankments

Non-cohesive soils do not allow localised scour to develop a vertical face, and surface erosion progresses more uniformly across the downstream face with a gradually flattening gradient which eventually reaches the crest. The presence of cohesive zones in the fill, such as a clay core, can retard the rate of erosion, typically by restricting further erosion of granular material until the cohesive element has been undermined and broken away by the flow.

(iii) Local geometry and other factors

Local discontinuities, such as changes in slope along the downstream crest breakpoint, provide a place for the initiation of erosion and lead to breaching of the embankment. Erosion has often been observed to be initiated at the road shoulder breakpoint and at the toe of the downstream slope. An even distribution of flow across the embankment is also less erosive than flow which becomes concentrated due to local variations in slope or low points in the crest. It is, therefore, imperative that road embankments designed for overtopping be graded horizontally.

(iv) Vegetal cover

Recent research by the United States Department of Agriculture (USDA) has been aimed at increasing the present permissible velocity neccessary for the stability design of grassed waterways. It also covers the computation of the extent of erosion damage and time to failure of grassed embankments for overtopping flow. Erosion failure of grass-lined embankments is classified in the following three categories:

- Local failure associated with major discontinuities in grass cover or boundary geometry.
- Direct physical destruction of the vegetal cover, such as grass plant or soil and root mat uprooting.
- Localised soil erosion leading to removal of the vegetal cover. Experience suggests that for a broad range of soil and grass conditions 60 mm can be taken as a realistic limit for erosion depth to signify complete failure of the protective grass lining.

2.2.4 Design

Current design practice relies strongly on engineering judgement, as there is no standard approach to the hydraulic design. Overtopping flow of embankments generally conforms to the principles applicable to flow in a steep channel (Bramley et al, 1987). Aspects of geotechnical stability are not dealt with here as it is regarded as an area where well established practices exist.

Bramley et al (1987) state that engineers should appreciate the advantages of protection being flexible. This enables the protection to accommodate minor movements due to settlement or to give early warning through its deformation of the incipient failure or loss of the underlying fill. The importance of giving careful attention to the design details of the protection at the crest and the downstream slope and toe is stressed, as these are key locations where erosion can be initiated and subsequent failure resulting.

The type of flow over the embankment as shown in Figure 2.18, is regarded as the basis for the design process, which would, inter alia, include consideration of potential seepage flow through the embankment. Since seepage through permeable embankments is naturally more pronounced than through relatively impermeable embankments, protection for the former requires more careful consideration to accommodate the seepage through-flow without adversily affecting the integrity of the protection layer.

Key hydraulic parameters which determine the type of protection are :

- Design unit discharge q over the crest

- Overtopping head and tailwater level in relation to the crest
- Design maximum shear stress or tractive force on the protection
- Design maximum velocity of overtopping flow
- Duration of the flow event

The generally accepted equation for computing discharge over an embankment for the free flow condition is :

$$q = CH_1^{3/2} \tag{2.15}$$

where

q = discharge per unit width ($m^3/s.m$)
C = coefficient of discharge for free flow
H_1 = total head above the embankment crest (m)

Alternatively the equation from "Hydraulics of Bridge Waterways" can be used, viz. :

$$Q = CLH_1^{3/2} \times \frac{Cs}{C} \tag{2.16}$$

where

Q = discharge (m^3/s)
L = length of inundated roadway (m)
C_s = coefficient of discharge for submergence

Where the depth of flow varies over the embankment crest, it is advisable to divide the inundated portions into sections of equal depth and compute the discharge over each reach separately.

The value of H_1 obtained in this equation is used in Figure 2.19 to determine the expected flow regime and, therefore, the extent and/or type of embankment protection required.

The physical processes governing the embankment erosion are closely related to flow-induced local velocity and effective shear stress on the embankment surface. However, all the current equations relate the discharge to the head and tailwater conditions and to date no equations have been developed to take into consideration the local velocity and shear stress variables over the embankment. These variables are very much non-uniform in this rapidly varied flow condition.

Examination of velocity data from research testing indicates that, for surface flow, the velocity over the downstream slope surface would be relatively constant and generally less than the depth-averaged velocity at the downstream crest breakpoint. Therefore, if the depth-average velocity was used, the actual embankment erosion would generally be less than the computed erosion. However, for plunging flow, the velocity over the downstream slope surface would generally be greater than the depth-averaged velocity and the converse would apply.

In the design of the protection, consideration should be given to various tailwater conditions which will occur over a range of discharges from low overtopping depth through to the design discharge. In this regard it should be noted that the higher the tailwater the less the risk of local damage. Erosion potential may be increased at the front of the hydraulic jump that occurs under free plunging flow and strong protection may be required in this zone.

As a general rule it is recommended that the protection on the downstream side be extended to the upstream side of the crest centreline. Also, the protection is generally continued downstream of the embankment to act as an armoured apron for energy dissipation. To save costs, the downstream protection can be terminated in a cut-off wall arrangement as a precaution against undercutting scour.

In developing methods for reducing or preventing embankment erosion during flood overflows, several protection systems have been identified and researched. The most promising protection system for preventing overflow erosion are those which incorporate modifications to the embankment surface as described in Table 2.8 by Powledge et al (1989), bearing in mind that most of the protection systems currently available on the market were designed for uses other than overtopping flows.

Protective system	Description
Riprap	Well-graded stone of a specified average size placed on embankment surface to a specified thickness
Concrete blocks	Specially shaped precast concrete blocks that are designed to interlock mechanically or that are tied together by cables that are run through the interior of the blocks
Vegetation	Grass established on embankment surface
Geotextile	Wide range of manufactured fabrics, mats, or larger-scale cells that can be incorporated within embankment surface before grass is established
Gabion mattresses	Uniformly graded stone contained in wire cells laid on the embankment surface
Concrete, roller compacted concrete and soilcrete	Cement and either imported aggregates or suitable in situ aggregate (soil)

TABLE 2.8 : EMBANKMENT PROTECTION SYSTEMS

Some use has been made of geotextile and concrete block systems, but these are still undergoing evaluation. The behaviour of each system is unique and has a different erosion resistance characteristic and mode of failure. The design of the proprietary systems are generally empirically based and may not necessarily result in a reliable solution under the particular conditions.

(i) Riprap protection

Riprap is currently not widely used as protection to embankments against overtopping flow. However, useful information on recent experimental studies and on present design practices is now available giving more confidence for its use against overtopping flow.

The flow of water over a layer of riprap produces steady and fluctuating forces of lift and drag, which, if they exceed the restoring forces due to gravity, will cause movement of individual rocks and possible failure of the protective layer may ensue. In order to overcome this, the ratio of flow depth to stone size needs to be kept small. In full-scale tests it was found that the unit discharge at which stones start to move is 73% - 79% of the unit discharge at failure q_f.

Various formulae for determining the sizes of stone that would be stable down embankment sides have been developed. Of these, the Safety Factors Method (SFM) is found to give conservative predictions for all slopes tested up to a maximum of 1:5, but are nontheless favoured.

$$D_{50} = 0{,}503 \times i^{0{,}43} \times q_f^{0{,}56} \qquad (2.17)$$

where

D_{50} = median size of angular stone (m)
i = embankment slope (m/m)
q_f = failure unit discharge (m^3/s.m)

Rounded stone are not recommended for use in the armoured layers.

It is recommended that the riprap layer have a thickness of 3 x D_{50}. By increasing the layer thickness, the stability of the protection, particularly in the case of smaller stone will be increased.

(ii) Concrete block protection

Although concrete blocks are used increasingly for river bank protection, their use in the protection against overtopping of embankments is still relatively rare. The available proprietary revetment block systems in South Africa include Armorflex and Terraforce G, neither of which have been tested for this particular application in South Africa. Consequently no design methods have been developed specifically for overtopping protection. In the case of the Armorflex system, careful consideration should be given to its uplift characteristics and the anchoring down of the armour layer with stakes, as well as the provision of edge beams.

(iii) Natural vegetation protection

Grass established on embankment surfaces is the most common form of slope protection against surface erosion. Although it is normally not intended to protect against overflow erosion, limited protection can be attained in this way. Research (Hewlett et al 1987) and overseas case histories show that embankment slopes constructed of cohesive materials and having a well managed dense grass cover can withstand overflow depths of up to 0,6 m and velocities limited to 2,1 m/s. However, for short duration flooding this velocity may be extended to 3,7 m/s.

Dense grass surfacing requires maintenance by fertilizing, and filling erosion spots.

(iv) Geotextiles

Geotextiles can be used effectively on embankment soils against erosion by establishing a dense grass cover in conjunction with it, whereby the geotextile is thoroughly integrated with the subsoil via a well developed root system.

Geotextile products that have been tested against overflow erosion are :

- Two-dimensional synthetic woven geotextiles
- Two-dimensional synthetic non-woven meshes
- Three-dimensional synthetic mats

A well designed system should be securely anchored, permit the establishment of grass and discourage subsurface flows, which can cause uplift through a build up of water pressure at the subsoil/geotextile interface. The three-dimensional open textured mats appear to offer suitable permeability and the best enhancement of the erosion resistance of natural grass. Geotextile systems installed with well established grass may withstand flow velocities up to 5,5 m/s as shown in Figure 2.11.

(v) Gabion mattresses

Gabion matresses, used as described in Section 2.1.4 in conjunction with a geotextile or graded filter bedding, can be used effectively on embankments against erosion from overtopping flow.

(vi) Concrete, roller compacted concrete and soilcrete

Construction suitable for overtopping protection includes conventional cast in situ concrete, roller compacted concrete and soilcrete construction. Sprayed concrete can also be considered when circumstances justify the associated high establishment cost of the special plant, equipment and specialized personnel to carry out the work.

Cast in situ concrete can be used as mass or reinforced concrete. In the lattere case it is usually in the form of a blanket slab constructed over the embankment. The concrete slab is generally 100 to 150 mm thick and reinforced with welded steel fabric or galvanized diamond mesh or similar fencing material. To ensure stability of the slab, it should be provided with a perimeter cut-off wall to safeguard it against undermining. Due to its lack of flexibility it is not generally favoured.

Roller compacted concrete is conventionally used for the construction of dam walls. However, the method can easily be adapted for the construction of an embankment acting as a weir to cope with expected overtopping discharge. If it is used to blanket an earth embankment, specialist advice shall be obtained to ensure that the correct construction method is followed.

Soilcrete, which is a soil cement mixture should be placed and compacted like ordinary fill material. This will require a mixture containing 3 to 5% cement, sufficiently watered for the mixture to be placed in layers and compacted with plate or roller compactor. The construction procedure to be followed should comply with the requirement of Section 2211 of the CSRA Standard Specifications for Road and Bridge Work. (1987). Thus, when used, it would replace the embankment or part thereof and not accept the form of a blanket over the earth embankment.

2.3 Vertical bank protection

2.3.1 Introduction

Banks which are steeper than the natural angle of repose of the soil, are ultimately liable to failure. All forms of vertical protection of soil banks should be designed to resist the erosive forces, the earth pressure and ground water pressure as appropriate. The earth pressure forces are generally much larger than the hydraulic loading due to wave action or currents, and consequently the latter are not accounted for in the normal design process.

Vertical bank protection comprises the construction of walls which can for convenience be classified into the following categories :

- Retaining walls
- Proprietary walls
- Piled and diaphragm walls

The type of wall best suited for a particular application is dictated by economic considerations, availability of materials and plant and the founding conditions. These criteria should be evaluated for each situation to decide on the most appropriate type of wall.

Of the above categories the piled and diaphragm type of walls are the most expensive to construct and would generally be considered where excessive scour and poor founding conditions are the dominant factors.

This volume does not cover the analysis and design of the respective walls, for which the reader should consult the appropriate reference works and text books.

2.3.2 Retaining walls

The retaining walls in the context of this document are subdivided into the following two types :

- Cantilever and counterfort walls, and
- Gravity walls

The former type of wall comprises reinforced concrete, mostly cast in situ, but occasionally also used in precast form. The latter type is predominantly constructed in unreinforced cast in situ concrete and to a lesser extent in precast concrete blocks or mortar brickwork.

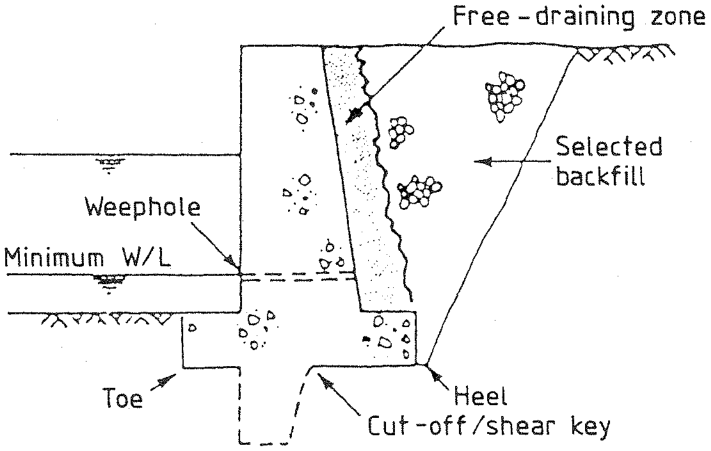
Notwithstanding the differences in the structural design of these walls, the acting forces and factors affecting their stability are the same.

For the purpose of this document only the factors affecting the forces and stability of the walls would be dealt with. However, the reader is advised to consult the appropriate specialized literature when designing walls.

(i) Factors affecting forces on walls

(a) Drainage provisions

Water pressures must be included in the forces acting on the wall, unless adequate drainage is provided. See Figure 2.20. For walls where drainage material is only provided on the back face of the wall with weep holes to relieve water pressure, hydrostatic water pressure must be accommodated with due allowance being made for seepage pressure.



Note:
Weepholes are essential features to reduce differential hydrostatic pressure across the wall. Locate at or below minimum external water level.

FIGURE 2.20 : DRAINAGE PROVISIONS

Hydrostatic pressures can contribute significantly to the forces to be resisted by a wall. Any measure taken to reduce the hydrostatic pressure should result in a more cost-efficient design.

The internal friction of the soil is also significant and any increase in this value will reduce the forces on the wall.

The essential feature of any method used to control the hydrostatic pressure on the rear of the wall is the provision of a free-draining material adjacent to the rear face of that wall. Granular fill is often used, but no-fines concrete, or a geosynthetic composite drain, may be used as an alternative. The free-draining layer must have adequate thickness and permeability to allow water to move vertically behind the wall without significant head loss, and consequent increase in differential hydrostatic pressure across the wall.

As with any competent backfill, use of granular fill at the rear of the wall is also advantageous where it replaces a weak material with a lower value of internal friction.

(b) Selection and use of backfill

The ideal backfill is a free draining granular material of high shearing strength. However the final choice of material should be based on the costs and availability, balanced against the desired properties.

In general the use of cohesive backfills is not recommended. Clays are subject to seasonal variations, swelling, and deterioration which all lead to an increase in the pressure on a wall. They are difficult to consolidate and long term settlement problems are considerably greater than with cohesionless materials. For cohesive backfills, special attention must be paid to the provision of drainage to prevent the build-up of water pressure. Free draining cohesionless materials do not require the same amount of attention in this respect.

The wall deflection required to produce the active state in cohesive materials may be up to 10 times greater than that for cohesionless materials (Ministry of Works New Zealand 1973). This, together with the generally lower values of shearing strengths for cohesive soils, results in the corresponding active earth pressures for a particular wall movement being higher for cohesive soils than for cohesionless soils.

(ii) Stability of walls

The possible modes of stability failures walls are subjected to are illustrated in Figure 2.21.

These are :

- Sliding due to lack of lateral restraint, usually obtained from either friction on the base or passive soil reaction.
- Excessive settlement movement due to low bearing capacity of foundation soil.
- Deep seated rotational failure of the surrounding soil mass, including the wall.
- Overturning due to inadequate, or poor disposition of, wall mass and/or soil restraining forces.
- Piping and loss of foundation material and toe support due to seepage flow leading to overturning. Loss of backfill through weepholes and cracks.
- Scour at the toe leading to overturning.

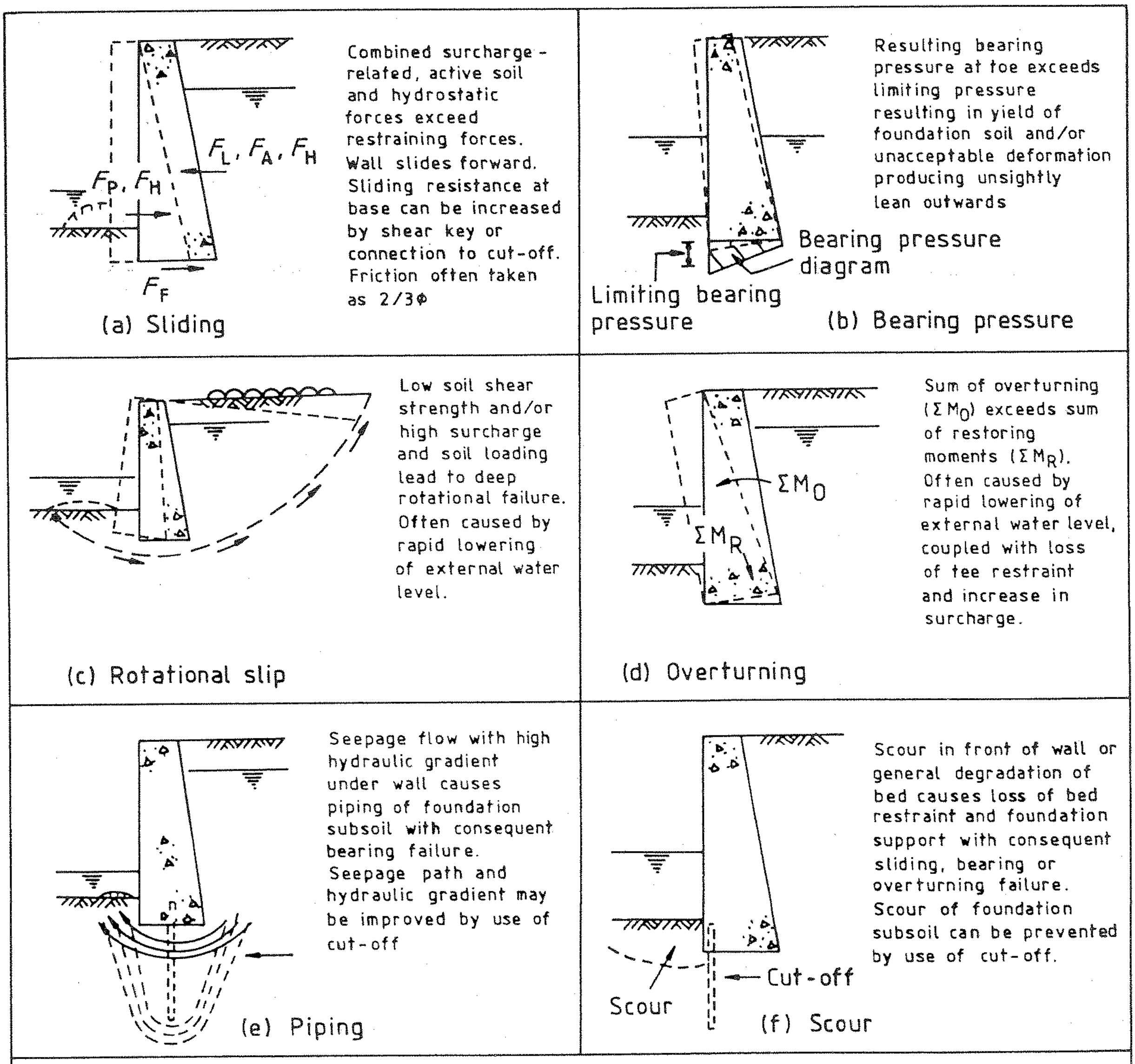
Of the above-mentioned modes of failure the main emphasis would be focused on scour i.e. protection work at the toe of the wall and on piping.

