THE MEKONG RIVER COMMISSION SECRETARIAT

Best Practise Guidelines for Structural Measures and Flood Proofing

The Flood Management and Mitigation Programme, Component 2: Structural Measures & Flood Proofing in the Lower Mekong Basin

December 2009

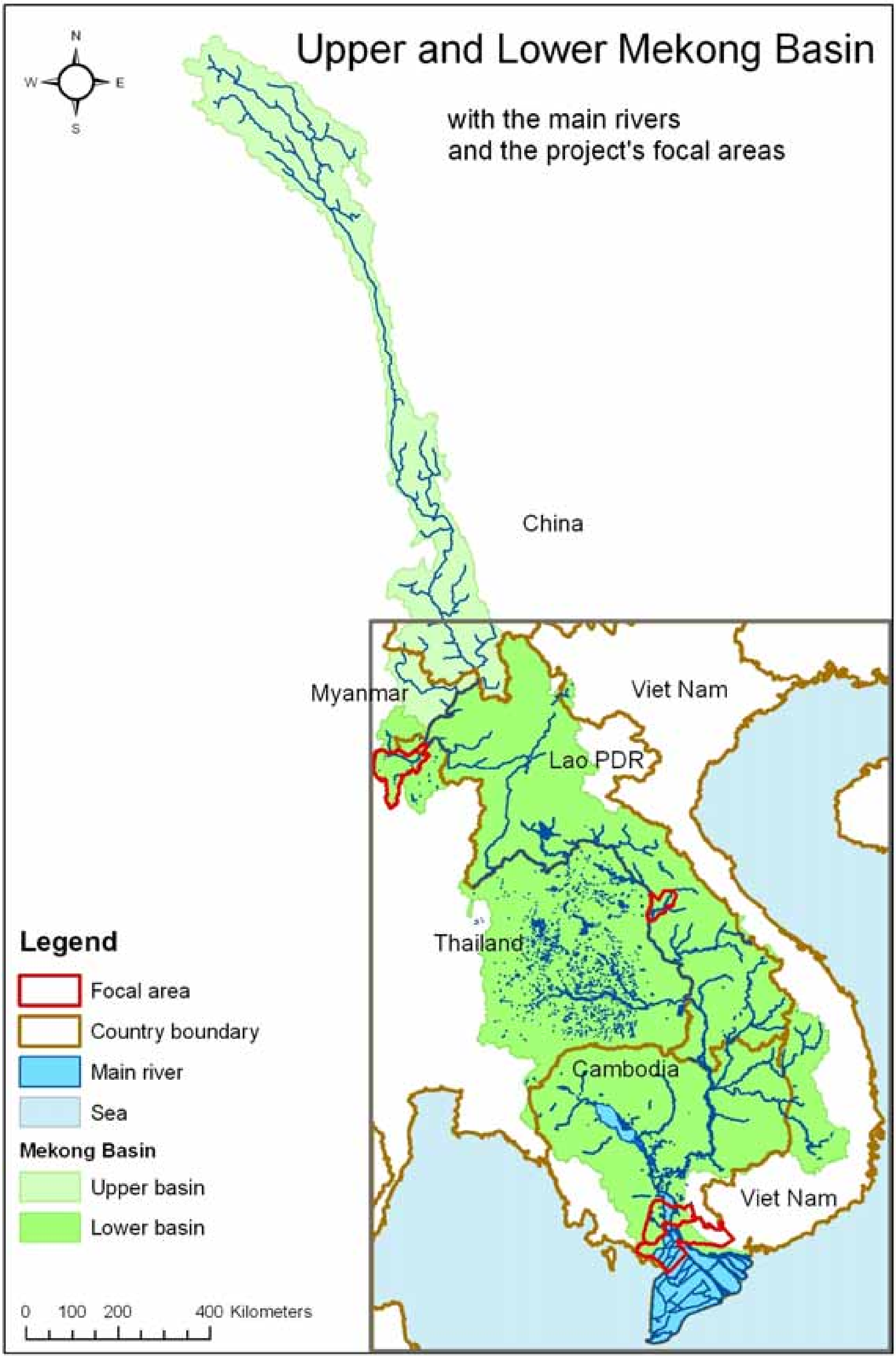
Draft Final Report

HASKONING NEDERLAND B.V.

UNESCO IHE

CONTENTS

APPENDICES



GLOSSARY

See Appendix 1.

ABBREVIATIONS

N.B. Abbreviations that occur only once and that are explained in the text are not included in the table below.

ADB	Asian Development Bank
ADPC	Asian Disaster Preparedness Center
BCM	Billion Cubic Meters
BDP	Basin Development Planning
BPG	Best Practise Guidelines
CBA	Cost Benefit Analysis
CBDRM	Community Based Disaster Risk Management
CNMC	Cambodian National Mekong Committee
d/s	downstream
DARD	Department of Agriculture and Rural Development
DSF	Decision Support Framework
EC	European Commission
EU	European Union
FHA	Flood Hazard Assessment
FMM	Flood Management and Mitigation
FMMP-C2	Flood Management and Mitigation Programme, Component 2
FPS	Flood Proofing System
FRA	Flood Risk Assessment
FV	Future Value (economic analysis)
GIS	Geographic Information System
HEC	Hydrologic Engineering Center
HH	Household(s)
IFRM	Integrated Flood Risk Management
IKMP	Information and Knowledge Management Programme
ISIS	Hydrodynamic simulator for modelling flows and levels in open channels and estuaries
IWRM	Integrated Water Resources Management
JICA	Japan International Cooperation Agency
KOICA	Korean International Cooperation Agency
LMB	Lower Mekong Basin
LMD	Lower Mekong Delta
LXQ	Long Xuyen Quadrangle (Vietnam)
MAFF	Ministry of Agriculture, Fisheries and Forestry
MARD	Ministry of Agriculture and Rural Development
MCM	Million Cubic Meters
MLUPC	Ministry of Land Management, Urban Planning and Construction
MONRE	Ministry of Natural Resources and Environment
MOWRAM	Ministry of Water Resources and Meteorology
MRC(S)	Mekong River Commission (Secretariat)
MSL	Mean sea level, the average (mean) height of the sea, with reference to a suitable reference surface
NAP	Navigation Programme (MRC)
NCDM	National Committee on Disaster Management
NEDECO	Netherlands Engineering Consultants
NMC	National Mekong Committee (NMCs are not part of the MRC 1995 Agreement, are structured differently in each country and are funded by their respective countries)
NPV	Net Present Value (economic analysis)

O & M	Operation and maintenance
PDR (Lao)	(Lao) People's Democratic Republic
PDS	Project Description Sheet (ProDIP)
PDWRAM	Provincial Department of Water Resources and Meteorology
PoR	Plain of Reeds (Vietnam)
ProDIP	Project Development Implementation Plan
PV	Present Value (economic analysis)
RFMMP	Regional Flood Management and Mitigation Programme
RN	Route Nationale (National Road)
SBF	Se Bang Fai (Lao PDR)
SIWRP	Southern Institute of Water Resources Planning
SWAT	River basin scale model quantifying the impact of land management practices in large, complex watersheds
TA	Technical Advisor
u/s	upstream
UNDP	United Nations Development Program
USD	US$
VND	Vietnamese Dong
VRSAP	Vietnam River Systems and Plains (hydrological/ landuse model)
WUP	Water Utilisation Programme

SYMBOLS IN FORMULAS

N.B. Symbols that occur only once and that are explained in the text are not included in the table below.

A	cross-sectional area of river (m2)
B	channel or river width at water surface (m)
C	Chèzy coefficient for hydraulic roughness (m/s)
c	cohesion (kN/m2)
c'	effective cohesion (kN/m2)
D	diameter or thickness of protection unit (m)
D_n	grain size diameter corresponding to n% by mass of finer (mm) particles
e	distance between pile axes (m)
F	force (kN)
Fr	Froude number (-)
g	acceleration due to gravity *(m/s2)*
H_{dcs}	design wave height (m)
H_s	significant wave height (m)
h	(local) water depth (m)
I	water level gradient (-)
I	hydraulic gradient in soil (-)
K_h	depth factor (-)
K_s	slope factor (-)
K_t	turbulence factor (-)
k	wave number *(11m)*
kg	permeability of geotextile *(m/s)*
ks	Nikuradse sand equivalent coefficient of roughness (m)
L	length of pile (m).
L_o	wave length in deep water (m)
L_f	fetch length (m)
n	cotangent of transverse bed slope (-)

O_n	opening size of a geotextile (µm)
p	permeability of groynes (-)
Q	water discharge (m3/s)
q	specific discharge (m3/sm)
R	hydraulic radius (m)
r	co-ordinate along bend radius (m)
S_g	spacing between groynes (m)
T	wave period (s)
t	time (s)
t	wall thickness (mm)
U	circumference (of piles) (cm)
u	depth-averaged velocity *(m/s)*
Ui	depth-averaged flow velocity at upstream boundary of control *(m/s)*
ii	cross-sectional and depth-averaged flow velocity (m/s)
U_b	bottom velocity (m/s)
V_{fa}	volume of falling apron per linear metre protected bankline (m3/m)
W_{fa}	width of falling apron (m)
Y_s	maximum local scour depth (m)
$Y_{(t)}$	time-dependent local scour depth (m)
γs	specific weight of solids *(kN/m)*
γw	specific weight of water *(kN/m3)*
γ	specific weight of solids in submerged condition *(kN/m3)*
Δ	relative submerged sediment density (-)
ΔH	head loss (m)
Δp	pressure gradient *(kN/m2)*
ε_s	angle of repose (degree)
ρ_s	density of protection material (kg/m3)
ρ_w	density of water (kg/m3)
σ	total normal stress *(kN/m)*
σ	effective normal stress *(kNlm2)*
τ	shear strength *(kN/m2)*
τ	f skin friction *(kN/m2)*
φ	*P* angle of internal friction (degree)
φ	*p'* effective angle of internal friction (degree)
ψcr	critical Shields parameter for initiation of motion (-)

SYMBOLS IN PAGE MARGINS

The FMMP-C2 guidelines contain in the left margins symbols for quick reference. The symbols indicate:

A. Type of text/ content;
B. A project stage.

A) The report texts have been categorised into four groups. These groups are as follows:

I) Project background/ Report info

Text on the FMMP-project and its background, or explanation on the report structure or content.

II) Theory

Theory behind the proposed/ applied methods and guidelines.

III) Example

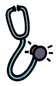
Example of the proposed/ applied methods and guidelines.

The fourth group comprises the remainder of texts and concern methodology and theory adapted/ applied to the Lower Mekong Basin, i.e. the guidelines. These guidelines are to be applied in one of the five project stages described below (B).

B) A project consists in general of five phases (see Appendix 2). Project FMMP-C2 encompasses almost exclusively Phase 2: Planning/ Development/ Design. This phase can be subdivided in the following five stages:

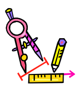
a) Preliminary/ prefeasibility study

b) Feasibility study & overall planning



c) Preliminary design

d) Detailed design & detailed planning



e) Construction/ bid documents

Any part of a guideline falling outside the scope of the five phases above will be marked with:

Sometimes more than one stage/ symbol may apply to a section.

CHAPTER 1

INTRODUCTION

1 INTRODUCTION

1.1 Guide to the reporting structure of the Flood Management and Mitigation Programme - Component 2, Structural Measures and Flood Proofing

Component 2 on Structural Measures and Flood Proofing of the Mekong River Commission's Flood Management and Mitigation Programme was implemented from September 2007 till January 2010 under a consultancy services contract between MRCS and Royal Haskoning in association with Deltares and Unesco-IHE. The Implementation was in three stages, an Inception Phase, and two Implementation Stages. During each stage a series of outputs was delivered and discussed with the MRC, the National Mekong Committees and line agencies of the four MRC member countries. A part of Component 2 - on 'Roads and Floods' - was implemented by the Delft Cluster under a separate contract with MRC. Component 2 prepared five Demonstration Projects which have been reported separate from the main products.

The consultancy services contract for Component 2 specifies in general terms that, in addition to a Final Report, four main products are to be delivered. Hence, the reports produced at the end of Component 2 are structured as follows:

Volume 1 **Final Report**

Volume 2 **Characteristics of Flooding in the Lower Mekong Basin**
Volume 2A *Hydrological and Flood Hazards in the Lower Mekong Basin;*
Volume 2B *Hydrological and Flood Hazards in Focal Areas;*
Volume 2C *Flood Damages, Benefits and Flood Risk in Focal Areas;*
Volume 2D *Strategic Directions for Integrated Flood Risk Management in Focal Areas.*

Volume 3 **Best Practice Guidelines for Integrated Flood Risk Management**
Volume 3A *Best Practice Guidelines for Flood Risk Assessment;*
Volume 3B *Best Practice Guidelines for Integrated Flood Risk Management Planning and Impact Evaluation;*
Volume 3C *Best Practice Guidelines for Structural Measures and Flood Proofing;*
Volume 3D *Best Practice Guidelines for Integrated Flood Risk Management in Basin Development Planning;*
Volume 3E *Best Practice Guidelines for the Integrated Planning and Design of Economically Sound and Environmentally Friendly Roads in the Mekong Floodplains of Cambodia and Vietnam*[1]*.*

Volume 4 **Project development and Implementation Plan**

Volume 5 **Capacity Building and Training Plan**

Demonstration Projects
Volume 6A *Flood Risk Assessment in the Nam Mae Kok Basin, Thailand;*
Volume 6B *Integrated Flood Risk Management Plan for the Lower Xe Bangfai Basin, Lao PDR;*
Volume 6C *Integrated Flood Risk Management Plan for the West Bassac Area, Cambodia;*
Volume 6D *Flood Protection Criteria for the Mekong Delta, Vietnam;*
Volume 6E *Flood Risk Management in the Border Zone between Cambodia and Vietnam.*

The underlying report is **Volume 3C** of the above series.

1 Developed by the Delft Cluster

1.2 The Best Practice Guidelines for Structural Measures and Flood Proofing

These BPGs are prepared as a checklist of planning, functional requirements, design criteria and specifications of specific flood control structural works including assessment and needed actions to be taken in case of increased flood risk.

The Best Practice Guidelines (BPG) for development and design of structural and flood proofing measures developed under FMMP-C2 to provide policy-makers, managers and FMM professionals in MRC and national line agencies with a common knowledge base to apply in:

- policy formulation,
- strategy and plan development and
- project design and evaluation for flood development and design of structural and flood proofing measures, in the Lower Mekong Basin (LMB).

Each member country of the MRC has its own policy and legal frameworks that will guide or regulate the planning, evaluation and implementation of structural and flood proofing plans and measures. The first step in preparing the BPG was the collection of existing relevant guidelines used in the countries.

In Cambodia there are no guidelines for bank protection, flood proofing and existing guidelines on urban planning and construction are very generic and do not include flood risk management. Some design guidelines have been developed under ADB-sponsored Flood Emergency Rehabilitation Project.

In Lao PDR sector guidelines have to be produced by the sector ministries. However, those guidelines have only been made for hydropower projects, roads and mining projects. Under a decree, the WREA, becomes responsible for the drafting of guidelines in Lao PDR although specific guideline for flood protection works is not foreseen. The new River Works Department would require guidelines for design, maintenance of bank protection as well as for preparation of technical specifications for river works.

In Vietnam there are guidelines for the collection of flood damage data in use by DDMFSC. Old guidelines date back to 1996. New guidelines prepared in 2006. For Mekong delta: Design of sea dikes 14 TCN-130-2001 standard applies. Guidelines for river dikes in Mekong Delta have not yet been finalized. For the design guidelines of river bank protection works reference is made to MARD. The Ministry of Construction (MOC) has design guidelines for flood proofing.

In Thailand the RID would also favour capacity building and training at stakeholder level: to create understanding about the concept of floods, the effect of human interventions, land use, soil saturation, risk awareness, guidelines for land use (planning), to avoid ‘adverse’ structures in flood prone areas, damage reduction at farm level, simple protection methods (sandbags), etc. The Public Works Department does not have plan or strategy according to the interviewed person, and there are no specific design guidelines for river bank protection and dikes.

Best practice guidelines for design, maintenance and operation of flood control structural measures in the Lower Mekong Basin shall be also harmonised and standardized. Structural measures shall be designed to similar level (frequency and risk) in the Mekong countries to ensure that the risk of failure of those measures will not generate flood transboundary impacts. This can be of special importance between Cambodia and Vietnam.

This report consists of three main parts:

- Guidelines for Flood proofing (Section 2),
- Guidelines for Development and Design of Bank Erosion Control Measures (Section 3),
- Guidelines for Flood Embankments / Dikes (Section 4).

1.3 The Best Practice Guidelines and project phases/ stages

In order to manage an engineering project properly, it is normally divided in project phases. Common is a division in the following five phases:

1. Initiation
2. Planning/ Development/ Design
3. Production/ Execution
4. Monitoring/ Control
5. Closure

The Best Practise Guidelines are almost exclusively applicable to Phase 2: Planning/ Development/ Design. This phase, its stages and the associated symbols used in the guidelines are elaborated in: Appendix 2.

CHAPTER 2

FLOOD PROOFING

2 GUIDELINES FOR FLOOD PROOFING

2.1 Introduction

Damage to infrastructure, roads, bridges, buildings, housing, equipment, and other components/ utilities are most commonly caused by floodwater inundation. Floodwaters disrupt communication links and inundate large settlement and urban areas, saturating the building materials and its contents. The floodwater is usually contaminated by a number of substances, such as sewage and other hazardous materials.

Flood proofing measures are considered flood vulnerability reduction measures in the context of IFRM. This can be justified because of the anticipated urban and infra-structural development in the Mekong basin, adding significantly to the investment value of property and services needing protection from floods.

The guidelines are intended to aid anyone involved in planning and design of buildings, infrastructure, disaster management and maintenance of key infrastructure (i.e. Roads and bridges), ensuring that loss of life and damage to those buildings and structures is reduced. Vulnerability analysis in flood-prone areas is carried out separately for different building types and for infrastructure, such as roads, bridges, water and electricity supply systems, sewerage systems etc.

In the process of developing the guidelines it is recognized that integrated planning of structural works requires a strong participatory process, with central, provincial, and local government agencies sharing the responsibilities with local stakeholders and their representatives. The guidelines may also assist agencies involved in planning and authorizing river bank protection works within the LMB.

2.2 How to use these guidelines

The primary target audience for these Guidelines is government professionals and leaders at all levels engaged in the planning and implementation of flood risk mitigation programmes. Other professionals and local government organizations in charge of urban planning, educators and authorities dealing with disaster management may also find these guidelines useful for widening the scope of their work.

Proper design of flood proofing measures for infrastructure or facilities and provision of the necessary equipment and flood proofing devices represent important components of a successful flood proofing program to be addressed in the guidelines. However, these actions alone cannot ensure success. It is still necessary for all measures to be properly installed within the limited amount of time that is available prior to flooding. Therefore, the best means of ensuring that this can be done is through the preparation and implementation of a flood emergency preparedness plan.

The guidelines can be also used in conjunction with preparedness plans to make them more comprehensive and specific. The plan must cover every aspect of the flood proofing procedure ranging from the initial receipt of a flood warning to post flood cleanup requirements. Each activity, in the guidelines, must be clearly specified in its order of occurrence, with enough detail to ensure that the personnel who will be required to perform these activities will know exactly what to do and how to do it. One of the items that the flood emergency preparedness plan must consider generally involves all personnel required to install the flood proofing measures. For those times when the structure is not occupied, the plan should include provisions for the efficient notification and assembly of personnel that are responsible for initiating all contingent and emergency flood proofing measures.

Public and community participation are also two important elements of success when implementing flood proofing measures. The measures should also be community-specific, integrated with existing disaster warning and response systems, focused towards information on prevention, mitigation and long-term recovery, established as on-going process, and addressed towards the most vulnerable people.

Co-ordination between the community agencies, representing its citizens and the village or district authorities, is essential for implementation of flood proofing measures. Specific tasks are vested with the community, while others belong to the domain of local authorities.

Apart from co-ordination, all concerned agencies need to develop knowledge and skills on how to use and apply the guidelines for reducing the possibility of damage and casualties during floods. Learning while doing is a practice accepted in community participation, but some formal training programs may be also required for the full benefit of using the guidelines.

2.3 Purpose and scope of the guidelines

The purpose of the Guidelines is to offer a set of planning and design approaches to promote the integrated use of known engineering, structural measures, rather than to propose ready-made solutions which depend on the flooding site-specific conditions of each particular area. Flood proofing guidelines are meant for line agencies and local governments and aiming at solutions that satisfy the requirements of environmental and economic sustainability. Preparation and utilization of the guidelines requires public participation.

Existing flood proofing measures in the Mekong floodplains is in most of the cases to the result of traditional indigenous expertise and technology, which deserves to be studied, improved and promoted. In some other areas the knowledge is to be developed in these guidelines and shared within the member countries adding new technologies and approaches.

Flood proofing guidelines are developed for riverine flooding and flooding in non-wave velocity areas. The main considerations are focussed on flood characteristics including depth, velocity, duration and rate-of-rise, and their effects on the various flood proofing techniques. Coastal flooding forces and phenomenon such as wave generated impacts or erosion are not addressed in these guidelines. The information presented in these guidelines has been developed specifically to reduce flooding risk problems associated with infrastructure (roads, bridges), residential and non-residential (industrial, commercial, and institutional) structures, or to minimize the damage caused by the waters that get in.

Flood proofing will be an initial step in reducing vulnerability to flooding in unprotected settlements and urban areas in unprotected floodplains. The guidelines will be applicable to different areas in the Lower Mekong Basin and preferable will be accompanied by a demonstration project for testing and evaluating various flood proofing measures.

Many factors influence the decision making process for determining the feasibility of flood proofing options. It is generally accepted that the most suitable solution would be one that:

- Provides for reduction in damages for the selected or required design level and does not result in increased damages to other property.
- Is responsive to all applicable floodplain regulations.
- Provides for the safety of persons on and adjacent to the site.
- Is cost effective with regard to installation, maintenance and operation of the system.
- Is acceptable to the property owner, employees and the general public with regard to operational efficiency and impacts on the surrounding environment.

The guidelines will serve to develop a flood proofing plan that can meet these performance goals. It is necessary to conduct a systematic evaluation of physical, social, and economic factors to determine its feasibility. In most situations it will be necessary to collect basic information on each of the major categories shown in Figure 2.1.

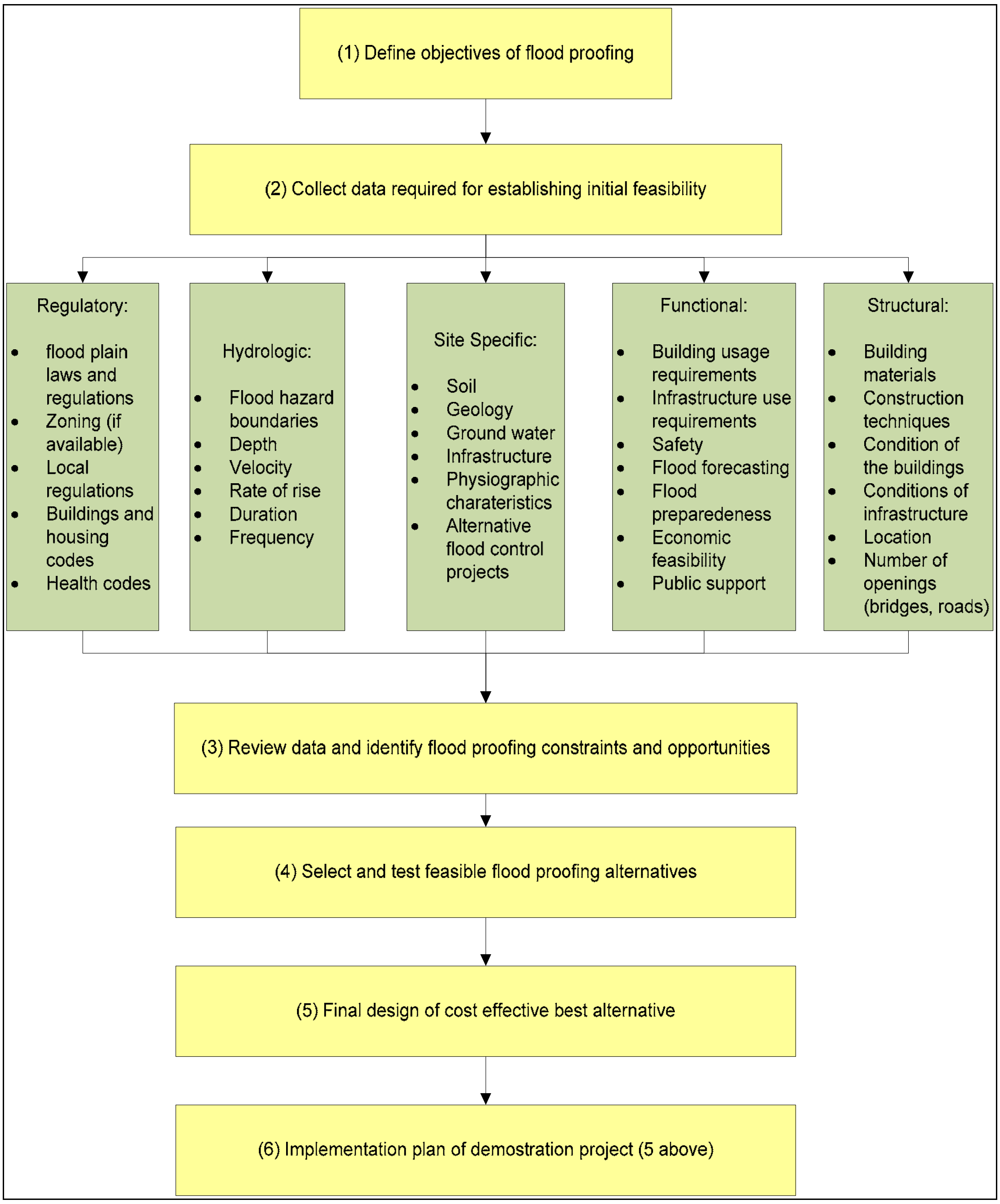


Figure 2.1 Flood proofing planning and design general process[2]

[2] Adapted from FEMA-flood proofing publication non-residential structures, May 1986

2.4 Definition of key concepts

2.4.1 Flood proofing in buildings

Vulnerability analysis of buildings is divided upon the type of building and includes evaluation of the resistance to the force of water (hydrostatic load, uplift, hydrodynamic load) and of the changes of material characteristics when immersed in water (quality of mortar, presence of fine sands and expansive clay at foundations). Public buildings that are used as population shelters must have floor space above the expected flood level. This can be done by constructing the building on natural or artificial high grounds, by placing the building on columns and stilts or by providing access from outside via staircase to the upper floors.

2.4.2 Flood proofing of infrastructure

Flood damage to infrastructure elements can be caused by direct water forces, by erosion, or by combination of both.

Roads and bridges may be damaged easily by scouring caused by high water flows in two ways: the foundation can be washed away and the construction itself can be compromised. A vulnerable part of road system is the crossing of culverts and bridges. Insufficient openings in bridges will lead to higher water levels upstream. The river bed upstream and downstream of the bridge should be consolidated by local scour prevention measures. Most techniques that prevent river bed erosion fix the stream bed by stabilising the embankment (by means of masonry or vegetation).

The physical damage of water supply systems is concentrated on the intake points and the locations where the main supply crosses riverbeds. The quality of potable water in conduits is affected by silting and pollution. The problem of water contamination can be easily solved by constructing the pipes above the flood level. The same principle applies to electrical supplies, sewer pipes and telephone lines. Elevation above flood level secures the continuity of operation of those systems.

For the purpose of this guideline, flood proofing of buildings should primarily be viewed as any method or combination of methods that serve to meet the elevation or watertight flood proofing standards accepted by member countries for non-residential structures. Many of these same concepts and methods can also be applied to existing non-protected construction to reduce or reduce or eliminate future flood damage.

Flood proofing techniques will be classified in the guideline on the basis of the type of protection that is provided as follows:

1. permanent measures (always in-place, requiring no action if flooding occurs);
2. contingent measures (requiring installation prior to the occurrence of a flood), and
3. emergency measures (improvised at the site when flooding occurs).

However, it should be recognized that these classifications are not always clearly defined. For example, a flood levee or wall would normally be considered to be a `permanent' protection measure even though the success of a particular floodwall design may be dependent upon installation of one or more gates to seal openings. The advantages and disadvantages of alternative flood proofing techniques and specific information that can be used to develop preliminary design concepts for the techniques will be also described in the guideline.

2.4.3 Permanent flood proofing

For the purpose of this outline guideline permanent flood proofing measures are those, which, once installed, require no further action to be taken when flooding occurs. In general, permanent flood proofing measures are most effective when used in areas that are subject to frequent flooding, relatively high flood depths, or where insufficient flood warning time is available to implement contingent flood proofing measures. For several reasons, permanent flood proofing measures are preferred over contingent or emergency type techniques.

The application of permanent elevated walkways greatly improves accessibility between houses and important public buildings, such as flood shelters. For proper functioning, these walkways have to be raised above the average flood level

Furthermore, consideration will be given to operation and maintenance costs associated with the permanent flood proofing system because there is no need to store or maintain parts and supplies that would be required for contingent and emergency flood proofing techniques, and there is no need to train and maintain manpower for installing the flood proofing equipment. Also, permanent flood proofing measures will often meet the minimum floodplain management requirements in cases of flood insurance policies.

There are also some disadvantages associated with permanent measures. Initial construction costs may be relatively high, particularly for some existing structures and for large floodwall or levee protection projects. Another primary disadvantage to permanent flood proofing is that adjustments made to prevent water from entering a facility may restrict access to and use of certain parts of the structure.

2.4.4 Contingent flood proofing measures

Although permanent flood proofing measures certainly have advantages in terms of providing protection from flood damages, they can also have some disadvantages such as restricted access and inefficient utilization of space. When these factors represent major obstacles to the application of permanent flood proofing techniques, the use of contingent flood proofing measures may be appropriate.

In the guideline contingent flood proofing measures are those that require some type of installation, activation, or other preparation immediately prior to the occurrence of a flood. These measures can consist of flood shields, watertight doors, and moveable floodwalls. In some cases, flood protection provided by levees, floodwalls, or waterproof cores will require access openings that must be sealed with shields or doors during flood events. Obviously, the success of this type of system is dependent upon the ability to install and secure the flood shields and other protective devices prior to flooding.

The primary advantages and disadvantages of contingent flood proofing systems will be highlighted in the guideline. One advantage is that components may be moved aside or stored during non-flood periods allowing full access to the doors, windows, and other openings. Nevertheless, although convenience, cost, and adaptability provide major incentives to the use of contingent flood proofing measures, there are several potential disadvantages that must be considered in the guideline. The major disadvantage is that a contingent system is subject to human error associated with applying the system's components.

2.4.5 Constructing barriers

Constructing barriers is an effective approach to stopping floodwaters from reaching the damageable portions of structures.

Two techniques are employed in constructing barriers. The first technique involves constructing free-standing barriers that are not attached to the structures. The three primary types of free-standing barriers used to reduce flood damages are:

- Berms
  A berm is typically an earthen structure, constructed from local compacted fill that stops flood water from reaching the building. To be effective over time, berms must be constructed out of suitable materials (i.e. impervious soils) and with correct side slopes.
- Levees
  Levees, which are similar to berms, are also earthen structures of compacted local fill. Levees are usually constructed along riverbanks to prevent the floodwaters from spilling over and flooding structures. Berms, on the other hand, serve the same purpose but usually are constructed closer to the structures themselves. Both berms and levees are generally appropriate for flood proofing a home where floodwaters are less than 6 feet deep.
- Floodwalls
  Floodwalls are usually constructed out of reinforced concrete and anchored into the ground. Floodwalls, because of their greater cost, are usually not used to protect homes. Berms, levees, and floodwalls may not be appropriate for homes with basements since they are more susceptible to under-seepage.

The second technique that can be used to construct a barrier against floodwaters is known as "dry flood proofing ."

- With this technique, a building is sealed so that floodwaters cannot get inside.
- All areas below the flood protection level are made watertight. Walls are coated with waterproofing compounds or impermeable sheeting.
- Openings such as doors, windows, sewer lines, and vents are closed with permanent closures or removable shields, sandbags, valves, etc.
- This flood proofing technique is appropriate only where floodwaters are less than 2m since most walls and floors in buildings will collapse under higher water levels.
- A professional engineer should be consulted when considering dry flood proofing since threat of collapse from hydrostatic pressure (the pressure on standing water) is a major concern with this technique.
- The dry flood proofing technique is not as successful on buildings with crawl spaces or basements since those structures are difficult to protect from under-seepage.

If barriers are not possible the alternative is to prepare plan for wet flood proofing which involves modifying a structure to allow floodwaters inside, but ensuring that there is minimal damage to the building's structure and to its contents.

- Wet flood proofing allows the floodwaters to enter the structure. The building is modified so that utilities and furnaces are protected or relocated to an area above the anticipated flood level.
- Wet flood proofing is often used when dry flood proofing is not possible or is too costly.
- Wet flood proofing is generally appropriate in cases where an area is available above flood levels to which damageable items can be relocated or temporarily stored. This approach is also appropriate for structures with basements and where other flood proofing measures will not be effective.

2.4.6 Emergency flood proofing measures

Emergency flood proofing to be described in the guideline includes techniques that can be initiated on relatively short notice using stored and/or natural materials to prevent flooding.

Emergency methods that will be presented in the guidelines include sandbag dikes, earth-fill crib retaining walls and stop log barriers. These techniques are characterized by their ability to be initiated on relatively short notice using previously obtained and stored materials.

The primary advantage of an emergency method is low cost. Sand and timber are the primary materials and although these measures labour intensive, volunteers are often used. These methods are most effective in flood areas where water velocities are low and depths are shallow, and where floodwaters rise slowly. A major disadvantage of emergency measures is that substantial advance warning is required to mobilize personnel and install emergency barriers.

2.5 Planning considerations for developing design of flood proofing measures

2.5.1 General

To develop an effective flood proofing scheme for infrastructure or a facility, several hydrologic factors must be properly evaluated. The factors to include in the guideline are related to the regulatory floodplain boundaries and the anticipated flooding characteristics for the site such as flood velocity, duration, rate of rise, and frequency. This type of hydrologic base data may be available from several line agencies or may have to be independently determined for the specific site.

For those areas where data is not available, hydrologic specialists can develop the necessary design information from site specific investigations. These may involve development of hydrologic relationships in some cases using knowledge of historical flood events and the physiographic conditions of the site and watershed.

Detailed information regarding the specific structural loading impacts that floodwaters can exert on structures needs to be provided to comply with required flood proofing performance criteria. Required flood proofing performance criteria will be included in the guidelines. Much of the information regarding design criteria, the properties of materials, the values of flood water design forces, and other considerations have been adopted from standard engineering references, building codes, and other documents.

A general overview of considerations associated with other hydrologic factors[3] that shall be considered when using these guidelines.

Successful application of flood proofing requires knowledge of:

- Techniques and materials
- Code requirements
- Engineering Design
- Certification (for elevation and non-residential dry flood proofing)
- Accurate risk estimation (mapping)

[3] Best Practice Guidelines for Flood Risk Assessment in the Lower Mekong Basin. MRCS – April 2009

Flood proofing, and the degree to which it is employed, is also affected by:

- Policy that facilitates, encourages and rewards flood proofing
- Availability of funding and/or financing
- Inspiration (access to completed flood proofing installations)
- Justification (quantification of damages avoided)

Policy considerations during planning include a full range of codes, ordinances and other regulations relating to the use of land and construction within floodplain limits. The term encompasses zoning ordinances, subdivision regulations, building and housing codes, encroachment laws and open area (space) regulations.

In many cases there are also sub-division regulations which are regulations and standards established by provincial or local units of government with authority granted under special powers or enabling law, for the subdivision of land in order to secure coordinated land development, including adequate building sites and land for vital community services and facilities such as streets, utilities, schools and parks.

2.5.2 Flood hazard boundaries

Official floodplain zoning maps showing the extent and boundaries of the primary and secondary flood hazard areas shall be prepared by the countries and officially approved to form part of regulations for floodplain management and flood proofing. Furthermore, the zoning map shall be also used in combination with a regulatory flood datum for determining the elevation above mean sea level/ PWD to which flood proofing protection shall be provided.

The primary flood hazard area is defined, for the purpose of this guideline, as the lands adjoining the river or a channel or watercourse that would be covered by flood water during a regulatory flood. Secondary hazard area is the land beyond the run out line of the regulatory flood that could be affected by higher floods and by undergroundwater travel, back flooding of sewerage, drainage, domestic water supply, and public utility systems, or cause other flood related problems during a regulatory flood.

The proper identification of flood hazard boundaries is significant in that these boundaries define the regulatory floodplain, and the relative extent of flood hazard within various floodplain zones. Flood hazard boundary classifications must be investigated to determine areas that may restrict the use of certain flood proofing measures such as areas identified as the regulatory floodway or areas that are subject to high flood velocities.