(a) Toe protection

The toe of the bank on which a structure is supported or alternatively the toe of the structure must be protected from undermining by scour which can be caused by stream flow, wave action or seepage. Toe protection can be achieved in two basic ways:

- By constructing a cut-off to withstand scour, i.e. extending the structure beyond the maximum estimated depth of erosion. This cut-off can also be used to increase the potential seepage path.

- By providing an armour skirt or apron of flexible protection across the bed from the toe of the bank. The method of protection should be able to adjust to scour and follow any bed erosion downwards, thereby continuing to render protection to the toe of the bank and/or the toe of the structure.



(a) Sliding

Combined surcharge-related, active soil and hydrostatic forces exceed restraining forces. Wall slides forward. Sliding resistance at base can be increased by shear key or connection to cut-off. Friction often taken as 2/3ϕ

(b) Bearing pressure

Resulting bearing pressure at toe exceeds limiting pressure resulting in yield of foundation soil and/or unacceptable deformation producing unsightly lean outwards

(c) Rotational slip

Low soil shear strength and/or high surcharge and soil loading lead to deep rotational failure. Often caused by rapid lowering of external water level.

(d) Overturning

Sum of overturning (ΣM_O) exceeds sum of restoring moments (ΣM_R). Often caused by rapid lowering of external water level, coupled with loss of toe restraint and increase in surcharge.

(e) Piping

Seepage flow with high hydraulic gradient under wall causes piping of foundation subsoil with consequent bearing failure. Seepage path and hydraulic gradient may be improved by use of cut-off

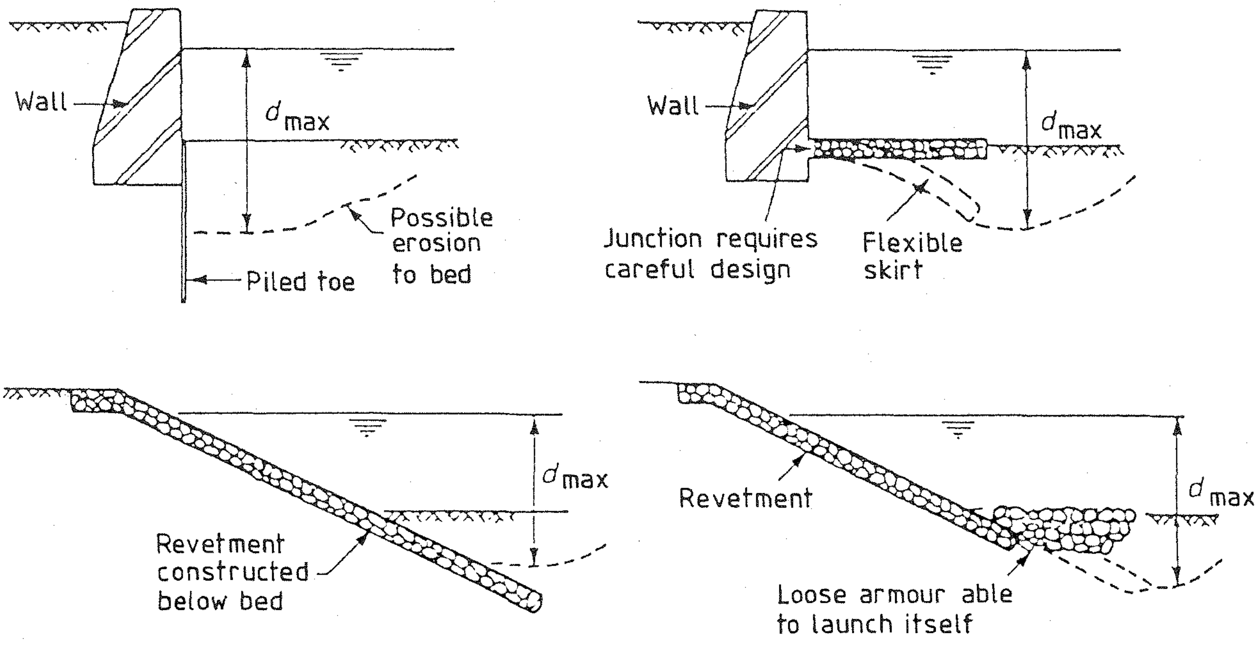
(f) Scour

Scour in front of wall or general degradation of bed causes loss of bed restraint and foundation support with consequent sliding, bearing or overturning failure. Scour of foundation subsoil can be prevented by use of cut-off.

With masonry, brickwork and mass concrete walls, it is normal practice to avoid tension in the rear face of the wall, except under extreme loading conditions. This implies that the resultant force in the wall remains within the middle third of the cross-section.

FIGURE 2.21 : MODES OF FAILURE OF CHANNEL WALLS

These methods are illustrated in Figure 2.22, which demonstrate the principles both for a wall and revetment. Flexible aprons perform best on cohesionless beds, where the underlying material scours and deforms to take up a relatively uniform slope. In cohesive beds, the process of scour does not provide a uniform slope and the structure should, therefore, be continued down to the maximum estimated scour depth.



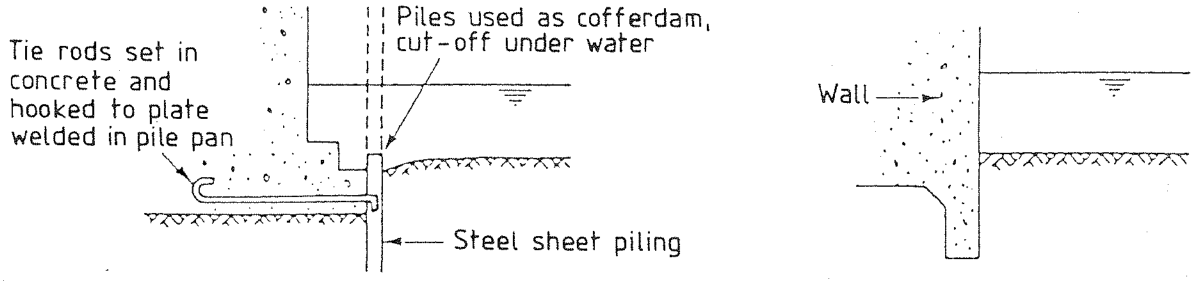
(a) Construction to withstand scour

(b) Construction to adjust to scour

FIGURE 2.22 : BASIC TYPES OF TOE PROTECTION. ESTIMATED SCOUR DEPTH, d_{max}

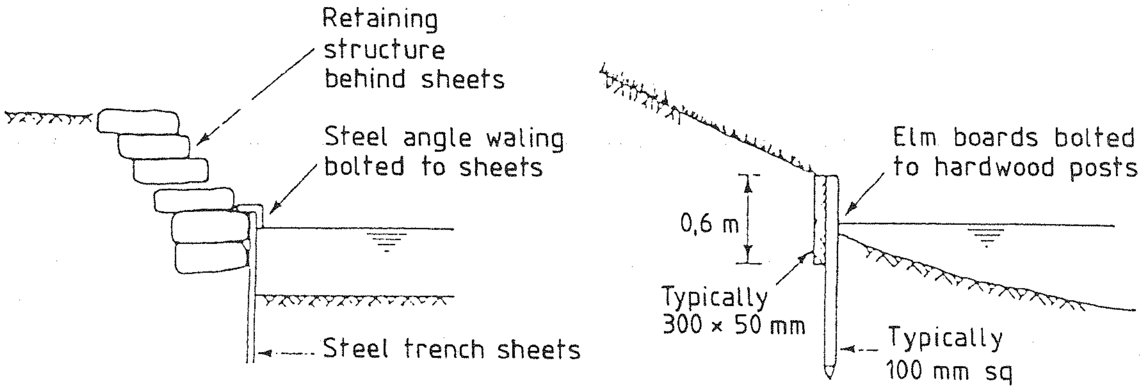
The depth of the cut-off must be such that it is taken an appropriate distance, generally at least 50%, beyond the maximum anticipated depth of scour below bed level, or alternatively to bedrock. The depth of scour has to be estimated on the basis of the bed material and flow pattern which pertain after the installation of the bank protection. If allowance has to be made for wave action rather than stream-flow velocity, then, as a rule of thumb, the allowance for scour should be no less than the maximum calculated unbroken wave height that can be supported on the normal water surface at the toe of the bank. Clearly, the scour due to wave action becomes less important as water depth increases.

The length of the armour skirt should also be based on estimated future scour and be sufficiently long to protect the residual slope as shown in Figures 2.22 and 2.23. In practice, a nominal length, allowing for at least 1 m of scour, is chosen. There are many different ways in which the toe can be constructed and detailed. Examples using different commonly available materials are shown in Figure 2.23.



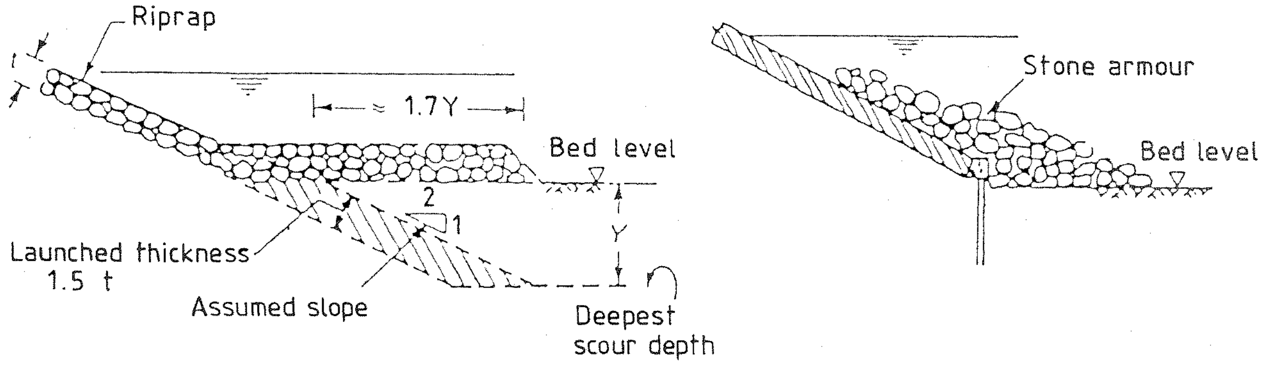
(a) Steel sheet piling to wall

(b) Extended toe



(c) Steel trench sheeting

(d) Timber posts and boards



(e) Launching apron for severe scour

(f) Protection to existing toe

FIGURE 2.23 : EXAMPLES OF TOE DETAILS

In the case of flexible toe protection, it can simply be continued across the bed if the bank protection is also flexible. The skirt should be completely flexible and adequately designed so as not to fail along the line of the toe.

(b) Piping

Due to seepage flow with a high hydraulic gradient through a permeable foundation under a structure, localized quick conditions are created and the liquified sand-water mixture moves through the surrounding subsoil as if it was flowing through a pipe. This erosion and consequential loss of founding material can ultimately lead to complete collapse of the structure (Capper & Cassie 1963).

Piping occurs particularly where beds or lenses of highly permeable sand or silt are surrounded by less permeable soils.

Very permeable material, such as coarse gravel, responds rapidly to water level changes. Consequently high pressure gradients do not occur, and piping is unlikely to present a problem. In the case of relatively impermeable material, the flow velocities will be very low and also not create a problem. However, with silty and sandy subsoils the risk becomes significant as they cannot respond quickly enough to avoid relatively high pressure gradients.

In designing structures that are likely to be affected by piping, the subsoil shall be carefully analized and the structure suitably modified to suit the conditions. The usual precaution is to lengthen the flow path by the provision of cut-off walls or the provision of filter blankets.

(iii) *Cantilever and counterfort walls*

Cantilever and counterfort walls are generally constructed in structural concrete of 25 MPa to 40 MPa strength, with the coarse aggregate varying between 19 and 38 mm in size.

Cast in situ walls should be built in the dry. The construction is normally dependant on the use of large equipment for moving the formwork panels and casting of the concrete.

Precast concrete walls, typically of the inverted T-section shape, are limited in size by the lifting equipment and are thus not used that often. The units are placed on a preshaped founding bed that has to be prepared in the dry, which in turn may require a cofferdam, dewatering and/or diversion of flow, depending on the circumstances.

(iv) Gravity walls

(a) Cast in situ concrete walls

Cast in situ concrete walls are relatively easy to construct and consequently are extensively used in rural areas.

The walls are generally constructed in mass concrete of strength varying between 15 MPa to 30 MPa. The aggregate size would normally be 38 mm, although plums between 15 kg and 55 kg with size varying between 150 mm to 500 mm may be used. Some mass concrete walls have light mesh to contain shrinkage cracking, while semi gravity walls have reinforcement on the backface, particularly at the footing/wall intersection or other construction joints.

The *restrictions to the construction* of the walls *are* the same as *for cantilever and* counterfort walls.

Mass walls are usually constructed with a batter on the rear face (Figure 2.20) for economy of construction. A nib or key may be formed integrally with the wall, if sheet piling is not used to this purpose. This will also assist in mobilising lateral restraint against sliding. Where the foundation soil is weak, toe and heel extensions are used to reduce the bearing pressure and spread the foundation load over a larger area.

(b) Precast concrete block walls

Precast concrete block walls comprise prefabricated blocks stacked on top of one another as shown in Figure 2.24. Although a firm and even foundation is required for this type of wall, it can tolerate an moderate amount of settlement by virtue of its flexibility.

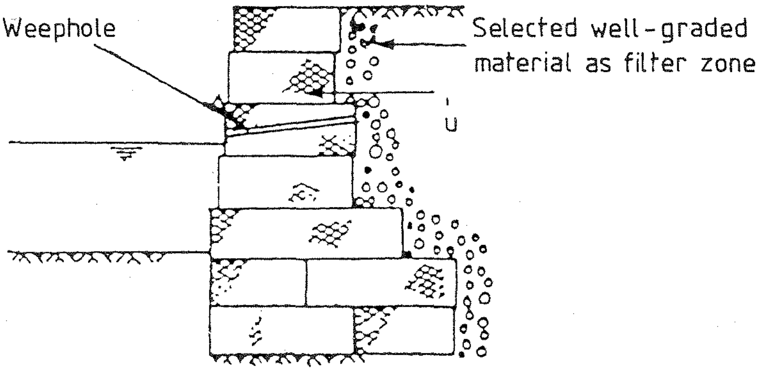


FIGURE 2.24 : PRECAST CONCRETE BLOCK WALL

The blocks are usually made out of mass concrete with reinforcing serving little purpose. In fact, where the concrete quality is poor and the cover to the reinforcement inadequate, reinforcement can be detrimental. The foundation levelling bed is usually made up from coarse gravel or weak blinding concrete.

Since there is no binding together of the blocks, stability analysis entails the checking against sliding and overturning at each block interface level. Resultant forces must fall inside the middle 1/3 of the wall at each level.

The sliding stability may be enhanced by sloping the foundation backwards. A batter of 1 : 10 is often used.

Although there may be gaps between the blocks, a full filter and weephole system should be employed to obviate differential hydrostatic heads as the gaps between the blocks may become sealed with fine grained soil.

Provided the founding material is firm and can be levelled out, this type of wall can be built under water with the aid of divers. The massive size of the precast concrete blocks dictates the use of heavy equipment for their installation.

(c) Brick or masonry walls with mortar joints

Situations may arise where aesthetic considerations demand a solution which is in keeping with the local environment. In such cases consideration should be given to the use of materials that blend in with the immediate environment.

Brick or masonry walls provide an aesthetically pleasing, though expensive, solution to river bank protection.

Engineering bricks, comprising dense and strong clay with defined absorption and strength limits, should be used for brickwork walls. The bricks should be resistant to sulphate attack, which is liable to occur under prolonged wet conditions, especially if the clay in the bricks contains sulphates.

Certain types of natural stone degrade rapidly on exposure and must be avoided in retaining walls. In many cases, the choice of stone will be dictated by availability and economic factors. In general, the strongest rocks, and those most resistant to weathering, are the most suitable for use. The lighter and more porous materials, which are more susceptible to weathering and degradation, should, if possible, be avoided in bank protection walls, as their use may result in an unacceptably short service life.

The durability of the mortar should be no less than that of the stone or brick used in the wall. For the type of walls considered here, a strong lime-free mortar mix, i.e. 1:3 cement/sand ratio, should be used. Such a rich mix will ensure increased resistance to sulphate attack, as well as minimize water absorption.

The design considerations for this type of wall are essentially the same as for the precast concrete block walls.

This type of wall should be built in the dry and the conditions previously described, apply in this case as well.

Failure of these walls is often caused by undermining of the foundations and, therefore, precautionary measures should be effected to counter this problem.

2.3.3 Proprietary walls

Proprietary walls include all the systems that have been patented and for which the primary products are available from selected suppliers. The most popular products are :

- Cellular containers
- Precast concrete blocks
- Soil-reinforced walls

These walls are special configurations of the gravity wall.

(i) Cellular containers

Cellular containers are manufactured from galvanized steel weldmesh or woven mesh. When the panels are assembled they form individual cells or a series of cells, as the case may be. These containers can be used with or without a liner, depending on the circumstances.

There are a number of proprietary systems available on the market, viz. Gabions, Concertainers, etc.

The advantages of this system as earth retaining structure and armour against erosion are :

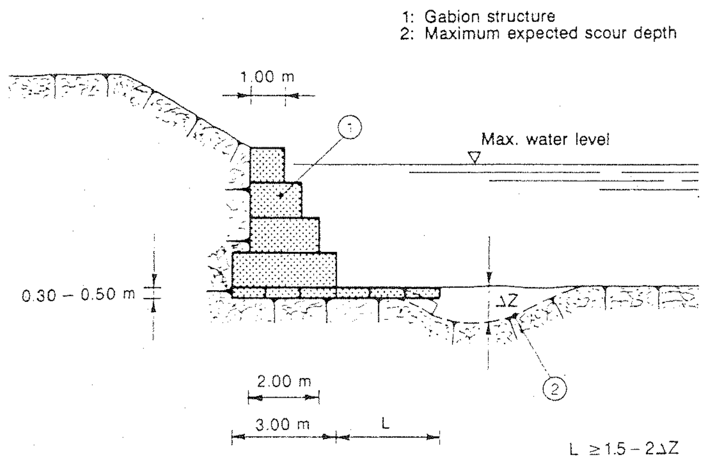
- Flexibility : The units are inherently flexible and tend to deform and bend, without loss of strength, to accommodate changes occurring in the subsoil.
- Permeability : The units are naturally permeable so that no provision for pressure relief need be made. However, Hemphill & Bramley (1989) comment that this permeability can permit excessive leaching of the fine material from the retained earth. A graded stone filter or a filter fabric is, therefore, required to retain the fines.

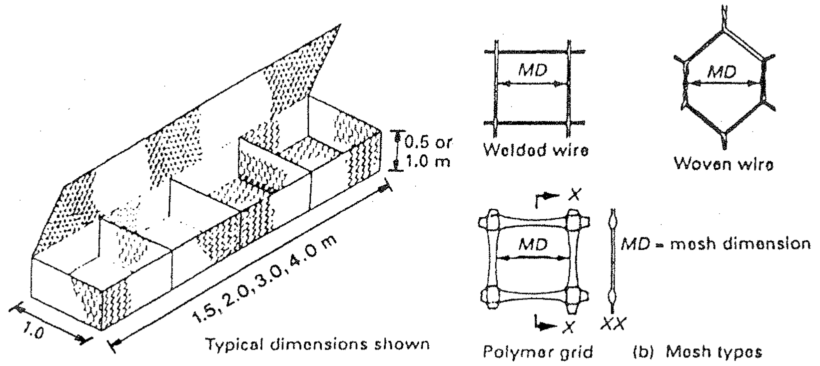
  They can be made reasonably impermeable by providing an impervious membrane, by grouting or by the injection of a mastic asphalt after filling. (Simpson 1991).
- Durability : Simpson (1991) states that a gabion is a heavy monolithic gravity unit able to withstand earth thrust. Its efficiency increases instead of decreasing with age, since progressive consolidation takes place as silt and soil collect in the voids and vegetation establishes itself.
- Versatility : Due to the range and availability of ballast materials and configuration options, the products can be effectively applied to many conditions.

The structure of the product lends itself to a gunite, plastered or grouted finish where pilferage or vandalism is likely. Permanent ballast such as stabilized earth, stone and concrete, can be topped up with fertilized topsoil and planted with vegetation. This would create an environmentally acceptable finish, while the root system would contribute towards stabilizing the structure.

When a stronger but still flexible structure is required and permeability not a factor, the cell can be grouted with a sand asphalt mastic. This modification combines the characteristics and performances of both materials. The cell retains its flexibility while the desity of the fill is increased and consequently the effectiveness of the protection. This protection is very suitable for revetments of the upstream facings of earth and rock fill embankments and banks, dikes and spillways.

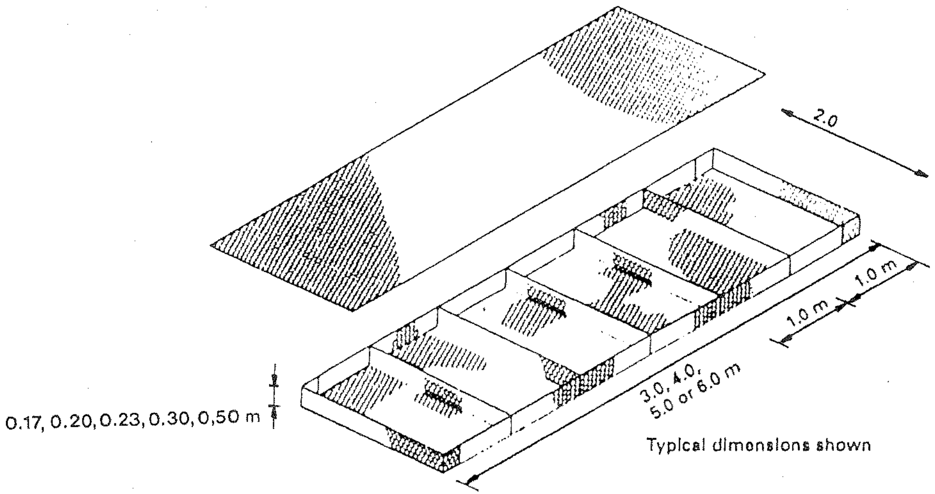
A typical detail of a gabion wall is shown in Figure 2.25. Except for dimensional differences the gabion mattress, described in Section 2.1.4, and the gabion box are the same product. Whereas the mattress is used exclusively as a lining, the box is used as an earth retaining member, with both having the same armour characteristics against erosion.





(a) Gabion box

(b) Mesh types



(c) Gabion mattress

FIGURE 2.25 : TYPICAL GABION DETAILS

CSRA Guidelines for the hydraulic design and maintenance of river crossings
Volume III, 1994, Committee of State Road Authorities, South Africa.

(a) Design

The stability analysis entails the checking against sliding and overturning at each block interface level. At each level the resultant force must fall inside the middle third of the wall.

The recommended parameters for the design of walls are :

- Density of granular fill - (1 800 - 2 000 kg/m^3)
- Density of rockfill - (1 600 - 2 000 kg/m^3)
- Resistance to Sliding - The ultimate friction factors for mass concrete on different foundation materials are also applicable to cells founded on the same materials.

Indicative unit weights of different types of rocks suitable for filling the cells are :

Type of rock	**Unit weight (kg/m^3)**
Basalt	2 900
Granite	2 600
Hard limestone	2 600
Sandstone	2 300
Soft limestone	2 200
Tuff	1 700

The unit mass of the cell can be determined from :

$$\rho_g = \rho_s (1 - n)$$

where

ρ_g = mass density of the cell (kg/m^3)
ρ_s = mass density of the stones (kg/m^3)
n = void ratio (n = 0,3 in most cases)

The design shall be carried out in compliance with the criteria recommended by the supplies and/or patent holder of the system concerned.

Where the cells are used for canal or river embankments, failure from a rapid draw down is most critical and, hence, the potential passive earth pressure in front of the structure should be ignored.

As these structures are normally mass structures, foundation pressures do not usually exceed the pressure under the natural embankment. Founding conditions can, where necessary, be improved by providing a concrete or gravel base in conjunction with a subsoil drain to remove entrapped water at the bottom of the base.

A common form of failure is inadequate toe protection leading to undermining of the structure. This can be avoided, either by providing a toe beam for small degrees of scour, or by extending a gabion mattress across part of the bed. The basic toe protection system, indicated in Figure 2.25, is a development of the gabion construction system. It is preferable to use the mattress as a base to the structure so as not to be dependant on manual tying of the mattress to the main structure.

It should be noted that, due to the rectangular prismatic format of the cells, they are not suitable for forming walls around sharp bends, especially if the wall comprises a multi-layered and rowed configuration. This is due to the overlapping bond between cells that cannot be maintained, especially when the cells in consecutive layers are stacked back from the previous layer below.

To prevent the fine material from behind the wall from leaching out and the formation of voids behind the cell, filters should be used. (Hemphill and Bramley 1989). While graded granular filters can be used effectively, the use of an appropriate geotextile fabric appears to be the favoured method.

When dealing with unconventional designs, it is recommended that expert advice be obtained.

In waters which have a pH value below 7 or above 11, PVC-coated wire should be used. Saline water and water polluted by many types of industrial waste or domestic sewage will corrode galvanized wire. In addition to the aggressiveness of the water, the location of the cell also has an effect on its serviceable life. Ferrous material subjected to alternative wetting and drying, as found in the splash zone, is extremely prone to corrosion (Hemphill and Bramley 1989).

Further cases where PVC coated galvanised mesh should be used, as recorded by Simpson (1991) include:

- Where the structure is in close proximity to the coast
- Where there is constant water velocity in excess of 2 m per second
- Where pure or soft water is present
- Where sulphate reducing bacteria are found

(b) Materials and construction

The materials to be used and the construction of gabions are well documented in Section 5200 "Gabions" of the CSRA Standard Specifications for Road and Bridge Works 1987.

The cells should be provided with diaphragms spaced at between 0,6 and 1,0 m intervals as appropriate.

The ballast may consist of sand, concrete, stone, gravel, mud or any other available materials that are suitable for the application.

The liners may comprise flexible fabric, mat or plastic film, metallic foil or laminate, geotextile fabric, fire retardant fabric, or any suitable product that would retain the ballast in the cell.

Where the exposed surfaces require additional protection, it can be achieved with sprayed concrete (gunite) or grouting.

When an asphaltic mastic mix is used for grouting, the following requirement must be met. The mix should comprise by weight approximately 66-73% river or crushed sand 0-3 mm, 12-16% asphaltic or calcareous filler (cement) and 15-18% bitumen. The mastic grount is mixed at 160-180°C at an asphalt plant, transported to the site in suitable vehicles and poured directly on the cell at 150-170°C.

(c) Maintenance

The integrity of cells is to a large extent enhanced by vegetation and the spreading of roots within the structure. With good root penetration a reasonable protection remains, even after the wires have corroded through. However, the growth of trees should be discouraged as the route system will ultimately disrupt the structure, unless the cell is intended to serve a temporary function only (Hemphill and Bramley 1989).

If at all possible, the cells should be protected against veld fires.

(ii) Precast concrete blocks

Gravity walls may be constructed from various shaped precast concrete hollow blocks which can be dry stacked together and backfilled with soil. In some specific instances certain block types are filled with concrete.

This type of wall can easily be shaped into complex curves in plan, as well as have continuously varying upper and lower profiles. The structure is relatively flexible and insensitive to foundation movements and, in most instances, is free draining. Whilst some blocks are built in an open lattice formation, others have interlocking qualities and are specifically designed for waterway bank protection. Standard design charts are available for the more common variety of proprietary walls. The charts allow for varying slope of wall and backfill, but do not provide for superimposed loads within a distance equal to the wall height behind the wall. Additional care should be exercised in the design of walls for heights greater than 2,5 metres.

Typical configurations of proprietary precast concrete walls are shown in Figure 2.26. The main advantages of this type of wall are the low cost, speed of construction and the construction adaptability. The system is very versatile and changes can be made at will to suit the site conditions.

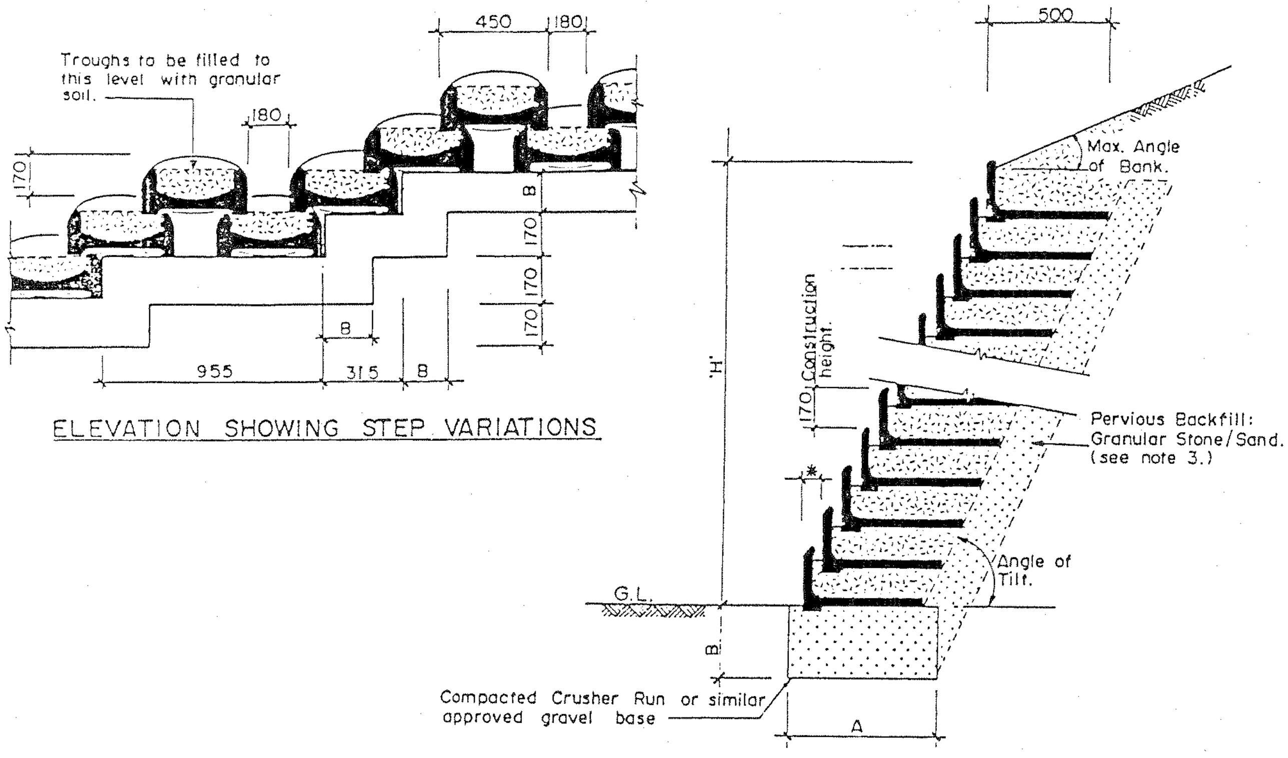
An added advantage is that in most cases the blocks are light and because they are stacked in the dry the walls can be erected speedily with unskilled and semi-skilled labour.

(a) Design

Special design procedures, methods and parameters are available from the developers of the respective products and these should be adhered to. However, the designer should control the stability of the structure by applying fundamental geotechnical principles to ensure that an adequate margin of safety against ultimate failure is achieved.

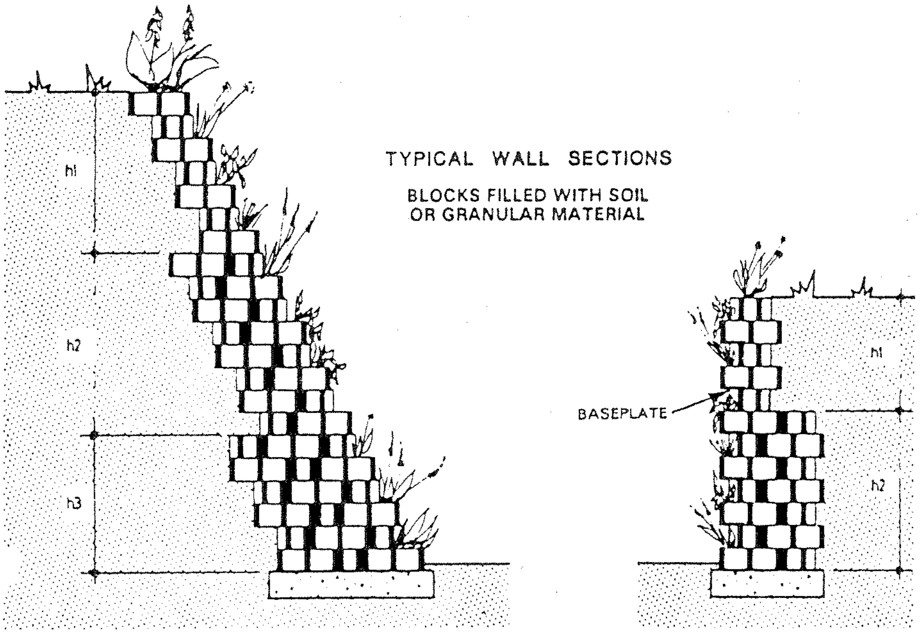
(b) Materials and construction

The precast concrete blocks are generally of unique design in accordance with relevant patent and are generally made from unreinforced concrete.



ELEVATION SHOWING STEP VARIATIONS

FOUNDATION TYPE 1



TYPICAL WALL SECTIONS

BLOCKS FILLED WITH SOIL OR GRANULAR MATERIAL

HEAVY DUTY RETAINING WALL

VERTICAL RETAINING WALL

FIGURE 2.26 : TYPICAL PROPRIETARY WALLS

Cast in situ concrete used in conjunction with the blocks is normally of 20 MPa strength.

The materials used and the method and quality of construction shall be strictly in accordance with those specified by the supplier and/or patent holder of the system to ensure that the guarantees are not negated.

Construction should be carried out under dry conditions which may require the construction of a cofferdam, the diversions of the water course and/or dewatering as appropriate.

(iii) Soil-reinforced walls

Soil-reinforced walls is the generic term categorizing an earth retaining structure in which a finite segment of earth behind the cladded exposed face of the wall is reinforced/mechanically stabilized by horizontal reinforcing strips.

The interaction of the reinforcing strips with the surrounding compacted soil produces an apparent cohesion in the direction of the strip and permits the fill to function as a homogeneous gravity structure.

The cladding on the exposed face is either connected to the reinforcing strips or integrated therewith. The cladding panels are normally of precast concrete, prefabricated steel panels, synthetic fabric or steel mesh, depending on the system.

There are a number of proprietary systems available on the market, viz. Reinforced Earth, Freyssisol, etc.

A typical example of this system is illustrated in Figure 2.27.

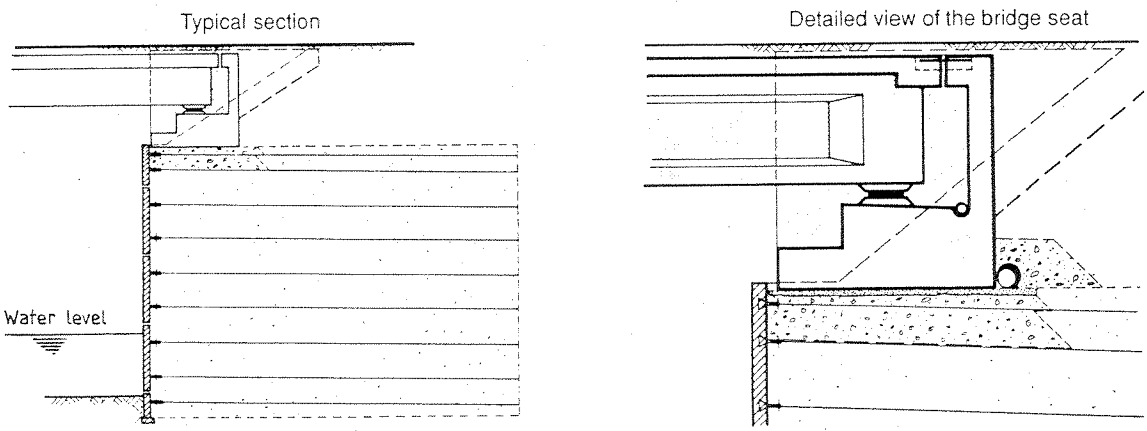


FIGURE 2.27 : TYPICAL SOIL-REINFORCED EXAMPLES

The cost advantage of a soil-reinforced structure over alternative retaining structures for wall heights in excess of 4 m often ranges between 20 and 50%. This advantage becomes even greater when poor founding conditions are present. An added advantage of the structure is that it can withstand a moderate degree of differential foundation settlement without loss of structural effectiveness.

(a) Design

For the design of these structures the conditions given in Section 2.3.3(ii)(a) apply mutatis mutandis. However, in most cases the design and detailing are undertaken by the supplier. For further information refer to Transportation Research Record 1242 "Innovative Earth-Retaining System".

(b) Materials and construction

The materials to be used and the construction of the proprietary system "Reinforced Earth" are well documented in Section 7200 "Reinforced Earth" of the CSRA Standard Specifications for Road and Bridge Works 1987.