In accordance with the country requirements, the design flood (1: x years) is to be used as the basis for flood proofing designs for new and substantially improved construction. Information can be also obtained zoning maps, or through analyses performed by hydrologic/ hydraulic specialists.

Zone classifications must be prepared for different areas in the floodplain[4], i.e. areas for flood proofing determined to be outside 500year or 1:100year floodplain (determined to be outside the 1% and 0.2% annual chance floodplains). However, it shall be always kept in mind that *mother nature doesn't read maps* and bigger floods can occur.

[4] Refer to section 2.4.7, flood mapping – BPG-FRA

2.5.3 Flood depth

The depth of flooding associated with the required regulatory flood or protection level is one of the primary factors that influence flood proofing design. This factor must be determined to design against overtopping of the system (freeboard consideration) and to formulate a design that can withstand associated loading pressures.

There is considerable variation among flood proofing techniques regarding the maximum flood depth for which each method can be applied. Elevation on fill has been used to protect against flooding depths in excess of 3 m depending upon the characteristics and availability of fill material. The upper limit of permanent and contingent closure systems is generally limited by the building's wall or floor strength and cost considerations.

Estimates of flood depths for a particular site can normally be inferred from flood studies or similar hydrologic reports or may be obtained from a flood profile. For flood proofing purposes, the depth of flooding may be calculated by subtracting the elevation of the lowest grade adjacent to the structure to be flood-proofed from the Base Flood elevation as determined from an appropriate flood profile, alternatively flood depths may be determined through site-specific evaluations or historical information.

2.5.4 Flow velocity

In addition to depth of flooding, velocity has a direct relationship to the amount of force applied to a structure by floodwaters. Water velocity also can result in higher depths of flooding on the upstream side of a building. An allowance for freeboard, particularly on upstream side of a facility, can address this concern. The velocity of flow also determines the force that could be applied to the structure through the impact of objects being carried by the flood. High velocities also have an impact on the design of levees or embankments that can be subject to local scour and lateral erosion. Experience has shown that flood proofing is generally not appropriate in areas where flood velocities exceed 2.5 m/s.

2.5.5 Rate of water rise

The rate of rise of a flood is an expression of how rapidly water depth increases during a flooding event. This factor is important when determining whether sufficient lead-time is available to permit the use of contingent flood proofing methods; and for designing appropriate emergency evacuation plans. The rate of rise of floodwaters can be derived from a streamflow hydrograph for the area under consideration that relates flooding depth to time. The rate of rise can be determined from the hydrograph by the slope of the hydrograph at the depth and time in question. Information required for determining rate of rise may be available from existing hydrologic studies, onsite investigations, local offices, or historical records.

2.5.6 Flood duration

The duration of a flood is an important flood proofing consideration because it affects the saturation of soils and building materials, seepage rates, and the amount of time facilities might be inaccessible. Flood-proofed structures that will be subjected to long periods of flooding must be carefully designed to reduce the risk of failure as a result of soil or building material saturation, internal pump system failures, or similar problems related to extended flood duration. The duration of flooding can be derived from an applicable streamflow hydrograph or, in some cases, from historical flood information. The depth at which damage from flooding begins at a particular structure can be plotted on the hydrograph. The amount of time that the water level remains above this elevation indicates the duration of flooding.

2.5.7 Frequency

The frequency of flooding must also be considered in flood proofing a structure.

Frequency of flooding is defined as the probability (in percent) that a random flood event will equal or exceed a specified magnitude in a given time period, usually one year. The frequency of flooding can be statistically determined using historical records of flooding at the location under consideration. The owner of a structure subject to a high frequency of flooding may choose to install permanent flood proofing measures instead of contingent measures to reduce operational costs and the chance for system failure resulting from an inadequate response.

2.5.8 Freeboard

In flood proofing measures the safety factor is usually expressed in metres above a certain flood level. Freeboard compensates for the many unknown factors that may increase flood levels beyond the calculated level, i.e. waves, debris clogging culverts or bridges, short history of known water levels, etc. Freeboard also provides a factor of added safety for when the actual flood levels are higher than that calculated for the "1: x year flood." The 100year flood level has a 1percent chance of being equalled or exceeded every year.

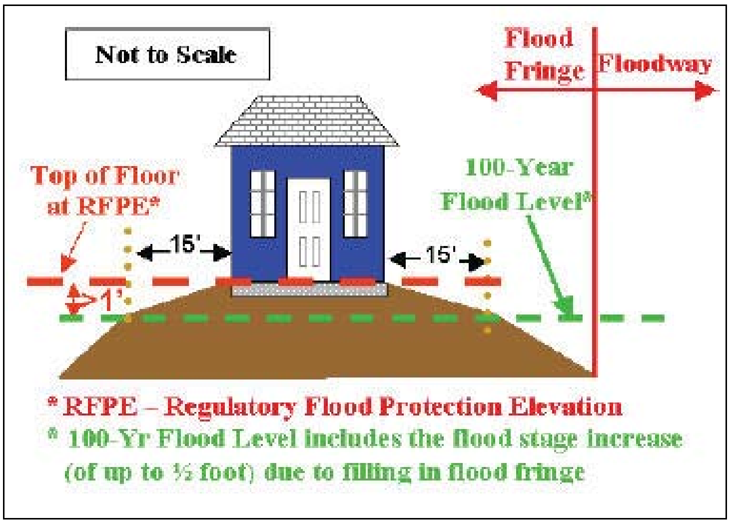


Figure 2.2 Example of freeboard following regulatory flood protection elevation[5].

Based on Figure 2.2 the minimum standards for infrastructure are defined by the regulations in each country for the type of structures in the floodplain/ floodzone classification established for the floodplain (1: 500 years, 1:100 years or less). If a return period of 100 years is used, the following minimum standards and definitions can be applied for planning and design:

- Regulatory Flood Protection Elevation RFPE = 100yr flood level + flood stage increase due to filling in flood fringe + 0.3m freeboard (min);
- Lowest floor (including basement) of structures is at RFPE or higher;
- Fill at 100year flood level (including stage increase due to filling in flood fringe) or higher extends at least 5 m in all directions;
- NO fill in floodway;
- Access road/ driveway shall not be lower than 0.6m below RFPE.

The greater the freeboard, the less depth of flooding that results in case of higher floods.

[5] Department of Natural Resources, Minnesota.

2.5.9 Site specific factors

In addition to the collection of information that defines the extent and characteristics of floodwaters, there are several other site-specific features that must be investigated as part of a pre-design analysis of flood proofing alternatives. The designer must identify flood proofing constraints and opportunities associated with geologic, groundwater, and soil conditions, existing infrastructure, and physiographic characteristics of the project area. This is important when design ring-dikes as flood proofing of small village or dwelling because of seepage due to permeable soil strata, especially for sustained high river water levels.

2.5.10 Geology, groundwater, and soil conditions

The selection and design of most flood proofing measures requires an evaluation of geologic, groundwater, and soil conditions. Although geologic features do not generally represent a key design factor in flood proofing design, basic data should be collected to identify any major geologic constraints including presence of karst (sink-hole) features, faults, or extremely shallow depth to bedrock. Likewise, the depth of the groundwater table in the area should be determined because a high water table in combination with flooding conditions could have a significant impact on foundation and floor system design.

Soil characteristics will often have a major effect on the selection and performance of flood proofing systems. Factors that are of primary importance include permeability, erosion potential, slope stability, and bearing capacity. Soil characteristics are particularly important in determining the feasibility of elevating structures on fill material, the construction of earth berm levees, and foundation design for floodwalls and elevated structures. Final flood proofing design must be based on site-specific detailed soil analyses conducted by a qualified soils engineer.

2.5.11 Infrastructure

Existing road and utility systems can influence the selection and design of various flood proofing measures. For example, levees and floodwalls must be compatible with road, or water-borne transportation systems; and elevated facilities must be designed so that they are accessible to people and materials. In addition, the flood-proofed facility must be designed so that it is compatible with existing utility systems. Information concerning existing and planned road and utility systems that may influence flood proofing design may be obtained from local and provincial planning agencies and utility companies.

2.5.12 Physiographic characteristics of the area(s)

An analysis of the various physiographic features of a proposed flood proofing site is an important step in the identification of the best location for a new building or the location of a floodwall or levee. Characteristics that should be considered include the size and shape of the land areas, site elevations, slope, and existing drainage patterns. The physiographic characteristics of an area may have a significant impact on the feasibility of flood proofing systems that require a substantial amount of space, such as levees and fill used to elevate a structure. In addition, levees and earth fills, road embankments must be carefully designed so that they do not create a significant constriction of flood flows, thereby increasing hazards for other facilities in the area. Physiographic features can be determined from topographic maps, floodplain studies, and on-site investigations.

2.5.13 Functional, operational and economic factors

Viable flood proofing alternatives must be responsive to the functional usage requirements of the structure, the safety of the structure's occupants, and the reactions of local officials and citizens to the proposed measures. In addition, the ultimate test of feasibility lies in the relative cost of the measure weighed against the economic benefits to be gained by taking action. The main factors to take into account are:

- Usage requirements. The function of the structures and buildings. Critical facilities such as hospitals or important stations cannot function properly if access is restricted by floodwalls or some other flood proofing technique. The current and future use of the structure must be carefully evaluated in deciding to what degree access can be limited and in determining how long the facility can be closed during a flood and how well the effects of the design flood being exceeded can be tolerated.
- Safety. The relationship of various flood proofing options to occupant safety must be evaluated in the pre-design phase. In situations where a flood-proofed facility is likely to be completely surrounded by floodwaters, provisions must be made for the evacuation of all personnel and residents before flooding affects the structure
- Flood forecasting. A flood forecasting system must perform two functions: first, it must determine when a flood is imminent, and second, it must predict when specific areas will be flooded.
- Economic feasibility. Once it has been determined that flood proofing is feasible in terms of regulatory requirements and the physical characteristics of the floodplain and the structure, it is possible to identify the flood proofing program that is most cost effective. A cost effective plan would be one where the total cost of flood proofing (installation, operation, and maintenance) is less than the amount of physical flood damages, lost earnings, and other economic impacts that are likely to occur if the structure, building and/or area is not flood-proofed.

2.5.14 General cautions applicable to flood proofing

The following considerations shall be taken into account:

- Most flood proofing techniques should be formulated and designed by experienced personnel (engineers or contractors) to ensure adequate consideration of all factors that could affect the techniques' effectiveness.
- Flood proofing techniques cannot be installed and forgotten. Maintenance must be performed on a scheduled basis to ensure that the flood proofing techniques adequately protect the structures over time.
- Floods may exceed the level of protection provided.
- Flood proofing may not change the instable characteristic of the environment and the generally risk adverse behaviour of the rural population. As such it provides little development perspective and is a less preferred option by local stakeholders compared with flood protection. Nevertheless, this may still remain as an option in areas where protection cannot be immediately envisaged.

2.5.15 Implementing demonstration projects

Specific demonstration projects will be investigated for the purpose of refining and supplementing data. This guideline is intended to serve as a general technical guide on the selection of alternative flood proofing techniques. It must be emphasized that the actual design and construction of the various techniques should involve the services of a registered professional engineer or architect or experienced contractor.

2.6 Performance criteria

The performance criteria shall represent objectives that should be achieved in the design of flood-proofed infrastructure, non-residential structures and associated service systems. These criteria are applicable to the permanent and contingent techniques described above and including:

- elevation on fill or supporting columns, piles, posts, piers, or wall section,
- watertight construction (through the use of interior and exterior membranes or sealants; integrally waterproofed concrete construction; and/or a full range of closure and flood shield assemblies), and
- the use of floodwalls and earth levees.

Performance criteria shall be structured to indicate the desired attributes of a flood-proofed structure without reference to specific construction techniques or materials. This format has been selected to facilitate and encourage the development of a full range of traditional and innovative designs that are equally effective in reducing flood damages.

Provisions included in the criteria to be provided represent the minimum design requirements for flood proofing of non-residential structures. It must be understood that these criteria are generally limited to design factors that are directly related to flooding conditions. Therefore, the following performance criteria can only be used in association with all applicable local building codes and regulations.

Some of the performance criteria to be taken into account can be listed as follow:

- strength,
- stability,
- scour and deposition of debris,
- permeability and storm drainage,
- electrical systems,
- flood proofing operations,
- rescue operations.

2.7 Raised areas/ platforms

The schemes of raised earthen areas or platform may be implemented to provide shelter to people and livestock of the flood affected villages, which get marooned frequently resulting in acute hardship due to disruption of basic civic amenities and communication links.

Since the FMMP C2 aims to alleviate suffering of the people, the benefits of the schemes may be considered as social benefits and therefore a rigid benefit cost analysis on the lines of flood management schemes may not be sole requirement reflecting only the tangible damages avoided. However, both direct and indirect benefits by implementation of the scheme may be assessed and properly projected to justify the investment.

2.7.1 <u>Selection of area</u>

These types of schemes are indicated for areas which suffer inundation of homestead areas of villages at least once in 5 years. Homestead areas should be identified on the basis of reliable flood records of the past 10 years also viz. demarcation could be done based on level of submergence shown on contoured index maps of the area, frequency of submergence/ duration established by reliable flood records. Selections of village for such schemes are also to be supported by certified statements of damages suffered year wise. A reliable damage assessment has to form basis for the investment.

2.7.2 Design criteria

Top of platform
Top level of the platforms should be 0.6m above the flood level for 25 years frequency if the platform is to be constructed in unprotected areas. In case the platforms are on the countryside of embankments the freeboard is to be reckoned above the maximum water level observed due to drainage blockage with the proviso that platforms are generally at the same level as the top of the protecting levee adjoining.

Size
Size of the platform may be determined on the basis of 40 to 50 m^2 for each family plus 10% for animal and fodder plus 20% for internal passages, water and sanitary installations. These provisions can be subject to alteration on the basis of actual experience in other flood prone countries.

Drinking water
Provision for tube well at the rate of one tube well for 20 families may be made.

Sanitation facilities
One block of 4 toilets may be provided for 25 families with suitable disposal of wastes.

Link road
All platforms shall be sited so that connection to nearest all weather road/ service roads of flood embankments to provide all weather access is possible. The link roads may have 20 cm brick soling and 3.5 m top width. Provision of water transport (rural IWT) may also be made for such platform clusters. Operational aspects have to be considered to ensure boat operation remains sustainable.

CHAPTER 3

BANK EROSION CONTROL MEASURES

3 GUIDELINES FOR DEVELOPMENT AND DESIGN OF BANK EROSION CONTROL MEASURES

3.1 Introduction

The need of river training and bank stabilization along the Lower Mekong basin in Cambodia, Laos, Thailand and Vietnam arises from the fact that along some river stretches the condition of the river channel is unstable, i.e. they are not in a state of equilibrium with the governing physical processes. Unstable rivers reaches undergo permanent and rapid changes of the water and sediment regime and, hence, adjustment in depth, slope, width and planform. Both, river training and bank protection measures, which are strongly interrelated, have the objective to ensure a safe and efficient transport of water and sediments (suspended material and bed load) through a certain defined stretch of the river. A safe and expeditious passage of flood discharge must be possible during high water stages with a minimum of negative effects on surrounding areas.

The Mekong River has a variable width and it is geologically controlled almost along the whole length and local geology and related rock outcrops are very important. In the Upper Mekong in Lao PDR and Thailand floodplain is only marginally developed along most of the Mekong river reaches, and often the river reaches the floodplain levels only during extreme floods. Only one reach near Nong Khai and Vientiane has been identified as fairly alluvial (a reach where the influence of the bed rock outcrops appears almost to be negligible). On account of soil fertility and accessibility, most of the basin's population is concentrated in these alluvial areas.

Like many other rivers, the Mekong erodes its banks in many points. Erosion at the Mekong has not reached alarming proportions but the considerations on: (i) international border and transboundary issues, (ii) sustained socio-economic development of the region, (iii) flood and protection of existing infrastructure and cultural heritage could require prompt attention at critical locations along the Mekong river banks.

Riverbank erosion is slowly acknowledged as one of the problems hampering effective flood protection at some stretches along the Mekong River however, not all the LMB countries have a clear concept of how to address river bank erosion. Furthermore, there is additional indication that existing river bank protection along one side of the river is a contributing factor of river instability on the other side. Most of the bank failure and erosion areas show that flood and rapid drawdown of water level are the dominant erosion cause of damage to the river banks.

Best Practice Guidelines for riverbank protection are intended to aid anyone involved in planning, design, operation and maintenance of river bank erosion control works, ensuring that the structures used are appropriated, installed and maintained in a timely and proper way. Furthermore that the values associated with the river and their surrounding environments are protected.

In the process of developing the best practice guidelines it is recognized that integrated planning of structural works also requires a strong participatory process, with central, provincial, and local government agencies sharing the responsibilities with local stakeholders and their representatives. The guidelines may also assist agencies involved in planning and authorizing river bank protection works within the LMB.

One important aspect to consider when preparing best practice guidelines is that general guidelines are not preferred by the countries. Best practice guidelines shall be specific and preferable accepted by national governments. During the present FMMP C2 Stage 1 a checklist has been prepared of main issues to be addressed when developing and designing bank protection works. This checklist is presented in this outline and should be examined by the relevant line agencies for assessing the need for modification or a supplement to existing guidelines in the riparian countries. The checklist refers to functional requirements, design criteria and general specifications of specific erosion control structural works including assessment and needed actions to be taken in case of increased erosion risk.

Special attention it will be given to transboundary impacts that may result from the use of different design criteria for similar structural measures in the member states. This refers especially to the application of different protection levels (frequency) for both diking schemes and for bank protection works.

There is also a need for developing a sound enabling policy, planning, and institutional framework to support the integrated management of flood and riverbank erosion risks encompassing all stages of disaster risk management. In many cases financing for infrastructure maintenance remains also largely insufficient to pursue adaptive incremental approach to effectively respond to the dynamic morphological environment.

Last but not least, the use of the best practice guidelines and development of erosion management plans needs a good organization that leads the process. The responsible line agencies requires engineering background that should be developed through systematic capacity building and training in terms of staff resources, technical and non-technical managerial capacities to comprehensively manage flood and riverbank erosion risks, management systems and business processes, and inter-line agencies coordination towards comprehensive risk management set in the context of integrated water resources management.

The following sections give the overview of issues that will be taken into account when planning and designing river bank erosion control measures.

3.2 Scope of the guidelines

Riverbank erosion is recognized as a perennial problem caused by dynamic channel shifting of the rivers flowing through the unconsolidated sediments of the floodplain.

The main objective of river bank erosion control works is to provide a clearly defined decision process that will identify a strategy and priorities to carry out riverbank stabilisation works along sections of the Mekong mainstream.

The achievement of the main objective has to be supported by a well established program of consultation to allow local community members to be involved in setting priorities for the bank protection works.

The overall process to achieve the main objective has to be carried out following an integrated approach as shown in Figure 3.1. It shall be kept in mind that integrated bank protection is the assimilation of three factors; cause of bank erosion, habitat, and risk; into the planning and design of a river bank protection project.

It is crucial to assess these factors at the onset; otherwise a bank protection project will not likely achieve ecological and structural success.

Objectives are typically somewhat general or qualitative. For example, objectives may be stated as “preventing further erosion of the river along a road or flood embankment” or “stabilizing the river bank to reduce loss of agricultural production”. In fact, there are usually a number of objectives with differing levels of priority.

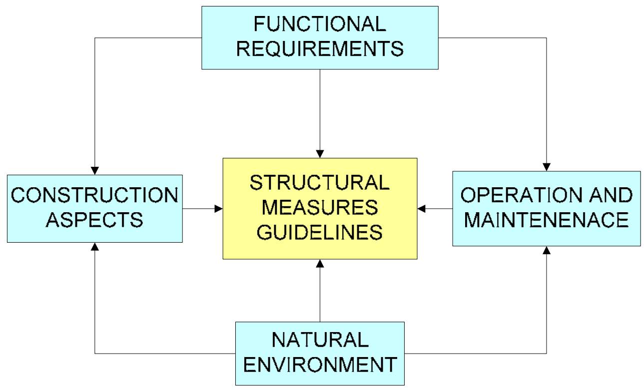


Figure 3.1 Integrated approach for planning and design of structural measures.

In order to bridge objectives with selection of erosion control techniques, it is important that design criteria are established. These criteria, considering risk and cost, and stratified according to relative priority, outline the objectives of the project and provide the foundation for making design decisions about the specific sizes and components of bank protection techniques.

First of all the guidelines have not the intention to cover all river hydraulics, morphological, natural and technical related aspects, which might be envisaged during planning, investigation and implementation of bank protection works. Instead, main issues regarding planning design and typical structural solutions developed on basis of risk and experiences of constructed projects or within demonstration projects are to be considered.

The guidelines are structured in a way that the concerned decision making levels, planning and design engineers as well as monitoring and maintenance teams are guided to arrive at safe and economic solutions.

Considerations on simplification and standardization of the planning and implementation process are crucial to allow for timely completion of bank protection structures within a restricted construction window and despite possible sudden changes of the river course.

The BPG is prepared for the river engineers, technical staff and planners for guiding design works of newly-constructed, as well as rehabilitation or upgrading river bank protection works to control erosion in the LMB.

The BPG is also applicable for planning and management of the related river bank protection works but is not compulsory to the other for other purposes.

Types of river bank erosion control or training works mentioned in this guideline include:
- Structures for improving the flow regime in the channel;
- Structures for river bank protection and stabilizing the channel alignment;

Application of the guidelines in projects with international involvement has to be based on the agreement of the related authorities. The BPG shall be used in consultation with other related guidelines and standards in each country.

3.3 Definition of concepts

In order to prevent or minimize the loss of valuable land, infrastructure and damage due to flood, several stretches of the Mekong river banks might need suitable protection against erosion. It has to be emphasized that with protection measures also the consequences must be taken into account, because a certain response of the river, i.e. morphological changes in the vicinity or even further away from a countermeasure, are to be expected. Protection of one place will possibly influence erosion and bankline shifting at other locations. From that point of view, only absolutely necessary measures should be considered.

The stability of unprotected river banks depends on a number of factors which have to be assessed carefully in the process of selection and design of suitable protection measures.

A reliable assessment of potential causes of bank failure is indispensable for the success of any measures, i.e. for the integrity of the selected bank protection system and thus the stability of the river bank. However, it is stressed that a certain proposed protection scheme can possibly affect the overall stability of the river or a river channel.

River bank erosion can occur in stable as well as in unstable rivers or river channels. Although the morphology of stable rivers is in a state of equilibrium with regard to the governing physical processes and is not expected to change significantly the general shape and dimensions, some local erosion and deposition is likely to occur, especially in meander bends.

Modifications of flow velocity, discharge, and sediment load and river morphology in unstable rivers channels are major factors initiating erosion and deposition. How fast the river responds to changes in boundary conditions depends on the natural stability of the subsoil (bed and banks) and the extent of changes. Successive erosion and deposition often leads to rapid changes in the river platform and slope. Increased meandering reduces the channel slope, whereas straightening of the channel through cut-off in most cases increases the local gradient. Any changes in bed elevations can also promote rapid bank erosion. Strong accumulation of sediments, followed by development of bars and islands promotes rapid widening and development of braided channels.

General surface erosion of river banks or along the river bed occurs if the driving erosive forces are exceeding the resistive forces of the individual grains or of the conglomerates in case of cohesive materials. The main impacts responsible for surface erosion at river banks are:

- current induced shear stress,
- wave loads (wind-generated waves; ship- and boat-generated waves),
- seepage (excessive pore pressure),
- surface runoff,
- mechanical action (desiccation, ship impact, activities of humans and animals).

Shear stress induced by current flow is the main hydraulic erosion factor. Although the primary current (velocity component in direction of the river course) is much larger, in river bends secondary currents, which are mainly generated by inertia forces, are the key factor for the asymmetric cross sectional profiles. The interaction of the system, hydraulic and geotechnical factors can be simplified as shown in Figure 3.2.

The provision of suitable tools for the planning, design and implementation as well as for appropriate monitoring and maintenance schemes of standardized bank protection measures in the Lower Mekong mainstream are the main objectives of this guidelines. The provided information is primarily based on the findings of consultations in the countries but also includes previous experience gained on rivers in the region and on similar rivers around the globe within other projects.

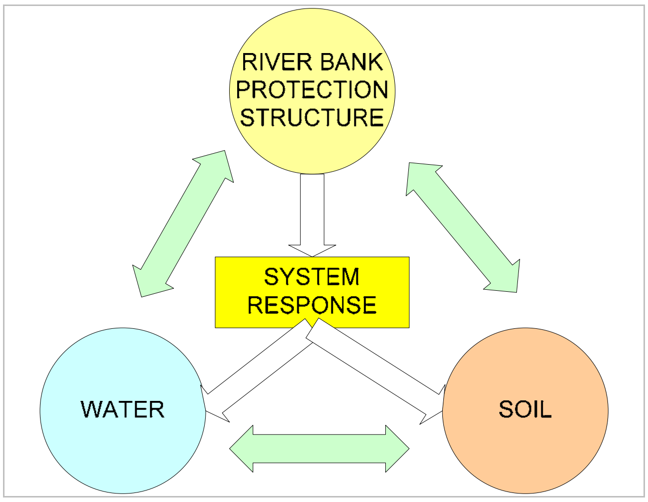


Figure 3.2 Interactions– soil – water structure and system response

3.4 Overview of design process

3.4.1 Introduction

This section presents an overview of the design process for structural measures, river training works in hydraulic engineering. This overview ranges from initial problem identification, boundary conditions and functional analysis, to design concept generation, selection, detailing and costing and includes an examination of construction as well as maintenance considerations.

The overview is supported by a more in-depth look at the structure types considered together with their potential failure modes and by a review of design approach illustrated with examples. A final remark considers the important area of Environmental (Impact) Assessment in relation to the design of river bank protection works.

3.4.2 Planning and design process

line agencies should seek appropriate advice when developing their river bank erosion plans. The advice should cover a range of specialties, depending on the needs of the project. It may include:

- Investigating channel characteristics of focal areas.
- A geomorphic assessment to evaluate how the river functions from a physical and hydraulic perspective (at a catchment, sub-catchment and focal area scale), and how it will respond to the proposed bank erosion works.
- Conduct erosion/ sedimentation study, including modelling for a certain frequency. Flood elevations are considered for final configuration of the channel.
- Determining hazard area from future channel migration.
- Developing mitigated channel migration hazard map(s). Fix the outer limits of future lateral migration.
- Evaluate whether the mitigated hazard area can be sub-divided into areas of severe and moderate hazard using calculated historic channel migration rates.
- An engineering survey to determine the suitability of the site to make sure the design of any structural measures is appropriate before works begin.

Planners and designers shall consider the primary design requirements of erosion control works. For any proposed river bank erosion works, the designer shall demonstrate that:

- Proposed bank stabilization will not adversely impact the river reach or development upstream and downstream.
- Demonstrate the stability of proposed bank erosion works. Local scour, long-term degradation, channel migration and bank erosion must be explicitly addressed in the bank protection design.

This section addresses the details of the overall formulation in a simplified flow chart form of design process logic diagram Figure 3.3 which relates the design process to the contents of this Best Practice Guidelines (BPG). This section also indicates the principles and methods which support the design procedure making reference as appropriate to other parts of the Standard. It must be recognized that the design process is a complex iterative process and may be described in more than one way. Therefore it is always recommended to follow a decision process diagram to identify a range of potential options that would be suitable for each situation of the river.

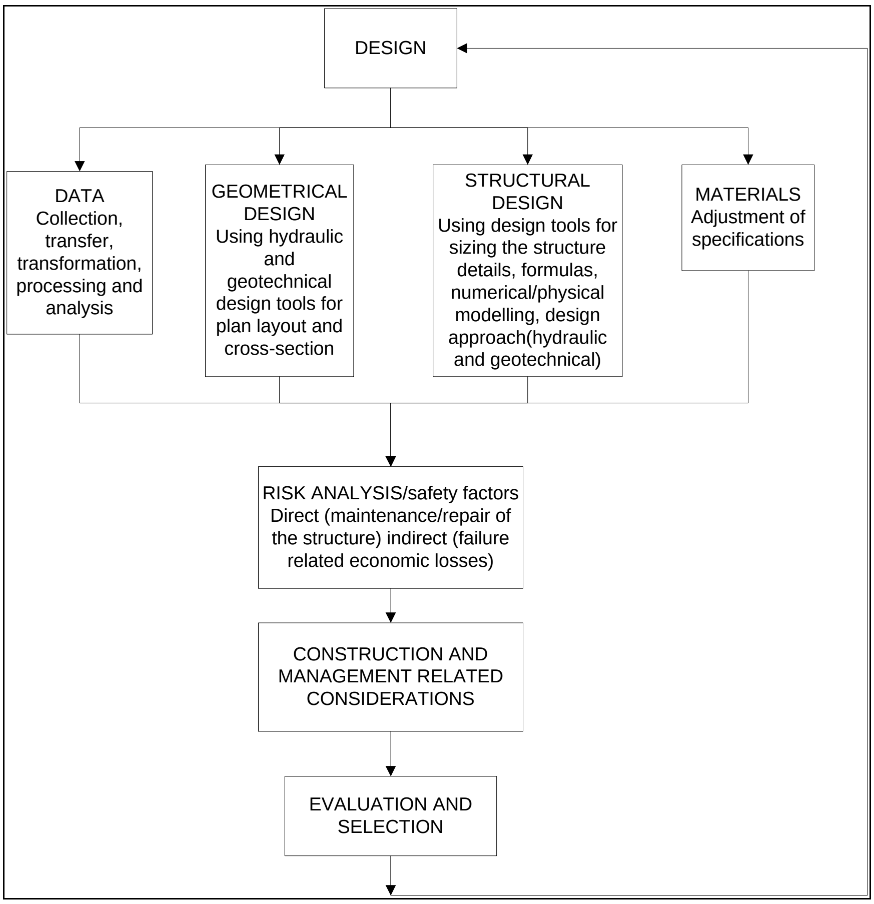


Figure 3.3 Design process with reference to the BPGs.

Look at a range of feasible stabilization options and determine the most appropriate. Take account of factors such as cost, material availability, aesthetics and ecological values when determining the most appropriate method of foreshore protection for each location.

Address the three preliminary questions about foreshore protection works:

- Is bank erosion occurring at the site? If erosion has occurred it may not necessarily continue — a new stable equilibrium (or regime) may be achieved. For example, bank erosion will stop when all of the erodible material below the water level has been removed and the bedrock is exposed. Equally, restrictions on navigation activity or changes to the flow rate upstream may stop erosion. A clear understanding of fluvial processes at the site is required, together with enough time to observe and assess the processes over time.
- Is bank erosion a problem at the site? Bank erosion may occur in situations where foreshore protection works would generally be unnecessary and uneconomical. In such cases it may be best to leave the bank to meander naturally, provided buildings, property or infrastructure is not threatened.
- Is the riverbank under investigation in a natural condition? In areas where dredging of the river or sand mining has already occurred, the flow behaviour can differ substantially from that of a natural section of river. When bank protection is considered necessary, specialist professional skills are required to assess the situation and provide the most suitable management option, taking account of engineering, ecological, aesthetic and economic factors.

Assuming the section of river channel being assessed is natural, the next step is to determine what is making the bank erode. This must be known before a stabilization strategy can be finalized and implemented.

If vessels traffic is the cause of wave attack, then management and control (e.g. limiting speed, designating areas where stopping and starting is prohibited) may be good options. Liaise with Waterways Authority to identify and assess options for managing IWT and vessel speed to reduce the erosion risk. The guidelines will recommend methods for determining the bank retreat due to wave action by ships.

Where protection works are needed, engage a hydrological, river training works and morphology engineers as a members of the design team, together with an environmental and a landscape architect (if needed in urban and tourist areas) to address all relevant issues.

3.4.3 Design stages and process

Follow a decision process diagram to identify a range of potential options that would be suitable for each situation of the river is shown in Figure 3.4.

Look at a range of feasible stabilisation options and determine the most appropriate. Take account of factors such as cost, material availability, aesthetics and ecological values when determining the most appropriate method of foreshore protection for each location.

Address the three preliminary questions about foreshore protection works:

- Is bank erosion occurring at the site? If erosion has occurred it may not necessarily continue — a new stable equilibrium (or regime) may be achieved. For example, bank erosion will stop when all of the erodible material below the water level has been removed and the bedrock is exposed. Equally, restrictions on navigation activity or changes to the flow rate upstream may stop erosion. A clear understanding of fluvial processes at the site is required, together with enough time to observe and assess the processes over time.
- Is bank erosion a problem at the site? Bank erosion may occur in situations where foreshore protection works would generally be unnecessary and uneconomical. In such cases it may be best to leave the bank to meander naturally, provided buildings, property or infrastructure is not threatened.

- Is the riverbank under investigation in a natural condition? In areas where dredging of the river or sand mining has already occurred, the flow behaviour can differ substantially from that of a natural section of river. When bank protection is considered necessary, specialist professional skills are required to assess the situation and provide the most suitable management option, taking account of engineering, ecological, aesthetic and economic factors.

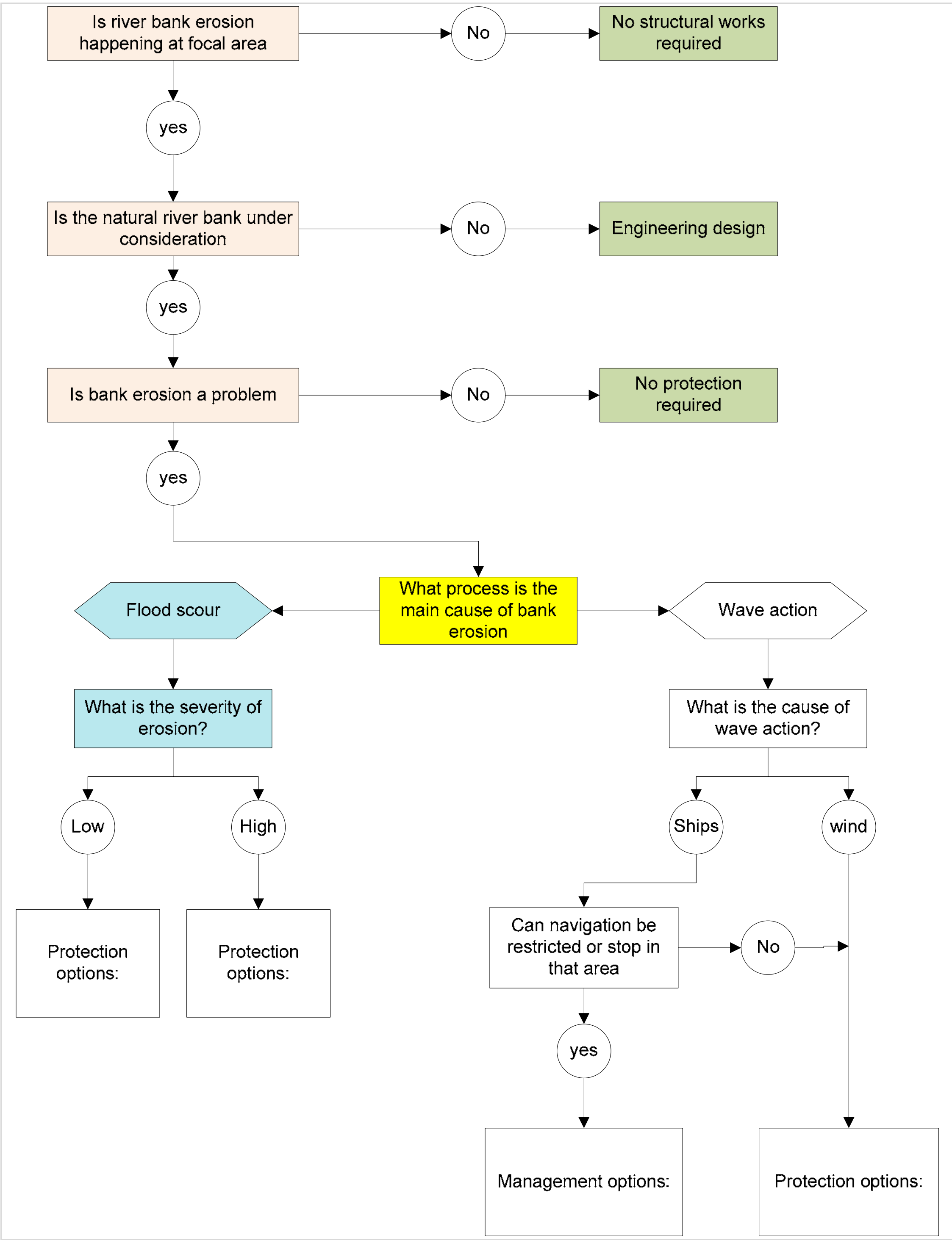


Figure 3.4 Decision process diagram

Assuming the section of river channel being assessed is natural, the next step is to determine what is making the bank erode. This must be known before a stabilisation strategy can be finalised and implemented.