For all other systems, the materials used and the method and quality of construction shall be strictly in accordance with those specified by the supplier and/or patent holder of the system.

The reinforcing strips are normally galvanized steel or synthetic fabric. Stainless steel is not recommended for this purpose. As the integrity of the structure is directly dependent on the reliability of the reinforcing strips, great care should be exercised to select a material compatible with the soil and aggressiveness of the environment.

Construction should be carried out under dry conditions which may require the construction of a cofferdam, the diversion of the watercourse and/or dewatering as appropriate.

2.3.4 Piled and diaphragm walls

These systems comprise the construction of vertical walls without the need to construct cofferdams, the diversion of the watercourse or to dewater the area. The systems can be used in restricted areas where other forms of walls may not be feasible. Unfortunately the systems are usually avoided because of their relatively high costs.

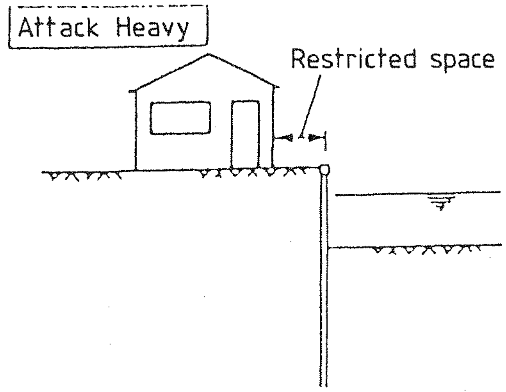
The most common systems in use are :

- Sheet piling
- Contiguous pile walls
- Diaphragm walls

Where the flexibility of a wall created with these systems becomes critical, it can be overcome by introducing horizontal ties comprising ground anchors or deadman anchors as shown in Figure 2.28.

(i) Sheet piling

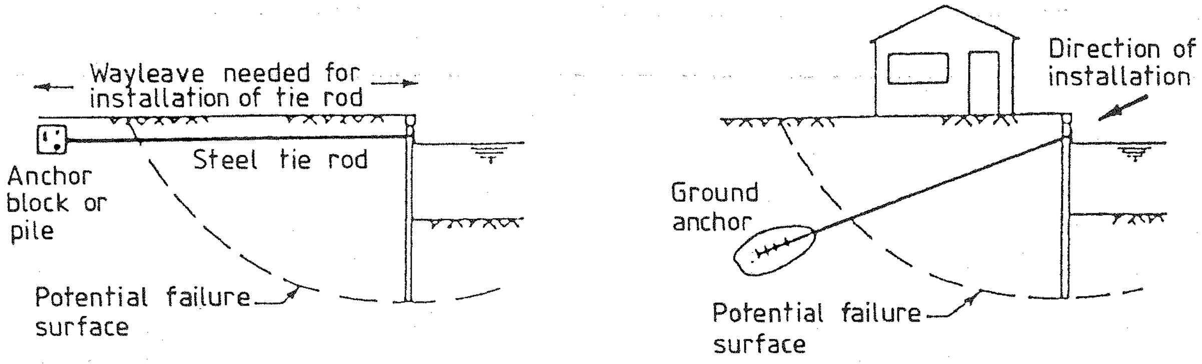
The most commonly used sheet pile is a patented interlocking steel profile. However, this is not a prerequisite for sheet pile walls and, depending on circumstances, other rolled steel sections can be used.



Notes:

1. Cantilevered wall relatively uneconomical in relation to anchored walls for retained heights in excess of about 3m
2. Restricted space for installation of tie rod may necessitate use of cantilevered wall
3. Anchorage must lie outside potential zone of failure

(a) Cantilevered wall



(b) Anchored walls

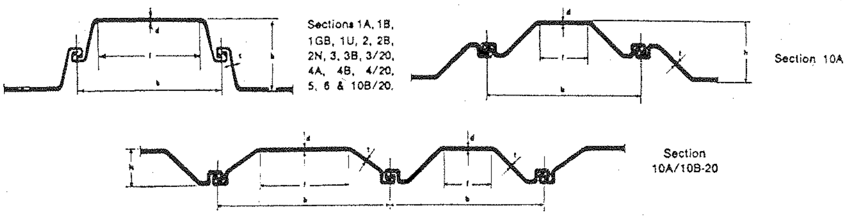
FIGURE 2.28 : TYPES OF PILED WALL CONSTRUCTION, CANTILEVERED AND ANCHORED

Under moderate corrosive conditions COR-TEN or WR-grade steel sheet piles can be used as permanent protection. Coated steel sheet piles are sometimes used, but as damage to the coating is unavoidable, they are not favoured as permanent protection.

As an alternative to steel, prefabricated concrete sheet piles are also used. See Figure 2.29. Reinforced concrete sheet piles are mainly used for marine and river structures by virtue of their lower cost and better resistance to corrosion.

(a) Design

Hemphill and Bramley (1989) record that unprotected steel piling in fresh water is likely to have an effective life of at least 60 years. The area of piling most prone to corrosion is that zone which is subject to alternate wetting and drying in the splash zone, allowing virtually unlimited exposure to water and oxygen. The effective life may be lengthened by reducing the stresses in the steel, which may be achieved by increasing the pile section, and thereby providing sacrificial thickness of steel for corrosion prior to the critical section thickness being reached. A rate of corrosion for ordinary steel of 0,05 mm/year is



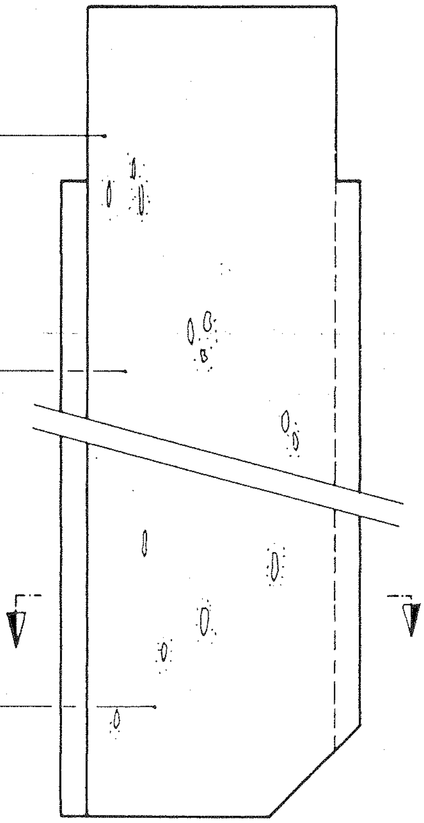
Section Through Typical Steel Sheet Piles Showing Interlocking Qualities

Typical Section Through Concrete Sheet Pile

Pile head is formed rectangular so as to accommodate driving helmet.

Typical section shown above.

Toe of sheet pile chamfered on one side to assist pile being driven to remain interlocked with previously driven pile.



Side elevation

FIGURE 2.29 : TYPICAL SHEET PILES

sometimes assumed as a rule of thumb for corrosion in river conditions, giving a reduction in thickness of 5mm for a 100-year service life. Consideration should always be given to the likely long-term water quality when assessing the corrosion risk.

The structural design of sheet piling is covered in numerous text books which should be consulted.

In the design, due consideration should be given to the potential scour depth at the toe, and the piling length specified accordingly to ensure stability under this condition.

(b) Materials and construction

Steel sheet piles are available in lengths of up to 25 m and are imported into this country, which tends to make them expensive.

Concrete sheet piles can be reinforced or prestressed, and are typically 600 x 250 mm in cross section with an overall length of 9 to 12 m. The piles are generally shaped to ensure interlocking between piles. The joints between piles can be grouted to improve the watertightness of the wall.

The presence of obstructions do create problems for the installation of sheet piles.

(ii) Contiguous pile walls

A contiguous pile wall comprises a row of soldier piles installed so that each pile is in contact or near contact with piles on either side of it. This technique is better suited to the softer collapsing soil conditions and provides a positive solution where design considerations dictate a conservative approach. Due to the likelihood of the fines leaching out through the gaps between the piles, the wall is not a watertight section, unless special precautions are taken to overcome this problem.

Under the correct circumstances, this type of wall presents an excellent and economical solution.

As the piles are cast in situ, a contiguous pile wall can only be built by forming the piles in the ground and excavating the canal afterwards. Therefore, they cannot be used for lining an existing canal without extensive temporary works.

(a) Design

The design principles for contiguous pile walls are similar to those for sheet pile walls.

(b) Materials and construction

The materials used in the construction are the same as used for cast in situ reinforced concrete work, except that the concrete mixes may be varied to suit the system.

The type of pile used for forming a contiguous pile wall can be any of the cast in situ systems, whether driven, augered or bored. However, the augered and bored systems are preferred and, unless there is a particular reason for wanting a driven pile, augered or bored piles should be used. A straight auger technique is normally used in non-saturated non-collapsing soils. Auger underslurry, augercast or temporary cased bored/auger piles are preferred in saturated collapsing soils.

Whichever pile technique is used, the installation sequence should be to first install odd-numbered piles, and when these have set, to proceed with the even-numbered piles. The spacing between the odd-numbered piles has to be carefully calculated so as to allow unobstructed installation of the even-numbered piles, whilst at the same time limiting the gap between piles.

To improve the watertightness of a contiguous pile wall the following techniques can be followed :

- To form a hole in the gap between piles with a rotary drill which is then grouted up.
- By installing the even-numbered piles that are cast in a cement/bentonite mix, back of the centre line. The odd-numbered piles are then installed, such that they cut into the cement/bentonite columns and thus creating a seal.
- Contiguous piles can be formed in a secant wall manner where even-numbered piles cut a secant into the odd-numbered piles, thereby creating a seal.

However, as installation of piles have a vertical tolerance of generally between 1 : 50 to 1 : 100, adjacent piles can be significantly out of line at moderate depths which could render the wall susceptible to scour damage.

Figure 2.30, diagrammatically illustrates the techniques described above.

(iii) Diaphragm walls

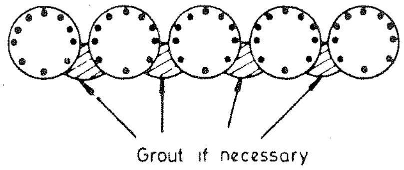
A diaphragm wall is formed by mechanically excavating a deep narrow trench in the ground under a bentonite slurry which replaces the removed soil. In turn, the bentonite slurry is displaced by the cast in situ concrete when forming the plain or reinforced wall.

This method is dependant on the thixotropic properties of the bentonite slurry which stabilizes the sides of the trench under its hydrostatic head. The slurry penetrates the soil pores for a certain distance from the excavated face, wherein it forms a gel, which makes the previously cohesionless soil cohesive and practically impervious. Simultaneously with this saturation, a skin of lining of gelled bentonite is formed at the soil/slurry interface which gives the excavated face stability.

(a) Design

Walls are generally designed to span vertically with discontinuity in horizontal reinforcement between panels. To achieve continuity involves considerable cost and intricate site work.

Contiguous Piles
Bored, Augered or Driven cast-in-situ piles installed adjacent to one another.



Secant Piles
Special case of a contiguous pile wall where the adjacent piles intersect one another to form a seal.
Applicable only to Bored or Augered piles.

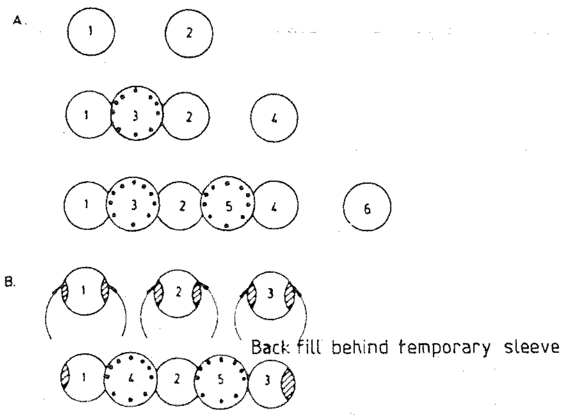
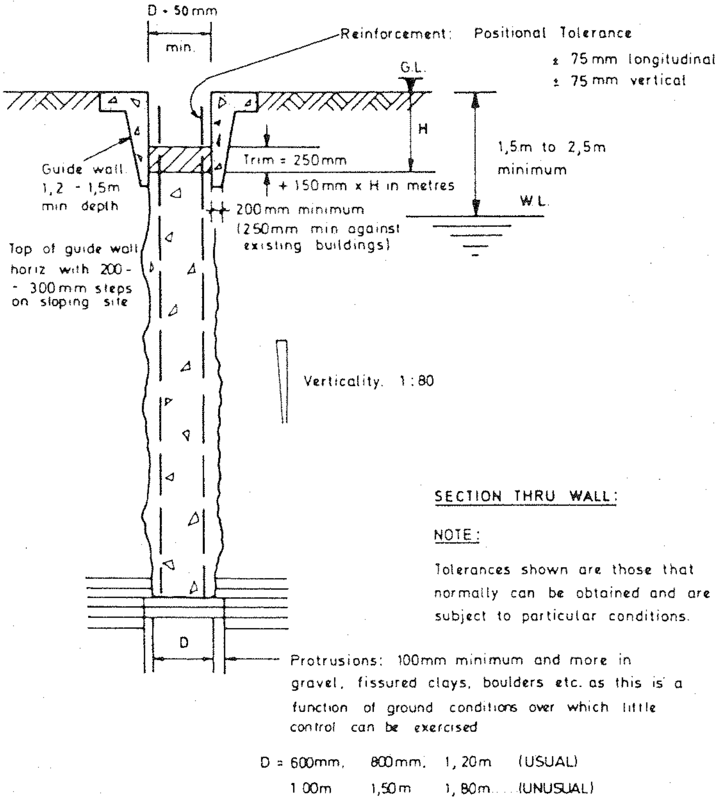


FIGURE 2.30 : CONTIGUOUS PILE WALL

The forces acting on diaphragm walls are similar to those of sheet pile walls and thus the same principles should be followed in their design.

The width of the wall depends on the design requirement and is determined by the width of the excavating grab. The diaphragm thicknesses commonly vary between 500 mm and 1,2 m, but diaphragms of 1,8 m width have been constructed successfully. (Frankipile 1986).

The length of panel excavated is also limited by physical considerations, such as the weight of the reinforcing cage and placing by tremie of concrete in a limited time. Lengths of individual panels and of any configuration of panel shape in plan are further controlled by the length of grab (2,2 m) and the necessity to follow certain digging sequences to ensure optimum accuracy in alignment of the wall. Refer to Figures 2.31 and 2.32 for typical dimensions.



SECTION THRU WALL:

NOTE:

Tolerances shown are those that normally can be obtained and are subject to particular conditions.

D = 600mm, 800mm, 1,20m (USUAL)
1 00m 1,50m 1,80m (UNUSUAL)

FIGURE 2.31 : SECTION THROUGH DIAPHRAGM WALL

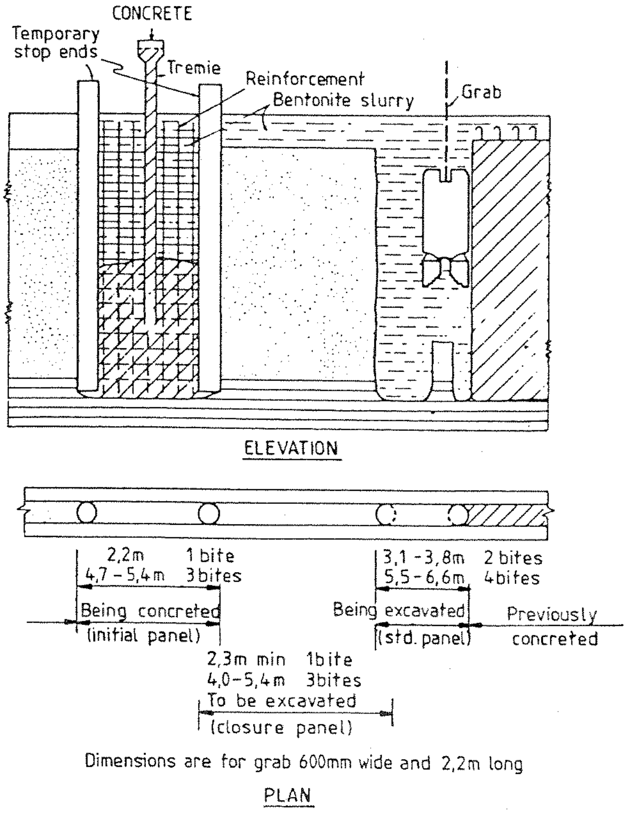


FIGURE 2.32 : SEQUENCE OF CONSTRUCTION

(b) Materials and construction

The bentonite slurry which is the key to this construction technique, comprises a mixture of bentonite powder and water. The bentonite powder which is a type of clay is supplied in bags from commercial sources.

Frankipile (1986) note that careful control should be exercised over the viscosity, density, sand content, "cracking" qualities and pH level of the bentonite slurry to ensure that the excavated particles are held in suspension, that hydrostatic pressure is provided, and that the thixotropic qualities of the bentonite slurry do not break down.

The concrete used in the construction shall be designed for placing by tremie under slurry. Normally an over-sanded pump mix concrete is used with 19 mm stones, slumps varying between 150 to 200 mm and a water/cement ratio not exceeding 0,60.

The cover over the reinforcing steel should preferably be not less than 75 mm.

For proper penetration of the slurry into the pores of the soil on the excavated faces, the hydrostatic head of the bentonite should be between 1,5 and 2,5 metres in excess of the ground water head.

When the trench is subjected to surcharge loading, it is necessary to restrict the length of panel excavated or to increase the density of the slurry, before commencing excavation of the trench.

If the depth and thickness of wall are not too large, precast concrete panels may be placed and grouted into position.

The recommended sequence of construction is illustrated in Figure 2.32 below.

When founding the wall, it is recommended that chiselling into sound rock be avoided. Instead, the wall should be cast on the rock and provision be made for subsequent drilling and grouting in of splice bars between the rock and the toe of the concrete wall, as shown in Figure 2.33.

Since diaphragm walls can only be built underground, they must be built either behind an existing wall or structure or inside a cofferdam.

As in the case with piling, the construction of diaphragm walls is of a very specialized nature and should be undertaken by specialist contractors only.

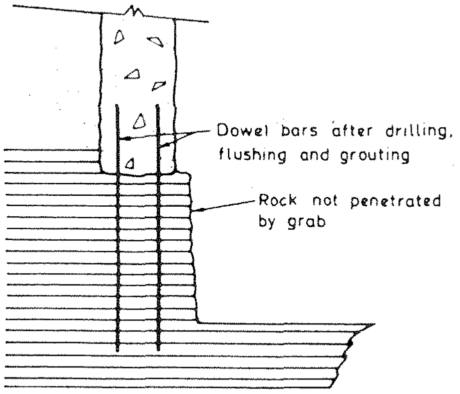


FIGURE 2.33 : TYPICAL SECTION ON ROCK

2.3.5 Transitions

A gradual transition between the natural or sloped embankment and the vertical protection should be provided to guard against local scour associated with sudden changes in the flow pattern. This protection should normally extend into a zone where the unprotected channel or embankment is not subjected to erosion.

Within the transition zones a system of protection should be selected that would provide intermediate hydraulic roughness ensuring a gradual transition in the velocity gradient.

In addition to the transition, it is good practice to turn the end of the vertical protection into the embankment to prevent erosion taking place behind it.

In Figure 2.34 examples are given of typical transitions between vertical walls and the natural channel. These also include solutions where it is economically unavoidable to provide a gradual transition from a vertical to sloping bank.

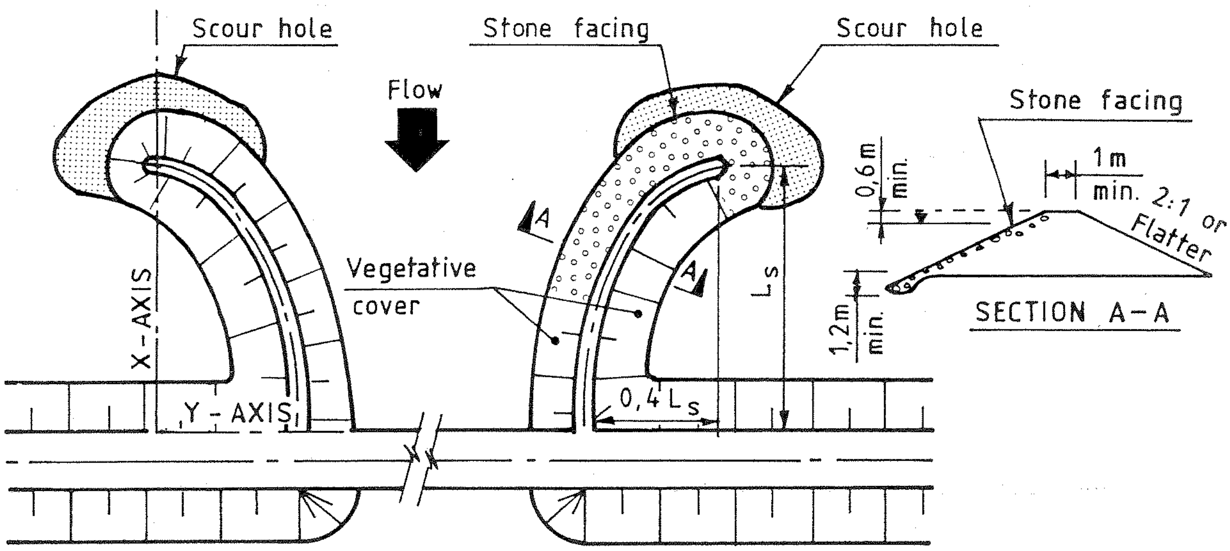


FIGURE 3.1 : TYPICAL GUIDE BANK

3.2.1 Design

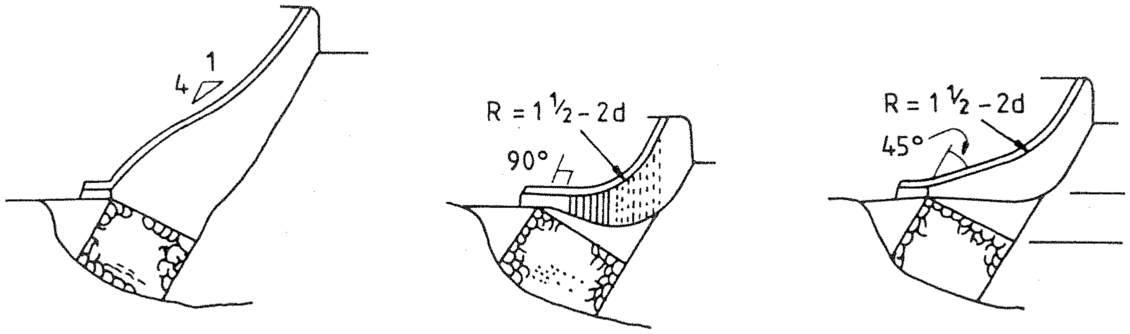
The primary factors dictating the functioning of guide banks are their orientation to the direction of flow, alignment with the bridge opening, plan layout, upstream length, cross-sectional shape, crest elevation and protection measures.

(i) Orientation, alignment and layout

In the design of guide banks preference should be given to orientate the guide banks at the junction with the bridge parallel to the flood flow direction and to align them with the abutments of the bridge so that the distance between the banks at the bridge is equal to the bridge opening.

Research shows that best results are obtained by using guide banks with a plan layout in the form of a quarter of an ellipse (Bradley 1970). This layout allows for a transitional constriction of the flow. For design purposes the length of the guide bank, measured perpendicularly from the approach embankment to the upstream nose, is denoted by L_s. The amount of expansion of each guide bank, i.e. the maximum offset, measured from the abutment parallel to the approach embankment, is 0,4 L_s. The layout can be determined by using Equation 3.1, which is the equation of an ellipse with its origin at the nose of the guide bank. The orientation of X and Y is shown in Figure 3.1.

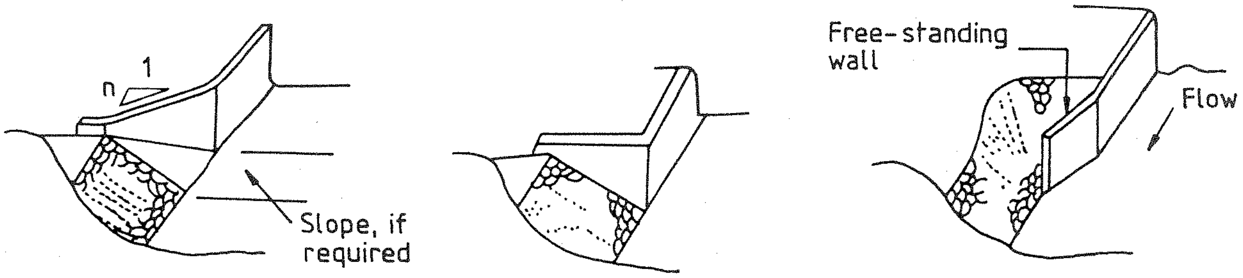
$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0{,}4\ L_s)^2} = 1 \qquad (3.1)$$



(a) Warped (gradual)

(b) Cylinder - quadrant

(c) Broken - back (with curvature)



(d) Broken - back

(e) Square - ended

(f) "Onion" outlet

FIGURE 2.34 : EXAMPLES OF TRANSITIONS BETWEEN VERTICAL AND SLOPING BANKS

3. INDIRECT PROTECTION

3.1 Introduction

Indirect protection consists of those methods introduced to control the flow paths and velocities in water courses to render protection or minimize scour to the channel. Some effective methods comprise :

- Guide banks
- Jetty fields
- Spurs
- Check dams

With these methods the objective is not to retard the flow velocity, but to reduce the velocity gradient to a minimum.

3.2 Guide banks*

The predominant cause of scour at bridges is due to the turbulences created at the supports by poor flow alignment towards the substructure. The resultant flow direction at a particular location from perennial low flows through to major floods may vary considerably due to the influences of the flood plains and bends in the water course.

Where bridge approach embankments encroach on to a flood plain, the flows from these areas are deviated from the natural course to flow parallel to the embankment towards the bridge opening. At the abutment the severity of the contraction is increased by this returning flow which tends to reduce the effective bridge opening. This flow can erode the embankment and increases the severity of scour at the abutment.

Guide banks can be used effectively in these cases to exclude the flow adjacent to the embankment and to guide the stream flow through the bridge opening and thereby prevent erosion of the approach embankment and scour at the abutments. These embankments are generally constructed contiguous to the upstream side of the structure, such that they align parallel to the river flood flow as shown in Figure 3.1.

A significant aspect of the guide bank, illustrated in Figure 3.1, is the transfer of the scour hole, which normally occurs at the abutments of the bridge, to the nose of the guide bank.

Deflection guide banks are another type that are generally placed strategically at some upstream positions from the bridge crossing where they are designed to deflect the main river flow in a desired direction.

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* In some publications these are described as "spur dykes".

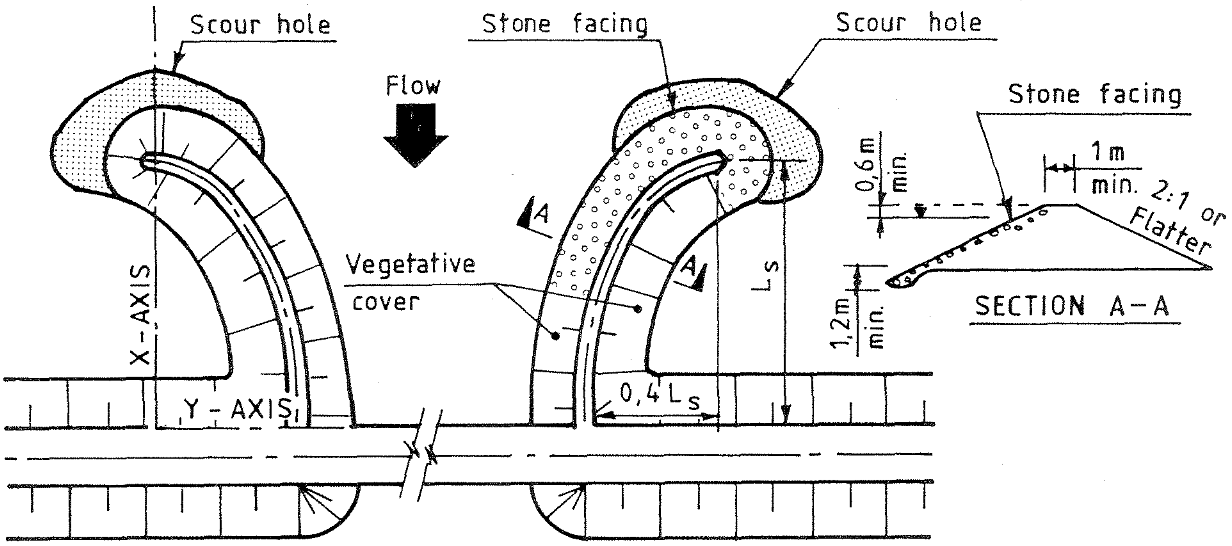

FIGURE 3.1 : TYPICAL GUIDE BANK

3.2.1 Design

The primary factors dictating the functioning of guide banks are their orientation to the direction of flow, alignment with the bridge opening, plan layout, upstream length, cross-sectional shape, crest elevation and protection measures.

(i) Orientation, alignment and layout

In the design of guide banks preference should be given to orientate the guide banks at the junction with the bridge parallel to the flood flow direction and to align them with the abutments of the bridge so that the distance between the banks at the bridge is equal to the bridge opening.

Research shows that best results are obtained by using guide banks with a plan layout in the form of a quarter of an ellipse (Bradley 1970). This layout allows for a transitional constriction of the flow. For design purposes the length of the guide bank, measured perpendicularly from the approach embankment to the upstream nose, is denoted by L_s. The amount of expansion of each guide bank, i.e. the maximum offset, measured from the abutment parallel to the approach embankment, is 0,4 L_s. The layout can be determined by using Equation 3.1, which is the equation of an ellipse with its origin at the nose of the guide bank. The orientation of X and Y is shown in Figure 3.1.

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0,4\,L_s)^2} = 1 \qquad (3.1)$$

or L_s = $(X^2 + 6{,}25Y^2)^{1/2}$

(ii) Length

The length of the guide bank L_s can be determined using a nomograph. For design purposes the utilization of the nomograph, which is presented in Figure 3.2, involves the following parameters:

Q = total design discharge of the stream (m^3/s)
Q_f = design discharge over flood plain (one side) measured just upstream of bridge crossing (m^3/s).
Q_{30} = design discharge in 30 m width of main channel measured adjacent to the abutment (m^3/s)
b = width of bridge opening measured between abutment faces (m)
A_{n2} = cross-sectional flow area below normal stage through the bridge opening (m^2)
$V_{n2} = \frac{Q}{A_{n2}}$ = average velocity through the bridge opening (m/s)
Q_f/Q_{30} = guide bank discharge ratio, which relates to the flow over one side of the flood plain to a specific portion of the flow under the bridge.

The nomograph should be used to determine the guide bank lengths L_s for banks greater than 15 m and less than 75 m. It is recommended that, if the nomograph indicates a length required to be greater than 75 m, the design should be limited to 75 m. Similarly, for guide banks less than 15 m, no guide bank is recommended. For bridges skewed at an angle of 45°, Bradley (1970) recommends that the forward guide bank be lengthened to 1,5 times the value given for L_s in the nomograph. For lesser skew angles ϕ, this forward guide bank may be lengthened in proportion, i.e. by $(\phi/45°)\ 0{,}5\ L_s$.

No specific recommendations exist for the length and layout of downstream guide banks. Where such guide banks are required, engineering judgement should be used to determine their orientation, alignment and layout. Due care should be exercised to ensure a gradual velocity gradient.

(iii) Cross-section and crest elevation

The recommended cross section for guide banks has slopes of 1 : 2 or flatter with the crest having a minimum width of 1 m, while a crest width of 3 m is regarded as a practical minimum for construction purposes. The upstream end of the guide bank should preferably be round nosed.

Guide banks are normally designed so as not to be overtopped at the design discharge. In general, a minimum freeboard of 600 mm above the design high flood level (DHFL) is considered desirable (Richardson and Richardson 1991). However, most guide banks are regarded as sacrificial and may be partially submerged during a high flood.

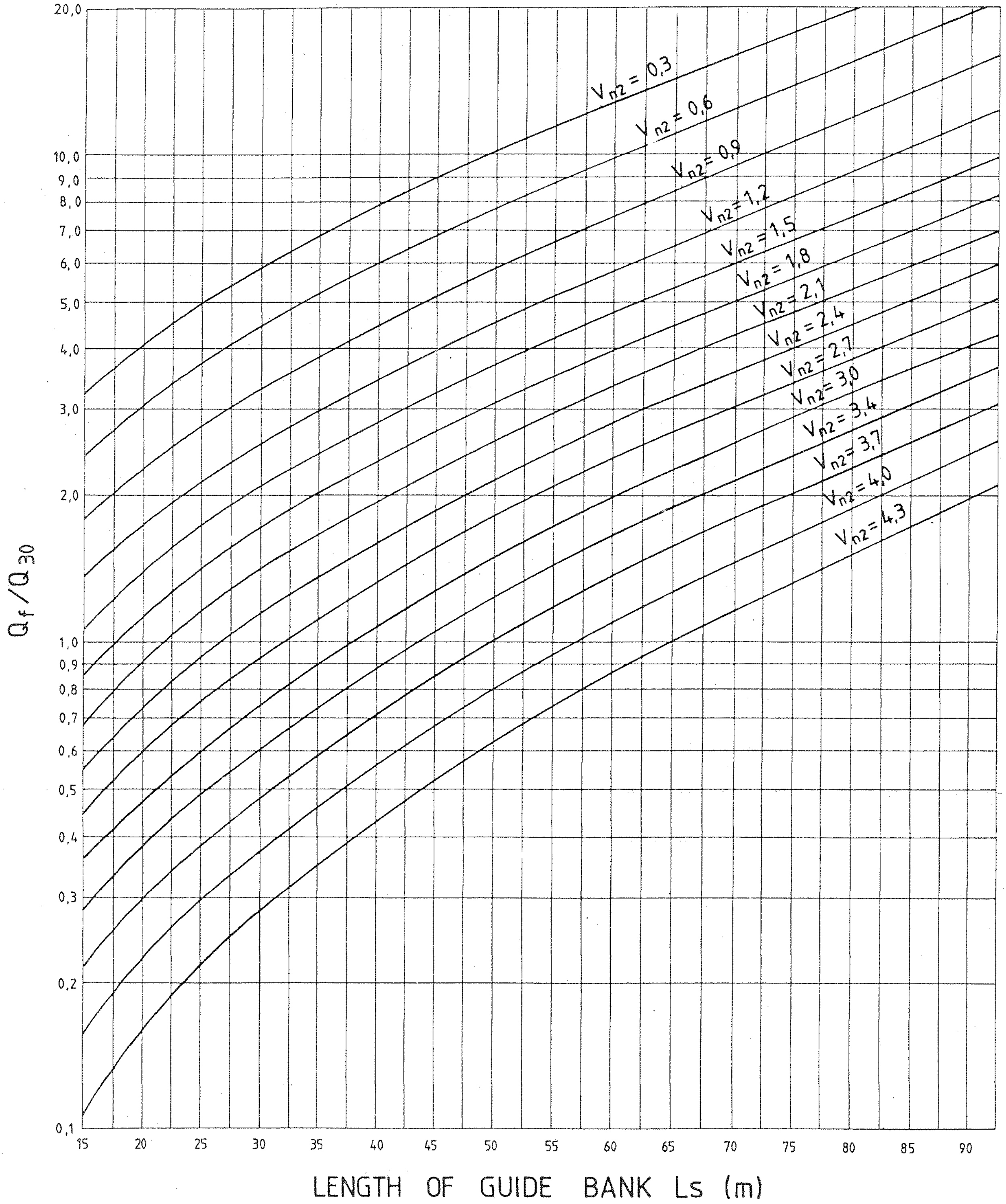

FIGURE 3.2 : NOMOGRAPH TO DETERMINE GUIDE BANK LENGTH (FHWA 1990)

3.2.2 Materials and construction

The materials used for the construction of these structures and the methods of construction are usually the same as those for road embankments. The armour layers required should be in accordance with the requirements specified elsewhere in this document for the particular protection.

3.3 Jetty Fields

The jetty field is an effective and economical system for the stabilization of banks and rectification of channels in alluvial rivers and can readily be adapted to cope with a wide range of bank erosion conditions existing along the channel of aggrading rivers.

When a river flows through erodible material protected by riverine vegetation, the equilibrium shape of the river channel will often be narrow and deep. If the natural vegetation is subsequently removed through extensive farming on the flood plain, the banks will erode and the channel will become wider and shallower, until a new equilibrium cross-sectional shape, associated with unprotected banks, is reached. The position of this channel within the flood plain invariably changes with time and the banks will continue eroding at some locations with a concomitant undesirable deposition taking place elsewhere, especially under inundation of the flood plain during floods. Bank erosion commences as soon as the flow in the river occupies the full width of the channel with erosion becoming progressively worse as the river rises, reaching a maximum when the river approaches full bank discharge.

To limit potential erosion and for reclaiming eroded areas, a jetty field is constructed along the river bank at the sections where the bank is most vulnerable to attack. A typical layout of such a jetty field is illustrated in Figure 3.3.

3.3.1 Design

The steel jack, which forms the main element of the jetty field, consists of three lengths of angle iron sections joined together at right angles at their centre and laced with wire, as shown in Figure 3.5. These jacks are placed in rows, i.e. in winglines and diversion lines, within the area to be protected and connected with cables anchored at their ends. The layout of these jacks within a jetty field is illustrated in Figure 3.3.

The system is designed to trap debris on the jacks and connecting cables, which in turn restricts the flow velocity within the field, causing the deposition of silt. As a result the re-establishment of natural vegetation is encouraged, which leads to a further slowing of the flow velocity and acceleration of the silt deposition and consequent reclamation process.

In this manner a stable new river bank is formed which induces the flow to return to the original channel (Alexander 1976).