If vessels traffic is the cause of wave attack, then management and control (e.g. limiting speed, designating areas where stopping and starting is prohibited) may be good options. Liaise with Waterways Authority to identify and assess options for managing IWT and vessel speed to reduce the erosion risk. The guidelines will recommend methods for determining the bank retreat due to wave action by ships.

Where protection works are needed, engage a hydrological, river training works and morphology engineers as a members of the design team, together with an environmental and a landscape architect (if needed in urban and tourist areas) to address all relevant issues.

Furthermore, design of river erosion control or training works has to be done in accordance with current regulations of the government of each country for construction of this type of projects.

Problems and needs shall be determined in detail on the basis of river morphology, flood impacts, transport requirements and environment and, alternative solutions developed. The effects of various alternatives are explored, and the project or program which best meets the objectives in a cost effective way is selected. Such a project must also be environmentally and socially acceptable to the public.

Project documents shall include:

1. Survey/ investigation reports
2. Design/ study report
3. Design drawings
4. Calculation sheets

The project documents have to cover and reflect thoroughly content of the work for each stage of the project. The reports have to be well prepared, clear and binded, signed and stamped in accordance with the regulations in force.

The study/ design reports have to be concise, truthfully reflecting the study problems. Analyses, assessments, recommendations and alternatives have to be mentioned in the report.

In the BPGs the following approach can provide a thorough review of the design process, which consists essentially of three stages:

- *Conceptual design stage:* evaluating the condition of the site under consideration, determining the impact of erosion control options and selecting the most suitable stabilisation treatment.

- *Pre and feasibility stage:* collating detailed design information and defining predominant loads. Environmental, cost and accessibility factors need to be assessed and the most appropriate solution identified.

- *Detailed engineering stage:* consists of studies for reviewing engineering feasibility studies and updating them as necessary to ensure that feasibility designs are still valid at the time of implementation. Morphological conditions and flood protection requirements change with time, and therefore engineering studies should be reviewed and updated. Design studies largely involve refining and detailing designs developed at feasibility level. Therefore, detail design studies serve as the basis for the preparation of plans, tender documents and specifications for construction.

The construction drawings shall be prepared on the basis of actual information and recent survey maps (not more than 1 year old). For areas with active erosion showing large seasonal variations, the survey map of the current year shall be used. According to the nature and scale of the different components of the project, different drawing scales may be adopted for the survey sheet of the construction area.

Moreover, the planners and design teams shall include in the process all main stakeholders and consider the following recommendations:

- Consult with other countries and MRC programmes to discuss options for riverbank stabilisation during the design process.
- Check the Basin Development Plan (BDP) for the focal area before planning stabilisation works.
- If required for sustainability of the works develop a river erosion control plan incorporating any BDP requirements for the particular section of river.
- Adopt adaptive protection as the preferred option where possible. Only use constructed protection where the erosion is severe and the energy too high for a adaptive solution.
- Prepare detailed design documents for the stabilization works where they are required.
- Obtain all approvals from the concerned agencies, the relevant provincial and other authorities.
- Pre-qualify suitable contractors with demonstrated experience in implementing similar river bank protection works.
- Implement project management arrangements to ensure close supervision of the works so that the design objectives are met and the environmental impacts are minimized.

3.4.4 Primary requirements

For any proposed river bank erosion works, the designer shall demonstrate that:

- Proposed bank stabilization will not adversely impact the river reach or development upstream and downstream.
- Demonstrate the stability of proposed bank erosion works. Local scour, long-term degradation, channel migration and bank erosion must be explicitly addressed in the bank protection design.

One of the most difficult but important aspects of the design process is moving from the site and river reach assessment to the selection of an appropriate solution.

Screening matrices can be developed to assist the user in the selection of bank protection measures that:

- perform adequately to meet bank protection objectives;
- are appropriate with respect to mechanism(s) of failure and site-and reach-based cause(s);
- are considered with an understanding of the potential impacts caused by each technique;
- are selected in order of priority that first avoid, second minimize, and lastly compensate for impacts.

This consideration results in accepting or rejecting alternatives. Throughout the process of identifying a solution, the question should always be posed whether the best course of action might involve none at all.

Information about river bank protection techniques applicable within the Lower Mekong Basin shall be part of the data from line agencies. The available techniques can be divided into several functional groups. For each technique, the following information shall be provided in the guidelines:

1. Description of the technique;
2. Application (typical application, variations, emergency, site and reach limitations);
3. Effects;
4. Design;
5. Habitat considerations (mitigation requirements for the technique or mitigation benefits provided by the technique);
6. Risk (risk to habitat, adjacent properties, and reliability/ uncertainty of the technique);
7. Construction considerations (material required, timing considerations, cost);
8. Operation and maintenance needs;
9. Monitoring considerations by case studies;
10. Examples (typical drawings, site example, description, photographs); and
11. References.

3.4.5 Boundary conditions

Groups should decide on the problems they will tackle and the objectives for resolving them. These decisions will be based on the analysis of the survey data. Each objective should show how the bank erosion problem will be managed in order to achieve the group's vision for the river bank erosion and flooding. Relevant landowners and river users should be consulted to ensure no problems are missed and to make sure there is consensus on how the problems will be managed.

For short-term measures to be effective and not only temporary, they should fit into an adaptive programme for which sustainable institutional and financial arrangements are very important towards a long term solution. The execution of riverbank stabilisation works can therefore only be effective if it is immediately associated with a commitment for monitoring, maintenance and progressive implementation of adaptive works (if necessary).

In conjunction with identification of the problem, all of the boundary conditions which influence the problem and its potential solutions must also be identified. These boundary conditions are of various types and include aspects of the following:

- planning policy (including environmental impact aspects);
- physical site conditions;
- construction and maintenance considerations.

The chosen national design/ safety policy of each country shall serve for determining in certain way the required level/ quality of the design (guidelines/ standards/ codes, process, dimensioning techniques, materials, specifications and the construction techniques).

Planning policy aspects involve political, legislative and social conditions and include a definition of acceptable risk of failure/ damage/ loss of life and acceptable/ desirable environmental impacts. Environmental impact aspects are discussed in different guidelines as these tend to be predominantly boundary conditions to the solution and as such effectively become part of the iterative design process, evaluating the potential beneficial or adverse effects of each proposed solution.

Determination of physical site conditions of principal concern and possibilities for data collection will be described in some detail in Section 3.5. Principal site conditions concern bathymetry and morphology, hydraulic conditions and geotechnical conditions.

The extent of their influence on the (proposed) river bank erosion protection structure must be recognized. It is important to recognize over what parts of the structure, and for what time duration, various environmental influences apply. For river training works structures (including those subject to wave attack and shoreline protection structures, three approximate exposure zones can be distinguished as seen in Figure 3.5.

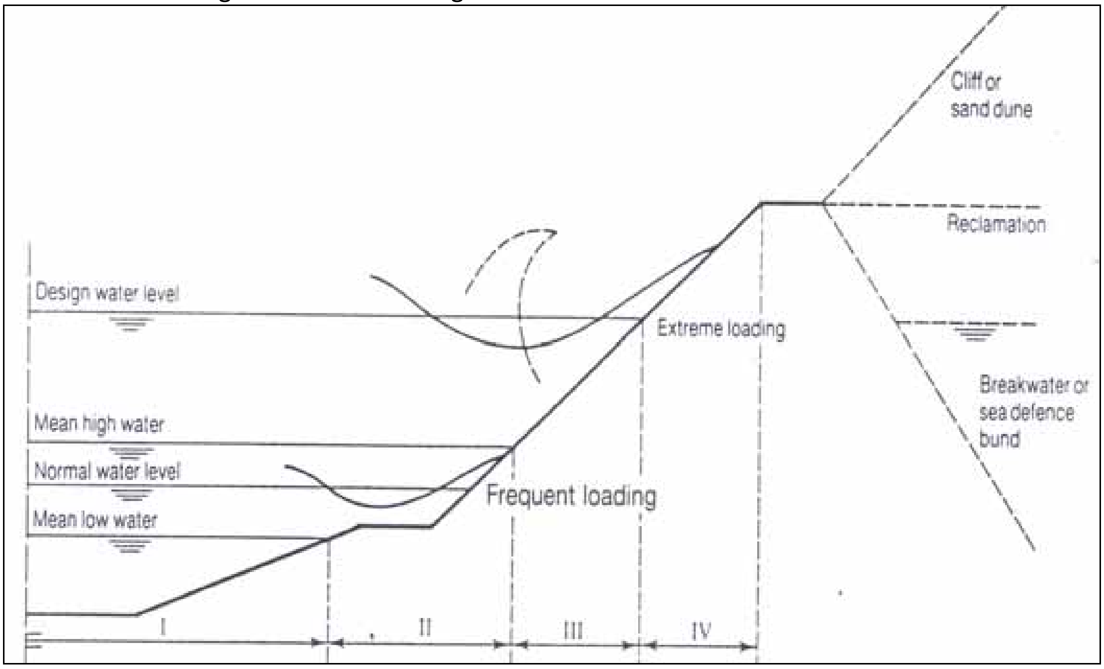
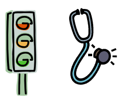


Figure 3.5 Exposure zones in river training works structures.

- the zone I, permanently submerged (this zone can be heavily attacked by currents and scour but the frequency of such attack reduces as one goes higher up the slope);
- the zone II, between MLW and MHW where the wave-loading is of importance for the long-term behaviour of structure;
- the zone III, above design level, where there should only be risk of damage due to wave run-up and overtopping and human activities.



Typical questions to be answered in the planning stage are:

- What data are available and needed,
- is it possible in view of time and cost constraints to collect (more) data, and
- will (physical or mathematical) model testing be a solution?

Construction and maintenance entail a need to know additional boundary conditions. These must be acknowledged in the stage of planning and design and as such, like many aspects of environmental impact, become part of the iterative design process.

The division of the structure into loading zones shall be made not only with direct connection for ensuring safety against failure, but also with overall ensuring that the appropriate materials, and execution and maintenance methods, are identified for each zone.

3.5 Assess river condition

Concerned line agencies should survey the river to assess its condition and identify sites that are stable and healthy or degraded by river bank erosion. Surveying the river alignment and river banks will make it easier to determine what works need to be done and their priority, as well as the resources needed to do the works. The data collected will usually include information about remnant native vegetation, stream conditions, erosion, and bed and bank stability. The guidelines shall indicate the issues likely to be faced when surveying the river and shall outline the data that should be collected. It also shall contain pro-formas that can be used to record the survey data.

The following river characteristics will be taken into account:

- Geology and local geological controls,
- Hydrology,
- Bed material,
- Longitudinal slopes,
- Planform characteristics,
- Channel dimensions,
- Hydraulic roughness,
- Sediment transport,
- Bed topography,
- Bank erosion rates.

The level of river studies required shall include:

- hydrological, hydraulic and (geo)morphological analysis,
- river bank (erosion rate) monitoring,
- topographical and bathymetric surveys, and
- geotechnical investigations for selected demonstration projects.

These studies actually aim at gradually completing the information collected and analyzed within the scope of the guidelines, so that a data bank will be established. The hydrological, hydraulic and (geo)morphological analysis and available mathematical models can be used to study river bank erosion patterns and to evaluate the potential of the river development for different purposes.

3.5.1 Data collection

line agencies should seek appropriate advice when developing their river bank erosion plans. The advice should cover a range of specialties, depending on the needs of the project. It may include:

- Investigating channel characteristics of focal areas.
- A geomorphic assessment to evaluate how the river functions from a physical and hydraulic perspective (at a catchment, sub-catchment and focal area scale), and how it will respond to the proposed bank erosion works.
- Conduct erosion/ sedimentation study, including modelling for a certain frequency. Flood elevations are considered for final configuration of the channel.
- Determining hazard area from future channel migration.
- Developing mitigated channel migration hazard map(s). Fix the outer limits of future lateral migration.
- Evaluate whether the mitigated hazard area can be sub-divided into areas of severe and moderate hazard using calculated historic channel migration rates.
- An engineering survey to determine the suitability of the site to make sure the design of any structural measures is appropriate before works begin

3.5.2 Hydrology and hydraulic data

Hydrological and hydraulic data, many river characteristics are closely related to the discharge variation which is characterized by the duration curves of the different stations along the Mekong River. Furthermore, the longitudinal slopes along the river to be derived from the gauge readings both for low flow and for flood conditions. Finally, the discharge and stage data is to be used in combination with sediment transport data.

If sufficient data are not available or regarded as insufficient in quantity and/or quality for the planning, design, monitoring and maintenance of riverbank stabilization works, additional and systematic topographic hydrographic and bathymetric surveys need to be carried out. They should cover those parts of the river that are directly influenced by the river bank erosion. The exact extent of these surveys will be determined during the execution of the projects in detail.

Hydrographic surveys might comprise the measurement of typical cross-sections at regular time intervals and several blanket surveys to establish the topography of the riverbed. Discharge measurements might be carried out at convenient sites or taken from river stations where rating curves are available.

The survey data will be imperative to planners and designers, for the database required for monitoring and maintenance. Discharge measurements might also be carried out during flood stages. Furthermore, any hydrographic surveys programme shall concentrate on bankline and bed-changes during the flood seasons. The following physical phenomena might be investigated:

- scour in outer bends to calibrate prediction methods and to allow for extrapolations to extreme floods
- also sediment transport rates, flow velocities and flow fields will be measured in the present study
- bedform dimensions to extend the prediction methods for dune heights and bed level distributions to large water depths and to observe bed conditions during floods and receding season

Hydrographic surveys should include the low water season and continue during the flood season in order to monitor seasonal changes in the river processes. Furthermore, before, during and after construction of the river stabilization works, high quality bathymetric surveys and high accuracy positioning system are required as well.

The 'raw' data obtained through desk studies and field surveys will be processed and compiled per discipline and/or activity in a database, in formats as required for models and river engineering analysis.

In order to guarantee the timely execution of surveys and its quality it is necessary to review and assess the arrangements for implementation and refining necessary survey instruments to meet the requirements. Special attention will be paid to maintenance and calibration of the survey equipment and preparation of plans for repair and upgrading.

3.5.3 Planform data

Data on river planforms, data of river planform changes are available in the form of: (i) maps, aerial photographs, and satellite images. Changes in planform can be estimated in two steps. First for the whole of the Mekong River, where it is a border river between Thailand and Laos. A comparison was made between the river planform in the early seventies from satellite images with topographical maps. Second the river stretches downstream from the border between Lao PDR and Cambodia and the Mekong Delta in Vietnam.

One of the most important sources of data concerning the bed and bank topography of the Mekong River is series of bathymetric maps. Updated bathymetric maps can be used for an assessment of the width and the depth of the river, the occurrence of islands, values of the braiding index and the sinuosity and the thalweg location of the river. The cross-sectional profiles can be also derived from the bathymetric maps. Finally, the influence of bedrock outcrops could be assessed with the bathymetric maps in combination with available aerial photographs.

3.5.4 Sediment data

Data of the bed material size along the Mekong River is not readily available. Additional bed samples will have to be collected and analyzed to obtain a better insight in the grain size values and the distribution for characterization of river stretches and for preparing projects.

Results of sediment transport data and measurements in the Mekong River comprise suspended load transport measurements only.

Suspended load data is available at some stations along the Mekong. The results of the sediment transport data will be used in several ways. First they will used to determine the discharge range for which most sediment is transported. This discharge is sometimes considered as the most decisive for the morphological processes in a river. The Mekong, however, is characterized by a prolonged period of low discharge values and a fairly long period of floods. It appears therefore more logic to consider the river as a two-stage river. Secondly, sediment rating curves can be developed to determine the yearly sediment transport rates. Finally, the sediment transport measurements can be used to determine which (alluvial) sediment transport predictor fits best to the conditions of the Mekong River.

The results of the sediment transport data will be used in several ways. First they will used to determine the discharge range for which most sediment is transported (known also as dominant discharge). This discharge is sometimes considered as the most decisive for the morphological processes in a river. The Mekong, however, is characterised by a prolonged period of low discharge values and a fairly long period of floods. It appears therefore more logic to consider the river as a two-stage river. Secondly, sediment rating curves can be developed to determine the yearly sediment transport rates. Finally, the sediment transport measurements can be used to determine which (alluvial) sediment transport predictor fits best to the conditions of the Mekong River.

3.5.5 Geotechnical data

Geotechnical investigations are required at the projects locations, for a pre-feasibility design level and for the characterization of the river bank. It is expected that additional secondary field data can be obtained from the countries. For medium and large scale projects, the list of surveys and laboratory tests to be carried out will be prepared for each project.

The soil characterization of the river stability of the river banks is an essential step towards a successful bank erosion protection. Bank stability comprises both micro and macro stability. Typical modes of river bank failure like slip failures and block failures are shown in Figure 3.6.

Although bank protection works serve to increase the micro stability of the river bank, it may be expected that the bank protection works will also increase the macro stability of the bank.

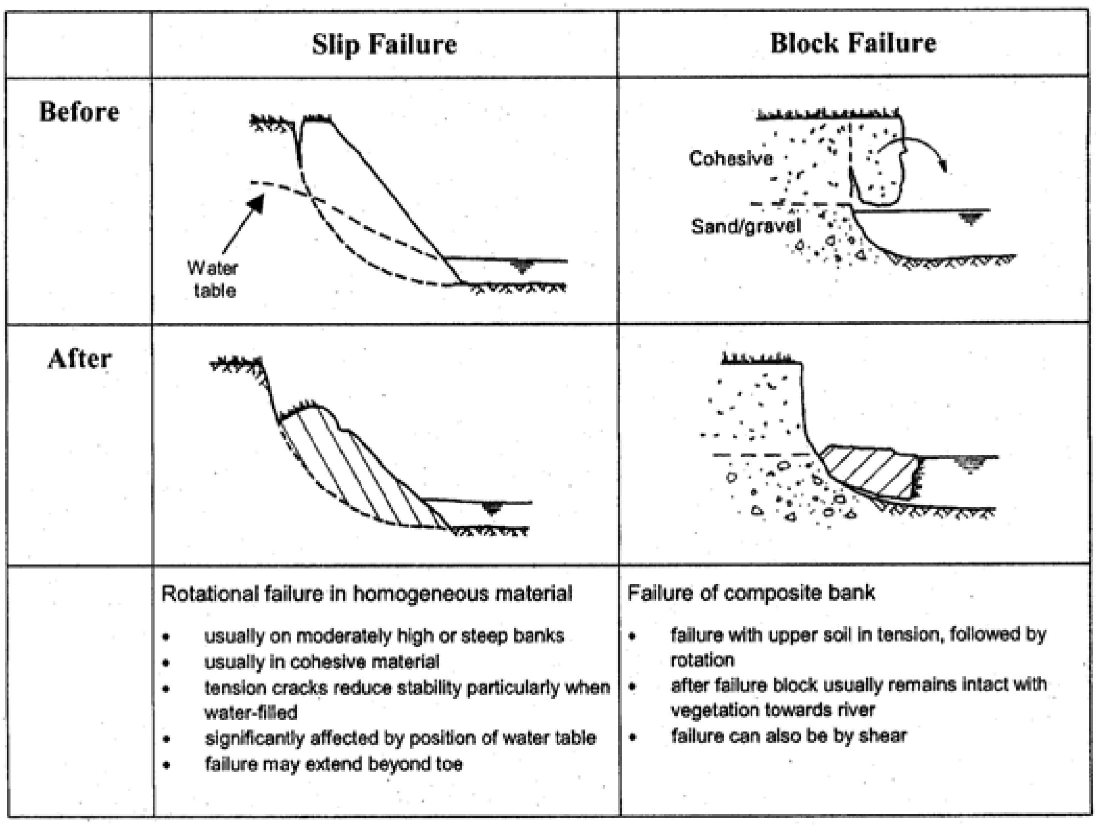


Figure 3.6 Typical modes of river bank failure (adopted Hemphill and Bramley, 1989).

A geotechnical site investigation and laboratory testing, in conjunction with site visits and a study of topographic and bathymetric survey results, is required prior to conduct geotechnical investigations to determine the mechanisms of slope failure and to establish the general design parameters.

The results obtained from the geotechnical field works and laboratory tests will provide the necessary information in order to establish the design parameters for the bank protection works (macro-micro stability). Therefore, the output will be focused on:

- Establishment of design parameters for safe slope design;
- Identification and establishment of the general subsoil characteristics along the river stretches under study;
- Analysis of slope failures and slope stability analysis for slip circle failure and compound failure, if required, under static and dynamic conditions and drawing up recommendations for safe slope angles;
- Recommendations for geotechnical requirements in the guidelines of river bank protection works.

3.5.6 Geo-morphological condition of the Mekong (LMB)

The Mekong is a major river carrying large amounts of water during the monsoon season. Different from many other large rivers, the Mekong does not flow in an alluvial plain and along the Thailand and Lao PDR the river is affected by local geology. The result is that at only a few locations the river is free to shape its own course and dimensions although many of its parameters are still controlled by faults, rock outcrops and other geological features. Rocky outcrops can be observed in the lower part of the Mekong River downstream of Kratie in Cambodia. The influence of the geological controls disappears downstream of Kampong Cham.

Another complication in the Mekong River is the variability of the discharge throughout the year. The ratio between the highest and the lowest (average monthly) discharges is in the order of 5 to 10, whilst the duration of the flood and low flow season is about 3 months and 4 months respectively. The river can therefore be described as a two-stage river for which many of the in-channel characteristics are highly dependent on the discharge. The last also implies that the geo-morphology of the river is always adjusting to the changing discharge conditions.

Available data on discharges along the Mekong River within the Lower Mekong Basin is given in *Annex 1, Flood Hazards in the Focal Areas, to the Stage 1 Evaluation Report, August 2008*.

For the purpose of planning and design of structural measures controlling lateral migration of the river due to erosion, the analysis of bankline changes is very important. This can be done by determining the historical changes in the river alignment by using maps, satellite images and aerial photographs. The analysis of bankline changes and erosion rates is to be linked to the local geology or geotechnical characteristics of the river banks.

The understanding obtained from the analysis of banklines shall be subsequently used for making predictions of the movement of the Mekong at priority sites for controlling erosion and damage to flood control infrastructure, roads, agricultural land, urban centres, etc.

In addition, for proper planning and mitigation of potential effects downstream and upstream of sites to be protected, to look at the possible effect of planned river bank erosion works on adjacent river reaches.

Minimum bed levels near planned river bank erosion control works have also to be mapped because these are referred to as scour. The development of scour holes has to be taken into account in order to be able to predict the maximum scour depth in future at the places where river bank erosion works are implemented. A distinction has to be made between the different types of scour at the sites to be protected. This is by itself a dedicated topic of river morphology to be investigated prior to design, especially at locations where there are strategic infrastructure to be protected against river erosion.

Possible causes of bank erosion are outer bend erosion, constriction scour and erosion due to the presence of rocky outcrops. From existing bathymetric maps in Thailand, Lao PDR and Cambodia, it can be concluded that river bank erosion does not necessary combine with the existence of outer bend scour holes. The reason for this is that they can possibly not be formed because of the geological substratum and the rocky outcrops.

In the upper Mekong reaches in Thailand, Lao PDR and Cambodia, erosion along outer bends in the Mekong can consists of two mechanisms, the first is the overdeepening of the channel along an outer bend of the channel, and this causes the river bank to fail easily and the outer bank to be eroded. This can be referred in the Mekong as the vertical erosion. The second type occurs when there is no outer bend scour hole present along the bank of the river. This can be caused by several reasons. The most likely and also the most common in the Mekong River is the presence of rock in the subsurface which prevent overdeepening along the outer bank. In this case some erosion can still occurs when the bank fails and can be called the horizontal erosion. In the Mekong delta erosion is mainly dominated by outer bend scour.

3.5.7 Data on implemented river bank erosion control works

Monitoring of river works already implemented along the Mekong River is necessary for making decision on mitigation measures at locations where erosion could have been induced by constructed river works. This can be done by the line agencies in each country by preparing a systematic inventory of river works and monitoring the changes in bank erosion rate along the river banks in the proximity of those works. The inventory shall contain river characteristics such as water level variation, presence of sand bars, flooding depth and when it happens. In addition, the condition of the banks prior to the construction of river works and after construction and the possible impacts on the same bank or opposite bank shall be also recorded.

In this BPG the advice is to characterize the river bank as: stretches or sites without erosion and sites with visible erosion, and to map those sites using the standard river chainage established in the LMB. The process of monitoring shall be maintain over a number of years because the changes of planform of the Mekong river are anyhow considered minor and they proceed at low rate. Hence also the effect of bank protections works will become noticeable at other locations only after a long period of time only.

3.6 Planning policy, policy analysis

3.6.1 Purpose

The main purpose of river bank erosion control works is to provide a clearly defined decision process that will identify a strategy and priorities to carry out riverbank stabilisation works along sections of the Mekong mainstream.

Structural measures shall form part of an integrated bank protection plan with the assimilation of three factors; cause of bank erosion, habitat, and risk; into the planning and design of a river bank protection project. It is crucial to assess these factors at the onset; otherwise a bank protection project will not likely achieve ecological and structural success. Solving a bank protection problem begins with clearly stating the objectives of a project at a certain focal area. Objectives are typically somewhat general or qualitative. For example, objectives may be stated as “preventing further erosion of the river along a road or flood embankment” or “stabilizing the river bank to reduce loss of agricultural production”. In fact, there are usually a number of objectives with differing levels of priority.

In order to bridge objectives with selection of erosion control techniques, it is important that design criteria are established. These criteria, considering risk and cost, and stratified according to relative priority, outline the objectives of the project and provide the foundation for making design decisions about the specific sizes and components of bank protection techniques.

Furthermore, a brief description of the types of river stretches as well as of the physical and geological aspects of the Mekong River for general planning which includes underlying erosion mechanism at river banks shall be followed by a description of potential structural measures taking into account the focal information and the possibility of applying standardized structures on basis of the predetermined hydraulic and morphological boundary conditions. This will allow introducing standardized structures to control erosion and to prepare a categorization defining: i) the range of expected impact loads ii) the importance of the protected area and risk.

This is indispensable to prevent from overdesign, but also to exclude projects of strategic of both international and national importance (i.e. bridges, protection scheme around main urban centres, cultural heritage, international boundaries, etc.) from the simplified planning procedures. For this purpose it is recommended to structure categories (SC) of measures, with increasing project relevance from SC1 to SC4 as follow:

- SC1: Minor measures and structures that can be copied by traditional means and/or ad-hoc measures.
- SC2/SC3: Erosion prevention within identified priority areas of valuable assets. Limited structural damages keeping the primary function may be tolerable, and adaptations to meet changing requirements are generally feasible.
- SC4: Measures for objects of extraordinary importance and/or most severe and complex hydraulic and morphological conditions. Damages are not acceptable.

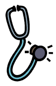
To define the application of standard protection structures and to allow for economic design four structure categories (SC) can be defined, along the Mekong River, as suggested in Table 3.1.

Table 3.1 Recommended classification of structure categories

Structure Category		Depth average velocity (m/s)	Design wave height (m)	Total scour depth (m)
1	Light	< 1	<0.25	< 3.0
2	Moderate	>1 -2.0	0.25 – 0.5	3.0 – 5.0
3	High	>2- 3.0	0.5 – 1.0	5.0 – 10.0
4	Extreme	>3.0	>1.0	>10

3.6.2 Management issues and objectives

Establish a program of consultation to allow local community members to be involved in setting priorities for the bank protection works.



Groups should decide on the problems they will tackle and the objectives for resolving them. These decisions will be based on the analysis of the survey data. Each objective should show how the bank erosion problem will be managed in order to achieve the group's vision for the river bank erosion and flooding. Relevant landowners and river users should be consulted to ensure no problems are missed and to make sure there is consensus on how the problems will be managed.

Projects in river engineering will normally have to meet the requirements of some governmental policy. Governments or local authorities may initiate the design of a structure because of their responsibility for the management of river bank erosion, flood protection, inland water transport, river port facilities etc. Private sector companies may do the same as part of the investment into a new economic development area. In both cases a number of procedures will have to he followed, related to established planning requirements, legislation, decision making procedures and, if required, benefit-cost analysis for financing of the project. The designer must attempt to mobilize available expertise from all interested parties and disciplines and ensure that each is fully involved so that delays can be avoided at a later stage.

A policy analysis should be carried out to identify the various interests of and/or constraints imposed by the social and economic environment in general and authorities in particular. The analysis must primarily serve the goals of the project. In this respect, relevant parties must be involved, but it should be realized that choices are, in principle, not made by the designer.

Key policy matters that will have a very significant impact on the design and which are often predefined include:

- acceptable risk of failure/ damage/ loss of life;
- maximum benefit-cost (b/c) ratio or rate of return for the project (finance);
- environmental assessment.

Many aspects of planning policy are not pre-defined at the start of a project and in many cases permission ("planning permission") by the relevant governmental organization to proceed with the project -if given- is subject to constraints which are imposed once a scheme concept has been presented for approval. Therefore it is necessary to involve in a preliminary stage of the design process the decision makers, authorities, politicians, public and any groups or individuals, who may have an interest in the existing problem and/or the way to solve it (including planning, design, construction and management of a structure). Experience has shown that a technical solution to a problem may not be accepted, by any of the parties of interest, if it is presented as an independent and pre-defined closed solution. The background of the various interested parties should be acknowledged as these may relate to various social, (individual) political and/or economical interests.

The impact of some constraints imposed by planning authorities, organizations or other countries can be considerable. Whilst a project can usually be effected using a variety of structures, materials, equipment and labour, the interested parties and/or the planning authority may seek to limit the freedom to choose from some of the available options and this limitation may have a significant influence on the design, construction and maintenance of the future river training or bank protection works. Aspects which may be important for acceptance by authorities, line agencies or the general public are:

- adequate measures to limit or minimize negative effects of the scheme on existing structures, e.g. ferry landings (jetties). drain outlets;
- social acceptability, e.g. use of local skills;
- recreational acceptability;
- environmental acceptability;
- ecological acceptability;
- general aesthetics.

In this engineering guideline only the technical solutions are discussed and more in particular those based on the different types of materials. Important elements and considerations of such an analysis are:

1. proper description of the problem;
2. generated alternatives should give credit to each of the objectives relevant in solving the problem (e.g. flood, bank shifting, environment);
3. representative involvements of all interested parties at an early stage of the design:
4. criteria and weightings to be used in a subsequent Multi-Criteria Analysis of solutions:
5. the final choice is often submitted to a political decision and is not done by the project team.

This being achieved, the designer may later, having reached the stage where he needs to compare solutions and to select a suitable design, propose a number design options to the interested parties, They can then judge these options using their own previously defined and agreed criteria matrix. Once a decision is made on the preferred option or options, this should then he written up in a formal document of agreement that will also cover 1 and 2 from the above list. Once in place. such an agreement will then allow final design and detailing to proceed, covered by a general political acceptance.

3.7 Acceptable risk

Throughout the design process, it is important to understand and evaluate the many types and levels of risk associated with a river bank protection project. A risk assessment should consider both the risk of continued bank erosion and the risk associated with the bank protection project with respect to property, habitat, and public safety. All bank protection projects contain some level of risk. For example, a bank protection project may be effective at lower flows, but may fail as a result of a larger flood.

The acceptable risk of failure/ damage/ loss of life, once the design parameters for a river training works or hydraulic structure are exceeded is a central boundary condition to any design both for serviceability and ultimate limit state conditions.

In a probabilistic design, taking a calculated risk is rewarded by the benefit of a cheaper design. Because the consequences of failure can be significant, attention should be paid to which of the parties involved is to carry the risks and enjoy the benefits respectively. In the design stage also construction and maintenance/ management should he included in these considerations.

According to a common definition, risk is the product of failure probability and the consequences of failure. Consequences of failure can (and often are) expressed in terms of a capital cost. The first factor, the probability of failure, can be defined quite objectively as the probability that the functional requirements are not met. However, an objective quantitative definition of the consequences is not always easy. In fact, only direct consequences of failure such as structural damage can he calculated. Other consequences are multi-dimensional and may be difficult to relate explicitly to the structure in concern. So a generally agreed scale and units to measure consequences may be impossible, Examples of other possible consequences (with different dimensions), so excluding direct structural damage are:

- social stress,
- loss of human life,
- human injuries,
- loss of property,
- loss of investments,
- loss of (expected) future income.

These consequences and losses can be categorized as done in Table 3.2.

Table 3.2 Categories of losses

	Quantifiable (tangible)	**Unquantifiable (intangible)**
Direct	Repair, replacement and rehabilitation of structure, Structure-related repair. rehabilitation and replacement of other objects	Loss of human life Injuries Loss of irreplaceable matter Environmental damage
Indirect	Failure-related lack of production at the structure Failure-related lack of production in the vicinity of the structure. Lack of production due to failure-induced disruption of economic system	Suffering and disruption of social system Stress, fear and increased susceptibility to disease

Sometimes an acceptable risk level for these various losses is proposed by the owner of the project or by the society. A risk evaluation can be done by comparing with an agreed predefined risk level.

3.8 Functional analysis

An essential stage in the design process is the analysis of the functions the river training and bank erosion control works have to fulfil in order to remove the problem(s). It should be decided which functions are essential. In conducting the analysis one has to see whether there are any undefined elements in the problem, which should be taken care of.

The outcome of the functional analysis is a set of functional requirements for the future structure. The degree to which this structure will perform satisfactorily depends to a considerable extent on the requirements thus defined.

Functional requirements can be defined in relation to the structure as a whole and in relation to the component parts of its cross-section. The main functions which river training works structures as a whole can potentially solve are listed in Table 3.3. In addition, river training work structures may form the exposed boundary of a larger structure to which a major function can be assigned and here the function of the structure is to prevent the larger structure from being affected by hydraulic loadings from the sea or a river. The functional requirements for the river training works structures as a whole will largely determine their plan layout.

Table 3.3 Principal hydraulic functions of hydraulic structures (CUR Report 169).

Function of rock structure	Rock structure providing one or more functions												
	← marine structures →				← dams →			← river structures →			← others		
	Break-water	Seawall	Groyne	Offshore Break-water	Gravel beach	Res.dam protect.	Closure dam	River guide bunds	River spur dikes	River bank protect.	Sills, weirs	Bed protect.	Canals
(1) Shelter from waves/currents for vessels	*			(*)									
(2) Sediment trap to preserve navigation channels	*		*										
(3) Flood protection			*					*					
(4) Prevention of erosion of bank, coast or shoreline	*	*	*	*	*		*		*	*			
(5) Prevention of undermining of foundation												*	
(6) Protection against impact, including ballasting													*
(7) Hydropower													
(8) Water management							*						
(9) Flow guidance								*					
(10) Water conveyance													*
(11) Navigation	*												*
(12) Control of flow (over/through)											*	*	

Notes: Item (3) includes reducing presently too frequent inundation of low-lying ground and properties, as well as providing an adequate defence against extreme storm-surge events. Item (4) may include protection of erodable cliff formations, wind-blown dunes of beaches where the littoral processes are leading to denudation rather than accretion of beach material. In the latter context it is important to note that man-made influences may be leading to erosion, such as wave reflections generated by existing sea walls

The functions of component parts of a structure are best appreciated by an example, as listed in Table 3.4. The table shows the component parts of a river training works along with the primary functions which they perform. Examination of the functions of these component parts reveals that they fall into two categories:

- Functions related to the primary function of the structure;
- Functions related to the necessity of maintaining the structural integrity of the protected structure; i.e. the cover layer of a groyne fulfils a primary function in that it prevents or significantly attenuates damage by currents and waves.

Having considered the functions required for various structures and structural components (see Table 3.4), the other main aspect of functional analysis is to consider at what times over what durations and, if appropriate, at what rates these functions need to be fulfilled. Compliance with these requirements is perhaps best expressed in terms of risk of non-performance which in turn will enable them to be expressed in terms of reliability, serviceability, risk of loss or injury and an overall management strategy. Practical examples of such translation of functional requirements include:

- Serviceability example: the requirement a prescribed flood should be able to pass at certain river reaches without eroding its banks and flood embankments, except for a given extreme event (exceeding a given return period);
- Maintenance example: the requirement of a maximum amount of expenditure on maintenance during a prescribed period of time.

Table 3.4 Functions of typical component parts of a rock structure.

Component	**Function**
Scour or bed protection/ falling apron	Prevention of erosion
Sill	idem, to prevent sliding of subsoil into scour hole
Core	Attenuation of wave transmission, support to armour layers
Berm/ toe	Geotechnical stability Attenuation of (wave) overtopping Provision of additional geotechnical stability
Underlayer	Provision of stable footing to armour layer Filtration Erosion protection of subsoil/ core In-plane drainage Regulating course Separation and reduction of hydraulic gradient into subsoil/ core
Armour layer	Prevention of erosion of underlayers by currents and wave action Wave energy dissipation
Crest	Attenuation of wave overtopping Access for maintenance

Functional analysis should also consider possible changes of functions or importance of functional requirements, within the projected lifetime of the structure in order that appropriate flexibility can be provided to the envisaged structure and its planned maintenance.