Alexander (1976) suggests a spacing for the jacks on the winglines of 4 to 6 m with a wider spacing on the upstream winglines to allow some debris to pass through to the more closely spaced jacks on the downstream lines. Winglines should be spaced between 40 and 80 m centres on curves and further apart on straighter sections of the river.

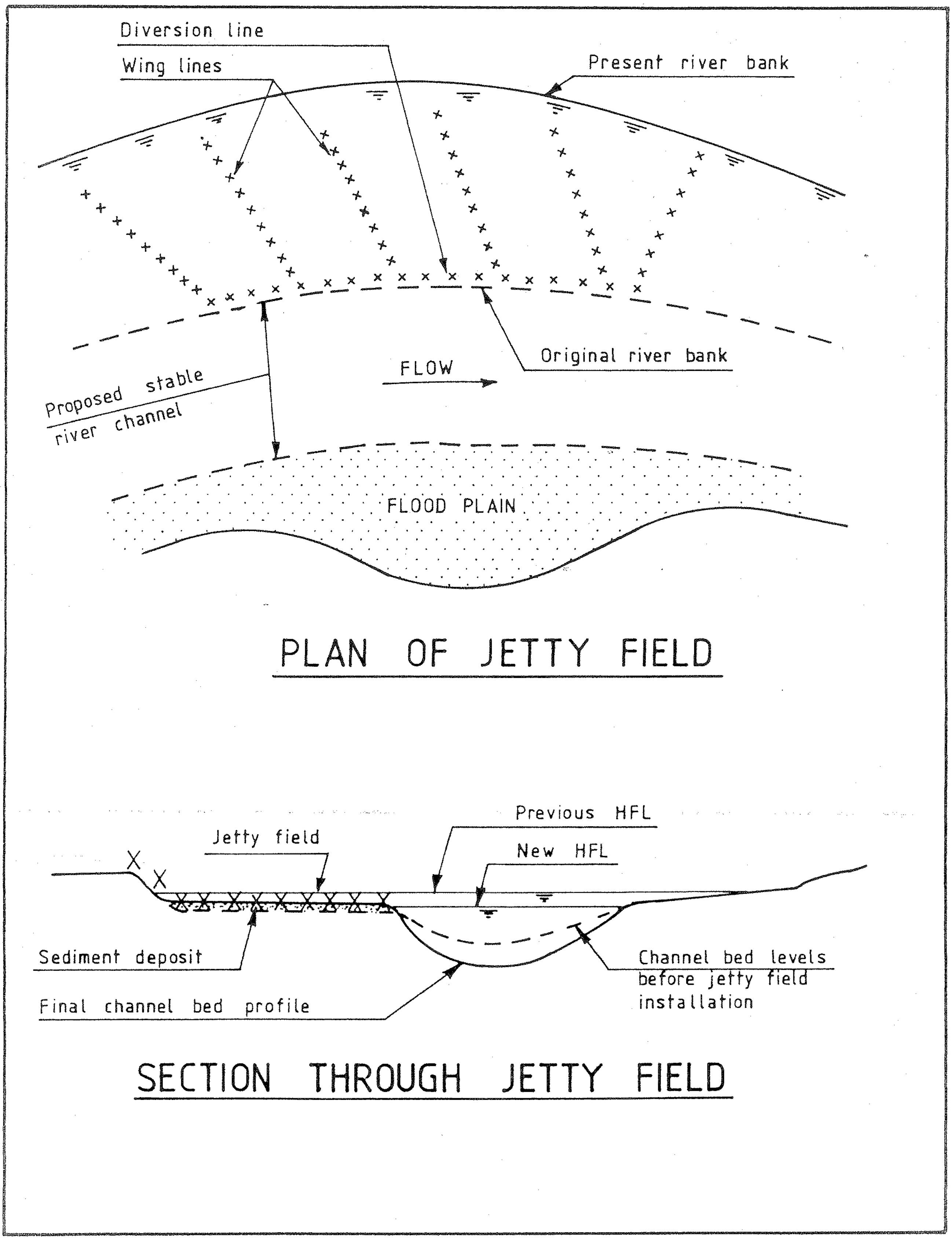

FIGURE 3.3 : SECTION TROUGH JETTY FIELD

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Ideally the winglines should be aligned so as to have an angle of attack of between 20° and 45° to the direction of flow, as illustrated in Figure 3.4, and be located such that the flow across the flood plain is intercepted by between 4 and 6 winglines.

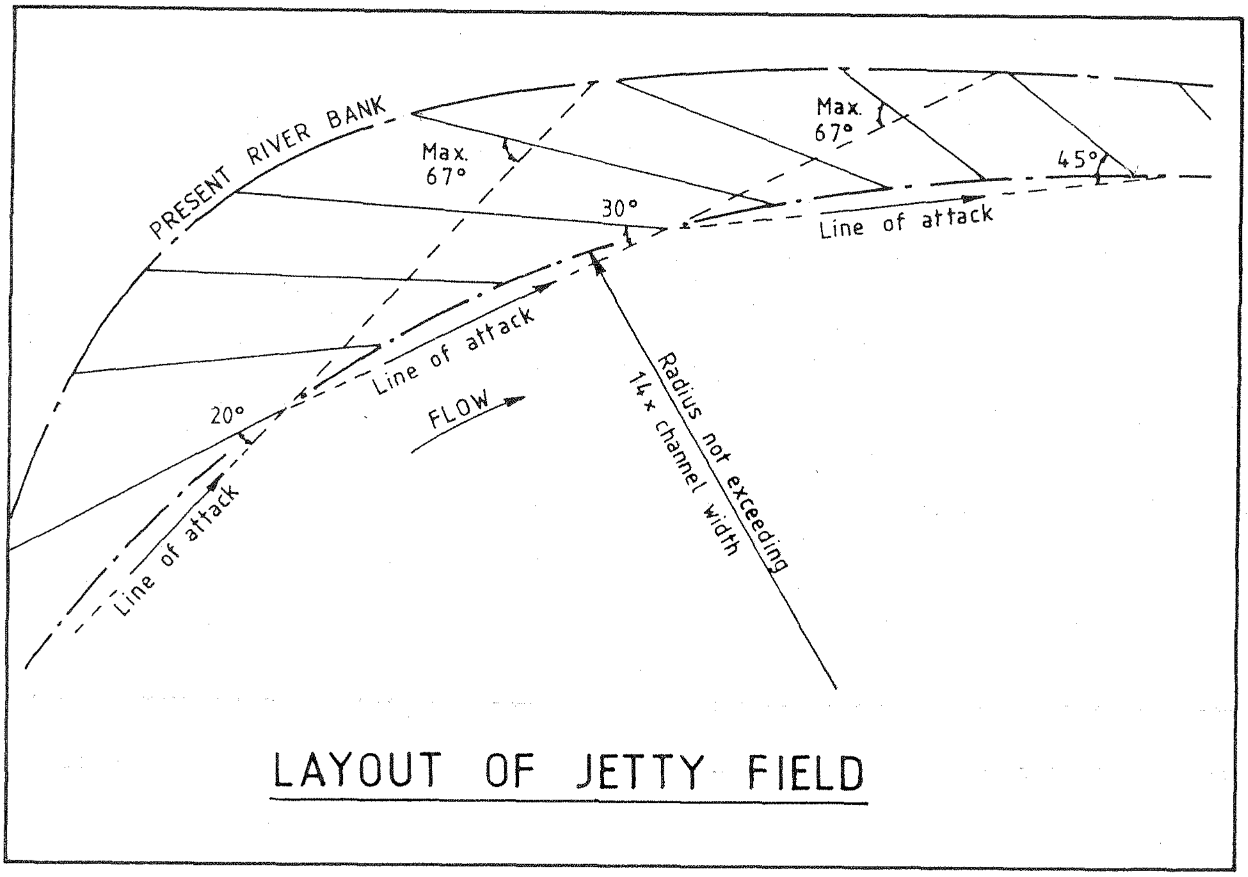

FIGURE 3.4 : LAYOUT OF THE WINGLINES IN A JETTY FIELD

3.3.2 Materials and construction

The jacks are made up of three lengths of angle iron sections bolted together at their centre to form a three-dimensional cross (tripod). The angle iron sections could be either 5 m long - 100 x 100 x 8 mm or 4 m long - 80 x 80 x 6 mm. These angle iron sections are laced together with 4 to 5 mm diameter galvanized steel wire at approximately 0,6 m centres. The individual jacks in the jetty field are linked by two 10 mm diameter galvanized steel wire ropes that are securely clamped to the centres of the jacks, as shown in Figure 3.5.

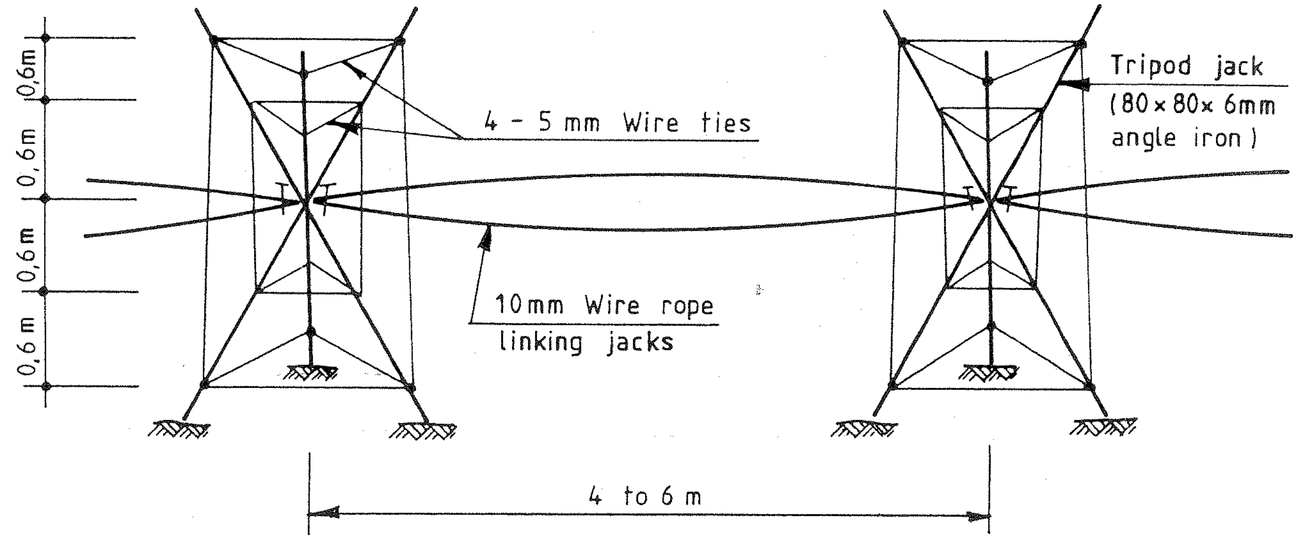

FIGURE 3.5 : DETAIL OF STEEL JACK

To ensure stability of the system all lines should be adequately secured in place by "dead-men" anchors located below the scour depth at the ends of the respective lines. The "dead-men" anchors can comprise railway sleepers or any other suitable object. To further enhance the stability it is recommended that the winglines be clamped to the diversion lines.

On the downstream ends of the lines, drag anchors could be used in lieu of the "dead-men" anchors.

To further enhance the effectiveness of the jacks, brushwood can be fastened to them and more particularly to those in the winglines at the downstream end of the jetty field.

Should high velocities be expected or experienced, or if it is not possible to align the winglines at a shallow angle of attack, the lines could be allowed sufficient slack to form an arc with maximum deflection not exceeding one fifth of their length.

3.3.3 Maintenance

Minor settlement of the diversion lines are likely to take place due to undercutting as the river establishes its new alignment, and some damage to the jacks may be expected, particularly during the first couple of floods. Where severe erosion still occurs after completion of the jetty field, additional lines may have to be erected within the affected areas.

3.4 Spurs[**]

Spurs are obstruction dykes formed in the watercourse to :

- Arrest meander migration of the stream
- Channel a poorly defined stream into a definite channel
- Direct the flow along a less erosive path
- Lessen the skew angle of flow through a structure
- Reduce the required flow opening under a structure
- Protect the banks of a watercourse by improving their stability

Spurs can comprise either permeable or impermeable structures depending on their intended function. Deflector spurs, diverting the primary flow currents away from the bank in a more desirable direction, should be of impermeable construction, while retarder spurs, intended to retard the flow velocity at the bank and diverting flow away from the bank, could be of the semi-permeable type.

When considering the introduction of spurs within the flood plain, the ecological impact thereof should be carefully assessed.

3.4.1 Design

To achieve the intended effect, spurs are normally used in groups, although single spurs can be used in special applications.

The extent over which the spurs should be provided along a channel bank is illustrated in Figure 3.6. The common mistake made in the protection of streambeds is to provide protection too far upstream and not far enough downstream. From research (Richardson and Richardson 1991) it has been established that the protection should extend at least for a distance of 1,0 times the average width of the channel top on the upstream side and for a distance of 1,5 times the average width of the channel top on the downstream side from the reference lines shown in Figure 3.6. However, due to influences of numerous factors on these lengths, they should be regarded as indicative of the minimum protection only and the designer should apply sound engineering judgement, based on knowledge of the flow patterns occurring at the specific site, to establish the appropriate limits of protection.

It is recommended that the spurs be orientated at an angle of approximately 90 degrees to the desired bank line. The leading spurs should, however, be angled downstream to provide a smoother transition of the natural flow lines and to minimize scour at the nose.

The spacing of spurs is a function of the spur length, spur angle, and the degree of curvature of the bend. To ensure that the flow, as it expands from the nose of the upstream spur, does not intersect the bank to be protected before intersecting the next spur, see Figure 3.7, the following formula is recommended for determining the spur spacing :

[**] In some publications these are described as "groynes".

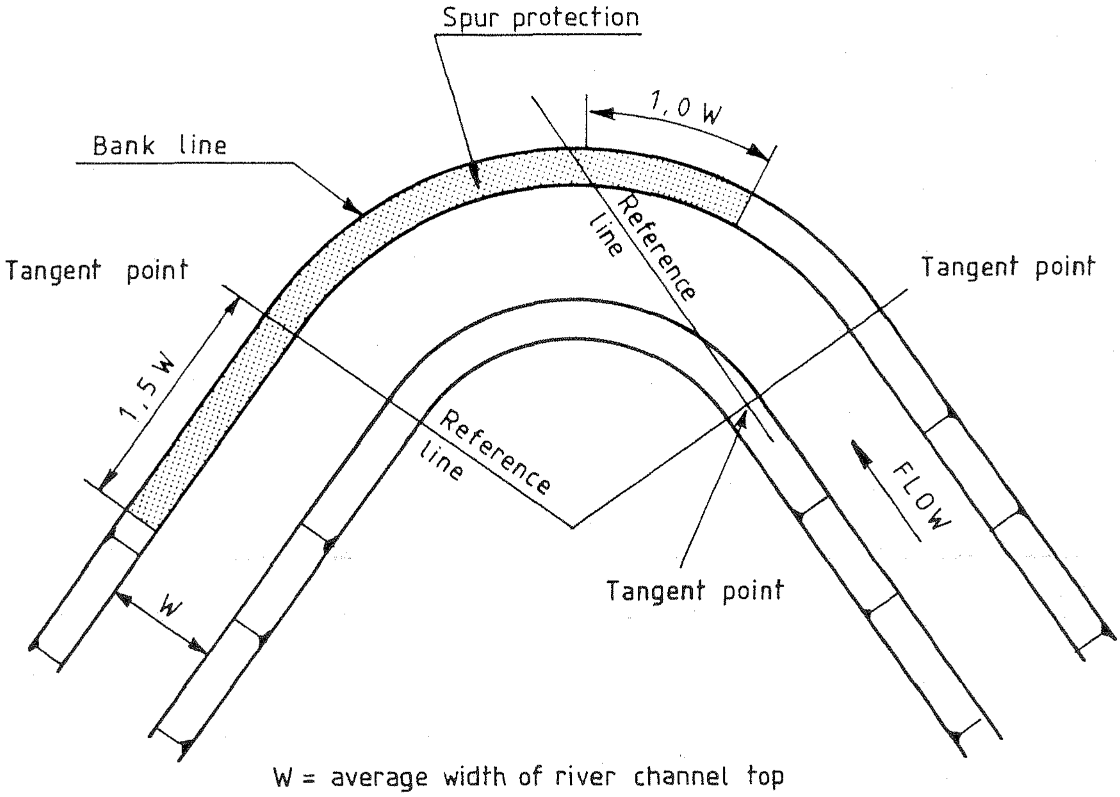

FIGURE 3.6 : EXTENT OF SPUR PROTECTION REQUIRED AT A CHANNEL BEND

$$S = L_e \cot \beta \tag{3.2}$$

where

S = the spacing between spur toes (m)
L_e = effective length of spur from the bank line (m)
β = expansion angle downstream of spur tips (degrees)

The flow expansion angle β, shown in Figure 3.7, is the angle at which flow expands towards a bank downstream of a spur. This angle is a function of the permeability of the spur and the ratio of the spur length to the channel width (FHWA 1990). See Figure 3.8. The angle between the spur centre line and the direction of flow also influences the expansion angle, but to a lesser extent.

Impermeable spurs are generally designed such that their height does not to exceed the bank height. Where stream flow levels are greater than or equal to the bank height, impermeable spurs should be equal to the bank height. If river flow levels are lower than the bank height, impermeable spurs should be designed so that overtopping of the spurs will not occur. The crest of impermeable spurs should slope downward away from the bank line to avoid overtopping at the bank and possible damage to the bank.

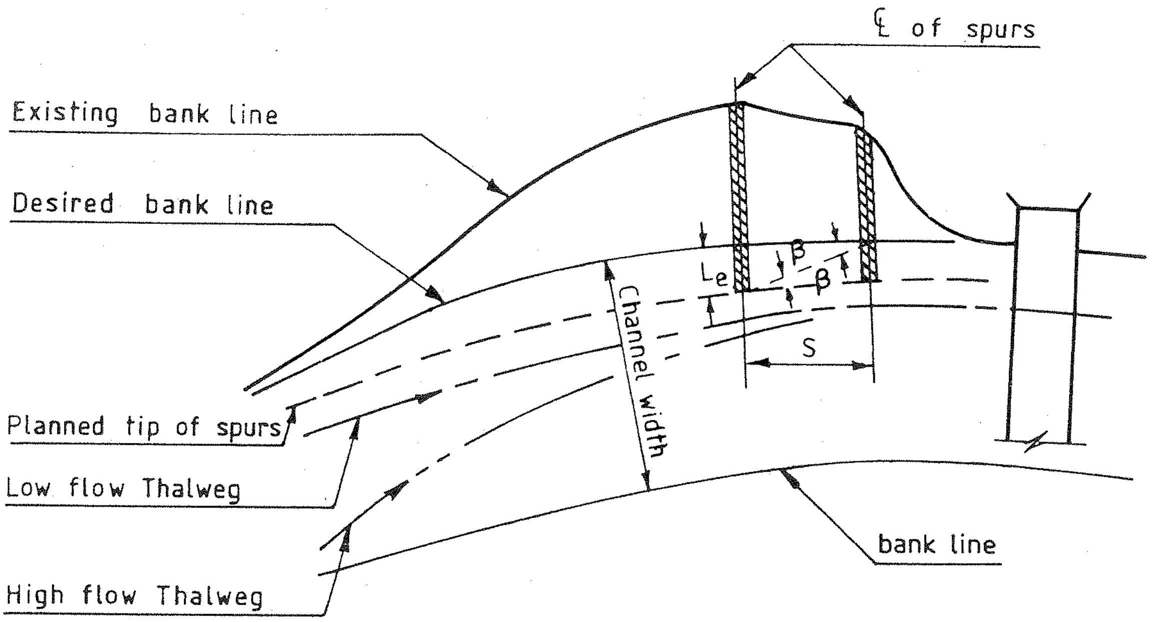

FIGURE 3.7 : ILLUSTRATION OF SPUR SPACING AND EXPANSION ANGLE

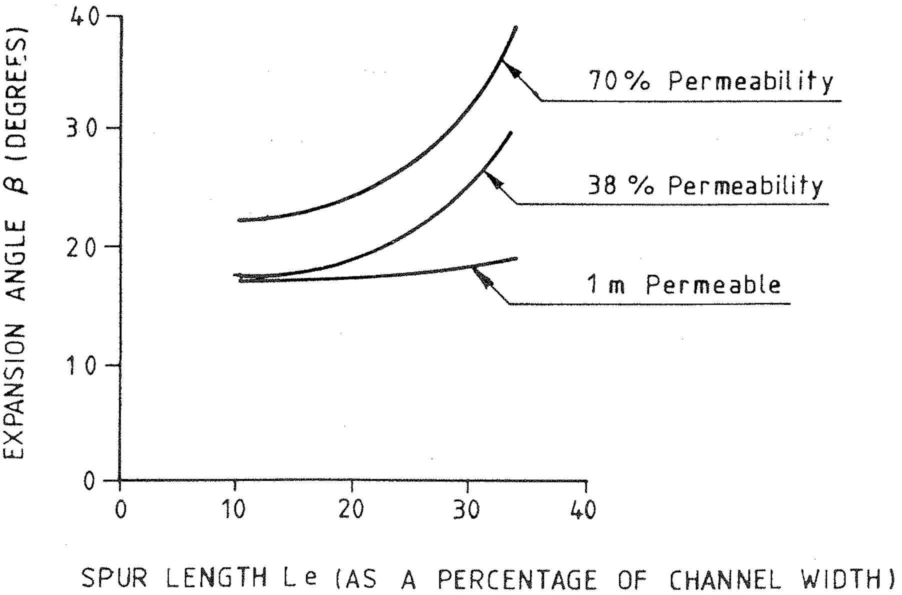

FIGURE 3.8 : RELATIONSHIP BETWEEN SPUR LENGTH AND EXPANSION ANGLE FOR DIFFERENT PERMEABILITIES

Permeable spurs should be designed to a height that will allow large debris to pass over the top. Highly permeable spurs such as dolosses are dependent on light debris being trapped on the spur to make them less permeable. The crest of permeable spurs is generally level, except where the bank height requires a sloping crest.

Scour around spurs will be less for permeable than impermeable spurs. Riprap should, therefore, be provided on the up- and downstream faces and the nose of impermeable spurs to prevent erosion. The riprap should extend to a depth of at least 1,5 m below the river bed. Where the spur would be submerged at the design flow, the riprap should extend over the crest. If the spur is higher than the design flow level, the riprap should extend to at least 0,5 m above the design flood level.

3.4.2 Materials and construction

The materials used for the construction of spurs and the methods of construction are the same as those required for guide banks and/or the respective armour protection as appropriate. Thus, spurs can consist of riprap, sheet piles, gabions, concrete, dolosses, jetties with the jacks being either treated timber or steel, etc. The choice of the material eventually used, is dependent upon many factors and the designer should exercise sound engineering judgement in selecting the most suited material for a particular application.

3.5 Check dams

Check dams, consisting of channel drops or weirs, are used downstream of road structures to control vertical instabilities by arresting the progression of head cutting and general channel bed degradation.

3.5.1 Design

The check dam must be designed structurally to withstand the forces of water and soil, assuming that the scour hole is as deep as estimated. For this reason it may prove more economical to construct a series of check dams of lower height. A typical vertical drop resulting from a check dam is diagrammatically illustrated in Figure 3.9. (FHWA 1990). For lack of a better equation, the Veronese Equation (3.3) is recommended for estimating the depth of the scour hole downstream of the drop for both the submerged and unsubmerged conditions. (Richardson and Richardson 1991).

$$d_s = K H_t^{0,225} q^{0,54} - h \qquad (3.3)$$

where

d_s = local scour depth for a free fall (m)

q = discharge per unit width ($m^3/s.m$)

H_t = total drop in head, measured from the upstream to downstream energy grade line (m)

h = tail water depth (m)

K = 3,60

H_t is computed using Bernoulli's equation for steady uniform flow.

The above equation does not take into account grain size of the bed material.

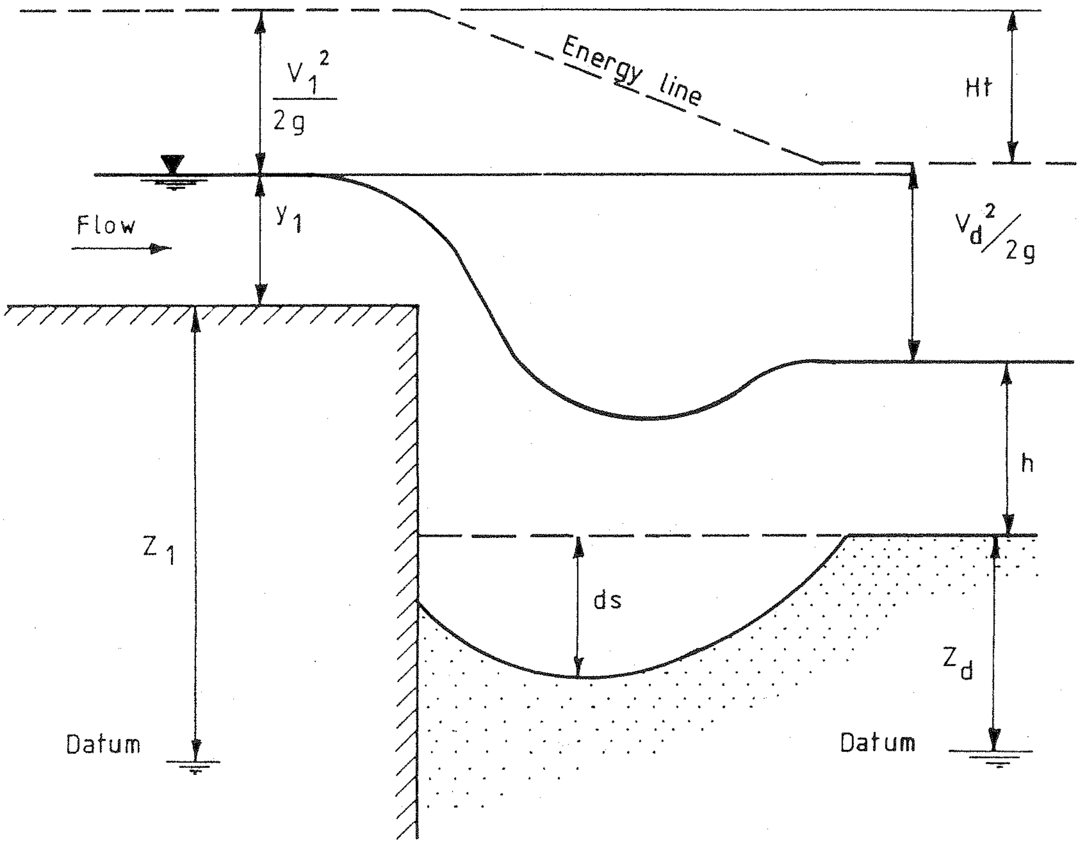

FIGURE 3.9 : SCHEMATIC OF A VERTICAL DROP CAUSED BY A CHECK DAM

3.5.2 Materials and construction

Check dams are generally constructed from riprap, concrete, sheet piles, gabions or treated timber piles.

To overcome lateral erosion of the channel embankments, it is recommended that a revetment be placed on the stream bank along the area of attack.

As a result of the vertical drop and concomitant turbulences and dissipation of energy, check banks can initiate erosion of the banks and channel bed downstream thereof. By introducing energy breakers downstream of the check dam, maintenance can be reduced considerably.

4. ROCKFILL EMBANKMENTS

Although guidelines are given for determining the stability of rockfill embankments under different flood conditions, it must be emphasized that they are not exhaustive. The designer is encouraged to become familiar with the techniques that are described in the reference. The Dutch are recognised as the leaders in this field.

4.1 General

Rockfill should be considered as an alternative solution to the conventional dense earthfill embankment and is preferred wherever :

- The foundation conditions are poor
- The embankment is designed to be overtopped during excessive flooding.

When correctly designed the rockfill will form a resistant barrier against overtopping. A rockfill embankment, designed to overtop, will act as a broad-crested weir where $W_r/D_{50} > 10$ as shown in Figure 4.1.

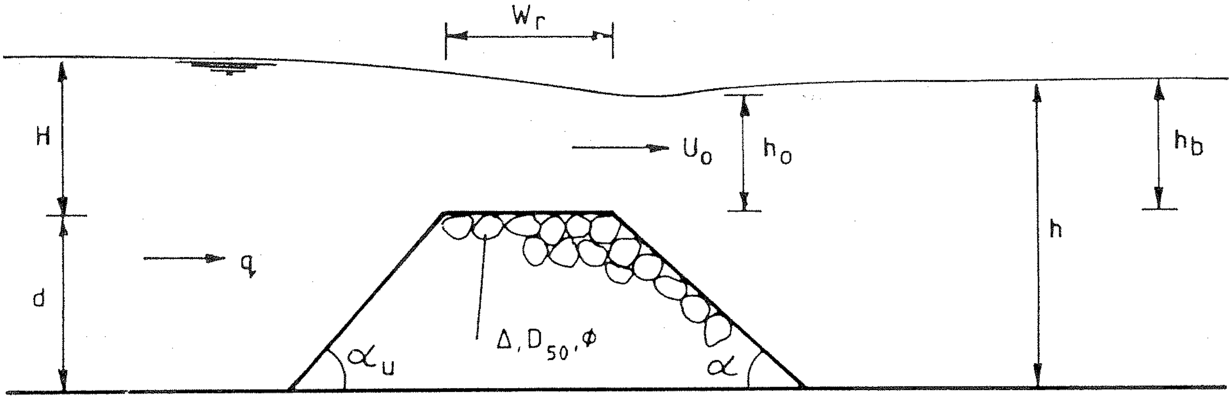

FIGURE 4.1 : ROCKFILL EMBANKMENT AS BROAD-CRESTED WEIR

Based on recent investigations, improvements in the stability design of rockfill embankments under flood conditions have been made by introducing new stability parameters. In this regard four typical flow conditions are identified, all with reference to the tailwater elevation as illustrated in Figure 4.2.

They are :

- Low embankment flow
- Intermediate embankment flow
- High embankment flow
- Through flow

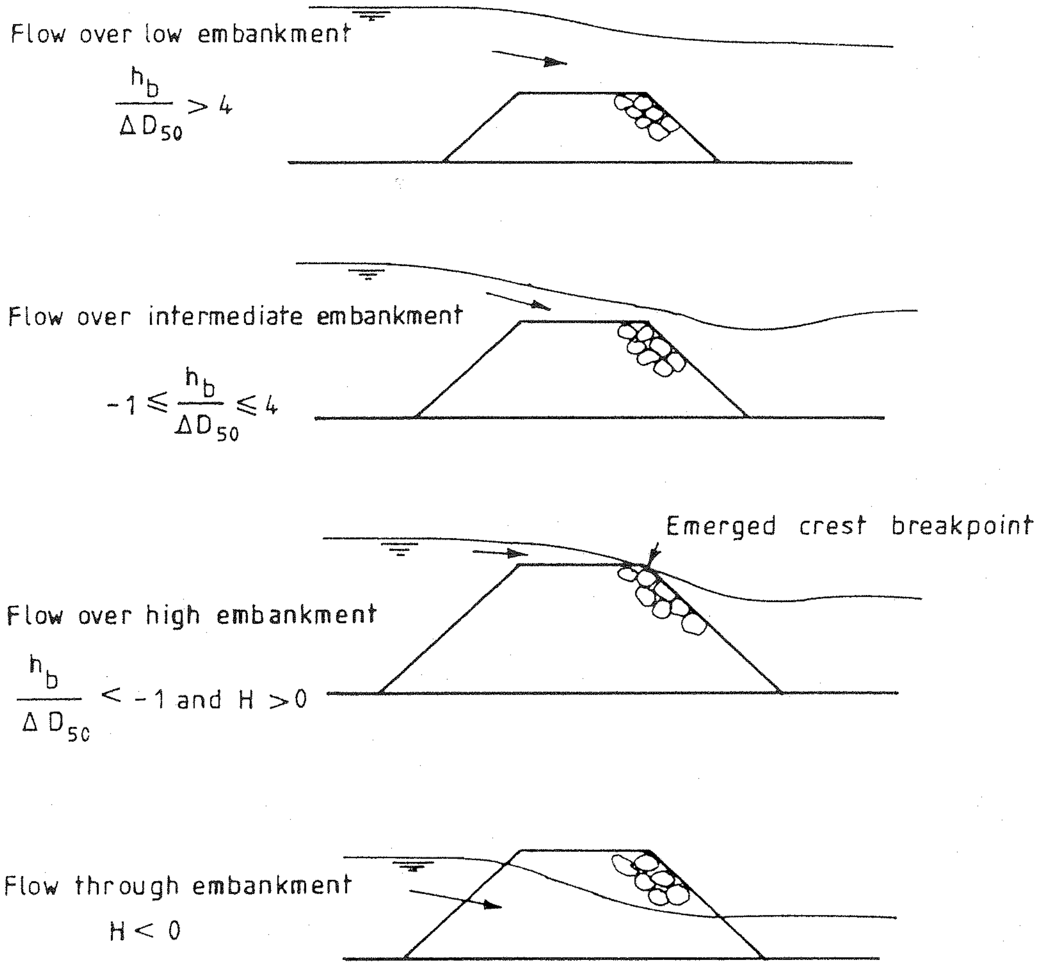

FIGURE 4.2 : TYPICAL FLOW CONDITIONS

Overseas research on rockfill embankments/dam walls has been focused on what is called the vertical closure method of construction, i.e. construction of the embankment in horizontal layers using the end-tipping method. The information contained in the research should assist in the design of rockfill embankments when using rockfill material that provides a mechanical interlock when compacted.

As the design methods for rockfill dam walls are equally applicable to the design for rockfill embankments, the designer is referred to the research literature on the former for design criteria and guidelines. These criteria should be regarded as only indicative, as there are still many uncertainties in the theoretical approach.

4.2 Materials

Ideally the rock for the embankment is blasted quarry stone that is hard and will not be susceptible to excessive weathering and disintegration on exposure to the atmosphere or water. The size, grading and shape of the rock is important for stability, and rounded boulders should, therefore, be avoided. To ensure that the fines in the underlying soils are not washed out and transported away, the rock in the lower layers should be well-graded.

4.3 Design

For the design of rockfill embankments, it is recommended that the four typical flow regimes are analysed.

- Low embankment flow, i.e. $h_b/\Delta D_{50} > 4$: drowned flow
- Intermediate embankment flow, i.e. $-1 \leq h_b/\Delta D_{50} \leq 4$: free flow
- High embankment flow, i.e. $h_b/\Delta D50 < -1$ with $H > 0$: emergence of downstream crest breakpoint
- Through flow, i.e. $H < 0$: the full discharge passes through the embankment

Disturbance of the stability of the rock embankment, due to the proximity of structures, is expected to be minimal when the embankment is in the intermediate or high embankment flow regime. In the low embankment flow regime, the negative influence on the stability will increase, but no quantitative information is available to support this notion at the moment. The stability of the adjacent rockfill bank face itself can be assessed from the research literature.

4.3.1 Low embankment flow

A very simple empirical relationship for the critical velocity for threshold damage has been developed by Izbash (1932), viz. :

$$u / \sqrt{g \, \Delta \, D_{50}} = 1{,}7 \quad \textit{for a packed/compacted stone on the crest}$$

and

$$u / \sqrt{g \, \Delta \, D_{50}} = 1{,}2 \quad \textit{for a dumped/loose stone on the crest}$$

where

u = critical flow velocity (m/s)
g = gravitational acceleration (m/s^2)
Δ = relative density of stone
D_{50} = median stone diameter (m)

Although the influence of roughness for different overflow depths is ignored for determining the critical velocity, these expressions are, nonetheless, widely used overseas.

4.3.2 Intermediate embankment flow

When the rockfill embankment is raised further relative to the fixed tailwater level, the flow regime will become free flow with the velocity down the downstream slope increasing to supercritical. For this situation the most appropriate stability parameters are :

- The upstream water depth above the crest compared to the stone dimensions, i.e. $H/\Delta D_{50}$
- The downstream water depth compared to the embankment height, i.e. h/d
- The stone size compared to the embankment height, i.e. the permeability parameter : D_{50}/d

The designer is referred to the literature listed in the references for detailed guidelines :

4.3.3 High embankment flow

Much information is available from investigations into the stability of rockfill embankments subjected to overflows for this regime. The actual flow in this situation cannot be characterised by the traditional uniform flow approach because of the fundamental influence of roughness.

Linford and Saunders (1967) investigated the overflow of rockfill dams with an impervious sealing on the upstream slope and along the crest. Much attention was paid to stone arrangement and it was found that a close packing by hand of edge-placed stones is capable of withstanding a flow of more than three times that of the flat-placed arrangement (Figure 4.3). The latter was found to be even less stable than the conditions achieved with a dumped stone layer. In addition, it was found that the critical discharge for rounded boulders was 35% less than that for quarried rockfill.

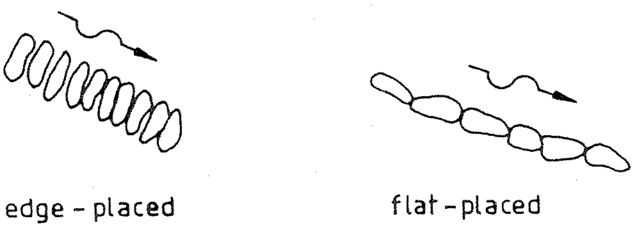

FIGURE 4.3 : STONE PACKING ARRANGEMENTS

A simplified relationship for critical discharge for the stability of the stones was proposed, viz. :

$$q_t = 0{,}84\ W_s^{0{,}5}\ (1{,}9 + 0{,}8\ \delta - 3 \sin \alpha) \qquad (4.1)$$

where

q_t = critical discharge ($m^3/s.m$)
W_s = average stone weight for D_{50} size (kN)
δ = packing factor ranging from 0,6 for natural (dumped) packing to 1.1 for manual (hand) packing
α = angle of downstream slope

By substituting $W_s = \rho g D_{50}^3$ in Equation 4.1, assuming $\rho_s = 2{,}7$ ton/m^3 and introducing $\Delta = (\rho_s - \rho_w)/\rho_w$ the equation reduces to :

$$q_t = 1{,}95\ (\Delta D_{50})^{1{,}5}\ (1{,}9 + 0{,}8\ \delta - 3\ \sin\alpha) \qquad (4.2)$$

where

Δ = relative density of the stone
ρ_s = mass density of the stone (t/m^3)
ρ_w = mass density of water (t/m^3)
D_{50} = nominal 50% on medium size stone (m)

It should be noted that, for this flow regime, the rockfill in the embankment should always be relatively pervious. If not, a minor overtopping can lead to a total collapse of the embankment.