Examples of the need to change or modify initial functional requirements or to introduce new functional requirements during the lifetime of a structure may include:

- changing hydraulic and morphological boundary conditions, such as changing alignment, rising water levels, river discharges, changing scour/ sedimentation rates, increasing IWT traffic, changing availability of local materials for maintenance and labour, etc;

3.9 Types of river bank protection works

3.9.1 General

In order to prevent erosion of riverbanks, suitable counter measures are required. These may be single or combined structural and non-structural measures. In general, three relevant concepts of erosion counter measures are existent:

- River training measures, which are intended to influence the flow conditions or channel properties downstream of the man-made intervention (active measures). River training works are structures constructed to protect the erodible materials in the river bed and banks from currents or waves that result in erosion and deposition of sediments. Erosion of the river bank is responsible for shifting of the bankline into the floodplain causing lost of land and infrastructure.
- Structures, which are aimed to decrease the hydraulic impacts directly in front of an area to be protected (partly active and passive measures), e.g. groynes, spur-dikes, hardpoints. They are classified as alternative indirect methods for protecting the riverbank by deflecting the current away from the bank.
- Structures to protect the actual bankline without relevant active interference on the flow (passive measures), i.e. revetments.

Either of them must be designed properly to resist hydraulic loads and to prevent the river channels from uncontrolled changing. These guidelines target on suitable tools for planning and implementation of permanent structural active and passive bank protection measures, e.g. groynes and revetments.

The types of river bank protection works described here are suitable for the middle and lower reaches of the river characterized by mild slopes and flow velocities in the range of 0.5 to 3 m/s and never greater than 5 m/s. Furthermore it applies to alluvial rivers with grain size characteristics D_{50} between 0.01 and 20.0 mm. River bank erosion control works in upper reaches of the river with high slopes and coarser materials are not part of this guideline.

River training and bank erosion control measures comprise permanent and recurrent measures (temporary), which are built either on the mainland, the floodplain, attached islands or which are built as floating structures.

3.9.2 Revetments

Revetment Structures are considered passive bank protection measures consisting of primarily armoured structures or armour layers preventing a bankline from erosion but which do not create significant interference with the passing flow. The hydraulic influence on the local flow condition is limited to changes in bed roughness.

Typical passive measures are revetment structures, which are built more or. less parallel to the flow to form an artificial sloped or vertical river bank. Vertical structures, e.g. retaining walls of stone-filled gabions, sheet-pile walls or similar, are not recommended for extremely mobile rivers, since these allow for high flow velocities directly at the structure toe (influence of water depth) followed by excessive scour, thus requiring substantial and expensive toe protection.

Structures, constructed on a slope of the riverbank or an embankment must be designed with a suitably chosen gradient and an adequately sized toe protection to support the revetment and to protect against scouring. Dependent on the prevalent soil to be protected against erosion and the external hydraulic loads by current and waves, many different types of revetments regarding the cross-section (berms, varying steepness, etc.) and the applied components are existent. Due to the differences between the river water level and the groundwater table additional pore pressure and subsoil is initiated, which may induce mass failure of banklines, especially after rapid receding of the river water level. Typically revetments have a cross-section as shown in Figure 3.7 and they are the most commonly used structures for protecting the riverbank from erosion because they provide a direct form of protection to the riverbank.

3.9.3 Groynes/ spur dikes

Spur dikes and groyne structures are applied to constrict the width of the river during the low season for improving inland navigation conditions and can serve also as a multipurpose structure providing indirect protection to the river banks during the high flow season by deflecting the main current with high flow velocities away from the riverbank. Spur dikes and groynes are constructed in series of structures varying in length and position along the riverbank as shown Figure 3.8. In between the groynes sedimentation will take place depending on the amount of suspended sediment load in the river.

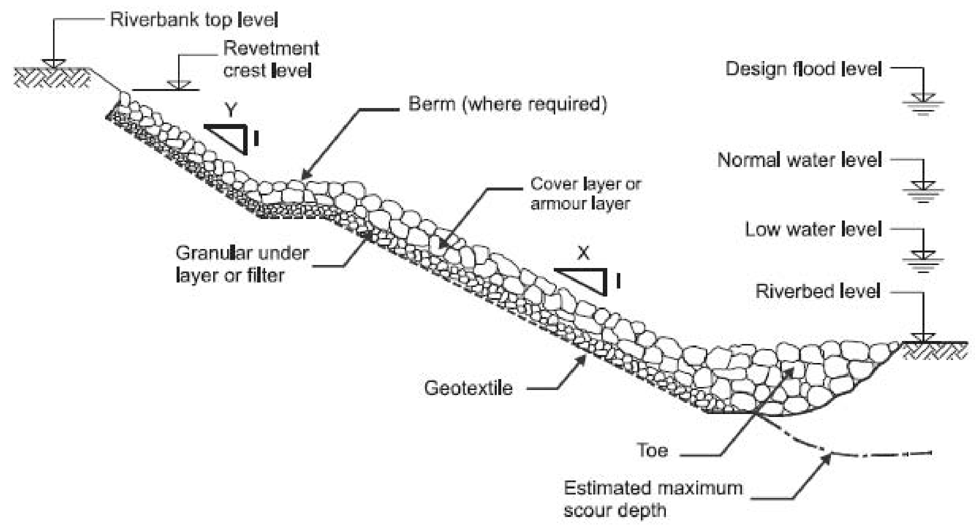


Figure 3.7 Typical cross-section of revetment for riverbank protection (CUR Rock Manual, 2008)

Spur dikes and groynes are suitable structures to control riverbank erosion but because of the effect of constriction of the river width, the flow velocity around the head of the groyne is higher than in normal conditions; therefore the structure generates scour and they can be damaged if there is not a proper design to protect against scour. Therefore a groyne requires heavier rock at the head. Groynes can be also constructed from various materials such as rock, concrete blocks and gabions depending on site specific requirements and availability of materials.

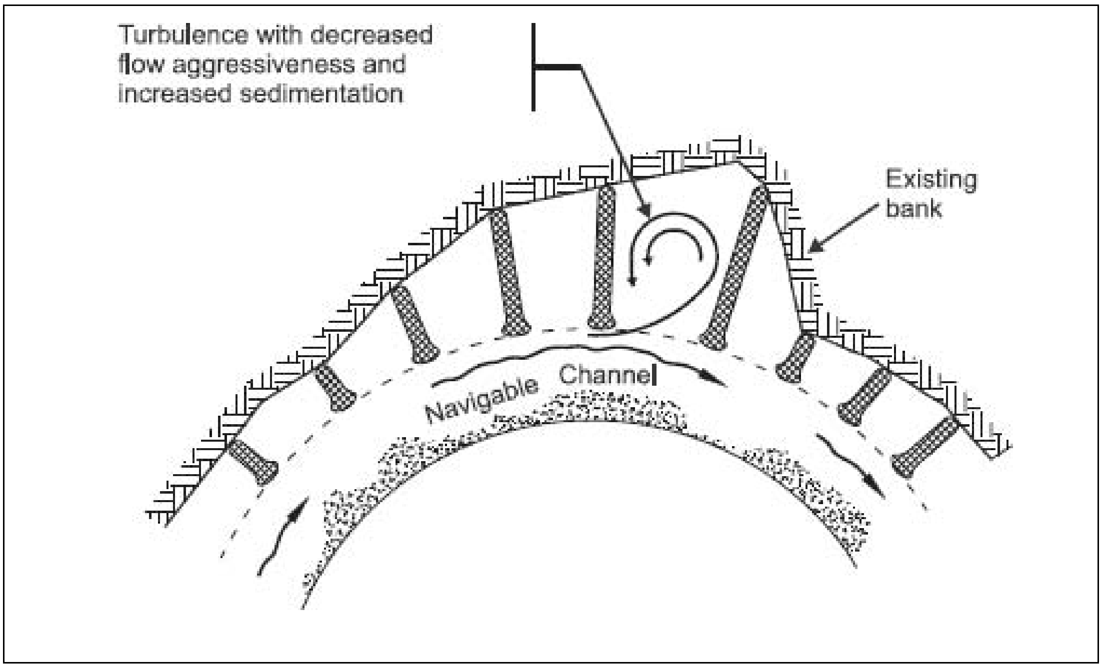


Figure 3.8 Example of field of groynes to improve navigation and protect the riverbank from erosion (CUR Rock Manual 2008).

From the point of view of implementation of groynes or revetments a decision shall be made based on costs and the need for construction of a full revetment.

3.9.4 Hardpoints

Another type of structures used for controlling riverbank erosion consists of hardpoints. Hardpoints are an erosion control structures consisting of stone fills spaced along an eroding bankline. Figure 3.9 shows an example of hardpoint constructed along the Mekong River. The structures protrude only short distances into the river channel and are supplemented with a root section extending landward into the bank to preclude flanking, should excessive erosion persist. The majority of the structure cannot be seen as the lower part consists of rock placed underwater, and the upper part is covered with topsoil and seeded with native vegetation. The structures are especially adaptable in long, straight reaches not subject to direct attack.

Short groynes can be also used as hardpoints to help to deflect the current from the river bank to be protected or to limit the length of embayment upstream of structures such as bridge abutments by restricting erosion. Hardpoints can provide cost effective protection but they do not provide full protection. Hardpoints can be also structures located adjacent to a river, such as buildings, monuments, bridges or levees that change the direction or rate of channel migration by interfering with the river’s movement.

Natural hardpoints exiting along the river can be integrated into strategies for riverbank stabilization. This is usually done at the time of development of IWRM plans including reducing the risk of riverbank erosion and breaching of flood embankments.

Figure 3.9 Hardpoint constructed along the Mekong for controlling riverbank erosion.

3.9.5 Structures to increase roughness and reduce current

Another type of measure suitable at some locations where the riverbank is retired from the main river channel during the dry season consists of retarding structures (known as porcupines in the Indian Subcontinent) or devices placed parallel to embankments and river banks to increase roughness and decrease the stream velocities and preventing erosion.

These structures can consist of pile retards be made of concrete, steel or timber. The design of timber pile retards is essentially the same as timber pile groynes. They may be used in combination with bank protection works such as riprap. The retard then serves to reduce the velocities sufficiently so that either smaller riprap can be used, or riprap can be eliminated.

3.10 Generation of alternative solutions

3.10.1 Criteria for selection

The next step in the design process is the generation of alternative design concepts to meet the boundary conditions and functional requirements. As indicated in the design logic diagram, this process draws on a wide range of technical experience and knowledge, much of which is summarized in this guideline, and covers the following principal areas:

- environmental assessment;
- determination of materials sources and properties;
- understanding the relevant hydraulic and geotechnical processes;
- structure-specific design methods;
- construction considerations;
- maintenance considerations.

The process of concept generation will also highlight the need for refinements in predictions of the physical site boundary conditions and this would normally be the stage to complete the appropriate data collection and analysis.

3.10.2 Materials availability and properties

Materials clearly represent a fundamental consideration in the generation of alternative solutions for a structure's cross section. In materials evaluation, available sources and types of materials must first be established. It is also required to make an assessment of existing sources of materials (rock, gravel, boulders, sand, cement, geotextiles, geobags, etc.). Choices must be made between local, easily available rock and other materials to be imported from some distant source. Also worthy of consideration in this context are alternatives to rock (concrete and industrial by-products), rock with stability improvement (by asphalt or colloidal cement grouts) and composite rock systems (gabions and mattresses).

The ability of riprap rock layer or revetment to resist the erosive forces of channel flow depends on the interrelation of the following stone and channel factors:

- stone shape, size, weight, durability, gradation; riprap layer thickness; and
- side slopes, roughness, shape, alignment, and slope.

The bed material and local scour characteristics determine the design of toe protection (falling apron), which is essential for groynes or revetment stability. The bank material and groundwater conditions affect the need for filters between the riprap and underlying material. Construction quality control of both stone production and riprap placement is essential for successful river training works and river bank protection. Riprap protection for flood-control channels and appurtenant structures should be designed so that any flood that could reasonably be expected to occur during the service life of the channel or structure would not cause damage exceeding nominal maintenance. While the procedures presented herein yield definite stone sizes, results should be used for guidance purposes and revised if appropriate, based on experience with specific project conditions.

Rock size computations should be conducted for flow conditions that produce the maximum velocity at the rip-rapped boundary. In many cases, velocities continue to increase beyond bank-full discharge; but in some cases backwater effects or loss of flow into the over-banks results in velocities that are less than those at bank-full. Channel bend riprap is conservatively designed for the point having the maximum force or velocity. For braided channels, bank-full discharges may not be the most severe condition. At lesser flows, flow is often divided into multiple channels. Flow in these channels often impinges abruptly on banks or levees at sharp angles. Precise guidance is lacking in defining design conditions for braided channels, although a correction factor for velocity is suggested.

In determining rock properties the intrinsic properties of the rock such as, density, porosity, degree of weathering and strength will first need to be determined, density being vital for early design work.

Production-affected properties must also be assessed to review the practical range of weight, size, shape and grading that is available. Durability of rock may vary and this factor can be incorporated into the design.

3.10.3 Understanding the physical processes

Understanding the hydraulic, morphological and geotechnical processes which may be involved in the design of any river training works solution that may be generated is clearly essential in designing both the plan layout and structure cross-section to meet the functional requirements. The designer must recognize the need to combine the bathymetric, hydraulic and geotechnical boundary conditions with the parameters of the structure (i.e. geometry and properties of materials).

Once these being defined, the hydraulic (i.e. max flow velocity and wave height) and geotechnical interactions (i.e. slope angle and pore pressures) have to be determined. The structural interactions (deformations and rock displacements) can be determined with the consequent loadings on the structure, together with the associated strength or resistance. In this way the eventual responses are determined, which may be interpreted in terms of damage or failure.

3.10.4 Mathematical models and remote sensing

It will increase insight in the morphological processes in the river system and provides specific information for estimating the general scour. Mathematical modelling (one and two dimensional) in combination with the geo-morphological and geological provides understanding of the short-term and long-term planform development of the river. It will also help to understand more site specific river processes such as the impact of rock outcrops, sand bar formations, geometry and celerities and the consequences of river existing and new bank protection works on the erosion patterns. This can be done by using aerial photographs, satellite images, and geotechnical investigations. If required field data shall be used for calibration of models.

Mathematical modelling is also very useful for determination and definition of hazard areas. The process shall be carried out based on:

- By means of modelling results;
- By means of historical records (rates of erosion m/year*). Based on past events and existing information the stretch on the main stream is identified as “special river erosion area”;
- Delineate hazard areas on floodplain and establish setback requirements, (depending on flood frequency adopted).

3.10.5 Structure-specific design methods

Some of the physical processes and design methods involved may be structure-specific and preparation of a range of alternative solutions for the identified problem will necessarily draw on experience of performance of existing structures. The designer must be fully aware of the functional possibilities and limitations of the various structures that he considers. In considering various solutions (e.g. a revetment versus rock groynes versus a longitudinal guiding dike for protecting a river bank or for deepening of a navigable channel) the designer should be aware of the known potential failure modes for these structures and their component structural elements.

3.10.6 Comparison and selection

The selection of structural measures and priority ranking of the erosion sites to be protected can be based on a multi-criteria analysis. It is envisaged that this methodology must be refined to the extent a more precise ranking is required for different alternatives of structural measures.

Technically the multi-criteria analysis (MCA) can be developed as a package of activities grouped into three main phases of analysis:

- Problem and model structuring, including discussion with experts to identify key benefits,
- Evaluation, and
- Elaboration of recommendations.

It is suggested that the multi-criteria analysis will be incorporated in the guidelines and further developed within the existing line agencies practice. During the next phases of the FMMP-C2 it is elaborate and to carry out a training programme activity including (i) introducing the multi-criteria analysis within the existing line agencies, (ii) to train technical staff to carry out and regularly revise the analysis and (iii) to refine the method to the extent required.

Ideally, in the stage of evaluation and selection the designer will have produced several alternative schemes for which he has sufficient data to carry out a full analysis for the envisaged lifetime of the structure. However, this is not always the case and then it is more likely that the engineer will try to choose the optimum design by adopting a least-cost solution using his own source of limited data.

Returning to the ideal effective selection process, an outline design of each of the identified range of possible structures is necessary, using the tools described above. In this stage, the designer might use the more simple techniques available of which many are presented in this standard: rules of thumb, simple analytical methods and empirical equations and graphs.

Application of more refined techniques, such as mathematical and physical modelling, should be restricted to the stage of final structural design and dimensioning of special structures (high cost). These tools are generally not appropriate for use in the stage of comparison and selection or for small structures (low cost).

After a series of concepts have been generated, for example the range of groynes or revetment cross-sections, unrealistic or uneconomic concepts can be eliminated in a selection process involving both objective and subjective criteria. Impractical concepts will be eliminated by the objective engineering criteria concerning:

- performance or the capability to meet the functional requirements;
- materials availability;
- construction time and equipment available;
- maintenance.

The selection process will also include more subjective criteria such as:

- risk level assessment during construction and operation, comparing assessed risk levels with acceptable risk levels;
- comparison with political, social and legal conditions;
- environmental impact assessment;
- complexity of operation and maintenance, relative to local technological experience and resources.

Results of a policy analysis can be combined here with objective criteria of functional performance to carry out a screening of the more economic solutions using Multi-Criteria Analysis (MCA). The advantage of MCA is that it is the only known method of including both quantitative and qualitative criteria in one assessment. An example of MCA is given here below in Figure 3.10.

3.10.7 Prioritization of river erosion problems

Preparing a prioritization schemes shall be based on scores assigned to the threatened resources in a focal area and the potential problems. It shall be a systematic process that leads to ranking of river stretches based on the significance of bank erosion and affected infrastructure and structures.

Multi-Criteria Analysis (MCA)

The method basically consists of making a matrix, with the various alternatives listed horizontally (I to V in Table 1) and the selection criteria listed vertically (A to F). An appreciation of alternatives is made with respect to each criterion. The appreciations are expressed in a mark using a predefined scale (e.g. integers from 0 to 10) as shown in the example of Table 1.

Table 1 *Example scoring table of a MCA (adjustment of weight factors, see Table **2)***

	ALTERNATIVES				
CRITERIA (+ **weight factors)**	I	II	III	IV	V
A rock volume (1)	2	5	1	1	8
B environmental impact (4)	7	7	6	2	2
C construction time (2)	9	0	5	4	1
D maintenance (3)	8	8	3	1	1
E risk level (3)	6	3	10	7	5
F initial cost (3)	6	5	3	6	6
resulting appreciation:	108	81	83	59	54

Introduction of weighting factors improves the method and these factors can be adjusted by agreement (i.e. within the project management team or with a group of interested parties as part of a policy analysis process.

A more objective approach to determine weight factors, which can also be carried out by the project management or policy analysis team, is to assign priorities for all combinations of criteria. For example, assigning 1 to each dominating criterion thus leaving the other with 0, weight factors can be found by adding all 1's for each criterion (see Table 2). Alternatively, use of 2,1 and 0 also anticipates to equally important criteria (which are then assigned 1).

Table ***2*** *Adjustment of weight factors for MCA*

CRITERIA	A	B	C	D	E	F	weight
A	-	0	0	1	0	0	1
B	1	-	1	1	0	1	4
C	1	0	-	0	0	1	2
D	0	0	1	-	1	1	3
E	1	1	1	0	-	0	3
F	1	0	1	0	1	-	3

Figure 3.10 Example of an MCA.

3.11 Final design and detailing

3.11.1 Background

Having selected an appropriate solution to the identified problem, the final design and detailing can proceed, taking into account all the previous design thinking. At this stage further alternatives may appear, but these will generally be minor variations on the basic option that has been selected arising from the interplay of optimization of functional efficiency and minimization of total cost. Variations and adjustments will tend to concentrate on minor details of plan layout and detailing of the cross-section.

The final design essentially consists of a series of calculations and model tests to check and adjust as necessary all details of the structure and to produce tender documents and a design report.

Before commencing the final design, a decision must be taken as to whether to proceed on a deterministic or probabilistic basis. Preliminary designs at the stage of generation of alternative solutions will preferably have been carried out using deterministic or simple probabilistic methods, whereas in the final design process, depending on the importance of the structure, a more thorough probabilistic approach may be adopted. Probabilistic methods have the advantage of providing the designer with a quantifiable list of probabilities, the interrelation of which is identified in a so-called fault tree or failure tree. Knowledge of the significance of individual failure mechanisms in relation to the overall functioning of the structure is particularly important in the structure optimization process.

The hydraulic and geotechnical tools used to check and adjust the hydraulic and structural performance in the final design will be a combination of established theoretical and empirical approaches along with mathematical and physical modelling (as indicated in Chapter 5).

The calculations and model tests will have the objective of ensuring that the final structural design meets all the functional requirements given the physical site conditions and other boundary conditions. In this latter regard, it will naturally incorporate all the latest information on boundary conditions; particularly in relation to physical site conditions (results from surveys commissioned earlier in the design process may only become available at this stage).

The process of checking and adjusting the selected design will involve each of the sub-processes involved in the stage of generation of alternative solutions, but carried out to a greater degree of refinement.

Evaluation of the functional (hydraulic and structural) performance is carried out with corresponding limit states. Limit states are associated with loading conditions, the exceedance of which will lead to a significant decrease in performance. Probabilities of failure hence refer to probabilities of exceeding a given limit state, as is explained below using some examples.

3.11.2 Limit state conditions

In the final design the functioning of the structure under design conditions is evaluated. These design conditions may be determined by either one the categories of performance:

- performance under extreme conditions: Ultimate Limit State (ULS)
- performance under normal conditions: Serviceability Limit State (SLS)

Here the ability of the structure to survive extreme conditions is checked. This is done by evaluating all failure mechanisms likely to occur under the specified extreme conditions. In this case the limit state is the Ultimate Limit State (ULS), defining collapse or unacceptable serious deformations of the structure for conditions exceeding the ULS. Examples of such limit states are (i) soil stress conditions leading to sliding and (ii) wave heights causing breakage or displacement of armour elements.

The performance is evaluated under the `normal' or daily conditions, which the structure will be exposed to during most of its lifetime. In this case the limit states are the Serviceability Limit States (SLS). These limit states define (mostly hydraulic) conditions, the exceedance of which will disable various activities or services provided by the structure.

In addition, the long-term performance of the structure under 'normal' conditions (analogous to fatigue) will be evaluated here (i.e. degradation of a scour protection, deterioration of armour elements).

Also during construction specific construction-dependent loading conditions may be expected, which (part of) the structure is exposed to. Also these conditions, eventually affected by hydraulic interactions with the structure, can be decisive to the design. Examples are: (i) the strong local current in a closure gap during construction of a groyne or guiding dike, (ii) wave attack on a filter layer under construction and (iii) surcharging the structure by driving with heavy construction equipment.

In ensuring that both the whole structure and its component parts have complied with ULS and SLS requirements, it is suggested that a list of aspects be prepared and a check made to ensure that limit state criteria for each aspect are satisfied. Such a check list might include for a typical river training works constructed with rock aspects such as:

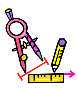
- overall plan geometry (i.e. side slopes, crest level);
- armour (waterfront face, toe, crest, rear face);
- underlayers and filters;
- core design and foundation drainage;
- arrangements at limits of or transitions between parts of the structure.

With limit-state criteria including aspects such as:

- run-up, overtopping and armour stability;
- filter criteria;
- pore pressures for geotechnical stability;
- wave transmission;
- allowance for settlement;
- avoidance of outflanking.

3.12 Design documents

On completion of design and detailing, there will be two main products: a design report and a set of tender documents.



The design report will contain a summary of the design process as described above but specific to the structure in question, explaining the reasons for the various choices made. It will generally include the following key components:

- description of selected structure and selected process;
- materials to be used, reasons for selection and anticipated method of production and transport to site;

- description of how the selected structure meets the functional criteria up to defined limit states;
- safety factors or probabilities of failure in various hydraulic and geotechnical failure modes;
- construction methods and equipment envisaged;
- description of maintenance strategy as agreed with the owner;
- environmental (impact) statement;
- cost estimates.

The tender documents will be a standard form and as usual contain drawings, specifications, bills of quantities, and conditions of contract. Both specifications and bills of quantities should as far as possible be prepared in order to avoid unrealistic or impractical requirements or measurement techniques being imposed, whilst still retaining proper control over the quality of the construction work.

3.13 Quality assurance and control

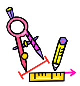
Quality assurance is a style of management or a management philosophy which -when properly applied- affects every aspect of working life. Its application within the construction industry is now well-established and growing. Quality assurance must therefore be covered in separated guidelines, although space precludes more than a brief treatment. The user is strongly advised to consult the relevant national and international standards on the subject.

Quality control systems are essential as a part of the quality assurance measurement philosophy, in order to be able to guarantee successful completion and lifetime functioning of a structures to control or stop river bank erosion. A typical quality control system will consist of four main elements:

- a set of technical specifications;
- measuring systems and procedures;
- quality control or comparison of standards and;
- results of measurements; procedures to correct or change the production process.

In order to provide for a proper quality control in a project, the project quality assurance system and organization should be capable of integrating the various quality control systems. It should be noted that tender documents produced at the detailed design stage should provide a sound basis for quality control during the construction contract.

3.14 Schedule of works

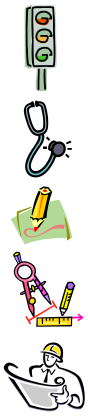


The schedule of works should include:

- A list of the planned works, along with their intended timelines and costing.
- A detailed description of the methods to be used for each of the works.
- A series of drawings and aerial photographs or satellite images that show the location and extent of all planned works. The drawings shall be clear and unambiguous.
- Detailed plans of all erosion control river works, and other structures.

General statements about the construction methods to be used can be given in the body of the design and implementation plans. However, detailed descriptions of the proposed works should be included in the works schedule on a section-by-section basis. This is especially important if the works involve vegetation clearance, in-stream works, river bank works (e.g. rip-rap, gabions, groynes, vegetation, etc), labour intensive work or the use of machinery. Each of the aforementioned descriptions should be linked to a map.

3.15 Existing construction capacity



It is envisaged that additional information is required on the existing structural measures and river bank protection construction capacity, characterized by number of contractors, skilled and unskilled labour, available equipment and experience with similar types of works. In some cases, the existing construction capacity may be a constraint for implementation of structural measures and rehabilitation of existing ones. On the other hand, this constraint may be taken into account when selecting the type of pre-feasibility design for structures, river bank protection and rehabilitation works required.

This is especially true when considering the opportunities to involve local people in the construction of such works: by adjusting the design to locally available skills (and materials), the river bank protection and rehabilitation works may be implemented using local resources only, potentially increasing the construction capacity for the more conventional type of design.

3.16 Monitoring of river bank protection works

Monitoring the river stretch in the focal area before, during and after the project will give an indication of the success of the project and the maintenance needed.

If properly planned, members of the local community, schools and the local authorities may be able to help.

Several techniques can be used to monitor the success of a river bank erosion control works. The most common practice is to set up a program of regular inspections by a qualified person to monitor the effectiveness of the works or management program, and implement remediation works if necessary.

Line agencies can assess the condition of the river using existing country river survey guidelines. The guidelines shall encourage groups to examine all components of the river, including the riparian vegetation, cross-section, erosion and sediments. Photo-points (photographs taken from fixed locations) can also be used to show the before and after condition of the river, and to monitor long-term changes after the project has been completed.

3.17 Provision for maintenance

The aim of the FMMP-C2 best practice guidelines is to improve the long-term condition of the river. Therefore, all groups must show how they will maintain the improvements they make to the river after measures have been implemented. Maintaining the works is particularly important in the Mekong at stretches that can degrade quickly and the money spent improving them can be wasted if there are no follow-up works.

Long-term maintenance of the river can be funded by arrangements that share the costs equitably between the beneficiaries. For example, a preferred arrangement could be a river works district, which provides a framework for collecting the funds and administering and managing the maintenance. Generally, it is recommended that local councils establish river works districts and set up special council committees to administer them.

3.18 Recommended management plans

Effective riverbank protection and stabilisation planning helps protect assets such as urban areas, cultural heritage sites, agricultural land, infrastructure and riparian zone vegetation on or near the riverbank.

The general recommendations for bank erosion management in focal areas, river reaches and demonstration projects shall include the following elements:

- Adopt a hazard corridor.
- Formulate or amend existing flood control regulations and policies to include river management policies.
- Regulate all new developments within the hazard corridor by requiring a special use permit.
- Establish a no-build zone close to the river banks (habitable structures and businesses should be setback at a minimum distance from the top of the bank.
- Request erosion study to certify that proposed developments will not be affected by river bank erosion over the period of planning.

3.19 Peoples participation

It is essential that concerned agencies obtain the support and agreement of the majority of landowners along the river when developing their river erosion control plans. Council staff can advise community groups about the other interest groups and individuals that should be consulted. These people should be consulted to determine the likely effects of the plan on them and to obtain their consent.

It is also important to consider land use in the floodplain and to link the plan to other management plans in the basin (i.e. BDP).

If river erosion control plans are being developed for other focal areas of the river they should be linked and integrated with each other. Groups or agencies should work out how they can co-ordinate their works with those of nearby groups. Groups or agencies working along the same river or in the same region should be encouraged to share resources and integrate their plans.

3.20 Design tools: design of the protective layer

3.20.1 Introduction

The guideline methods given in the following sections of this standard can be used for dimensioning the protective layer units of the river training works such as upper cover layer and falling and launching aprons, respectively. In the given formulae loads from current and waves are considered, from which the larger result must be used for dimensioning of the protective layers. For stone filled mattress systems generally both, the mattress thickness and the nominal size of the mattress filling must be determined.

3.20.2 Revetments: characteristics

Alongshore revetments are mostly used in regulation works and shoreline protection. Alongshore revetment is composed of three parts: toe, the revetment, and crest. The toe is to protect slope from erosion, and also a foundation for the revetment to base on. The crest is to protect the revetment from erosion by surface flows and other impacts. The revetment is the part that connects toe and crest, the revetment is used to protect the bank from erosion by flow, wave, and hydraulic pressure and seepage flow.

In general, two main types of revetments are distinguished: rigid and flexible revetments.

- Rigid revetments are made of concrete (plain, reinforced or precast slabs), cement mortar, soil cement, sheet piles (steel or timber), brickwork or stone and mortar. They are mainly impermeable unless water and soil movement is possible through the joints or special pressure-relief holes.

- Flexible revetments are made out of riprap (loose, bound or grouted stone), concrete blocks (loose, interlocking, cable connected or anchored) fabric and other containers (bag blankets, fabric mattresses, tubes, wire, bamboo or polymer gabion baskets and mattresses) bitumen (asphalt, bound or grouted stone or willow) and many other materials (old tires, oil drums, etc.).

The recommended procedure of revetment design consists of:

- assessing hydraulic loads,
- assessing the erosion resistance of the subsoil,
- designing the stable cover layer,
- designing the sub-layer if required,
- designing transitions between different systems and elements of protection.

Before giving guidance on the types of revetment most suitable for each case, it is useful to define a few major characteristics that affect the overall behaviour of revetments. One of these concerns the very nature of the revetment, whether it is essentially formed by living plants (bioengineering), or by hard units (structural or engineered revetments) or by a combination of both types (biotechnical engineering). Another important characteristic is the flexibility of the revetment, defined as the ability to maintain good contact with the underlying soil during gradual settlement.

A revetment such as a concrete lining is considered rigid. It has, in fact, little capacity to accommodate variations in the base soil without cracking, in spite of being able to follow bank contours effectively before it cures.

Riprap is an example of a flexible revetment because of the ability of the individual stone units (each of them very rigid) to rotate next to its neighbours while remaining in close contact. Also, because riprap is formed by more than one layer of units, when erosion of the underlying soil occurs, the individual units can easily move to fill small gaps and holes. Flexible revetments are generally advisable for protection work, and particularly so for situations where soil instability is expected and where maintenance is to be kept to a minimum.

Design for the toe has to be met following requirements:

1. The structure of the toe and construction material has to be:
   - Resistant to be moved by flow, and bed load transport,
   - Adaptable to the changes of river bed at the construction site,
   - Resistant to scour of flow,
   - Easy for construction and maintenance.

2. The level of the toe should be chosen depending on bed level, expected scour and construction method. For some small works, the apron is located 0.5m higher than low water stage with 95% of exceedance.

3. The toe protection can be made of riprap stone, concrete blocks, fascine mattresses and rock. As alternative toe protection in case the available stone does not meet the required size, gabions can be used instead of rock. Guidelines for calculation of required toe protection are given in the following sub-sections. The diameter of the protective elements (stones, concrete blocks or gabions) can be determined by the stability criteria in Sections 3.20.8 to 3.20.12.

3.20.3 Destructive forces: currents

The comparison of different methods regarding the calculation of unit dimensions of revetment cover layers and toe protections (Pilarczyk, 1990) show only marginal deviations within the range of application for most of alluvial rivers and can be applied to Vietnamese rivers as well. Therefore, the widely used Pilarczyk method (1990) is recommended, because it includes the turbulence intensity by an empirical coefficient (yet merely in a very rough and qualitative way), still keeping a certain practical simplicity. It was initially developed from laboratory and field tests mainly on rip-rap, but, introducing coefficients considering the specific properties of different protection layer materials, also allows for the application on other types of revetments. The general formula for the design against current loads is given by

$$D_n \geq \frac{0.035\bar{u}^2}{(1-n)\Delta_m 2g} \frac{\phi_{sc} K_t K_h}{K_s \varphi_{cr}}$$

where: D_n = characteristic size of the revetment cover layer material (single unit size for loose elements, thickness of mattress systems) (m)

$\bar{u}$ = depth averaged flow velocity; if replaced by ub = 0.6 $\bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of Kh = 1.0 must be applied (m/s)

Δ_m = relative density of submerged material = $(\rho_s - \rho_w)/\rho_w$

G = acceleration due to gravity (= 9.81) (m/s^2)

Φ_{sc} = stability factor for current

Ψ_{cr} = critical shear stress parameter

K_T = turbulence factor

K_h = depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio:

$$K_h = 2\left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$$

where: k_r = D_n for relatively smooth material

K_s = bank normal slope factor:

$$K_s = \sqrt{1-\left(\frac{\sin\alpha}{\sin\varepsilon_s}\right)^2},$$

neglecting the longitudinal slope of the bank or structure, which is reasonable for Vietnam rivers and a conservative assumption slope angle of bank or structure angle of repose considering the material specific internal friction:

α = slope angle of bank or structure (^{O});

ε_s = angle of repose considering the material specific internal friction (^{O}).

Table 3.5 shows the range of values for the stability factor and the shields parameters that are generally used in the design of river training works structures and Table 3.6 the recommended values for turbulence factor.

Table 3.5 Stability factor Φ_{sc} and Shields parameter Ψ_{cr} for various cover materials.

Protective layer type	Stability factor Φ_{sc}		Critical shear stress parameter (Shields) Ψ_{cr}
Cover layer	Continuous protection	Exposed edges, transitions	
Randomly placed, broken rip-rap and boulders	0.75	1.5	0.035
Concrete blocks, cubical shape, randomly placed in multilayer	0.8	1.5	0.035
Concrete blocks, cubical shape, hand placed in a single layer	0.65	1.25	0.05
Concrete blocks cable connected	0.5	1.10	0.06
Wire mesh mattress/ gabions	0.5	1.00	0.07
Gabion/ mattress filling by stones	0.75	1.5	0.09

Table 3.6 Turbulence intensity factor KT (current).

Turbulence intensity	K_T Gabions, mattresses	K_T Others
Normal turbulence in rivers	1.0	1.0
Non-uniform flow with increased turbulence, mild outer bends	1.0	1.5
High turbulence, local disturbances, sharp outer bends	1.0	2.0

For calculation of the required thickness of stone or brick filled mattress systems or other interconnected units the relative density (1-n) Δm, considering the volume of the voids between the individual filling elements, must be applied instead of Δm. The percentage of voids in the mattress fill and between interconnected blocks can be estimated to:

n = 0.4 for stone filled mattresses,
n = 0.15 for brick filled mattresses,
n = 0.1-0.3 for cable connected block mattresses.

The minimum thickness of stone filled mattresses should not be chosen smaller than $1.8D_{50}$ (with D_{50} = Dn/0.85) or than 15 cm. Besides the stability of the whole mattress, the weight of the individual stones should be sufficient to prevent from excessive movement and thus loads on the wire mesh material.

The required nominal diameter D_{50} of the filling material can also be calculated by taking the respective stability coefficients for mattress filling (Δ_m, $\Phi_{sc,}$ Ψ_{cr}, ε_s) into account. The minimum size of the stones must be larger as compared to the width of the wire mesh material.

3.20.4 Destructive forces: waves

Also for wave induced impacts on armouring units, several theoretical calculation methods are available for the design. Due to the various input parameters involved in the different methods a direct comparison is rather difficult. The more universal formula by Pilarczyk (1990) allows for calculation of different structure components and includes the breaker type specific dynamics of the wave impact by introducing the breaker similarity index The minimum dimensions for the stability of the cover material under wave attack can be deter-mined as follows (Pilarczyk, 1990):

$$D_n \geq \frac{H_s \xi_z^b}{\Delta_m \psi_u \phi_{sw} \cos\alpha}$$

where: D_n = characteristic size of the revetment cover layer material (single unit size for loose elements, thickness of mattress systems; m)

- Δ_m = relative density of submerged material = $(\rho_s - \rho_w)/\rho_w$
- Φ_{sw} = stability factor for wave loads
- Ψ_u = system specific stability upgrading factor bank normal slope angle
- ξ_z = wave similarity parameter
- H_s = significant wave height (m)
- α = slope angle of bank or structure (^{0})
- b = wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

ξ_z = wave similarity parameter $= \tan\alpha \dfrac{1.25 T_m}{\sqrt{H_s}}$;

where: T_m = mean wave period (s)

The wave similarity parameter determines the type of wave breaking, which is decisive for the actual wave impact. The formula is restricted to values $\xi z \leq 3$ and $\cot\alpha \geq 2$, i.e. to plunging breakers, which generate high local pressure heads. Otherwise overestimation of the unit size is likely, because the dynamics of the breaking process are diminishing.

As for the design against current attack, the required thickness of stone or brick filled mattress systems must be calculated on basis of the relative density (1-n) Δ_m, considering the volume of the voids between the individual filling elements. The minimum thickness of the mattress as a unit should be larger than 1.8 Dn. Stability coefficients for wave attack are shown in Table 3.7.

Table 3.7 Coefficients for the design of various cover materials against wave attack.

Revetment type	Stability factor for incipient motion Φ_{sw}	Stability upgrading factor Ψ_u	Interaction coefficient b
Randomly placed, broken rip-rap and boulders	2,25 - 3.00	1.0 – 1.33	0.50
CC-blocks, cubical shape, randomly placed in multi-layer	2.25 – 3.00	1.33 - 1,50	0.50
CC-blocks, cubical shape, hand-placed in single layer chess pattern	2.25	1.50	0.67
CC-blocks, (cable connected)	2.25	1.80	0.67

Revetment type	Stability factor for incipient motion Φ_{sw}	Stability upgrading factor Ψ_u	Interaction coefficient b
Wire mesh mattresses/ gabions	2.25	2.50	0.50
Gabion/ mattress fillings by stones	2.25	2.50	0.50

3.20.5 Rip-rap revetments: design against currents[6]

The first step is to calculate the diameter of the armour unit by the formulas given in Section 3.20.3. Typical value of some of the coefficients to be used in the calculation of the D_n is given in Table 3.8 and Table 3.9 below.

Table 3.8 Value of some coefficients.

Coefficient	Unit	Value
Turbulence intensity K_t		
- Normal turbulence in rivers		1.0
- Non-uniform flow with increased turbulence, mild outer bends		1.5
- High turbulence, local disturbances, sharp outer bends		2.0
Angle of repose	(^{0})	
- Geotextile		20
- Granular		25
Material density		
- Rock ρ_s	(kg/m^3)	2600
- Water ρ_w	(kg/m^3)	1000
Stability factor		
- Continuous protection		0.75
- Exposed edges, transitions		1.25 (1.5)
Critical shear stress Parameter ψ_{cr} (Shields)		0.035

Table 3.9 Value of slope factor.

Embankment	Slope	1: 2.5	1: 3	1: 3.5	1: 4	1: 5
	Angle	21.8^o	18.4^o	15.9^o	14^o	11.3^o
Rip-rap on geotextile filter mat (ε_s = 20^o)		-	0.385	0.599	0.707	0.820
Rip-rap on granular filter (ε_s = 25^o)		0.477	0.665	0.761	0.820	0.886

[6] Government of Bangladesh Ministry of Water Resources – WARPO. Guidelines and Design Manual for Standarized Bank Protection Structures. Pilot Project FAP21. December 2001

3.20.6 Rip-rap revetments: specification and construction details

For broken stone material: nominal diameter $D_{50} = D_n/0.85$. For stone or boulders this relation may be used for dimensioning and should be verified for detailed design.

In case of surface or pattern grouting, the nominal diameter found from wave loads (which are in general decisive for the stability of grouted material) can be reduced to:

$D_{50\,(grout)} =$ $0.9D_{50(rip\text{-}rap)}$ for surface grouting;
$D_{50\,(grout)} =$ $0.6D_{50(rip\text{-}rap)}$ for pattern grouting.

The typical grading envelop for rip-rap material (recommended by PIANC, 1997) is shown in Figure 3.11.

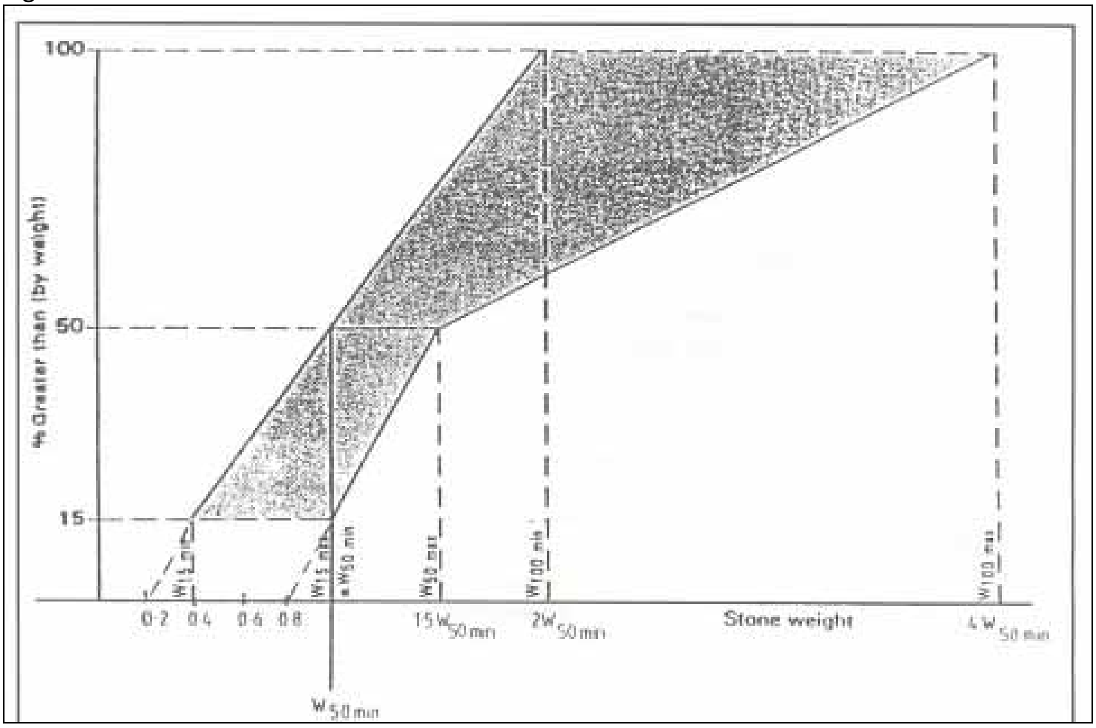


Figure 3.11 Rip rap grading envelope.

W_{50} is weight of a single unit of nominal diameter (kg):

$$W_{50\min} = D_n^3 \rho_s = \left(\frac{D_{50}}{0.85} \right)^3 \rho_s$$

Rip-rap with dimensions varying between 0.20 and 0.40 m is widely available in north of Lao PDR and Thailand and, to a lesser extent, in the southern parts and in the Mekong Delta. Rip-rap (or rock) is exploited and stored from quarries at various locations.

Rock specified for construction of revetments and groynes along the Mekong can be applied as per the following categories:

- Grade 1: nominal grade D50=0.15m; 5 - 40 kg
- Grade 2: nominal grade D50=0.25m; 10 - 50 kg
- Grade 3: nominal grade D50=0.25m; 10 - 90 kg
- Grade 4: nominal grade D50=0.25m; 1 - 60 kg
- Grade 5: nominal grade D50=0.30m; 1 - 200 kg

3.20.7 Rip-rap revetments: design against waves

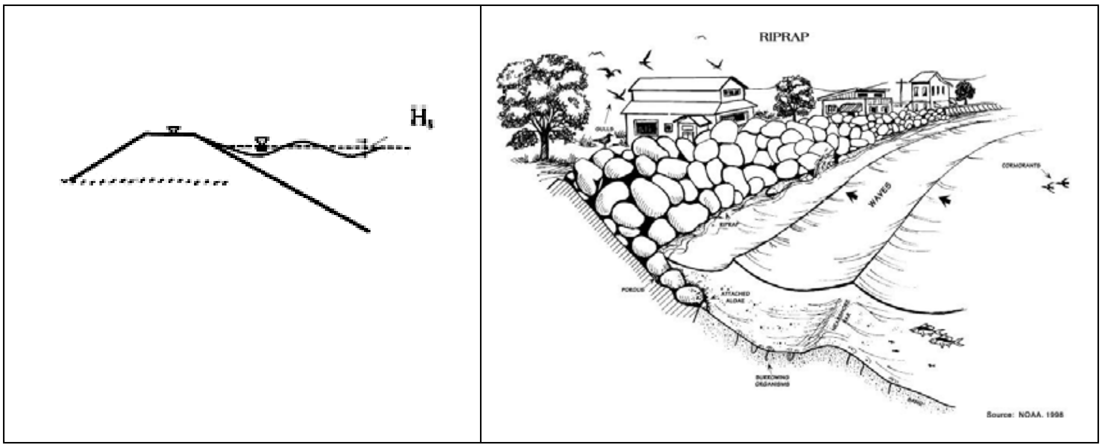


Figure 3.12 Riprap protection under wave attack.

Diameter of the armour stone is determined by the formulas in Section 3.20.3. The recommended values for coefficients to be used are given by Table 3.10.

Table 3.10 Value of some coefficients.

Coefficient	Unit	Value
Stability factor for incipient motion Φ_{sw} (3)		2.25
Stability upgrading factor Ψ_u (4)		1.0
Interaction coefficient b		0.5
Angle of repose		
- Geotextile	(^{o})	20
- Granular	(^{o})	25
Material density		
- Rock ρ_s	(kg/m^3)	2600 - 3000
- Water ρ_w	(kg/m^3)	1000 - 1025

Notes: (3) for maximum tolerable damage of a two-layer system on granular filter Φ_{sw} = 3.0;
(4) for a two layer rip-rap system (no damages); in case of certain damages are tolerated the upgrading factor might be increased to Ψ_u = 1.33.

3.20.8 Cube-shaped concrete blocks: design against currents

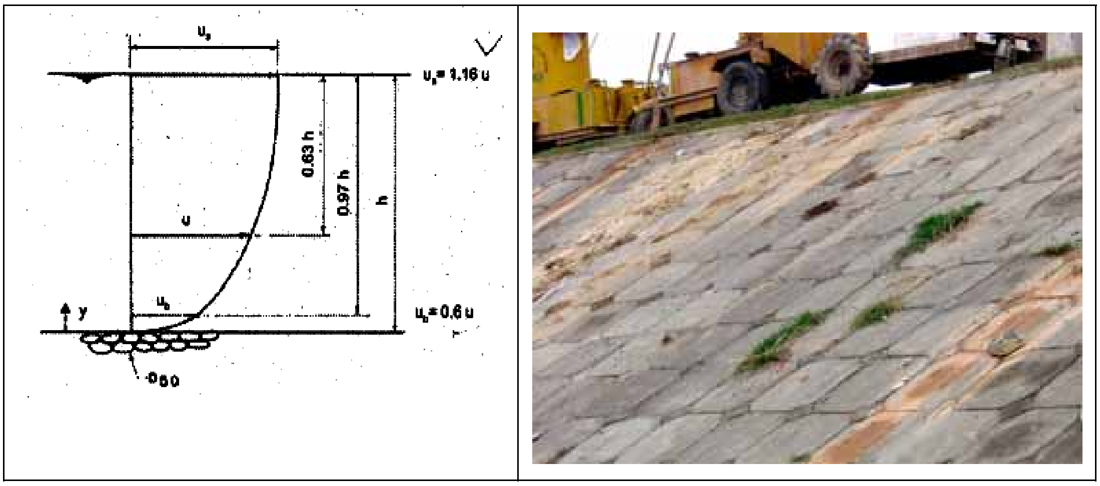


Figure 3.13 Cube-shaped concrete block revetment (hand placed in a single layer).

The diameter of the armour unit is calculated by the formulas in Section 3.20.3. Table 3.11 shows some recommended coefficients for the calculation of D_n; the values of the slope factor can be found in Table 3.9.

Table 3.11 Value of some coefficients.

Coefficient	Unit	Value
Turbulence intensity K_t		
- Normal turbulence in rivers		1.0
- Non-uniform flow with increased turbulence, mild outer bends		1.5
- High turbulence, local disturbances, sharp outer bends		2.0
Angle of repose	$(^0)$	
- Geotextile		20
- Granular		25
Material density		
- Concrete (gravel aggregated) ρ_s	(kg/m^3)	2400
- Water ρ_w	(kg/m^3)	1000
Stability factor		
- Continuous protection		0.65
- Exposed edges, tran-sitions		1.25
Critical shear stress Parameter ψ_{cr} (Shields)		0.05

3.20.9 Cube-shaped concrete blocks: specifications and construction details

To increase the stability of such block work (revetment), the gaps between individual CC-blocks should be filled with smaller gravel material (for increased interlocking effect).

The material density of concrete units made from coarse aggregates may vary between ρ_c = 1,980 to 2,400 kg/m3 dependant on the quality of the aggregates and the mixture of concrete. Adequate care and control during concrete production is important for the stability of individual element. Frequent laboratory or in-situ analysis of the physical stability is required.

3.20.10 Cube shaped concrete block protection: design against waves

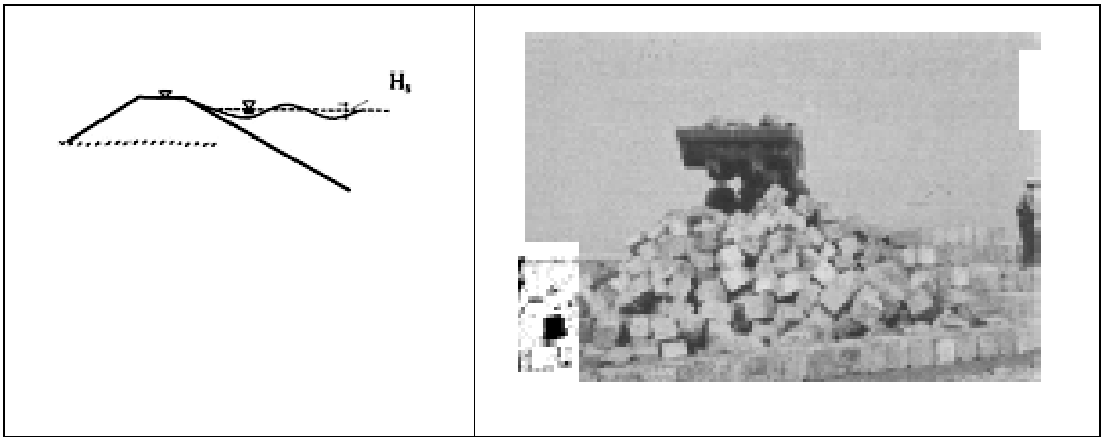


Figure 3.14 Riprap protection under wave attack.

The diameter of the armour stone is determined by the formulas in Section 3.20.3. Recommended coefficients are shown in Table 3.10, only the ‘interaction coefficient b’ is in this case 0.67 (instead of 0.5).

3.20.11 Stone-filled mattress systems: design against currents

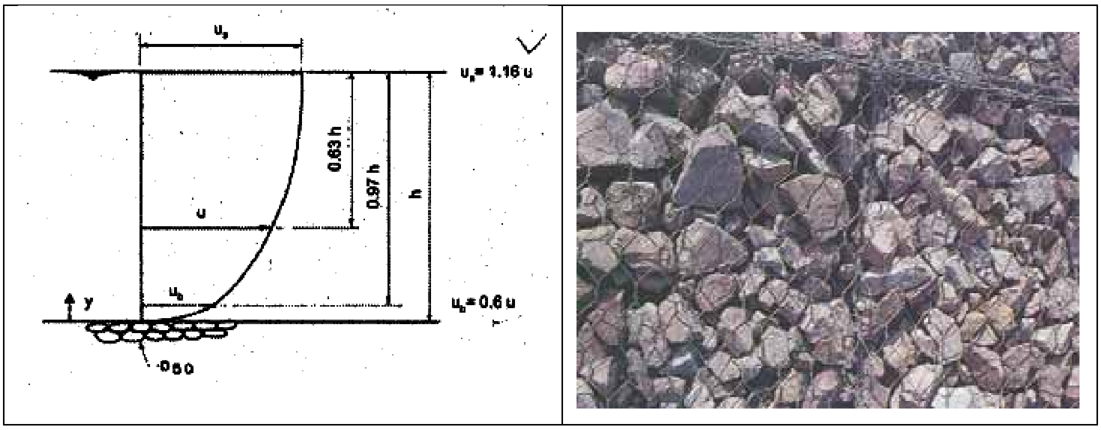


Figure 3.15 Gabion revetment.

Thickness of the armour layer is calculated by the formulas in Section 3.20.3. Recommended coefficients are shown in Table 3.12, Table 3.13 and Table 3.15.

Table 3.12 Value of some coefficients.

Coefficient	Unit	Value
Turbulence intensity K_t		
- Normal turbulence in rivers		1.0
- Non-uniform flow with increased turbulence, mild outer bends		1.0
- High turbulence, local disturbances, sharp outer bends		1.0
Angle of repose	(^{0})	
- Geotextile		20
- Granular		25

Coefficient	Unit	Value
Material density - Rock ρ_s - Water ρ_w	 (kg/m^3) (kg/m^3)	 2600 1000
Stability factor - Continuous protection - Exposed edges, transitions		 0.5 1.0
Critical shear stress Parameter ψ_{cr} (Shields)		0.07

Table 3.13 Value of slope factor.

Embankment	Slope	1: 2.5	1: 3	1: 3.5	1: 4	1: 5
	Angle	21.8^o	18.4^o	15.9^o	14^o	11.3^o
Gabion on geotextile filter mat (ε_s = 20^o)		_	0.385	0.599	0.707	0.820
Gabion on granular filter (ε_s = 25^o)		0.477	0.665	0.761	0.820	0.886

Table 3.14 Value of angle of repose and density of material.

	Angle of repose	Density ⍰$_s$ in kg/m^3
Gabion/ mattress fillings by stones	45^o	2600

In case of very high design flow velocities (u_b > 3 m/s) or large wave heights (H > 1m) a granular sub-layer with minimum thickness of 0.2 m should be provided between geotextile filter mat and wire mesh mattress.

3.20.12 Stone-filled mattress systems: specifications and construction details

Wire material and anchoring: Besides the sufficient weight of the mattress a proper interlocking between the individual mattresses and appropriate anchoring of the mattress elements is most important. The diameter of the wire material should be 4mm minimum, the anchor and interconnecting cables should be chosen to 10mm (strand-wire). In case wire mesh mattress systems are applied as launching apron only proprietary box gabions (i.e. RENO) should be used.

3.20.13 Filter layers: granular filters

The properties of granular filters depend significantly on the particle size. The filter criterion relates the grading of the filter to that of the subsoil. If the filter also has got drainage function, it is necessary to check for filter uniformity to ensure that internal migration of fines does not occur. Various authors have developed minimum filter requirements. Pilarczyk (1990) defined the following criteria regarding the relation between characteristic grain sizes of the subsoil D_s and the filter D_r.

$D_{15f} < (4 \text{ to } 5)D_{85s}$ stability criterion
$D_{15f} < (4 \text{ to } 5)D_{15s}$ permeability criterion
$D_{50f} > (20 \text{ to } 25)D_{50s}$ segregation criterion

$$C_u = \frac{D_{50}}{D_{10}} < 10$$ no migration

$$C_u = \frac{D_{50}}{D_{10}} > 20 \qquad \text{susceptible to migrate}$$

where: C_u = coefficient of uniformity
D_X = diameter according to x% undersize by mass taken from grain size distribution (mm)

To achieve the required filter characteristics it might be necessary to use more than one granular layer. In that case the filter has to be built in successively coarser layers starting from the underlying soil. The first layer must retain the base material, whereas the outer layer must be stable against the revetment armour layer. The minimum thickness of any granular filter is normally taken as 2 to 3 times the maximum particle size for each layer, maintain a minimum overall thickness of the granular filter of 150 mm. Wherever practicable, the granular material must be carefully compacted to minimize settlements.

3.20.14 Filter layers: geotextile filters

The main design parameters for geotextile filters to be look up are the retention criterion and the permeability criterion, which define the capability of the material to retain the existent sub-soil without clogging and to allow unhindered water transport trough the membrane. Besides the required functional characteristics of the geotextile in context with the existing sub-soil properties, certain stability standards shall be considered, which have to be defined with respect to the planned use and which might have further implications on the construction techniques to be employed. Specific properties of geotextiles are available from product sheets of the respective manufacturers. Minimum standards of geotextiles recommended for different segments of standardized structures shall follow defined guidelines by PIANC method (1987). The PIANC design procedure involves the following steps shown in Figure 3.16

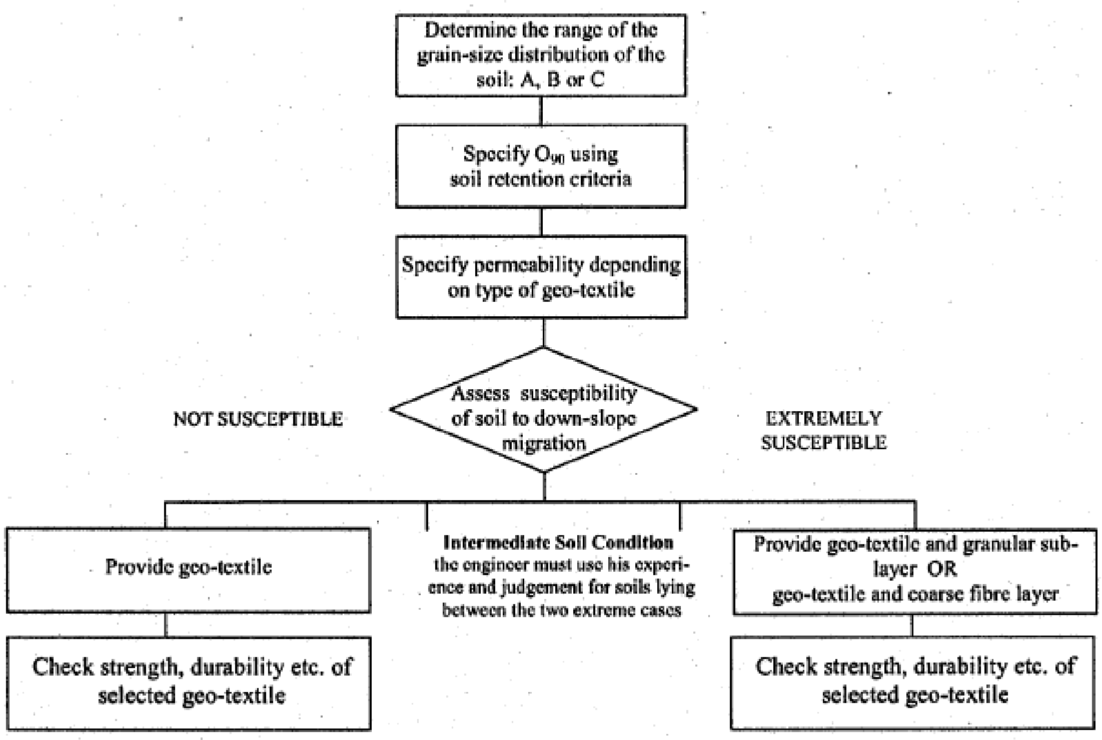


Figure 3.16 Design procedure for a geotextile filter (adopted from PIANC, 1987).

3.20.15 Geotextile filters: determination of the grain-size distribution

The grain-size distribution curve for design must be determined following international standard regulations, to allow for calculation of the various design parameters. Due the fact, that the filter characteristics of geotextile are mainly influenced by the fine compartment of the grain-size distribution (grading curve), the PIANC method categorizes the soil by the screen fraction smaller than 0.06 mm grain size. Typical grain-size distributions for different soil categories A, B and C are given in Figure 3.17.

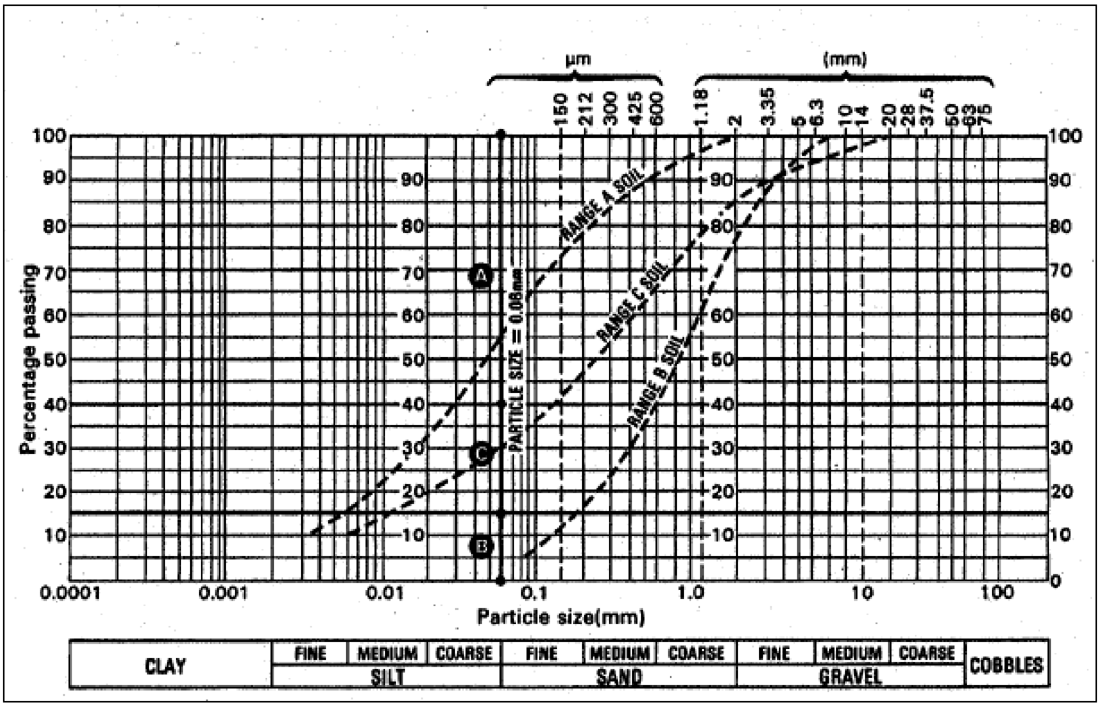


Figure 3.17 Typical grain-size distribution for different soil categories (PIANC, 1987)

Range A: *40% or more of the soil "particles are smaller or equal to 0.06 mm*
Range B: *15% or less of the soil particles are smaller or equal to 0.06 mm*
Range C: *between 15% and 40% of the soil particles are smaller or equal to 0.06 mm*

3.20.16 Geotextile filters: design for soil retention

The capacity of a geotextile in terms of soil retention is characterized by the effective opening size 0_{90}, which is defined by the equivalent diameter of a grain fraction which is retained to 90% by the filter mat in a sieving test. This value is normally provided in the product information sheet provided by the manufacturer. The soil retention is strongly influenced by the dynamics of the impact; therefore different regulations are given for moderate stationary current and for potential highly dynamic hydraulic loads (i.e. wave impacts). The minimum requirements for the geotextile filter to be considered for dynamic load conditions, characterized by high turbulent flow and wave attack, are specified in Table 3.15. The given retention criteria are only applicable for 0_{90} values determined by the wet sieving analysis as defined by Swiss standard SN640550.

Table 3.15 Soil retention criteria (adopted from PIANC, 1987).

Grain size range	Retention criteria
A (amount of fines ≤ 0.06mm larger than 40%)	O_{90} $< d_{90}$ (*) $< 10\ d_{50}$ < 0.3mm
B (amount of fines ≤ 0.06mm smaller than 15%)	$O_{90} < 1.5d_{10}\sqrt{C_u}$ O_{90} $< d_{50}$ < 0.5 mm
C (fines ≤ 0.06mm between 15% to 40%)	As range B

*) if the soil exhibits long-term stable cohesion, then $O_{90} < 2d_{90}$ is applicable.

3.20.17 Geotextile filters: design for permeability

The geotextile filter must maintain a long-term permeability equal or larger than that of the prevailing soil. Shortly after installation a reduction in the fabric permeability due to clogging and blocking will occur, which depends on the pore structure and thickness of the material as well as on the grain structure of the soil. In general, the permeability of the geotextile material is acceptable if

$$\eta k_g \geq k_s$$

where: η = material specific reduction factor
k_g = permeability of the geotextile (m/s)
k_s = permeability of the soil (m/s)

If k_s is not available from laboratory tests, it can be approximated by the empirical relation (Hazen, in Tomlinson, 1996):

$$k = 0.0116 D_{10}^2$$

The reduction factor η for needle-punched and other non-woven fabrics thicker than 2 mm (measured at a normal stress of 2 kN/m^2) is defined as a constant:

$$\eta = \frac{1}{50}$$

3.20.18 Standardization of rock gradings

Information on the grading or the size or mass distribution of rock and granular material in general may be needed in design for a number of reasons. The most important are the hydraulic stability of armour stone, filter rules, choice of construction method and equipment and application of quality assurance.

The particle mass distribution is most conveniently presented in a percentage lighter by mass cumulative curve where W50 expresses the block mass for which 50% of the total sample mass is of lighter blocks (i.e. the median mass) and *W~* and *W„* are defined similarly.

The overall steepness of the curve indicates the grading width and a popular quantitative indication of grading width is the W_N/Wi, ratio or its cube root which is equivalent to the D_{x5}/Dn ratio determined from the cumulative curve of the equivalent cube or sieve diameters of the sample'. The stone sizes defined by D85 and D15 play an important role in the design of filters.

The following ranges are recommended for describing the grading widths:

Table 3.16 Rock grading widths.

Gradation	$D_{85} / D_{15} = (W_{85} / W_{15})^{1/3}$	W_{85} / W_{15}
Narrow or 'single sized'	1.2 to 1.5	1.7 to 3.4
Wide	1.5 to 2.5	3.4 to 16
Very wide	2.5 to 5.0+	16.0 to 125+

There are many advantages of introducing standard grading classes. These mostly concern the economics of production, selection, stockpiling and quality control from the producer's viewpoint. The proposed standard gradings for armour are relatively narrow. This can result in increased selection costs but this cost will often be completely offset by the possibility of using thinner layers to achieve the same design function. It is convenient to divide graded rock into:

Table 3.17 Rock gradings sizes.

Heavy gradings	for larger sizes appropriate to armour layers and which are normally handled individually;
Light gradings	appropriate to armour layers, underlayers and filter layers that are produced in bulk usually by crusher opening and grid bar separation;
Fine gradings	that are of such size that all pieces can be processed by production screens with square openings (i.e. less than 200 mm).

Standard gradings are more or less essential for fine and light gradings. However, for heavy gradings it is not difficult to define and produce gradings other than standard. For example a grading between 1 to 3 tonnes and 3 to 6 tonnes may be chosen for a certain structure when to go to the safer 3 to 6 tonnes range involves an 'excessive' layer thickness.

A consistent scheme for defining grading requirements for standard grading classes is given in the Dutch standard NEN 5180. The straight-section envelopes for the standard gradings including fine gradings can be shown in Figure 3.18 if some assumption is made about size-weight conversion. In this Figure, the fine gradings refer to rock with density of ρ_s = 2.7 - 3 and a shape factor of F_s = 0.6 relating sieve to cubic sizes. The standard fine and light gradings are produced by screens and grids and sometimes with eye-selected top sizes in the 60 to 300 kg class. Due to the poor screening efficiency that occurs in practice means that a correction factor would be needed in addition to the theoretical relationships for sieve size *z* and minimum thickness *d*, should the chart be used to indicate the combination of screens and grids that would produce the standard gradings.

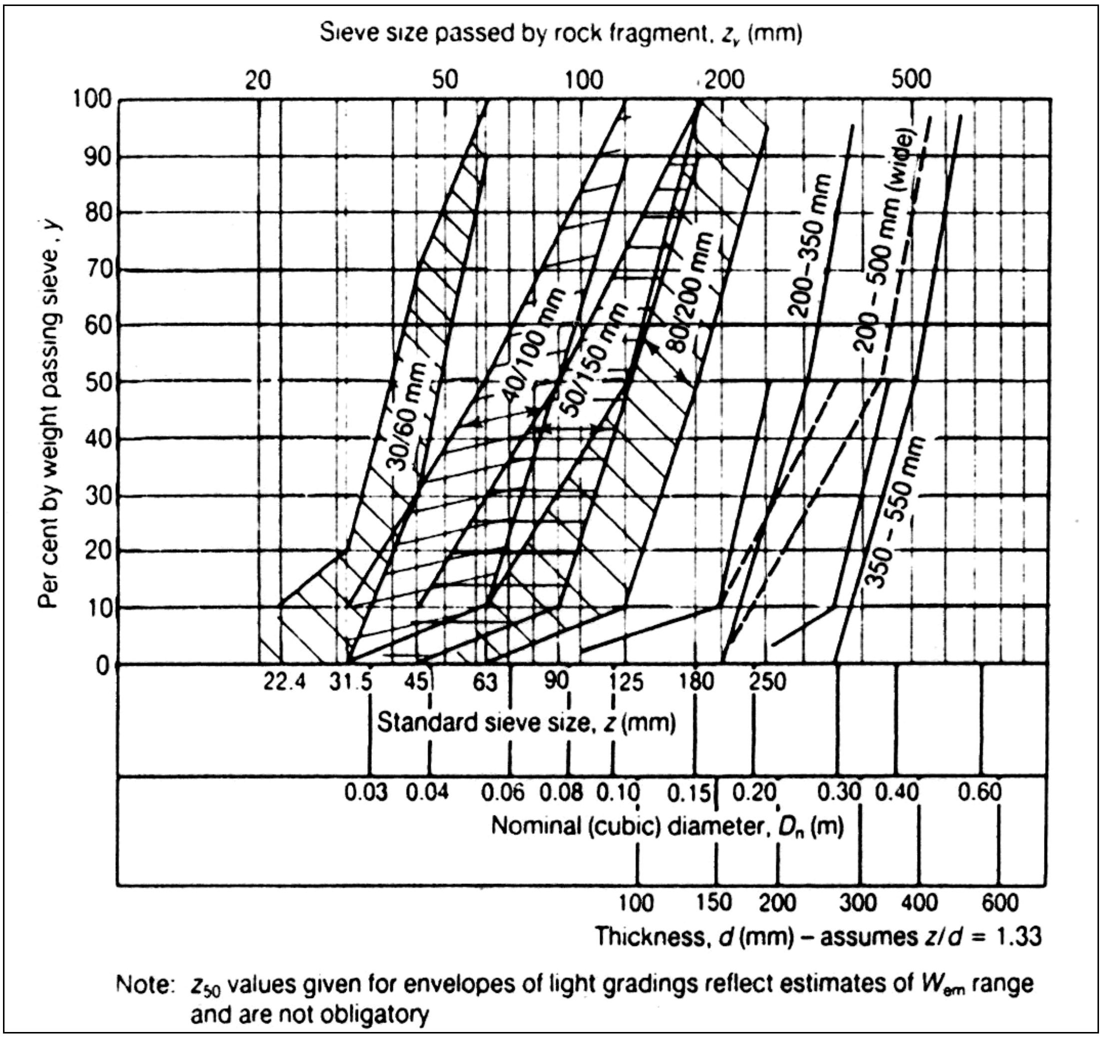


Figure 3.18 Weight and size relationships for the standard gradings.

3.20.19 Revetments: design and materials

Structure and construction material have to be:

- Resistant/ stability against flow and wave action,
- Stable against internal erosion of the bank caused by seepage flow. (It has been found that failures of banks are often a result of inadequate design or construction of the underlayer, rather than being due to instabilities in the cover (or armour) layer.
- Resistant to be damaged by floating objects on the river or human activities

The revetment is usually made of randomly placed rip-rap, pitched stone, grouted stone, interlocking concrete blocks, and bitumen. Selection of the construction material depends on the availability of materials, conditions of geology, hydraulic regime, landscape requirements.

The revetment that is constructed of placed stone should comply with following:

- Slope of the revetment must be stable in terms of geotechnical stability (static and dynamic loading). The method to determine the safety factor is given in Section 3.25.
- The armour stone have to be well-placed. The size of stone should be large enough to resist the action of waves and/or currents. The size of the armour can be determined by given formulae in Sections 3.20.5to 3.20.10 and 3.20.20.
- A granular or geotextile filter has to be constructed under the armour stone layer. Granular filters are usually around 0.15 ~ 0.25m thick. Geotextiles filter has to be durable and with sufficient strength to resist tearing. Guidelines for design of filter layers are presented in Section 3.20.11 to 3.20.17.