4.3.4 Through flow

In practice the through flow regime is not normally a critical stage for stone stability, as the maximum stone dimensions are generally dictated by the overflow conditions.

Prajapati (1981) found the threshold unit discharge or critical discharge for the onset of instability in rockfill embankments for throughflow to be a function of tailwater depth and stone size. The application of this theory is, however, restricted to rockfill placed at the angle of repose of 1 : 1,25. Research results indicate that:

q is proportional to (tailwater depth) $^{1/3}$ and (stone size) $^{7/6}$.

At this stage there is no applicable formula for determining q.

4.4 Failure mechanisms

A rockfill embankment may fail because of mechanisms as illustrated in Figure 4.4.

From a design point of view it is important to assess the margin between the threshold condition and a partial or total collapse of the embankment. Emphasis is, however, placed on the threshold or extensive damage margin. External damage due to overflow and throughflow causing deformation of the embankment profile usually results from the displacement of stones.

Wave action combined with the overflowing current may decrease the stability of the embankment. If current attack is dominant, the decrease in stability is probably mainly due to the instantaneous surge in discharge during passage of a wave which will be decisive in creating instability. Failure due to wave attack is similar to that described for breakwater stability in Section 2.1.8.

The stability behaviour of the embankment should be carefully assessed on the basis of the embankment geometry and its porosity. For a large rockfill embankment crossing a major river flood plain or estuary, it is recommended that a model study be undertaken.

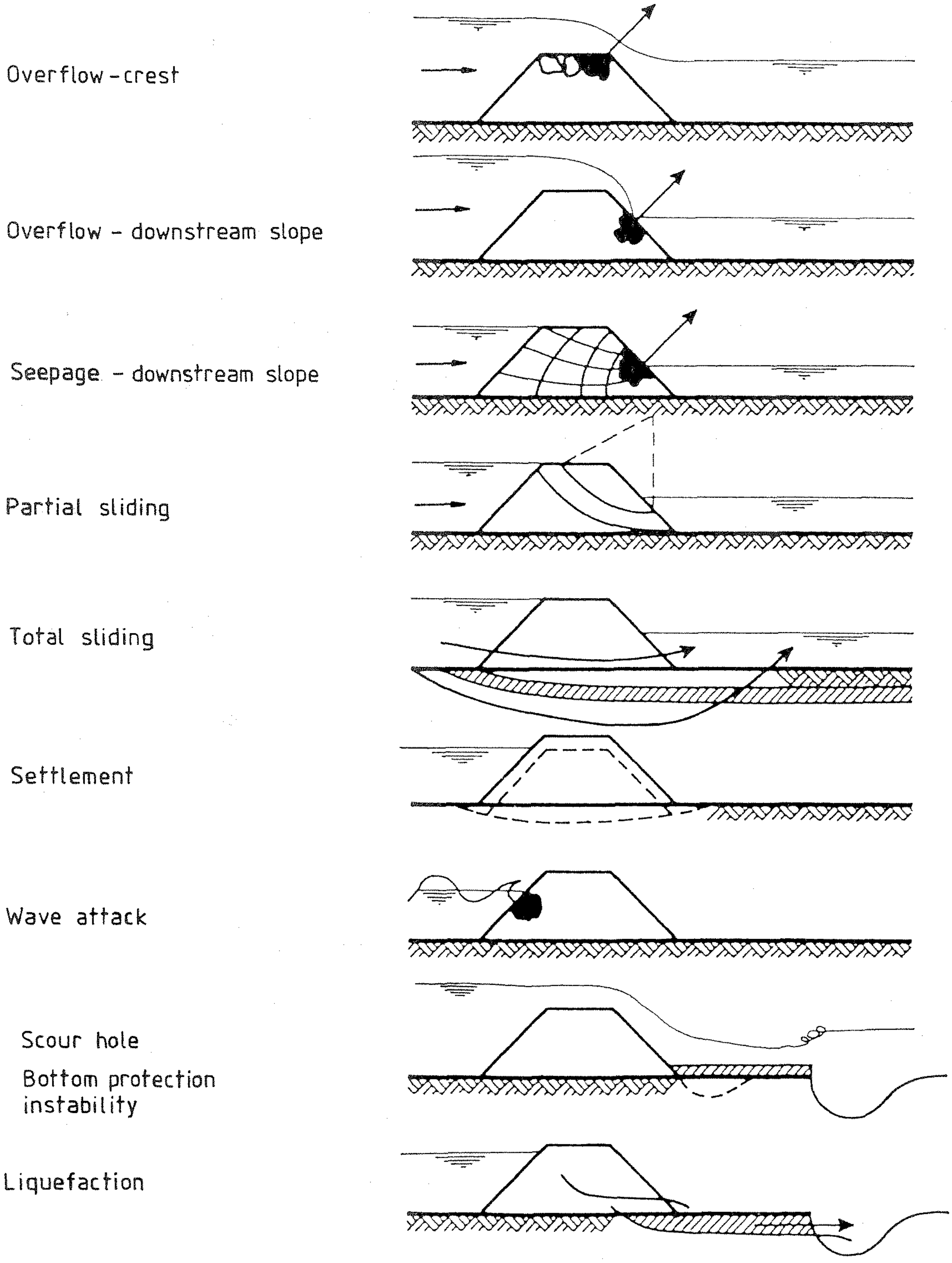

FIGURE 4.4 : ROCKFILL EMBANKMENT FAILURE MECHANISMS

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4.5 Bed protection

To prevent undermining of the embankment, bed protection on both sides of the embankment is generally required when the bedding comprises cohesionless material. The bed protection should extend over a sufficient distance to ensure that the region of scour hole formation is a sufficient distance from the embankment as not to threaten its stability. The scour hole depth should be minimized by providing riprap type protection in conjunction with a stone filter to prevent washing out of the fines from the bedding.

These aspects, although decisive for the bed protection design as a whole, are beyond the scope of rockfill embankment design and the designer should consult the appropriate literature.

4.6 Construction

The rock is generally end-tipped and spread and compacted by means of a bulldozer into layers of thickness ranging from 0,5 to 2,0 m, depending on the size of the rock and the compaction equipment available. The stone sizes used should preferably be limited to approximately two-thirds of the layer thickness.

5. REFERENCES

Abt, SR and Johnson, TL. 1991. *Rip-rap design for overtopping flow.* Journal of Hydraulic Engineering, ASCE, Vol. 117, No. 8, August, 959 - 972.

African Gabions (Pty) Ltd. no date. *Channelling works.* African Gabions.

African Gabions (Pty) Ltd. no date. *Terramesh systems - a solution for soil reinforcement.* African Gabions.

Agostini, R et al. 1988. *Flexible gabion and reno mattress structures in river and stream training works - section two.* Officie Maccaferri S.p.A. - Bologna - Italy, 27 - 30.

Alexander, W. no date. *Restoration of damaged river channels.* Department of Environmental Affairs, 1 - 6.

Alexander, WJR. 1976. *An introduction to river mechanics and channel stabilisation.* Department of Water Affairs, Division of Hydrology, Technical Note 70, March, 3 - 30.

Alexander, WJR. 1971. *Suggested use of steel jetty fields for river channel stabilisation in the Gamtoos valley.* Hydro, August, 1 - 5.

Beach Erosion Board. 1961. *Shore protection planning and design.* Beach Erosion Board. Technical report No. 4.

Blake, KRK and Millard, BED. 1993. *Prevention of scour below the Mdloti Bridge.* Proceeding Annual Transportation Convention, Pretoria.

Bradley, JN. 1970. *Hydraulics of bridge waterways.* In : Herr, L.A. (ed), second ed. U.S. Department of Transport, 51 - 55.

Bramley, ME et al. 1987. *Protection and provision for safe overtopping of dams and flood banks.* CIRIA, Project Report No. 2, November.

Brogdon, NJ and Grace, JL. 1964. *Stability of rip-rap and discharge characteristics, overflow embankments, Arkansas River.* U.S. Army Corp of Engineers, Vicksburg. Technical Report 2-6500.

Bruun, P and Günbak AR. 1977. *Stability of sloping structures in relation to* $\zeta = \tan\alpha / \sqrt{H/L_\theta}$ *risk criteria in design.* Coastal Engineering.

Burcharth, HF. 1980. *Full-scale trials of dolosse to destruction.* Proceedings XVIIth Coastal Engineering Conference, Sydney.

Butfield, A. 1987. *Designing in reinforced soil.* Civil Engineering, August.

Capper, PL and Cassie, WF. 1963. *The mechanics of engineering soils.* E and FN Spon.

CERC. 1984. (Coastal Engineering Research Centre). *Shore protection manual*, Vol. 1, Ch. 2. Vicksburg, USA.

Charles, JA and Soares MM (1984). *Stability of compacted rockfill slopes.* Geotechnique (34) Volume XXXIV, No, 1, March 1984. The institution of civil engineers, London.

Chen, YH et al. 1987. *Development of a methodology for estimating embankment damage due to flood overtopping.* Federal Highway Administration, Report No. FHWA/RD-86/126, March.

Concrete Systems International cc. *Laboratory testing of the sliding/shear resistence of Loffelstein concrete block retaining walls.*

Concor Technicrete. 1990. *Armorflex erosion control system.* Concor Technicrete Armorflex brochure.

Crowhurst, AD. no date. *The use and performance of gabions in large scale flood control structures.* Maccaferri Gabions Inc., 4 - 4, 4 - 12.

CSIR. 1989. *Wind data for four South African harbours, design conditions.* CSIR. Report EMA-C8909.

Committee of State Road Authorities. 1987. *Standard Specification for road and bridge works,* 1st ed. CSRA.

Darbin, M et al. 1988. *Durability of reinforced earth structures : the results of a long term study conducted on galvanised steel.* Proceedings of the Institution of Civil Engineers, Part 1, 1988.

Department of Public Works. 1960. *Bank and shore protection in California highway practice.* Department of Public Works - Division of Highways, September.

Department of Public Works. 1970. *Bank and shore protection in California highway practice.* State of California Department of Public Works.

Dobbie, CH and Partners. 1980. *Report on bank protection in rivers and canals.* Hydraulic Research Station, Oxfordshire, July.

El-Khashab, AM et al. 1987. *Modeling materials for protecting earth weirs.* National Conference on Hydraulic Engineering, ASCE, Proceedings 1987, 576 - 581.

FHA. 1990. *Stream stability at highway structures.* FHWA Department of Transport, Washington. Engineering circular No. 20. (Draft).

Frankipile. 1986. *Guide to piling and foundation systems.* Second edition.

Fuchsberger, M. 1971. *Diaphragm walls.* University of Surrey Department of Civil Engineering, Summer school in foundation engineering, September.

Fulton, FS. 1977. *Concrete technology.* The Portland Cement Institute.

Graziano, F et al. (1990). *Design of rip-rap to protect bridge piers from local scour.* Public Roads, Vol. 54, No. 2, September, 193 - 198.

Hemphill, RW and Bramley ME. 1989. *Protection of river and canal banks - a guide to selection and design.* CIRIA Water Engineering Report.

Hewlett, HW et al. 1987. *Design of reinforced grass waterways.* CIRIA, Report 116.

Hillen, JG and Pilarczyk, KW. 1986. *Hydraulic design criteria for rockfill closure of tidal gaps - vertical closure method.* Delft Hydraulics Laboratory, M1741, Part IV, July.

Holtzhausen, AH and Zwamborn, JA. 1990. *Stability of dolosse with different waist thicknesses for irregular waves.* Proc. 22nd ICCE, Delft.

Hudson, RY. 1961. *Wave forces on rubble-mound breakwaters and jetties.* U.S. Army Engineer Waterways Experiment Station. Miscellaneous Paper 2-453. September.

Huntington, WC. 1957. *Earth pressures and retaining walls.* John Wiley and Sons.

Hydraulic Engineering Circular 18. *Evaluating scour at bridges.* Federal Highway Administration, Washington.

Institution of Civil Engineers. 1951. *Earth retaining structures.* Civil Engineering Code of Practice No. 2. Prepared by Civil Engineering Codes of Practice Joint Committee.

Kantey and Templar. 1986. *Sildur Loffelstein precast concrete gravity retaining walls.* Concrete syetms International cc.

Kluger, JW. 1988. *Survey methods of breakwaters.* CSIR Report T/Sea 8801.

Kovacs, Z. 1974. *Scour and protective measures at drainage works - guide for design.* Van Niekerk, Kleyn & Edwards, 30.

Laursen, EM. 1984. *Assessing vulnerability of bridges to floods.* Second Bridge Engineering Conference. Washington, Vol. 2.

Linford, A and Saunders, DH. 1967. *A hydraulic investigation of through and overflow rockfill dam.*

Meyerhof, GG. 1951. *The ultimate bearing capacity of foundations.* Geotechnique. Vol. II.

Meyerhof, GG. 1953. *The bearing capacity of foundations under eccentric and inclined loads.* Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering.

Meyerhof, GG. 1957. The ultimate bearing capacity of foundations on slopes. Proc. 4th International Conference on Soil Mechanics and Foundation Engineering.

Meyerhof, GG. 1963. *Some recent research on the bearing capacity of foundations.* Canadian Geotechnical Journal, Vol. 1. No. 1.

Maynord, ST et al. 1989. *Rip-rap design.* Journal of Hydraulic Engineering, ASCE, Vol. 115, No. 7, July, 937 -949.

Maynord, ST. 1988. *Stable rip-rap size for open channel flows.* U.S. Army Waterways Experiment Station, Technical Report HL-88-4.

Meigh, AC and Hobbs, MA. no date. *Soil mechanics*. Civil Engineers Reference book, 3rd edition. Newnes-Butterworths.

Ministry of Works New Zealand. 1973. *Retaining wall design notes.*

Naval Facilities Engineering Command. 1986. *Foundations and earth structures.* Design Manual DM-7.02.

Neill, CR. 1975. *Guide to bridge hydraulics.* Project Committee on Bridge Hydraulics, Roads and Transportation Association, 121 - 132.

Parola, AC and Jones, JS. 1991. *Sizing of rip-rap to protect bridge piers.* Transportation Research Record No. 1290 Volume 2, 276 - 280.

Perrier, H. 1986. *Use of soil-filled synthetic pillows for erosion protection.* Third International Conference on Geotextiles, Vienna, 1115 - 1118.

Powledge, G. et al. 1989. *Mechanics of overflow erosion on embankments, II : Hydraulic and design considerations.* Journal of Hydraulic Engineering, ASCE, Vol. 115, No. 8, August, 1056 - 1075.

Prajapati, JJ. 1981. *Model Studies on throughflow rockfill structures.* XIX Congress, New Dehli. Subject D, Paper No. 12, 267 - 281.

Rajaratnam, N. et al. March 1983. *Flow near groin-like structures.* Journal of Hydraulic Engineering, ASCE, Vol. 3, No. 9, March, 463 - 479.

Rankin, D. 1982. *Stabilising stream channels by river training and interaction with the environment.* The Institution of Engineering, Australia, Paper C1273, 135 - 141.

Reinforced Earth Company. *Reinforced earth.* Reinforced Earth Co., Washington D.C.

Richardson, JR and Richardson, EV. 1991. *Countermeasures for scour and stream instability at bridges.* Transportation Research Record, No. 1290, Volume 2, 269 - 275.

Rooseboom, A. et al. 1983. *Road drainage manual.* National Transport Commission, second edition, 4 - 30 to 4 - 32.

Rooseboom, A. et al. 1992. *Sediment transport in rivers and reservoirs.* Water Research Commision Report No. 297/1/92.

Rossouw, J. 1984. *Review of existing wave data, wave climate and design waves for South African and Namibian coastal waters.* CSIR Report T/Sea 8401.

Rossouw, J. 1989. *Design waves for the South African coastline.* PhD Thesis, University of Stellenbosch.

Schlosser, F and Vidal, H. 1969. *Reinforced earth.* Traduction de l'article "La Terre armée" publié dans le Bulletin de Liaison des Laboratoires Routiers - Ponts at Chaussées - n°, November.

Simpson, G. 1991. *Gabions.* The Civil Engineering Contractor, May.

Stevens, MA. et al. 1984. *Rip-rap stability analysis.* Transportation Research Record No. 950, Volume 2, 209 -216.

Stephenson, D. 1979. *Rockfill in hydraulic engineering.* Elsevier Scientific Publishing Co., New York, 215.

Terzaghi, K and Peck, RB. 1967. *Soil mechanics in engineering practice.* John Wiley and Sons.

Thomson, J. 1986. *Erosion control.* Landscape design, August, 41 - 44.

Transportation Research Board. 1989. *Innovative earth-retaining systems.* Transportation Research Board 1242. Washington, D.C.

Tschebotarioff, GP. 1951. *Soil mechanics, foundations and earth structures.* McGraw - Hill book company.

US Army Corp. of Engineers. 1981. *Appendix G - demonstration projects on other streams nationwide.* US Army Corp. of Engineers, Volume 1 of 2, December, G-10.

US Army Corp. of Engineers. 1984. *Shore protection manual.* Department of the Army Waterways Experiment Station. Volume II, 7 - 205 - 7 - 237.

Van Wyk, S. Deputy Director. 1991. *Grasses for vegetation of embankment slopes.* Research Institute for Reclamation Ecology, Potchefstroom University, Private interview, September.

Weather Bureau. 1975. *Climate of South Africa. Part 12, Surface Winds,* WB 38, Pretoria.

Worman, A. 1989. *Rip-rap protection without filter layers.* Journal of Hydraulic Engineering, ASCE, Volume 115, No. 12, December, 1615 - 1629.

Wyman R. et al. 1983. *Cardiff flood protection.* Concrete, January.

Zwamborn, JA and Beute, J. 1972. *Stability of dolos armour units.* ECOR Symposium S71. Stellenbosch.

Zwamborn, JA. 1979. *Analysis of causes of damage to sines breakwater.* Proc. Coastal Structures 79, Alexandria, Virginia.

Zwamborn, JA. 1980. *Measuring techniques, dolos packing density and effect of relative block density.* CSIR. Research Report 378.

Zwamborn, JA. et al. 1980. *Dolosse; past, present, future? Proceedings XVIIth* Coastal Engineering Conference Sydney.

Zwamborn, JA and Phelp D. 1989. *Structural tests on dolosse.* Proc. Seminar on Stresses in Concrete Armor Units, Vicksburg, USA.

Zwamborn, JA, et al. 1990. *Redesign, repair and monitoring of the Table Bay main breakwater.* Proc. 27th PIANC Congress, Osaka.