Revetments that are constructed of grouted stone have to meet following requirements:

- The size of stone can be determined as in Section 3.20.2 to 3.20.10 with 25% reduction to coefficient d_0, and drainage holes have to be added.
- Besides above mentioned requirements the grouted stone has also to meet requirements to resist against up-lift force as indicated in the mentioned sections.

Revetments constructed of concrete units (blocks or slabs) have to meet following requirements:

- Pre-casting/ in-situ casting concrete/ reinforced slabs can be used to armour the bank slope after finishing the filter layer. Banks that are protected with concrete/ reinforced concrete slabs should have construction joints; the joints shall be filled with bitumen.
- Dimensions of the concrete units must be large enough to ensure stability conditions under the attacks of waves, currents and its combination. The most interested parameter is thickness of units which normal to revetment slope.

Grouted stone revetment and concrete slabs revetment need to be checked for uplift/ floating resistant:

$$P_n > \eta d_b \gamma_b \cos\alpha$$

where:
- P_n = uplift/ floating pressure of water on the armour unit (T/m^2)
- α = angle of the bank slope with the horizontal
- d_b = the thickness of the slab (m)
- γ_b = unit weight of concrete (T/m^3)

The crest level of the revetment can be extended 1m (free board) above the maximum run-up wave at design water level, while the lower edge of the crest connect direct to armour layer. When using stone to protect the bank it is needed to find the sufficient size of the stone for to withstand attack from each individual action current, wind wave and wave induced by ship/ wind, and then the maximum size will be chosen. The width of the crest should be (1.0 ~ 3.0) m wide. The structure of the crest may be the same as the one designed for the revetment.

3.20.20 Falling/ launching aprons: design and materials

Taking into consideration the possibility of prevailing non-cohesive and cohesive soil materials (soil stratification), for river training works a redundant system, combining loose protection elements (falling apron) and interconnected protection units (launching apron) is recommended.

The design concept regarding the toe protection of river training works is based on the objective to build all structure components on a dry fluvial plain or completely under water. In general, the design implies that the scouring and undermining process of the developing scour hole in front of the structure initiates the deformation process of the toe protection. At the estimated maximum scour depth, the falling and launching apron is assumed to cover and stabilize the slope of the scour hole formed on the bed river profile, preventing from further erosion reaching the main structure.

After completion of the construction works the toe protection must remain stable under the existing flow conditions. The most important concern is that no larger part of the material is transported in flow direction.

Subsequent to the articulation and reformation of the material along the scour slope, the elements must be able to resist shear stresses of the existing current. At this location the depth averaged design flow velocity u_{toe} can be approximated by the hydraulic design velocity selected for design.

The required size of rock or individual CC-blocks can be computed by the formulae of Pilarczyk given in Section 3.20.3. It is recommended to apply a minimum block size D_n = 0.3m.

Following the assumptions made for the calculation of the minimum volume required to cover the expected equilibrium (maximum) scour hole, a geometrical solution based on the scour profile and multiplied by a safety factor can be applied. The required volume V_{FA} of a scour protection per metre can be estimated as:

$$V_{FA} = 1.5 D_n \sqrt{5} y_{BL} C_{FA}$$

where: V_{FA} = volume of rock/ blocks in the falling apron per liner metre protected length (m^3/m)

D_n = rock diameter or block size ($1.5D_n$ is the proposed layer thickness after scouring without voids, in m)

Y_{BL} = vertical distance between the base level of falling apron at time of construction and the deepest point of the expected design scour hole (m)

C_{FA} = flow attack coefficient: 1,5 (moderate flow attack)
1.75 (strong flow attack)

The term $\sqrt{5} y_{BL}$ describes the simplified area of the landward scour profile in m^2 per linear metre, assuming a 1V in 2H stabilized scour slope (fully developed equilibrium scour and stabilized slope). The recommended construction base level of the falling apron is set at bed level at the moment of construction or below in case dredging is carried out, in case this precondition is modified due to other site specific reasons, it must be taken into account in the computation of the required material quantity. The design of a typical falling apron design is schematically shown in Figure 3.19.

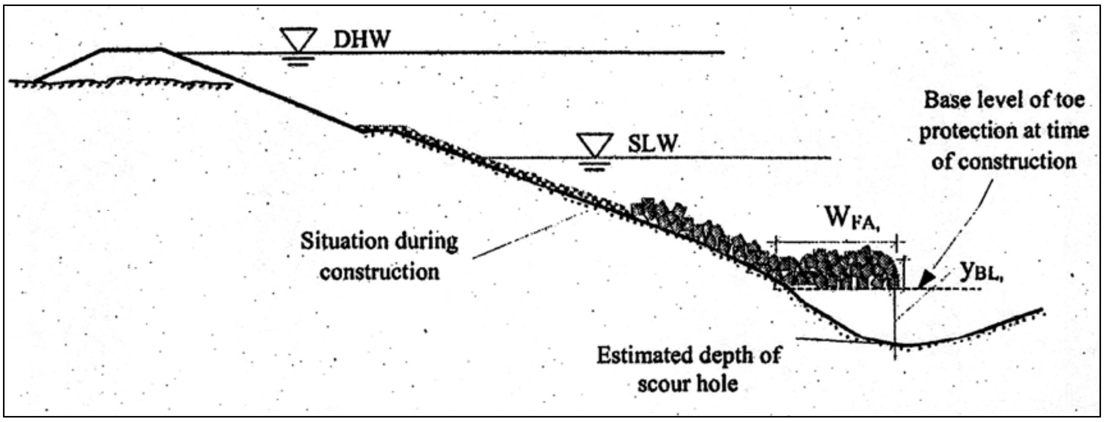


Figure 3.19 Toe protection of a revetment for under water construction.

In case, reliable construction techniques for under water placement of toe protection material under water and flow velocity conditions are at hand with experience contractors, this approach is preferred. The underwater construction of a falling apron has got certain advantages due to the possibility to work with a smaller Y_{BL} depth and therefore the material required for the actual falling apron is much smaller.

3.21 Impermeable groynes

Groynes, as river training works, can be considered as partly active and passive measures. The decisive criterion in this regard is the structure permeability which will he discussed below. Groynes are built perpendicular or at a certain angle to a riverbank, protruding into the river. The main objective is to deflect the flow away from critical banks or reduce the width of the river, i.e. for controlling erosion or to establish and maintain safe navigation channels.

Groynes inclined in upstream direction are called repelling groynes, because of their ability to divert the flow away from the structure (Figure 3.20). In contrast, attracting groynes point downstream and attract the flow towards the structure's head and thus to the river bank (Figure 3.21). Therefore, this type of groyne should be placed at the inner bend of a river course to protect the outer concave bend. As demonstrated. In the Figures single groynes provide only local protection. For that reason, normally several groynes are combined to form a groyne field to increase the efficiency and to enlarge the stretch of protected river bank.

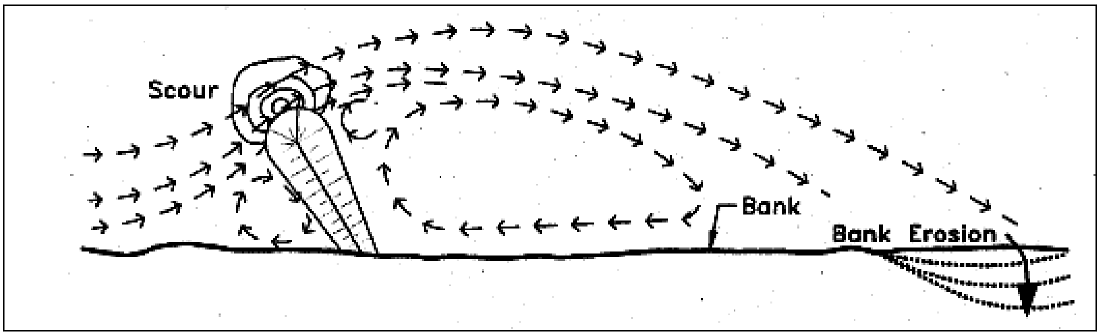


Figure 3.20 Repelling groyne.

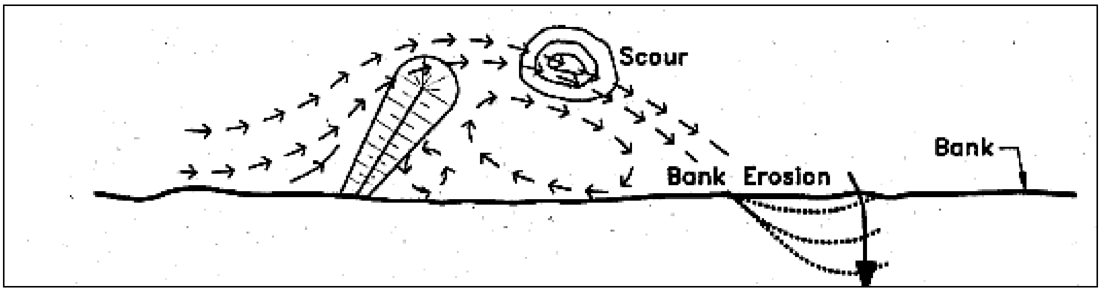


Figure 3.21 Attracting groyne.

Due to the fact that groynes act like a blockage to the river flow, the flow lines will merge in front of the groyne head resulting in high local velocities and scour. To reduce this effect, which might destabilize the groyne structure and normally requires massive scour protection, and to further improve the performance (i.e. the protection capability), a large number of differently shaped groyne heads have been tested over the last decades. Some alternative groyne head designs are given in Figure 3.22.

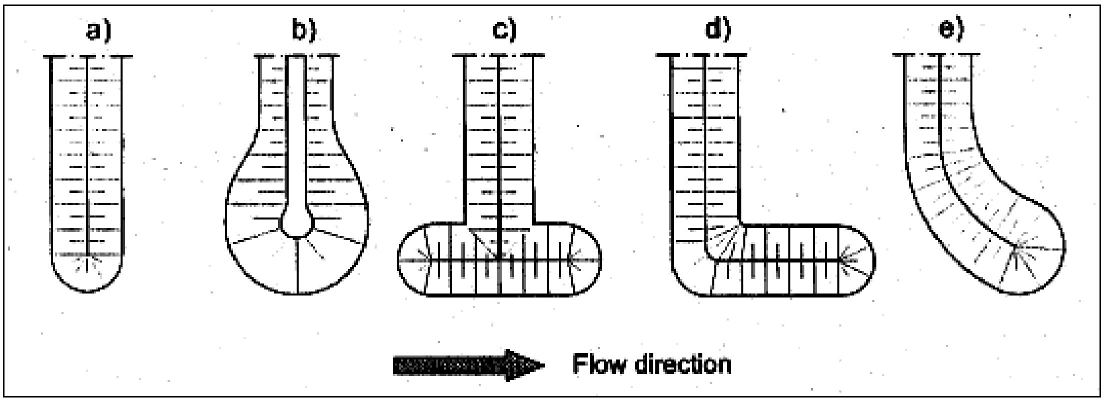


Figure 3.22 Typical groyne heads.

Straight groynes (Figure 3.22 a) without any extra head protection as compared to the trunk are most unfavourable regarding the head stability. To improve the scour resistance a more extended round head (b), a so-called mole-head, may be employed, which provides extra volume of scour protection material and a gentler transition between groyne head and river bed.

T-head (c) or L-head (d) groynes are introduced to give additional guidance to the flow, to improve the bank protection, to reduce the scouring at the groyne head and to increase sediment depositions downstream from the groyne. T-head and L-head groynes generally need strong cover layers.

Groynes with curved trunks are known as hockey shaped groynes (e). In particular if the groyne is curved in flow direction (inverted hockey shaped groyne), this type allows a reduction of the scour material required as compared to (c) and (d) if a strong attack at the head is expected. In addition, many combinations and specific designs of groyne heads are existent.

Impermeable groynes shown in Figure 3.23 can be built of local soil, stones, gravel and rock with suitable slopes at the shanks and the head or even vertical walls at the shanks, using steel sheet piles or pre-stressed reinforced concrete sheet piles. In case of an appropriately sloped earth-dam, the trunk and the head have to be protected by a cover layer placed on a suitable filter-layer. The main hydraulic disadvantage is the effect of flow separation at the groyne head, caused by the blockage of the flow. Therefore, special attention must be given to the toe protection at the head of the groynes, where extreme scouring occurs. In addition counter measures against the return currents, possibly attacking the bank downstream of a groyne must be considered. Falling or launching aprons have to be provided in these areas.

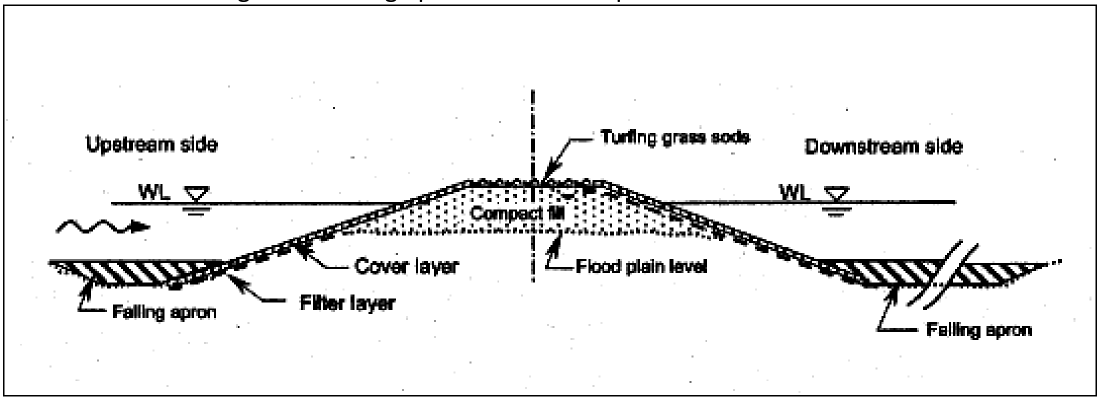


Figure 3.23 Typical cross-section of an impermeable groyne (non-submerged).

Structure of groyne can be divided into 3 main flowing parts:

- Groyne root is the part that connects to the river bank,
- Groyne head is the groyne end that is exposed to river flow,
- Groyne body is the part that connects the groyne head and root.

It depends on flow status, topographic, geotechnical conditions structure of the head and body can be different, but structure of root in most of the case is the same.

The structure of the groyne body should meet the following stipulations:

- For the trapezoidal cross section of riprap groyne the slope of the upstream side should be taken as 1:1 ~ 1:1.5 while the downstream side, 1:1.2 ~ 1: 2.5.
- The crest width of the groyne may be taken as 2~4m; under conditions of strong current velocity or with drift woods the value may be taken on the big side, under special cases, the width may be properly widened.
- Crest level of groynes in rivers with navigation is usually placed between 0.3 to 1.3 m above the mean yearly water level. For streams with considerable depth, groynes sloping from root to head may provide considerable savings in construction.
- In the light of design requirement, the groyne body should be firmly pitched and the top of it should be trimmed and meet the stable criteria.

The longitudinal slope of the groyne top should meet the following stipulations:

- The longitudinal slope of the groyne top may be taken as 1:100 ~1:300, for long groyne, the longitudinal slope should be adjusted in connection with the elevation of the bar shores.
- When the groyne is required to play roles at different water stages and does not contract the river bed too much the groyne may be designed with several longitudinal slopes.

The structure of the groyne root should meet the following stipulations:

- When the river bank is exposed to erosion, the groyne should be protected against scouring by water current; otherwise the groyne root does not need to be protected.
- The length of stone revetment for the groyne root may be determined by the geology of the bed and river bank and currents. It requires carrying out scour calculations for the design flood condition.

The structure of the groyne head should meet the following stipulations:

- According to the effect of the water current acting on the groyne head, the width of the groyne top may be properly widened within the range of 10 ~ 20m of the dike head.
- The dike head is recommended to be made as a smooth curve in plane the riverward slope of 1:2.5 ~ 1: 3 is recommended.

The diameter of the armour stone of groyne has to satisfy functional requirement of resistance against currents, wave action, human activities, etc. The required dimension of armour units for various types of protections can be determined as shown in Section 3.20.

In case the construction site is far away from the quarry, the groyne can be made with earth core and stone armour to reduce the construction cost. In this case, besides all above mentioned requirements following additional requirement have to be fulfilled:

- A filter layer has to be used between the earth core and armour layer. Design related calculations of filter layers can be made as explained in Sections 3.20.13 to 3.20.17.
- Construction of earth core needs to be taken with care to minimize the possibility of settlement/ collapse.
- In all the cases the groyne head has to be made of stone to again the flow and waves actions, and to ensure for stability of the groyne when the head is locally damaged.

Toe protection of groynes can be constructed using natural stones, concrete blocks, launching aprons or gabions. The sizes of protective elements are determined according to Section 3.20.

Layout design: determination of groynes alignment, spacing and length in relation to the channel width are presented in the following sections together with additional criteria.

3.22 Permeable groynes

3.22.1 Permeability of groyne

The permeability P of a groyne is defined by the ratio of open (non-blocked) area to the total area, which can be expressed by the quotient of internal width s and the distance e between the axis of two adjacent piles (P = s/e, see Figure 3.24).

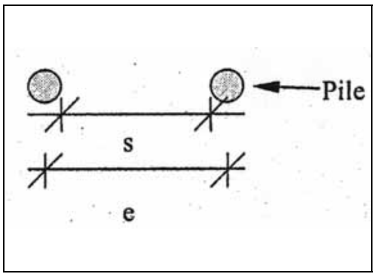


Figure 3.24 Definition of permeability.

In general, keeping the same target permeability, larger pile diameters are considered more economical as compared to smaller diameters because the material quantity and the construction time is reduced. A minimum permeability of P = 0.5 (50%) should be kept for constructional reasons (increased pile-driving resistance) if lower values are required, the groyne might preferably be constructed in two neighbouring rows in the respective parts.

3.22.2 Orientation of groynes

The deviation of the streamlines is mainly dependent on the structure induced blockage of the flow cross-section, which can be described by the length and permeability of the structure in relation to the geometrical channel properties: Consequently, for permeable groynes the actual shape (jockey head, etc.) of the groyne and the angle between groyne axis and bankline is of rather small importance as compared to impermeable spurs. Taking also into account the expected variations in the approach direction of the critical channel during the subsequent years after construction, it is economical (because $L_G = L_{G,eff}$) and appropriate from the hydraulic point of view, to implement the individual permeable groynes in bank normal direction (90°).

3.22.3 Groyne crest level

The crest level varies depending on the rivers and sometime depending on sections of specific river due to it depends on the purposes of the groynes. The crest level of the toe riprap has usually fixed level and sometime it is provided with a reverse slope from the structure part to the bankline in order to (i) reduce flow velocity due to higher flow near the bankline, and (ii) regulate the flow concentrating into the channel at low water.

In the groynes system, the crest level at the transition of downstream side is lower and equal to the upstream groyne crest level. In general, the slope of transition between the continuing crests should be in line with the slope of water line in the river. The results of modelling show that the combination of some longitudinal structures at the heads of groynes will help to make a smoother flow and consequently it is favourable for navigation.

Floating debris trapped by the piles will influence the blockage and subsequently the scour development downstream from the groyne. To reduce the influence of floating debris at high water levels, the groynes can be designed with a negative freeboard, i.e. they act as slightly submerged groynes, allowing the debris to float just above the crest of the piles. A compromise is given by designing a variable crest level along the groyne axis to keep the functional efficiency and reduce negative effects by trapped floating debris. In case of partly or completely submerged groynes (during high flood level) the installation of navigation signals at the groyne's head is obligatory.

3.22.4 Groyne length

The effective length of a groyne L_G is defined as the length projected on a theoretical line perpendicular to the river bank. For orthogonal groynes, the effective length and the linear groyne length are identical.

Assuming a natural scour slope (dependent on the existing subsoil) developing from the deepest point of the scour hole towards the bankline, the minimum groyne length can be calculated according to Figure 3.25.

With this, the minimum effective length of a permeable groyne in the central section of a groyne field is defined by (a) whereas an upper limit is given by (b) to reduce possible negative impacts at the opposite bankline. Both formulae have to be validated for main rivers in Vietnam. Therefore, the values obtained by the formulae are recommendations and it might be necessary to deviate from it.

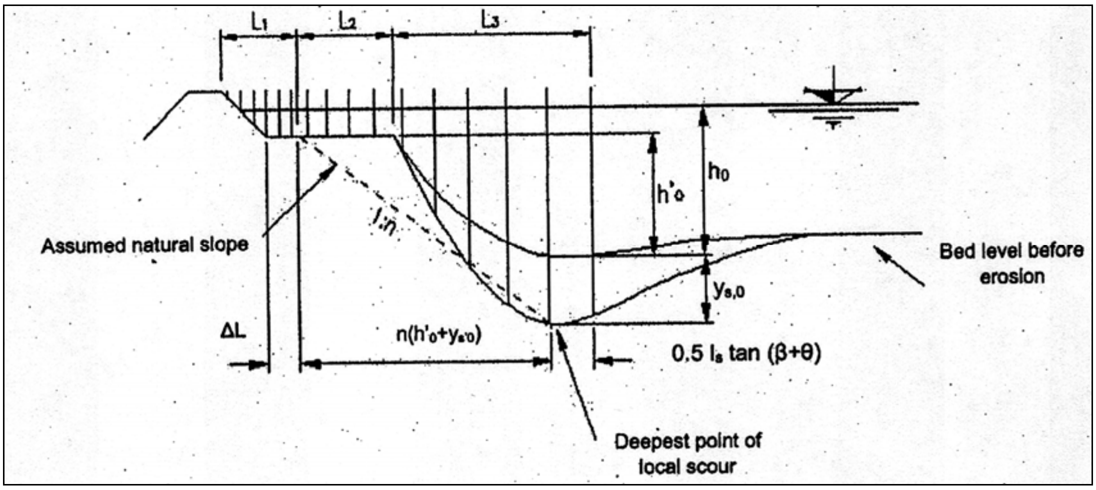


Figure 3.25 Assumed cross-section and scour development – permeable groynes.

$$L_G = \Delta L + n(h_0^{'} + y_{s,0}) + 0.5 l_s \tan(\beta + \theta) \qquad \text{(a)}$$

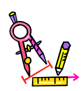
and

$$L_G \leq 0.2 \ to \ 0.4 B_{ch} \qquad \text{(b)}$$

where: L_G = effective length of groyne (perpendicular to the embankment, in m)
ΔL = safety margin (minimum 10m)
n = cotangent of natural slope of the bed material 1V : nH
h_o' = water depth at the thalweg referred to floodplain level (FPL, in m)
$y_{s,0}$ = maximum total scour depth related to the thalweg of the undisturbed river bed (m)
l_s = length of scour hole perpendicular to the groyne axis with first estimate is $l_s = 4\ y_s$ (m)
θ = angle of flow attack between flow line and bankline (^{o})
β = fictitious angle of flow separation (^{o})
B_{ch} = average width of the channel (m)

The assumed scour slope must be smaller than the angle of .repose of the bed material. Depending on the subsoil, values of 10 to 12 degrees are recommended. As a first approach, for sandy and non-cohesive soils, n = 5.5 can be chosen. If minor damages can be tolerated, n = 4 is acceptable. For cohesive soils this leads to a rather conservative calculation of L_G.

3.22.5 Spacing of permeable groynes



For a series of permeable groynes a wide range of recommendations regarding their optimal spacing exists. With certain exceptions, the executed and recommended spacing range between about 1.5 to 5 times the effective groyne length. From the economical point of view, the spacing should be as large as possible. However, the efficiency of the groyne field as a whole shall not be affected by too large spacing.

A groyne can be considered as a disturbance of the flow field, which diminishes in down-stream direction and finally becomes neutralized after a certain distance, called the relaxation length λ_w. The relaxation length follows from a linearized balance between the connective term and the friction term in a one-dimensional momentum equation, which is defined for a bank parallel flow as:

$$\lambda_w = c_s \frac{C^2 h}{2g}$$

where: λ_w = relaxation length (m)
C = Chézy coefficient ($m^{1/2}/s$)
G = acceleration due to gravity (m/s^2)
H = local water depth (m)
c_s = empirical coefficient for channel properties

Typical values for c_s are as follows in Table 3.18:

Table 3.18 Coefficient c_s.

Channel alignment	Scour hole	Coefficient c_s
Bend	deep	0.85
Straight	deep	0.70
Straight	no/ moderate	0.50

To allow for inclusion of potential oblique flow attack (θ ≠ 0) a fictitious separation angle β is defined by the relation:

$$\tan(\beta) = c_s 2g \frac{L_G}{C^2 h}$$

which is based purely on geometrical considerations as indicated in Figure 3.26, and the minimum spacing S_G between two adjacent groynes is given by:

$$S_G = \frac{2}{3} \frac{L_G}{\tan(\theta + \beta)}$$

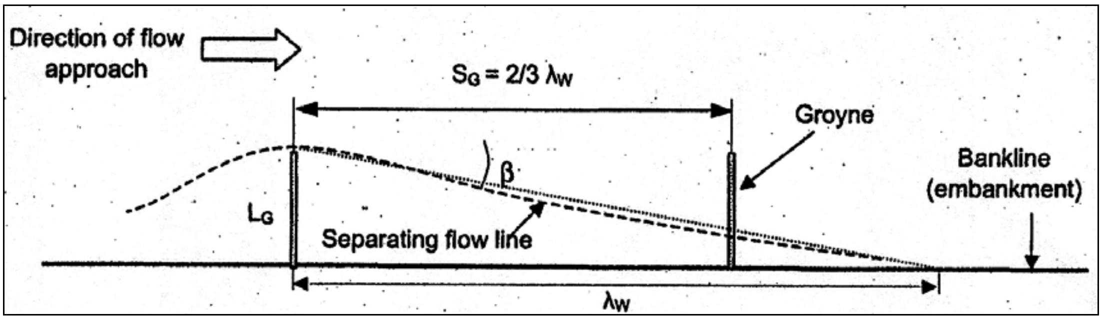


Figure 3.26 Theoretical separating flow line.

3.23 Guiding system (vanes) utilizing transversal circulation

The aim of creating or modifying the transverse circulation is to force the sediment and water to alter their flow direction, i.e. to control erosion/ deposition processes. This objective is obtained with the aid of a group of panels (vanes) set into the stream. They may be arranged in upper medium and/or bottom layers of the stream. Producing transverse circulation in the stream by means of surface panels set in upper layer of stream is shown in Figure 3.27. After passing the guiding system, movement of water current changes from the parallel to the helical one; surface currents (marked by solid lines) are deflected to the right, whereas the bottom ones (marked by dashed lines) are deflected to the left. Such a system may be used for cutting a convex bank of bend of small radius.

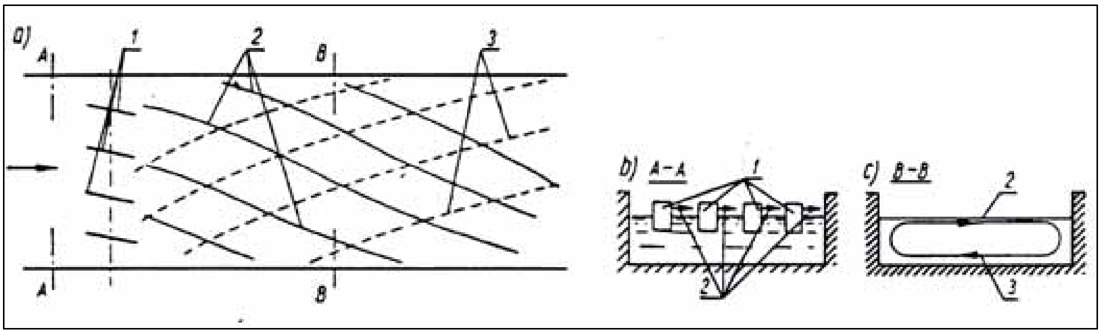


Figure 3.27 Guiding system consisting of four surface vanes.

(a) - Plan view, (b) - Cross-section A-A, (c) - Cross-section B-B.
1 - surface panels, 2 - surface streamlines, 3 - bottom streamlines.

However the guiding system will have efficiently functional only if it is designed to use for fixed situation of the riverbed, i.e. for the slightly changed water depth and attach angle of flow. For the design, therefore, the design water depth and attack angle are two most important parameters and normally selected corresponding to the longest occurrence period. Different values of attack angle will have corresponding different length. Recommended dimensions of guiding panels are as follows:

- at angle of attack α = 18 ~ 25^o a most practical length l = (1.0 ÷ 1.5*)h,* and at α = 12 ~ 18^o*, l* = (1.5 ~ 2.0)*h,* where *h* - water depth at the stage of the longest duration;
- the submerging depth *H* of a surface guiding panel or the height H of a bottom panel H = (0.2 ~ 0.5)h.

Similar recommendations were given by Altunin (1962), namely:
- 1 = (1.0 ~ 3.0)h on an average 1 = 2h;
- H = (0.2 ~ 0.4)h on a average H = h/3;
- α = 12^o ~ 72^o on an average α = 18.5^o.

The width of helical current induced by one single panel is equal to (1.0 ~ 1.2)h and the downstream length of influence (10 ~ 20)h.

As a result, guiding systems can be recommended for application to reduce sediment intake at the inlets to irrigation canals. They proved to be an inefficient measure of bank protection, especially for reaches with big water depths and velocities where the systems must be relatively large and not easy to exploit.

A critical design parameter is the vane height H. For the vanes to function optimally H must be less than about half the water depth h at the flow rates at which bank erosion occurs. The required number of vanes, N, was calculated by using:

$$N = \frac{2h\Phi bF}{C_L lH}$$

where: h = flow depth (m)
Φ = bend angle (^{o})
b = vane design section width (m)
C = L is the lift coefficient (0.5)
I, H = vane length, height (m)
H = vane height (m)

F = function given by:

$$F = \left(\frac{h}{H}\right)^{2/n} \left[(n+1) - (n+2)\frac{H}{h}\right]^{-1}$$

where: n = velocity profile exponent in the expression valid for a rectangular (the desired shape) cross-section:

$$\frac{u}{\bar{u}} = \frac{n+1}{n}\left(\frac{y}{h}\right)^{1/n}$$

where: u = point velocity at height y (m/s)
ū = depth-averaged mean velocity (m/s)

The value of n may be estimated from the following equation:

$$n = \frac{\kappa \bar{u}}{\sqrt{gIh}}$$

where: κ = von Karman's constant (≈ 0.4)
I = longitudinal slope of water surface

The calculated value of N guaranteed keeping the energy (longitudinal) slope constant (I = 0.00065 ÷ 0.00075), i.e. the same as before vanes installation.

Odgaard and Moscani (1987) give the following recommendations for future designs. Because of the sensitivity of the design to the approach flow conditions, the channel reach upstream from the first vanes should be stabilized to ensure that the approach flow angle remains constant. To ensure that an adverse flanking does not occur, the uppermost vanes in the system should be installed close to the bank, and their density should be greater than the theoretical. Vane systems should be designed based on total width of the channel to further reduce the transverse bed slope and near bank velocity, and provide more favourable conditions for the forming of a natural toe protection along the bank.

3.24 Scouring

Local phenomena like bends, constrictions and, in the case of the Mekong River, also bedrock outcrops may cause large scale scour depths in alluvial rivers. These scour holes may threaten the bank protection works already present or to be constructed in the future. This section analyses the various types of scour that occur in the Mekong River. The main purpose of this analysis is to provide tools to estimate the maximum scour depth for the different proposed bank protection works. The tools can be used to generate design conditions.

A distinction can be made between general scour and scour that occurs more locally. General scour is the reaction of the river on changes in its boundary conditions, like aggradation and degradation owing to accelerated soil erosion, sea level rise, cut-offs of bends etc. More localized scour can be distinguished in a number of different types, notably:

- *constriction scour*, caused by a local constriction of the width of the river bend scour, occurring along the outer bend of rivers, and being characterized by deep scour holes together with a point bar in the inner band;

- *confluence scour*, occurring in the reach downstream of the junction of two river reaches, like in the case of the confluence of a tributary with a main river *protrusion scour*, occurring when the bank of the river protrudes into the channel. like in case of a rock outcrop;
- *bedform scour*, related to the occurrence of deep troughs, downstream of dune crests;
- *local scour*, occurring near bank protection works.

For the Mekong River, along Lao PDR and Thailand in particular, constriction scour, bend scour, bedrock outcrop scour and local scour is important, and possibly also protrusion scour. Confluence scour is more common in braided rivers, may occur downstream of junctions with tributaries, but is in that case not near river banks. Bedform scour is not so important and not taken into account, as it is assumed that bedforms almost vanish in local scour holes.

There are two different ways for estimating the scour depth near structural measures. The first is based on the regime approach. For a river its depth is estimated with an appropriate equation. Next the regime depth is multiplied with a coefficient, which is based on experience. Hence, the different types of scour listed above are not explicitly accounted for (and computed), but the local conditions are reflected in the estimated values of the multiplication coefficient which indeed depend on local conditions. This first method has been used extensively in the Indian subcontinent. To the thus obtained estimates the general scour still has to be added.

The second method is a more recent development, based on the increased understanding of river processes and local scour phenomena. This method attempts to distinguish between the different types of scour and for each type a quantitative estimate is made. In a second step the different values are combined to arrive at the combined scour depth that will actually occur. The latter method has the additional advantage that it can be combined with a probabilistic design approach. In this guideline mainly the second method is used. For this purpose, the occurrence of the different types of scour is analyzed in the subsequent sections, whereby the main purpose of is to derive design formula applicable to the conditions in the Mekong River. Also how to combine the different types of scour is discussed. Scour estimates according to the first method are also made, to compare them with the ultimately obtained scour estimates.

The estimates of the maximum scour depth around groynes according to the regime theory approach are summarized in Breusers and Raudkivi (1991). Here the relevant part of their manual is quoted, whereby the notation is explained in Figure 3.28.

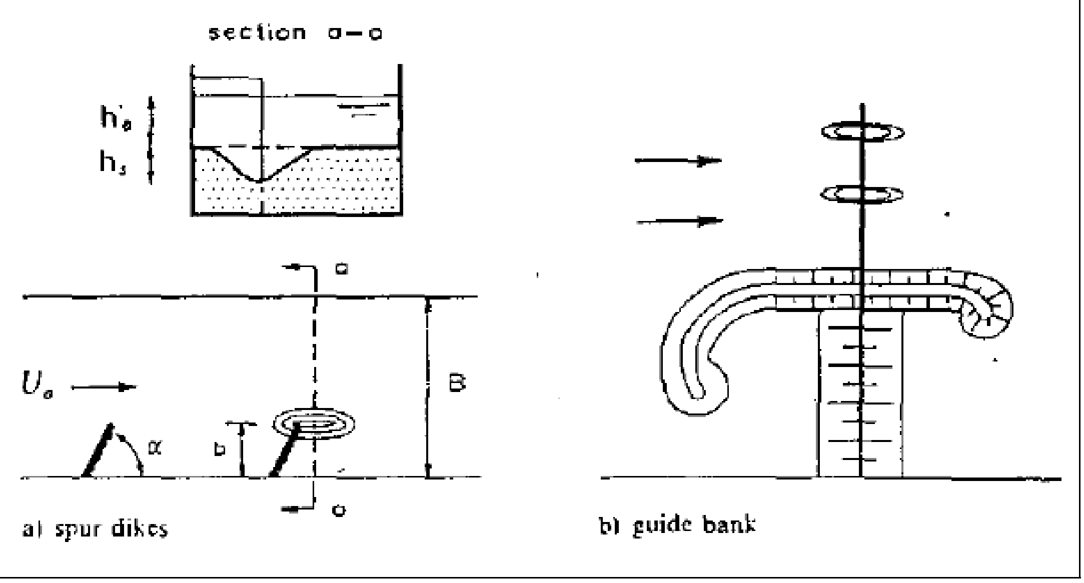


Figure 3.28 Definition sketch and notation used.

Inglis (1949) analyzed field data on the maximum scour depth observed near spur dikes and guide banks in India and Pakistan. He compared the total scoured depth, $h_0 + h_s$ with the three-dimensional Lacey regime depth h_{3r}, which can be obtained from the equation:

$$h_{3r} = 0.47(Q/f)^{1/3}$$

where: f = silt factor, defined by:

f = 1.76/d_{50}
where: d_{50} = in mm.

The ratio $(h_0 + h_s)/h_{3r}$ ranged from 1.6 to 3.9. Inglis (1947) recommended the use of the values as indicated in Table 3.19.

Table 3.19 Recommended values for $(h_0 + h_s)/h_{3r}$.

Conditions	$(h_0 + h_s)/h_{3r}$
Scour at straight spur dikes angled upstream α>90° with steeply sloping nose (1.5V: 1 H)	3.8
Scour at similar dikes but with long sloping noses	2.25
Scour at guide bank noses of large-radius	2.75

Ratios over the observed range should be used with judgement as to the severity of the river's attack on the structure.

Laboratory studies of these structures were performed in flumes with fixed vertical side walls and erodible beds, and they could therefore be compared with Lacey's two-dimensional regime depth, h_{2r}, defined by:

$$h_{2r} = 1.34\left(\frac{q^2}{f}\right)^{1/3}$$

where: q = discharge per unit width in the contracted section

The most useful study is the one of Ahmed (1953) which provided particularly information on the effect of the angle α on the depth of scour, and of Liu et al. (1961) whose results cover the widest range of the pertinent variables. Most of the available results are for spur dikes in the form of a vertical wall. Various other studies have added marginally to the limited information on the subject.

Ahmed presented his results on the basis of an equation with the form:

$$h_0 + h_s = Kq^{2/3}$$

which is compatible with the Lacey regime equation (K taking the place of $1.34/f^{1/3}$). His results for the effect of angle are summarized with other relevant factors hereafter.

Satisfactory results can be obtained from empirically determined values of K with a value of 2.0 ± 15% for a nearly vertical spur dike. Correction factors modify this result for various other conditions as follows in Table 3.20, Table 3.21 and Table 3.22:

Table 3.20 Influence of the spur dike angle on the coefficient K_1.

Spur dike/ groyne angle α	K_1
30	0.8
45	0.9
60	0.95
90	1.0
120	1.05
150	1.1

Table 3.21 Influence factor due to shape of the structure on coefficient K_2.

Shape of the spur dike/ groyne	K_2
Vertical board	1.0
Narrow vertical wall	1.0
Wall with 45^{o} side slopes	0.85

Table 3.22 Influence of position of the structure on the coefficient K_3.

Position of spur dike/ groyne	K_3
Straight channel	1.0
Concave side bend	1.1
Convex side bend	0.8
Downsteam part of bend, concave side, sharp bend	1.4
Downstream of bend, concave side, moderate bend	1.1

To an acceptable approximation, the combined use of the various factors K_1, K_2 and K_3 is recommended.

3.24.1 General scour

The general scour/ accretion in the alluvial stretch of the Mekong river is in the order of 8 cm per year[7]. The general scour/ accretion rate does not depend significantly on the (small) changes of the monthly dominant discharges.

Construction of dams will influence the general scour processes taking place at the upstream end of the alluvial stretches.

3.24.2 Constriction scour

Constriction scour occurs if the width of an alluvial river is constricted over a substantial length. This may be caused by i.e.:

- bank protection works,
- bridge approaches in the floodplains,
- buildings and even towns in the floodplains of the rivers.

The effect of the constriction will be that the bed level is lower and the water slope is usually smaller in the constricted reach. For constant discharge and steady conditions simple expressions for the increased depth and the related reduced slope can be derived (see Jansen, 1979) and assuming uniform conditions:

[7] Mekong River Bank erosion Study, NEDECO-SPAN-WDC (1995)

$$\frac{h_{cs}}{h_0} = \left(\frac{B_0}{B_s}\right)^{\frac{b-1}{b}}$$

$$\frac{i_c}{i_0} = \left(\frac{B_c}{B_0}\right)^{1-\frac{3}{b}}$$

where: h_{sc} = water depth in the constriction (m)
h_0 = original water depth (m)
B_0 = original width (m)
B_c = constricted width (m)
b = power of a simple transport equation (-)
i_c = water level slope in constriction (-)
i_0 = original water level slope (-)

The simple transport equation reads as follows:

$$s = au^b$$

where: s = sediment transport per unit width (kg/m)
u = current velocity (m/s)
b = coefficient

A graphical representation of the equations is given in Figure 3.29.

It is supposed that the width of the Mekong River is not significantly affected by the bank protection works such as revetments that will be constructed. The local depth of the river will therefore not change due to an increased constriction of the river at the given priority/ demonstration reaches. For the design bed levels it is important to know what the variations of the bed level will be. During high discharges the bed level will become lower in constricted reaches. The sediment is deposited in a wider reach downstream of the constriction. It will take some time until the maximum (calculated) scour is developed.

3.24.3 Outer bend scour

Outer bend scour is the scour that develops in the outer part of river bends. River bends are characterized by the so-called helical flow, causing sediment particles to move to the inner bend. This causes scour in the outer bend and deposition in the inner bend. By assuming that the centre-line values of the water depth are equal to the reach averaged values, the near bank water depth deformation (outer bend scour) can be calculated with the following equation:

$$H = h_0 A f(\theta_0) \frac{B}{2R_c}$$

where: H = water depth excess in outer bend (m)
h_0 = reach-averaged water depth (m)
B = channel width (m)
R_c = radius of curvature of the channel centre-line (m)
$F(\Theta_0)$ = weighing function for the influence of the sloping bed (-)
A_s = coefficient weighing the influence of spiral motion (-)

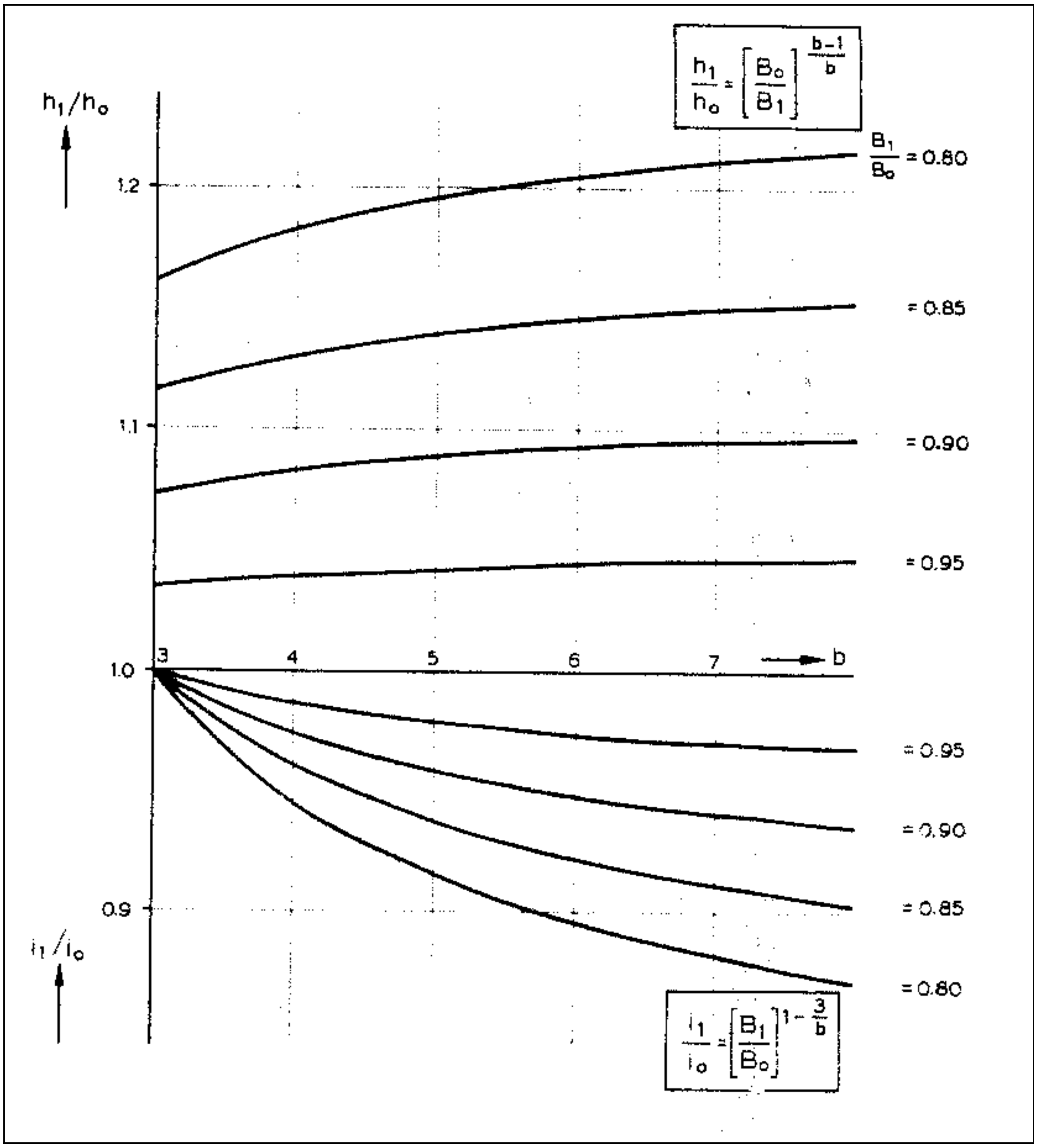


Figure 3.29 Effect of river bed constriction (Jansen, 1979).

The error induced by assuming the centre-line value of the water depth as to be equal to the corresponding reach averaged value is only small for the case of mildly curved channels. The coefficient weighing the effect of spiral motion can be computed as follows:

$$A = 2\alpha_1 K^{-2}\left(1-\frac{\sqrt{g}}{\kappa C}\right)$$

where: α1 = coefficient weighing the effect of channel curvature on the bed shear stress direction (-)
κ = Von Karman constant (-)
g = acceleration of gravity (m^2/s)
C = Chezy coefficient (-)

The coefficient α_1 weighs the effect f the deviation of bed shear stress due to the curvature of the channel. For rivers like the Mekong River a value of 0.8 can be used.

The weighing function for the effect of the transverse bed slope on the sediment transport direction reads as follows:

$$f(\theta) = \frac{0.85}{E}\sqrt{\theta}$$

where: E = calibration coefficient
Θ = Shield parameter, which can be computed with:

$$\theta = \frac{hi}{\Delta D_{50}}$$

The value of E has been derived experimentally from flume tests. However its value has been found to vary approximately a factor 2 for the computations of bed deformation in real rivers. The advised values for prototype conditions is E = 1.0.

Figure 3.30 presents the theoretical relation of the ratio between the outer bend depth and the average depth as a function of the two other relevant parameters. This relation was tested versus data from a number of bends in the Mekong River in the NEDECO study of 1995. It was concluded that a reduction factor should be used for the Mekong River.

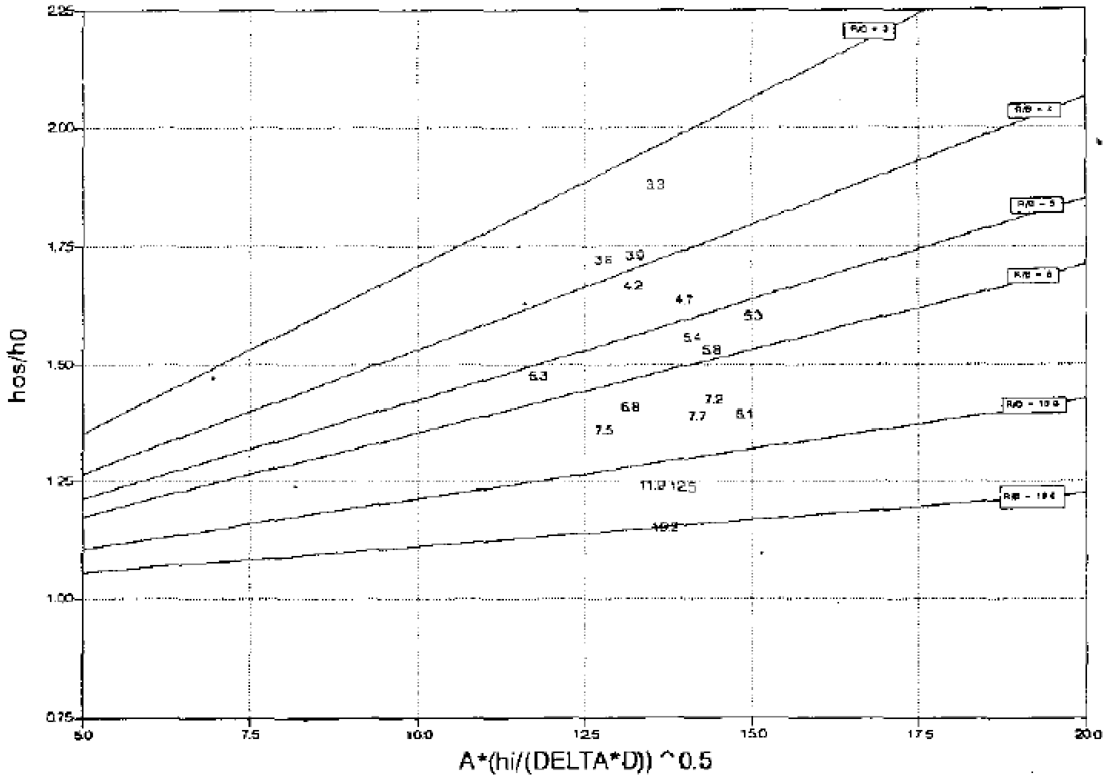
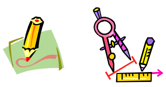


Figure 3.30 Theoretical and test of bend scour prediction.

3.24.4 Protrusion scour



The scour due to the protrusion of bedrock outcrops has not been studied extensively. This type of scour depends on the form and the extension of the protrusion and the turbulence that is induced by it. A prediction of the protrusion scour due to bedrock outcrops can only be carried out with information on the outcrop dimensions and the flow pattern around it.

3.24.5 Local scour

Local scour is the scour produced by man-made structures like groynes, abutments, piers, etc. It is caused by local eddies and additional turbulence being generated by these structures. Within the present study especially the local scour along revetments is of importance.

Along revetments local scour holes do occur even if the velocities are parallel to the revetment. This probably is due to the decrease in depth going towards the river bank and the difference in roughness of the revetment and the adjacent river bed, both phenomena producing horizontal eddies. Eddies in turn cause local scour. No specific design formulas are available for local scour along revetments.

From scale model investigation of the existing revetment along similar rivers like Mekong it has been found that' the scour depth along revetments could be calculated with the following equation:

$$h_{ls} = \varepsilon h_0$$

where: h_{ls} = scour depth due to local scour (m)
ε = coefficient (-)
h_0 = undisturbed water depth (m)

A value of 0.3 has been found for the coefficient ε for the revetments along similar rivers like Mekong. In this guideline the same value for this coefficient is recommended to be valid for the revetments along the Mekong River banks.

3.24.6 Combined scour

The combined scour can be calculated by following the 5 steps that are given hereafter. The k-factors and the scour depth values can be used for design purposes.

1. Constriction scour

The constriction scour depth during the design discharge can be calculated with the following equation:

$$h_{cs} = k_1 h_0$$

where: h_{cs} = constriction scour depth (m)
k_1 = local factor
h_o = initial waterdepth (m)

2. Bend scour

The outer bend scour can then be calculated with:

$$h_{bs+cs} = k_2 h_{cs}$$

where: h_{bs+cs} = bend and constriction scour depth (m)
k_2 = local factor (-)

3. Local scour

The local scour is calculated in addition as followed:

$$\Delta h_{ls} = \xi h_{bs+cs}$$

where: Δh_{ls} = local scour (m)
ξh = empirical factor (= 0.3)

4. General scour

With the mathematical morphological models (2D modelling) some preliminary morphological predictions can be made to check the autonomous morphological development of the Mekong River for the next years. The, resulting changes in bed elevation should be added to the total scour due to the combined effect of other types of scour.

5. Combined scour

The total combined scour depth, referenced to the water level in front of structure, can now be calculated as follows:

$$h_{ts} = h_{bs+cs} + \Delta h_{ls} + \Delta h_{gs} + \Delta h_{cs}$$

where: h_{ts} = combined scour depth in (m)

The combined absolute scour depth can be calculated as follows:

$$\Delta h_{ts} = h_{ts} - h_0$$

3.25 Slope geotechnical stability

3.25.1 Introduction

This section is not intended to provide the designer with a comprehensive knowledge of soil mechanics, but to remind him/her of the geotechnical factors that are relevant to the hydraulic design of river and channel revetments.

Soil is a natural aggregate of mineral particles which can be separated by gentle mechanical means, as opposed to rock where the minerals are connected by strong, permanent forces. Rock banks, unless they have been badly weathered, do not require protection against flow-induced erosion or scouring. There are two basic types of inorganic soil: cohesive soils and granular soils.

Among the many possible soil classification systems, classification by particle size is normally very useful since it is a simple way of identifying soils for preliminary assessments and gives an indication of their likely properties.

3.25.2 Relation between soil characteristic and stable slope angle

Table 3.23 presents different soil categories, drainage characteristics and the nominal particle sizes that are used to establish the limits between the categories.

Table 3.23 Different soil categories.

Soil	Size (mm)	Drainage characteristics
Clay	< 0.002 or 2μ (microns)	Impervious (intact clays) Very poor (weathered clays)
Silt	0.002 - 0.06	Poor
Sand	0.06 - 2.0	Fair
Gravel	2.0 - 60	Good
Cobbles	60 - 600	Good
Boulders	> 600	Good

In Table 3.24 values of the angle of internal friction are also presented for granular soils of various sizes and shapes, and for riprap. These values are approximately the same as the values of the angle of repose, which is the angle to the horizontal at which a heap of material will stand without support, for commonly used in revetment design to account for the reduced stability of particles placed on slopes, due to the component of their weight in the direction of the slope.

The coefficient for reduced stability (i.e. reduced critical shear stress) K_s is usually defined as:

$$K_s = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\phi}\right)^2}$$

where: α = the bank slope
Φ = the angle of repose of the bank material (see Table 3.24)

Table 3.24 Values of the angle of internal friction.

Material	Cohesion c (kN/m^2)	Angle of internal friction Φ$^{*)}$ (^{0})		
Clays - very stiff or hard - stiff - firm of stiff - firm - soft to firm - soft - very soft	 > 150 100 - 150 75 - 100 50 - 75 40 - 50 20 - 40 < 20			
Siky sand			27 - 34	
Granular soils		Rounded	Rounded and angular	Angular
Particle size D_{50} < 1mm 1 - 10 mm 10 - 100 mm		 30 30 - 32 32 - 37	 ~ 33 32 - 36 33 - 40	 33 - 35 33 - 40 ~ 40
Riprap			40 - 45	

*) For uncompacted sand, the angle of internal friction Φ coincides with the angle of repose. For riprap the angle of repose is typically between 35 and 42^0.

The modes by which banks can collapse are many and varied (e-g. deep rotational, shallow, planar failures) and depend on a number of factors too great to describe here in detail. The designer should refer to geotechnical engineering textbooks.

It shall be noted that pore water pressure is a major factor affecting soil strength. As a matter of fact, this statement can be broadened to include the weight of water itself, as water infiltrated into cracks causes not only a rise in pore water pressure but imposes an additional weight on the bank or structure. This increases its susceptibility to collapse.

Soils strengths also influence the stability of side slopes, which may be of particular importance in the construction phase. Typical side slopes for various soil types (underwater slopes are presented in BS 6349-5: 1991), as indicated in Table 3.25.

Table 3.25 Typical underwater slopes for various soil types.

Soil type	**Side Slope**	
	Still water	**Active water**
Rock	Nearly vertical	Nearly vertical
Stiff clay	45^0	45^0
Firm clay	40^0	35^0
Sandy clay	25^0	15^0
Coarse sand	20^0	10^0
Fine sand	15^0	5^0
Mud and silt	$10^0 – 1^0$	5^0 or less

Slip circle calculations (Figure 3. 1) of revetment can be determined following formulae:

$$[K] \geq \frac{\sum g_i \cos\alpha_i tg\varphi + \sum C_i l_i}{g_i \sin\alpha_i}$$

where: g_i = weight of the slice number i (T)

α_I = the angle between the vertical axis of slice i and the radius of slice number in (^{0})

φ = internal friction angle of the soil (^{0});

C_i = unit cohesive force (T/m)

l_i = length of the arc of slice number i (m)

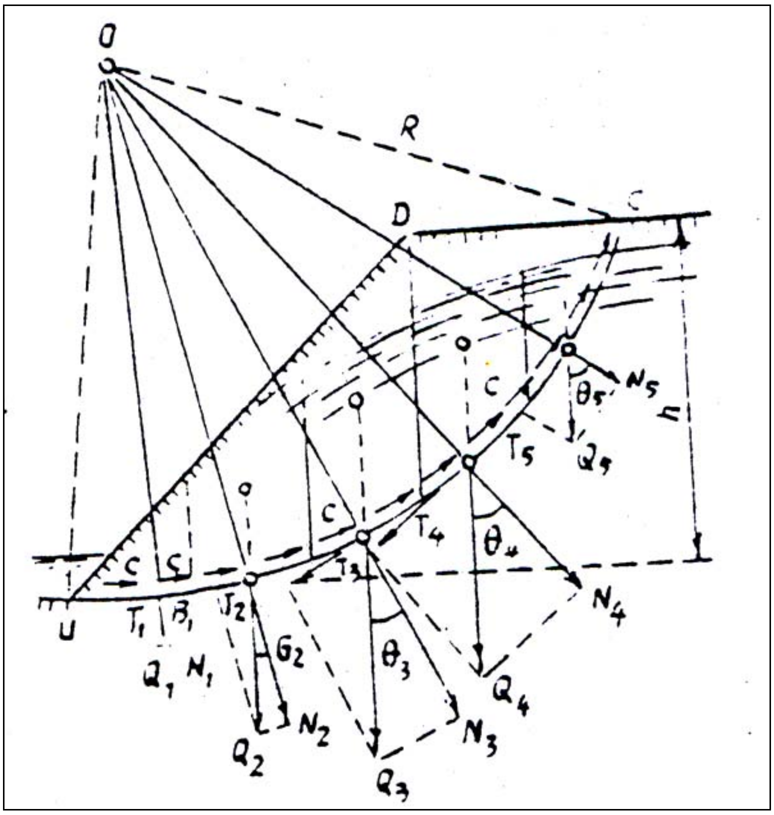


Figure 3. 1 Slip circle calculations of revetment.

In case of seepage flow, the part under the seepage line the forces have calculated under the saturated conditions.

CHAPTER 4

FLOOD EMBANKMENTS / DIKES

4 GUIDELINES FOR FLOOD EMBANKMENTS / DIKES

4.1 Introduction

The terms of reference of the FMMP-C2 call for the preparation of a set of guidelines to address the most suitable manner to develop, repair and maintain structural works aimed to reduce and mitigate flood along the Mekong River. Best practice is understood here as the most up to date, superior or innovative practice that contributes to the improvement or maintenance of the existing environment.

This guideline applies to a wide range of flood embankments. It covers all types of fluvial and coastal embankment, but does not include revetments or measures to protect against erosion. The technical content of the information provided is aimed at individuals within the flood risk management organizations in the countries who are responsible for planning, design and managing or monitoring flood embankments. It is assumed that readers will have an awareness of basic flood management issues but not necessarily technical knowledge of design and construction processes. This document does not attempt to reproduce detailed best practice guidance or national standards, but rather acts as a reference to it in order to highlight the elements of embankment design which are considered to be essential.

Coastal and river flood and flood/ road embankments are used to protect people, property and the environment from high flood and storm water levels. They are one of the most important type infrastructure used by the countries for flood management. Their effective performance during flood events is critical for the provision of sustainable flood risk management.

Flood embankments are usually built from locally available soils and gravels, and shall be built to high standards of construction (compaction and water tightness). Damage to infrastructure, roads, bridges, buildings, housing, equipment, and other components/ utilities are most commonly caused by floodwater inundation. Floodwaters disrupt communication links and inundate large agricultural, settlement and urban areas along the Lower Mekong Basin.

One of the common structural measures for dealing with the excess flow in Mekong River system consists of conveyance options (consisting of any solution that does not rely on storage). This category includes measures that require community participation, such as the construction of small or large scale dikes and the establishment of systems to warn local people of potential floods. In some cases the flood embankments are also designed to fulfil other functional requirements such as road embankments.

In some low lying areas, where small communities are settled, those areas can be successfully protected by encircling them with a flood embankment. This approach can be also treated as area flood proofing and can be applicable to small villages. This encircling dike solution can be useful to individual dwellings or farm buildings. These ring-dikes (empoldering) require further considerations on seepage and drainage due to permeable soil strata, especially for sustained high river water levels.

4.2 How to use these Best Practice Guidelines

Flood embankments are also called levees or dikes. In this guideline the words dike or embankment and levee will be used to illustrate the flood protection function along rivers. Only when the protection against wave attack and high tides is part of the function of embankments would the term sea defence be used. These sea defences are not part of this guideline.

The guidelines are intended to aid anyone involved in planning, design and operation of flood protection dikes, ensuring that loss of life and damage to agriculture, infrastructure, structures and urban areas is reduced.

In the process of developing the guidelines it is recognized that integrated planning of structural works requires a strong participatory process, with central, provincial, and local government agencies sharing the responsibilities with local stakeholders and their representatives. The guidelines may also assist line agencies involved in planning and authorizing structural measures for flood control in the Lower Mekong Basin.

Numerous criteria and issues must be considered in preparing the guideline for dike design. These may vary from project to project, and no specific step-by-step procedure covering details of a particular project can be established. However, logical steps based on successful past projects can be followed for dike design and can be used as a base for developing more specific procedures for any particular project.

Therefore the guideline will be developed to present basic principles used in design and construction of dikes and for the general guidance of design engineers. The guideline is not intended to replace existing national guidelines or the judgment of the experienced design engineer. The primary responsibility for proper dike design lies with the design engineer for the project.

The guidelines will convey best engineering practices in a typical situation and detail the issues or problems which a design engineer may need to resolve. In order for a dike to safely fulfil its intended function, the dike must also be constructed, operated and maintained properly. Supervision of construction or reconstruction of the dike by competent engineers is required to ensure that the dike will be built according to the approved plans.

4.3 Background on development of the Best Practice Guidelines

With the growing demand for protecting transport infrastructure in floodplains, urban areas, people and homes and increasing agriculture production; the issue of construction of flood control embankments in low lying areas is getting more attention lately.

The first step in preparing the best practice guideline for design of flood protection dikes is the collection of existing relevant guidelines for design used in the countries. In some of the countries there are design standards for communication infrastructure, roads/ flood embankments and bridges, flood proofing of buildings and settlements areas. During stage 2 of the project the FMMP-C2 team will collect existing guidelines from the countries for review and further discussion at national and regional level. Furthermore, the extent of successful application of existing national guidelines will be evaluated for making an analysis of planning and design topics that may require more attention in the guidelines.

Consultations with relevant line agencies will be carried out during the process of developing of the guideline. This will be focussed on government agencies that approve construction of new flood protection works, as well as activities on, through or adjacent to existing flood control works.

4.4 Key concepts, purpose and scope

It is recognized that the design of flood protection embankments/ dikes varies according to regulations for design conditions/ forces, foundation conditions, and construction materials. Design forces include height and duration of high water, flow velocities, debris, seepage, internal drainage, natural processes, etc.

This guideline main purpose is to assist line agencies in further improvement of embankments planning, design and therefore will incorporate a number of planning and technical requirements, including, but not limited to:

- The profile of the design flood;
- Freeboard for hydraulic and hydrologic uncertainty;
- Landside slope stability due to steady seepage;
- Waterside slope stability due to draw down;
- Surface erosion of slopes;
- Stream erosion of the waterside slopes;
- Seepage, uplift, and piping through or under the dike and structures;
- Internal drainage;
- Permanent access for inspection, maintenance, and monitoring;
- Practicality and economy of construction and dike maintenance; and
- Structures in and through dikes.

There are also numerous limitations on dike design due to the nature of the design standard, uncertainty in the determination of the design conditions and forces, and ongoing changes experienced in natural systems that affect operation and maintenance requirements. Those aspects need to be addressed in the guidelines as follow:

- Flood protection engineering cannot completely eliminate the risk of failure. For instance, while subsurface investigation is commonly undertaken for new dikes, there is an inherent variability in natural deposits that means perfect information is seldom available. While the engineer account for this in design practice, it is important to note that material behaviour can vary along a dike and anomalies can occur. Similarly, dikes themselves are constructed largely of natural materials as engineered fills with inherent limitations on quality control.

- There are also many older dikes that have not benefited from modern design techniques and technology which demand extra attention or repair because of uncertainties in construction practice.

- Dike management contains an essential continuing component of periodic inspection, performance monitoring and assessment, and maintenance aimed at identification and correction of problems both in advance of and during large flood events. For this reason, additional features are routinely incorporated in good practice embankment/ dike design to facilitate the practicality and economy of O & M. For instance, dike crests are constructed to function as roads for communication and patrol and maintenance, usually a minimum gravelled width of 3.6 to 5m with turnouts provided for maintenance vehicles.

- An important underlying assumption in embankment/ dike design is that there is continuing post construction management including periodic inspection, performance monitoring, routine repairs and maintenance, flood monitoring as well as emergency contingency planning in anticipation of failure or larger flood design events.

- Last but not least, in the context of the FMMP-C2, the aim of a risk analysis in flood protection is to get a systematic judgement of the flood risk and structural measures to reduce risk under cost-benefit aspects. As the input data for the risk analysis itself contains statistical uncertainties, it is inadequate to entirely rely on and communicate an absolute flood risk as a potential danger to the public. However, embankment/ dike sections can be compared and those sections can be identified where flood protection should be improved first. Comparing the risk due to different failure modes to each other, one can furthermore indicate the most cost-efficient measures to reduce risk and increase safety.

The probabilities of different failure modes of embankment/ dike systems are calculated and combined taking into account the statistical input data for geometric, hydraulic and geotechnical parameters.

4.5 Checklist

Dikes along the main river do not allow the flood water to flow on the floodplain or natural retention areas and therefore water levels will increase depending on the location of the dikes along the river. A dike reduces the area of the floodplain and protects areas behind them. Areas located downstream of flood protection dikes will increase due to the additional flooding. In all the aforementioned cases, the impact on flood behaviour requires the use hydrology, hydraulic and morphological changes by using available mathematical models.

Embankments along rivers or around specific flood prone areas like towns are part of what is called empoldering, and have been the standard solution for local protection against flooding through the centuries, in many river valleys and deltas throughout the world. There is nothing wrong with this solution provided that the river retains sufficient space (flood way, floodplain) for the discharge and storage of flood waves, the embankments are well maintained and flood levels are monitored. These points also indicate the weakness of a dike system. In flat low-lying areas, the river may require its storage and large floodplains at the time of floods (as is the case in the Mekong Delta), in which case only limited empoldering may be possible.

Structural measures of this type are considered flood risk reduction measures in the context of the FMMP-C2. This can be justified because of the anticipated urban and infra-structural development in the Mekong basin, adding significantly to the investment value of property, increase in agricultural production and services needing protection from floods.

Together with the height of the crest of a dike another difficult decision the designer has to make is fixing the horizontal alignment of flood embankments. This is influenced by the rate of lateral erosion or meandering of the river. Obviously, people and farmers would like to have the embankments as near as possible to the river, while the authorities responsible for maintenance do not like the idea of frequent rebuilding of embankment sections. The designer also has to bear in mind the need for (a) floodplain(s) along the river channel to enable flood waves to pass safely.

Dikes along the main river do not allow the flood water to flow on the floodplain or natural retention areas and therefore water levels will increase depending on the location/ spacing of the dikes along the river. A dike reduces the area of the floodplain and protects areas behind them. Flooded areas located downstream of flood protection dikes will increase due to the additional flooding.

In addition, the continuous periodic inspection and maintenance of embankments, together with a foolproof flood warning system, require a mentality of both people, farmers and local authorities, which can only develop in time. Assuming the embankment is structurally sound (slopes not too steep, no seepage underneath, no danger of slips, no settlement, no lapse in maintenance), it is mainly the height of the crest which determines its risk of overtopping.

Technical specifications and construction methods are not included in this guideline, although those are to be taken into account during the design. A separate guideline is required for this.

The line agency or proponent of a diking project shall consider the checklist and estimate the costs of design, construction of works and the time frame required to complete the project.

4.6 Planning and design process

The planning process includes suggesting embankment in suitable reaches of the main river and the tributaries with proper justification as to its effectiveness with respect to existing flood problem along with a time frame for its execution.

Designing them to cater to the flood discharge of 25, 50 or 100 years return period as per existing guidelines according to the importance of the area to be protected, Statement of the expected rise in water level, bed level, and flood slope in post embankment condition. Estimation of extent of area likely to be benefited by proposed embankments. Furthermore, examining the existing embankments and suggesting their raising and strengthening.

The assessment of existing flood embankments, the design of improvements or of completely new embankments, and the specification of management action all needs to be done in a manner that takes account of good practice and utilizes appropriate specialist skills.

Identification of natural detention basin water logged area. Study of the measures for drainage improvement viz. Channelization of river and the drains etc. interlinking of channels, digging new drains, providing anti flood sluices, providing adequate regulators in embankments, improving channel conditions by removing local obstructions by weed growth, cultivation etc., diversion of flood water into adjoining river system, examination of adequacy of waterways under road, railways and canals and increasing it suitably wherever required. Likely impact of drainage improvement work suggested, total benefits from them.

Flood management and planning, and design of structural measures are no longer based upon trial and error approaches, like in the pre-computer time. Currently, it is common practice to develop a mathematical model(s) of the flood processes in the area studied and to start with the generation of a well described reference state. Subsequently, a number of scenarios are simulated to show the effects of possible interventions relative to the reference situation.

Therefore for best use of the guidelines to be developed for FMMP-C2 it is expected that appropriate selection of such supporting mathematical models is also included in the design process.

For the appropriate choice of a routing/ hydraulic mathematical model to determine design parameters such as water level and flow velocities for selected extreme events, the following aspects are important:

- What are the physical processes taking place, e.g. flash floods, backwaters, tidal flooding etc. The nature of the flood processes determines what kind of model can be used.
- What sort of data is available and what is their quality. A good analysis of available data may reduce garbage content to a certain extent.
- Generally, a main issue in relation to flood modelling is the availability of topographical data and their quality.
- Suitability of the model to generate confidence in results obtained under extrapolated conditions, even in case of the availability of good sets of data to calibrate these models.

This is particularly important when dealing with floods. Usually one is interested in a range of events that rarely occurs and observations for such events are usually not available. In its application one expects that the model, which has been calibrated for more frequently occurring events, can be applied also for extreme events. As a rule, the better the physical basis underlying a model description, the more reliable such extrapolations are.

In modelling support to design flood embankments the following model types should be distinguished:

- Hydraulic routing models;
- Hydrodynamic models.

ISIS model is a generic modelling system for the simulation of unsteady flow in channel networks and serves as part of the MRC's Decision Support Framework (DSF). The model required improvements in the model schematization and calibration.

The model covers the Mekong Basin from Kratie to the South China Sea, including the Tonle Sap Lake and Floodplain, the Cambodian floodplains and the Vietnamese Mekong Delta.

A complete description of the available models used in the Lower Mekong Basin is given in the FMMP-C2 Main Report Stage 1, ANNEX-2 Modelling.

4.7 General design

4.7.1 Introduction

In this guideline the terms dike and embankment are used to defined structure whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. Embankments that are subject to water loading for prolonged periods longer than normal flood protection requirements, or permanently, should be designed in accordance with dam criteria rather than the dike criteria given herein. It shall be noted that embankments designed for flood protection essentially act as low-level dams for short retention periods. For the majority of the time, most embankments/ dikes are exposed to none or to low, hydraulic head and remain largely unsaturated. However, during flood events, an embankment may need to withstand a rapid rise in water level on the outward face, along with the corresponding changes to internal water pressure (and in some cases seepage) driven by the higher hydraulic gradients across the embankment.

Therefore, in order to achieve optimum design and performance, it is important to understand the nature and potential variability of typical flood embankments (function and form). This section introduces the generic components of a flood embankment (typical features) views how these may vary from site to site. Figure 4.1 shows some typical types and features of a flood embankment. These include:

- *Embankment body.* The main embankment structure providing the mass obstruction against flood water.

- *Toe of embankment.* The bottom of either the outward or inward embankment faces.

- *Inward face.* The embankment face exposed directly to water to varying degrees.

- *Outward face.* The embankment face on the landward side and hence not normally exposed directly to water, except under overtopping conditions.

- *Embankment crest.* The top of the embankment, typically flat and (ideally) several metres wide for safe access.

- *Berm.* Horizontal addition to basic trapezoidal cross-section to provide additional soil mass or for access. Generally on landward side.

- *Surface protection.* Sometimes termed 'revetment'. A protective layer covering part or all of any embankment face. The protective layer may be natural (e.g. grass), manmade (e.g. riprap, concrete) or a combination of different materials.

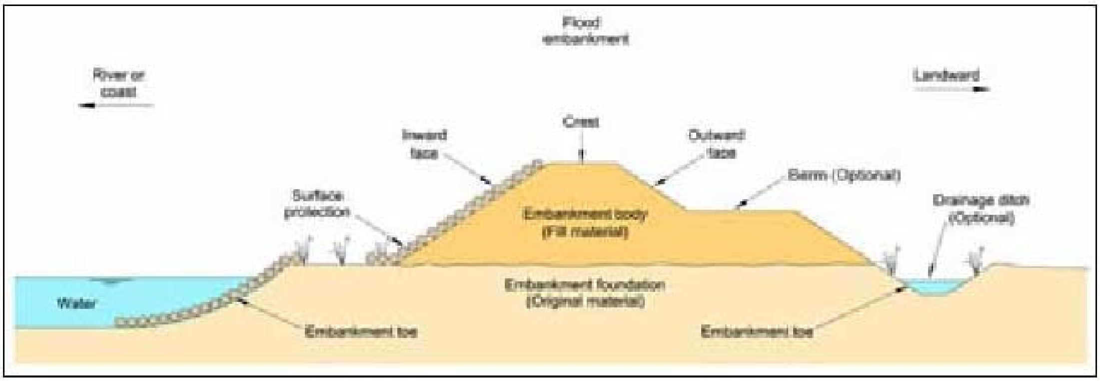


Figure 4.1 Typical features of flood embankments[8].

- *Drainage ditch*. They are typically found close to the outward toe of the embankment to drain any seepage and control water levels through the embankment. Larger 'depth' ditches may exist as a result of borrow areas used for embankment construction (i.e. embankment material taken directly from the ground behind the bank).

Principal function and forms of flood embankment are selected to ensure that the structure under a variety of situations will perform its principal function of flood protection.

The issue of embankment size is not addressed specifically here. It has been assumed that for each case, the embankment size is taken as 'fit for purpose'. This means that an embankment may range in size from perhaps 0.5m up to 5 - 10m or even higher. Whilst loading conditions on the embankment will change with size (i.e. water pressure etc.), the key embankment performance issues remain similar.

In addition to the principal function of flood risk management, the embankment typically also performs a number of secondary functions. These functions vary according to the site specific nature of the embankment but are important to be considered both at the design stage as well as during operation. Integration of a number of secondary functions of a flood embankment can make the difference between the acceptance or rejection of a proposed flood risk management scheme at the planning stage.

A flood embankment provides a barrier between a river and people. Consideration must, therefore, be given as to how people might legitimately access the river or coast across the embankment without an adverse affect on its performance. How this can be integrated into the design and access enhanced whilst preventing any damage to the embankment and how use can be made of accesses provided for maintenance purposes should be carefully considered.

It is important to understand the full role of the embankment at all possible stages in order to carry out a reliable design. The principal function of the embankment will always be to protect land from inundation; however the role of the embankment within a flood management system may not be immediately obvious when on site. The embankment may defend land immediately adjacent, but may also prevent floodwater from bypassing a line of defences, and consequently protect significantly greater areas remote from the embankment itself. The risk associated with

[8] Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme. R&D Technical Report FD2411/TR1. 2007

failure of the embankment may not therefore be immediately obvious and should be established when considering how the embankment and its related asset system may be designed, constructed, maintained or operated. The focus of this guide is upon ensuring the `immediate' performance of flood embankments.

The site-specific details that shall be considered in the structural design of dikes are:

- foundation conditions;
- dike stability with respect to shear strength;
- settlement, seepage, and erosion;
- available dike materials;
- available construction equipment; and
- available area for right of way.

Proposed cross-section designs shall be analysed for stability as it is affected by foundation and/or embankment shear strength, settlement caused by compression of the foundation and/or the embankment, external (surface) erosion, and internal erosion (piping).

4.7.2 Pre-design study

The design of flood protection structures such as embankments varies according to design conditions and forces, foundation, and construction materials. Design forces include height and duration of high water, flow velocities, debris, seepage, internal drainage, natural processes, etc. This implies that a number of technical requirements need to be met, including:

- The profile of the design flood;
- Freeboard for hydraulic and hydrologic uncertainty;
- Landside slope stability due to steady seepage;
- Waterside slope stability due to draw down;
- Surface erosion of slopes;
- Stream erosion of the waterside slopes;
- Seepage, uplift, and piping through or under the dike and structures;
- Internal drainage;
- Permanent access for inspection, maintenance, and patrolling;
- Practicality and economy of construction and dike maintenance, and;
- Structures in and through dikes.

Prior to undertaking a diking project, a pre-design study shall be carried out and include the following components:

- Identify existing flood control measures;
- Characterize the floodplain;
- Establish flood profile;
- Develop conceptual dike alignments and height;
- Identify the benefiting area of the project;
- Assess the impact of the proposed works on the environment;
- Assess the impact on existing agricultural, residential, commercial, and industrial sections within the boundaries of the flood prone area;
- Assess the impact of the proposed work on local drainage;
- Locate suitable local sources of construction materials;
- Prepare a preliminary benefit/ cost assessment of the project, including enhanced property values after the project; and
- Evaluate the hazards associated with the "do nothing" alternative.

Even well engineered and constructed flood protection structures have limitations due to the nature of the design standard used, uncertainty in the determination of the design conditions and forces, and ongoing changes experienced in natural systems.

It is strongly recommended that an experienced engineer carries out a preliminary survey, inspect and study the area using available mapping, obtain an inventory of the existing development from the local authorities and line agencies, and determine the feasibility of any embankment/ dike project. This type of initial assessment may save cost, time, and effort required during subsequent stages of design, and the project is more likely to meet the standards required by the approving line agencies in the countries.

4.7.3 Design high flood level

This is one of the most important criteria for design of flood dikes. In the design of dikes attention should be paid to the statistical - and model uncertainties of the river levels.

The standard design flood shall be the "designated flood" which means "a flood, which may occur in any given year, of such magnitude as to equal a flood having a certain year recurrence period interval, based on a frequency analysis of unregulated historic flood records or by regional analysis where there is inadequate streamflow data available.

Subject to availability of observed hydrological data, the design HFL may be fixed on the basis of flood frequency analysis (depends on government policies). In general, embankment schemes should be prepared for a flood of 25 years frequency in case of predominantly agricultural area and if the embankments concerned are to protect townships, industrial areas or other places of strategic and vital importance, the design HFL shall generally correspond to 100 year return period.

Subject to availability of observed hydrological data, the design High Flood Level (HFL) is normally fixed on the basis of flood frequency analysis.

In the case of embankments on both sides of the river, the design HFL shall be determined keeping in view the anticipated rise in the HFL on account of controlling the width of the river floodplain.

Where the flow of a large watercourse is controlled by a major dam, the designated HFL shall be set on a site specific basis."

4.7.4 Flood mapping

Flood risks maps are created to present information related to the (spatial distribution) of the flood risks on a map. These flood risk maps provide a basis for:

- Identification of flood prone areas;
- Identification of areas that have a large contribution to the flood damage and risk levels;
- Development of measures that effectively reduce the flood risk.

As such the maps provide a basis for the formulation of an integrated flood risk management strategy.

4.7.5 Floodplain regulation/ zoning

Preparation of map dividing the floodplain of the river system depending upon the severity/ risk of floods in different areas in a scale of 1:15000 or 10,000 or any nearer scale subject to the availability of map (Countries and line agencies may do this work and supply the map to the relevant authorities and people).

Carrying out survey works by competent authority for this purpose so that maps with a contour interval of 0.5m on 1:10,000~15,000 scales are available. Based on the flood risk maps prepared as above, the river system may be subdivided into various floodplain zones for identified human activities. Legislative enactment for the floodplain zoning by the countries may be enacted to enforce implementation of the envisaged activity in the various zones.

4.7.6 Field investigations

Once the dike project has been defined, whether it consists of constructing a new dike or upgrading or repairing an existing dike structure, in most cases a field investigation will be required to collect relevant information. A field investigation usually consists of an office review of all available geological, and other, pertinent information on the area of interest, an on-site survey, and subsurface investigation and testing. Some key factors affecting the extent of field investigations include:

- Construction and/or design experience in the area, particularly with respect to dikes;
- Consequences of failure involving life, property, or damage to the environment;
- Proposed final dike height;
- Expected foundation conditions (weak and compressible, highly variable along the alignment, potential under seepage and/or settlement problems);
- Borrow materials available (quality, water contents, variability); and
- Structures in dikes and/or utility crossings.

Field investigation tasks generally include the following:

- Office study - collection and study of topographic, soil, and geological maps, aerial photographs, boring logs and well data, information and performance data on existing engineering projects, etc.
- Field survey – reconnaissance of the proposed alignment and proposed borrow areas and note observations and geology of area, documented by written notes and photographs, including such features as: riverbank and coastal slopes, rock outcrops, earth and rock cuts or fills, surface materials, poorly drained areas, evidences of instability of foundations and slopes, emerging seepage and/or soft spots, natural and man-made physiographic features, etc.
- Interview locals or organizations with knowledge of the foundation conditions in the area.

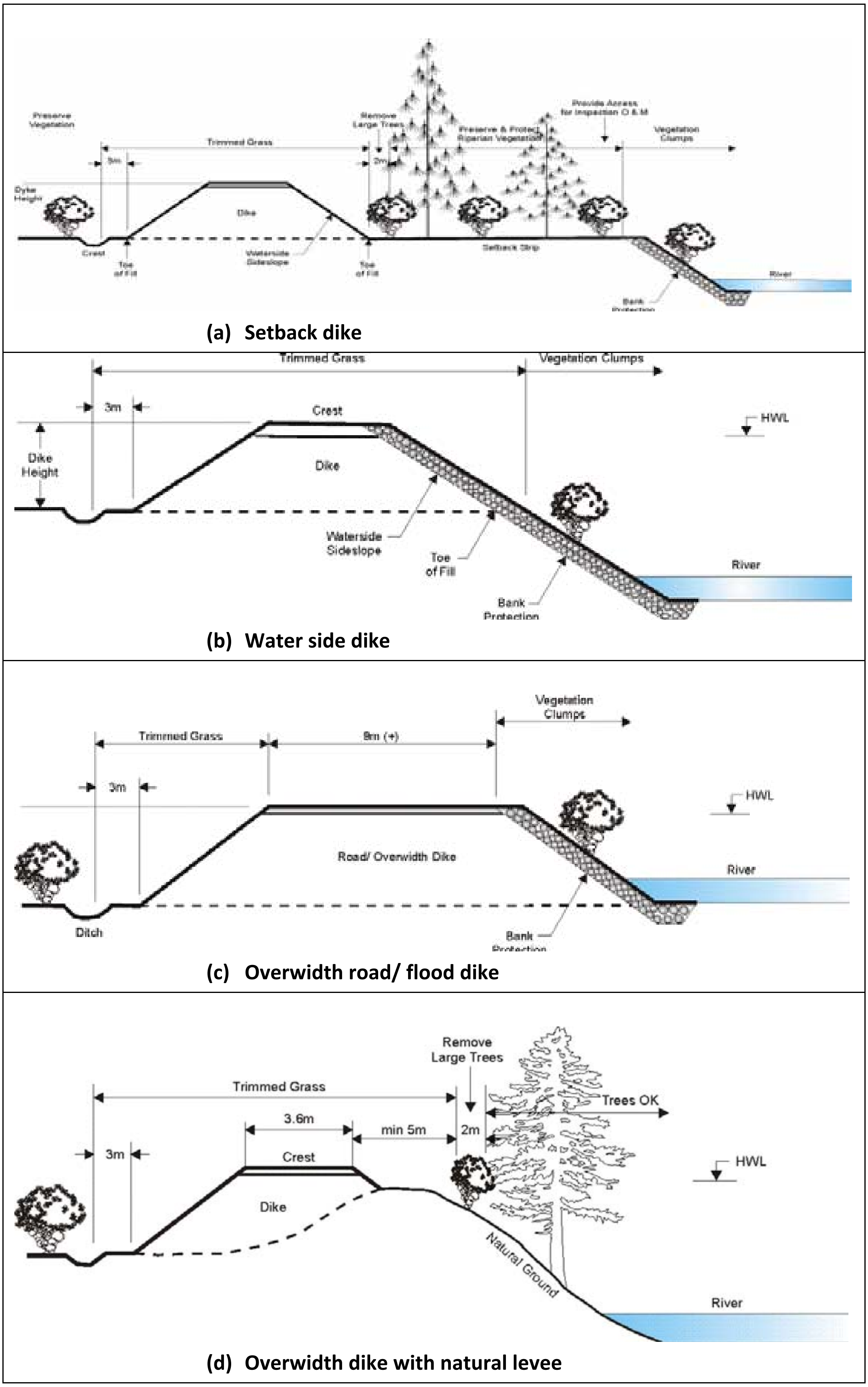
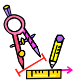


Figure 4.2 Alignment of the dikes relative to the river.

4.7.7 Geotechnical investigation

Subsurface investigation shall be planned by putting down test holes (auger, test pits, etc.), classify materials encountered, collect samples, water table observations, possible penetration testing (SPT's, CPT's, etc.), possible field vane testing, possible geophysical surveys to interpolate between widely spaced test holes, etc.

The geotechnical investigation programme shall also include a laboratory testing including moisture determinations, possible Atterberg limits, gradation analyses, consolidation tests, etc.

The extent of requirement for test boreholes and possible geophysical explorations can be based on information such as geologic maps, aerial photos, groundwater resources, prior experience in the area, and the general nature of the project.

Typical spacing of test holes usually varies from 50 to 300 m along the proposed or existing alignment, with closer spacing in expected problem areas. Test holes are normally laid out along the dike centreline with occasional test holes located near the toe of the proposed dike to provide additional information. At least one test hole shall be located at every major structure. If the dike investigation is carried out in phases, i.e. preliminary and design, additional test holes may be put down as required in the design phase of the investigation.

The depth of test holes shall be sufficient to locate and determine the extent and properties of all soil and rock strata that could affect the performance of the dike or other structures. The depth of test holes along the proposed alignment shall be at least equal to the height of proposed dike at its highest point but not less than 3 m below the existing ground surface. For example, with a 3 m high existing dike, test boreholes put down along the centreline of the dike would extend a minimum of 6 m depth and a minimum of 3 m depth for test boreholes put down at the toe of the dike.

Borehole depths shall always be deep enough to provide data for stability and seepage analyses of the dike and foundation. This is especially important when the dike is located near the riverbank. Where pervious or soft materials are encountered, at least some of the test holes shall extend through the permeable material to impervious material or through the soft material to firm material. Test holes at structure locations shall extend well below invert or foundation elevations and below the zone of significant influence created by the load. The test holes must be deep enough to permit analysis of stability and under-seepage conditions at the structure. In borrow areas, the depth of exploration shall extend about a metre below the practicable or allowable borrow depth or to the groundwater table. If borrow material is to be obtained from below the groundwater table by dredging or other means, test holes shall be at least 3 m below the base of the proposed excavation.

Appropriate field and/or laboratory tests shall be performed in order to aid in evaluating the strength, compressibility, permeability, and erosion resistance of the foundation soils, and the existing dike materials in the case of upgrading.

Also, appropriate laboratory tests shall be performed on samples of the proposed embankment materials in order to ascertain their suitability for use in the dike.

Geophysical exploration methods are a fairly inexpensive means of exploration and are very useful and recommended for correlating information between test holes in areas where they are generally spaced at fairly wide intervals.

4.7.8 Alignment and spacing

The following general items in this checklist on minimum spacing, alignment, etc., can be used to check the tendency of excessive encroachment of the natural floodplain of the river.

The alignment and spacing of embankments need careful consideration with respect to: 1) their vulnerability to the river, 2) the rise of high flood levels on account of reduction in flow area, and 3) increase in peak discharge due to reduction in floodplain storage by construction of the embankment. Finalisation of the alignment and the spacing with due consideration to the above factors and at the same time optimizing the benefit from the proposed embankment would need experience on the river behaviour and studies of the effects of the embankments along different alignments. In view of the widely varying nature of the rivers, no general recommendation about spacing of embankment can substitute the need for the above studies.

The alignment of the dike shall be selected with due regard to setback requirements, available land base for construction and site specific local constraints such as sensitive habitats.

In case of embankments on both banks of the river, the spacing between the embankments should not be less than 3 times Lacey wetted perimeter, given by the formula:

$$\sqrt{P} = 4.75\sqrt{Q}$$

where: P = wetted perimeter (m)
Q = design flood discharge (m^3/s)

In case of embankment on only one bank the embankment should not be less than a distance equal to 1.5 times Lacey's wetted perimeter from the midstream of the river.

Space permitting, a setback dike has benefits when compared to a waterside dike:

- Maintains natural wetland habitat and is environmentally sustainable;
- Provides a wider floodway with increased flow capacity;
- Reduces peak flood levels;
- Reduces flow velocity and bank erosion; and
- Reduces long-term maintenance costs due to less frequent flows against the dike slope.

Figure 4.2, (a) to (d), shows various dike sections relative to the watercourse[9] as follows:

Finalization of the alignment and the spacing with due consideration to the above factors and at the same time optimizing the benefit from the proposed embankment would need considerable experience of the river behaviour and studies of the effects of the embankments along different alignments. In view of the widely varying nature of the rivers, no general recommendation about spacing of embankment can substitute the need for the above studies.

4.7.9 Flow impingement

The issue of flow impingement is of prime importance when preparing the initial alignment of the dike. To the greatest degree practical, the dike shall parallel the direction of flow. In this manner, erosive stresses along the face of the dike during flood conditions can be minimized. By aligning the dike with the direction of flow, erosion protection requirements can be reduced.

[9] **Ministry of Water, Land & Air Protection,** Flood Hazard Management Section Environmental Protection Division Province of British Columbia – July 2003

If the alignment of the dike be such that flow impingement during a flood event cannot be avoided, erosion protection must account for flow impingement. As well, more intensive monitoring subsequent to flood events shall be undertaken. Generally, sharp bends towards the river side of the dike are not recommended.

4.7.10 Free board

The standard for river dike crest elevation results from the either the higher of 1 in 100 or 200 year instantaneous flow plus 0.3 m freeboard, or the 1 in 100 or 200 year maximum daily flow plus 0.6 m freeboard. For agricultural land, the higher of the 1 in 10 or 50 year instantaneous flow plus 0.3 m freeboard or the 1 in 10 or 50 year maximum daily flow plus 0.6 m freeboard is the recommended minimum level.

These criteria can change depending on high discharge or for aggrading rivers resulting in minimum free board of up to 1.8 meters over the design HFL.

4.7.11 Top width

The dike must be configured to enable maintenance vehicles, such as trucks, a reasonable radius of curvature for safe movement, without the wheels riding over the shoulder. Therefore, consideration shall be given at the planning stage to provide manageable curves for expected maintenance vehicles.

Generally the top width of the embankment should be of not less than 3.6m. The turning platforms shall be provided along the countryside of the embankment every kilometre.

The crest of the dike shall be sloped or cambered to promote drainage and minimize surface ponding. The running surface on the dike crest will permit maintenance vehicles and construction equipment access during wet weather without causing detrimental effects or presenting safety hazards for inspection and maintenance personnel.

4.8 Structural design

4.8.1 Introduction

The site-specific details that shall be considered in the structural design of dikes are:

- foundation conditions;
- dike stability with respect to shear strength;
- settlement, seepage, and erosion;
- available dike materials;
- available construction equipment; and
- available area for right of way.

Proposed cross-section designs shall be analysed for stability as it is affected by foundation and/or embankment shear strength, settlement caused by compression of the foundation and/or the embankment, external (surface) erosion, and internal erosion (piping).

When the embankment/ dike must be configured to enable maintenance vehicles, such as trucks, a reasonable radius of curvature for safe movement, without the wheels riding over the shoulder, the designer shall give due consideration to this at the planning stage to provide manageable curves for expected maintenance vehicles.

From experience and considering standard trucks, the radius of a curve shall not be less than 15m to allow efficient access of most heavy equipment. The speed at which a truck can round a curve is limited by the ability of the vehicle to resist centrifugal force tending to move the vehicle toward the outside of the curve. For dikes and embankments, a maximum speed of 20 km/h is recommended.

In case of designing road embankments, there are clear differences with the requirements of design for flood embankments. It is important to consider that new and existing road embankments reflect the following typical design philosophy and approach:

- Road embankments do not include design features, such as an internal impervious core and freeboard, required for a levee or other flood control structures;
- The fill material used in the construction of a typical road embankment is not a sufficient barrier against water; therefore, a road embankment is subject to piping, seepage, and infiltration; and
- Typical road embankment construction does not require the same level of geotechnical engineering analysis as required for flood embankment structures.

4.8.2 Fill settlement

Like uncontrolled seepage, settlement of a dike can result in failure of the dike, but more likely will serve to precipitate failure by another mode such as seepage or shear failure. Consolidation, shrinkage, and some lateral deformation occur over a period of time.

Settlement estimates can be made by the design engineer using standard analysis methods. Detailed settlement analyses shall be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, and embankments on compressible foundations. Where foundation and embankment soils are relatively pervious, most of the settlement will occur during construction.

4.8.3 Sudden drawdown

Analysis is based on the condition where a prolonged flood stage saturates at least the major part of the waterside embankment portion and then falls faster than the soil can drain. This condition only applies to the waterside slope.

4.8.4 Hydraulic gradient

The hydraulic gradient line of a dike should be determined on the basis of the analysis of the soils, which are to be used in the construction of embankments. However, the following guidelines are recommended:

Type of fill	Hydraulic gradient
Clayey soil	1 in 4
Clayey sand	1 in 5
Sandy soil	1 in 6

4.8.5 Side slope

The river side slope should be flatter than the underwater angle of repose of the material used in the fill up to an embankments height of 4.5 meter slope should not be steeper than 1 in 2 and in case of higher embankments slope should not be steeper than 1:3 when the soil is good and to be used in the most favourable condition of saturation and draw down. In case, the higher embankments are protected by rip-rap, the river side slope of earthen embankments up to 6 meters high may be 1 in 2 or 1 in 2.5 depending upon the type of slope protection.

In embankments constructed of sandy materials, the river side slope should be protected with cover of 0.6 m thick good soil.

For the country side slope, a minimum cover of 0.6 m over the hydraulic gradient line should be provided. For embankment up to 4.5 m height, the country side slope should be 1 in 2 from the top of embankment up to the point where the cover over hydraulic gradient line is 0.6 m after which a berm of suitable width with the country side slope of 1:2 from the end of the berm up to the ground level should be provided. For the embankments above 4.5 m and below 6 m heights, the corresponding slope should be 1:3. Normally berms should be of 1.5 m width. For embankments above 6 m height detailed design may be furnished in the project estimate.

It is usually preferable to have more or less free draining material on riverside to take care of sudden draw down. In case of high and important embankment stone rip-rap either dry dumped or hand placed and concrete pavements/ concrete blocks with geotextile and open joints are adopted to protect the embankments against draw down and/or erosive action of the river; in less important embankments where rip-rap is costly willow/ fascine mattress can be used.

Generally the side slopes and 0.6 m wide in top from the edges of the embankments should be turfed with grass sods. In embankments which are in imminent danger of erosion, necessity of protective measures such as slope protection by rip-rap and/or river training works should be examined separately.

An embankment should be provided with suitable soling over filter for proper drainage. For embankments protecting towns industrial area and places of strategic importance the necessity of providing all weather road surfaces of 3 to 3.5 m width should be examined to ensure maintenance works for reaches which are not easily accessible.

In order to provide communication from one side of embankment to other, ramps at suitable places should be provided as per requirement to obviate subsequent interference.

4.9 Confirmation of design criteria

The following criteria shall be confirmed by relevant authorities and local government agencies:

- Design high flood level (HFL)
- Freeboard (FB)
- Design dike crest (HFL + FB)
- Flood construction elevations for residential, commercial, and industrial development
- Floodplain zone boundaries

4.10 Protection of flood embankments

Damages at embankment structures along the river can be initiated by different causes, e.g.:

- uncertainties in subsoil conditions,
- irregularities or deficiencies in material qualities,
- substantial change in boundary conditions,
- underestimation of design loads,
- poor construction unintended use of structure (e.g. excessive live load).

These must be prevented as far as possible by adequate design, experienced contractors combined with appropriate construction supervision and through people awareness campaigns.

Nevertheless, all structure components have a certain risk potential throughout their life-time, which also depends on the quality of the monitoring activities. Some of the most typical failures are shown in Figure 4.3 other failures may also take, place, and might have even higher priority dependent on the local situation.

In order to ensure that consistent and acceptable standards of flood defence are maintained, the design shall incorporate to protect the embankment. Selecting the most appropriate measure(s) will require careful consideration of the embankment location, function(s) and loading. Protection measures are typically required to perform satisfactorily under a range of conditions. For example, whilst the embankment may remain unsaturated and retaining a relatively low water level for the majority of time, it may be required to withstand a rapid rise and fall in flood water level, in conjunction with heavy rainfall. An embankment can be also damaged by river erosion.

Protection measures typically have multiple functions as shown in Section 3 of these guidelines. For example, riprap or sheet piling along the toe of an embankment may protect against erosion of the bank, but it will also increase stability of the outward face and reduce seepage through the embankment. Similarly stone protection may protect against wave or flow erosion, but it may also increase stability of the embankment.

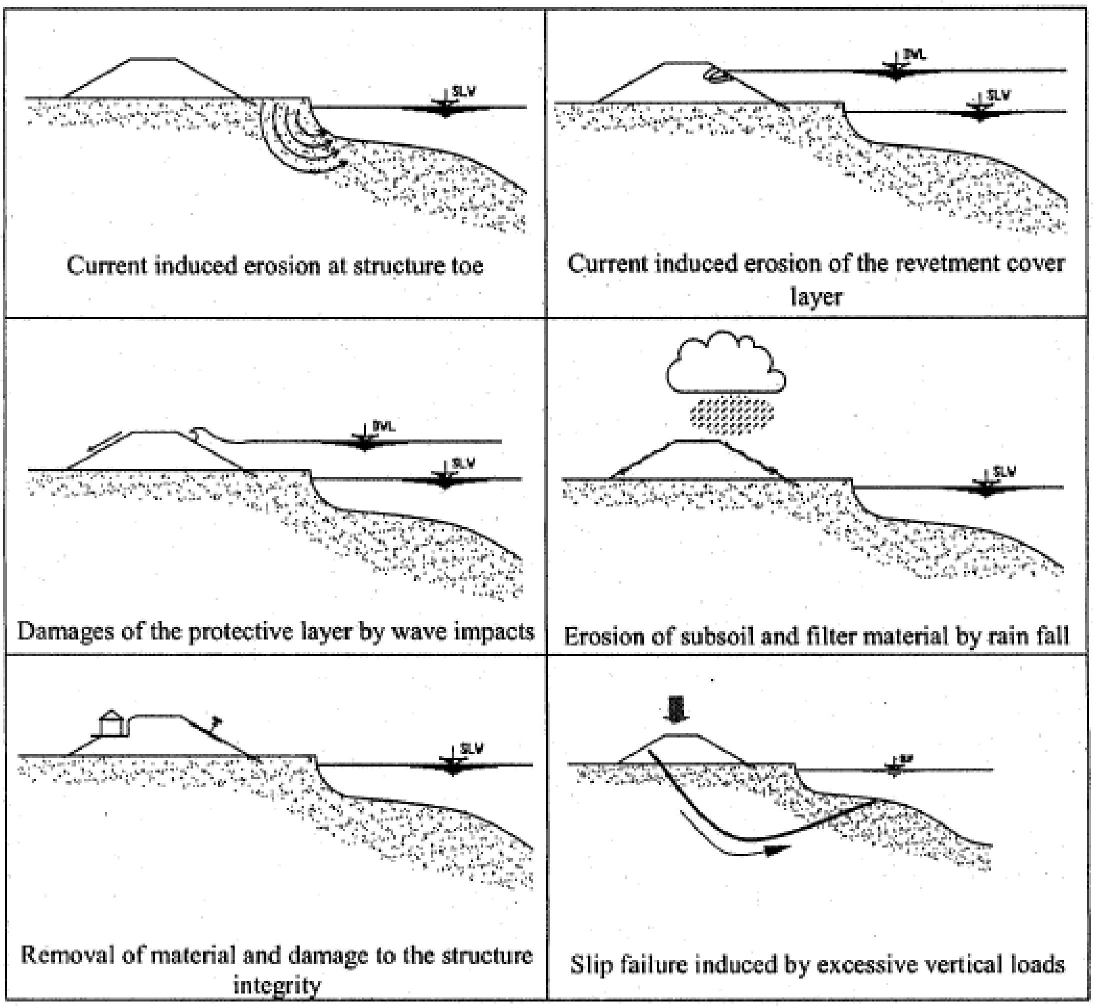


Figure 4.3 Typical failure of flood embankments.

When considering the design of protection measures it is important to consider:

- The dependency of bank integrity and stability upon the measure;
- How maximum value may be gained from different potential solutions.

If the toe of either slope is eroded or undermined, then it reduces the stability of the embankment face above and can lead to slipping of the embankment face, which can eventually threaten the embankment crest level and standard of protection.

Erosion protection needs to be constructed to withstand hydraulic processes that can cause erosion to the dike. The riprap layer is the primary protection against shear stress or erosive forces from flowing water which can act to remove material from the face of the dike (see Figure 4.4). In most of the situations, rock riprap is normally the most cost effective erosion protection material, due to its durability, history of use and availability. Riprap erosion protection is normally easy to repair, straightforward to construct and can withstand some dislocation of the armour without failing.

Any toe protection design should take into consideration channel morphology as well as likely flow velocities and wave action. Positioning a new embankment relative to a morphologically active river channel and rate of erosion is a careful balance between risk of scour and cost of land acquisition.

In case, the higher embankments protected by rip rap, the river side slope of earthen embankments up to 6 meters high may be 3H: 1V or 2.5H: 1V depending upon the type of slope protection. In any case the river side slope angle shall not be steeper than 2H: 1V for lower dikes.

Bank protection and erosion protection works to the dikes are also required along at inlet and outlet areas of culverts, bridges and pump stations.

Figure 4.4 Toe and inward face protection using rock rip rap protection.

4.11 Dike/ embankment access

Access points are essential for emergency access to the dike during high flow periods, for routine inspections and for regular maintenance of the dike. Access roads to the dikes shall be provided at reasonably close intervals in cooperation with regulatory agencies.

These roads shall be all-weather roads that will allow access for the purpose of inspection, maintenance, and flood fighting operations.

4.12 Preparation of project documents

- Design reports.
  The design report shall include an evaluation of the foundation conditions, the hydrologic and hydraulic design and structural stability of the proposed dike. The report shall be sufficiently detailed to accurately define the final design and proposed work as represented on the construction plans.
- Project drawings.
- Technical specifications and O & M Manuals.
  An important underlying assumption in dike planning and design is that there is a need for continuing post construction management including periodic inspection, performance monitoring, routine repairs and maintenance, flood patrolling as well as emergency contingency planning in anticipation of failure or larger than design events. Unfortunately, due to general economics and personnel limitations, this is not always the case. For this purpose, an O & M Manual must be prepared upon completion to provide a standard for the local authority.

4.13 Construction of flood embankments

Accurately predicting the performance of flood embankments and understanding potential breach initiation or other failure mechanisms under extremes in loading is difficult. Nevertheless knowledge about the type of fill material used to construct embankments and the method of the construction does allow the performance of the embankment to be considered in a rational manner and, if appropriate, analysed using principles of soil mechanics.

Many flood embankments are relatively old structures that have evolved over decades or even centuries from original constructions. In contrast with the modern construction of embankments for main roads and dam projects using heavy earth compaction equipment, many flood embankments have been built using low cost traditional techniques. These traditional methods have often evolved to suit local sources of fill material, which have been excavated from surface deposits or retrieved from river sediments. As a result, the construction of flood embankments can be highly variable across the flood prone areas in each country, and this can affect the performance and potential failure mechanism for embankments. Of these traditional construction methods, three common techniques are used as illustrated in Figure 4.5.

The designer shall note that embankment geometry varies according to type of material used and construction history. Ideally, an embankment should have a crest width of greater than 2.5m to allow access along the crest for operations and maintenance vehicles. Whilst the slope of inward and outward embankment faces might sometimes exceed 1 in 2 (according to construction material), stability problems will be encountered as the face is steepened. Poorly controlled maintenance activities can result in bank steepening through excessive removal of soil when cutting vegetation.

In case of embankments in areas subject to wind generated waves or ship induced waves, the slope of an embankment affects the way in which waves run up the face and potentially overtop the embankment.

Recent flood embankments are typically constructed in layers using standard compaction specification of the same kind to road construction as shown in Figure 4.5.

In cases where the fill material is considered to be too permeable, a less permeable core could be incorporated into the construction. An impermeable core is not often used, even where highly permeable fill materials such as quarry waste or silty sand is available. Nevertheless it would be feasible to design the core or cut off to control unacceptable internal seepage and inundation of water behind an embankment that could otherwise pose a threat to the long-term stability during long periods of flood. For example, the core may be built from a more impervious local material, probably with higher clay content, or cement bentonite cut-off wall.

In some situations (typical in the Netherlands, Germany and Denmark), sand embankments are protected by a layer of clay beneath the inward-facing revetment surface layer. In effect, the embankment has a porous but stable core into which seepage is prevented by an impermeable barrier of clay which itself is protected by some form of surface layer such as vegetation.

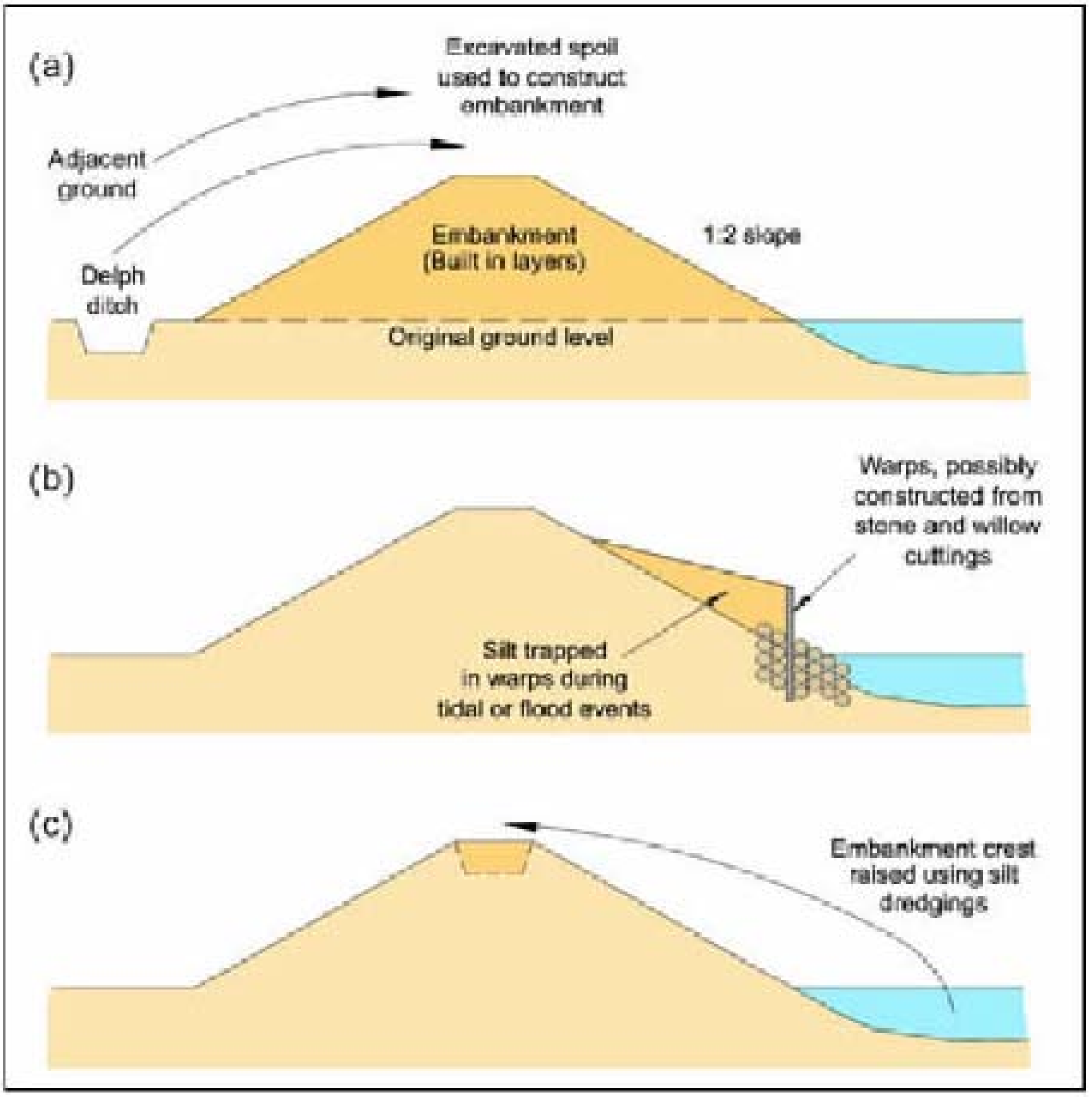


Figure 4.5 Traditional methods of construction (DEFRA, 2007).

A review of traditional earth-fill materials used to construct flood embankments found a wide range of soils and rocks used as fill material depending on the local geology and particularly the superficial deposits. In addition to providing a source of fill material, superficial deposits generally form the founding strata (i.e. foundations) for flood embankments and can strongly influence settlement and stability as well as sub-surface seepage.

Construction plans for embankments/ dikes shall be sufficiently detailed for evaluation of the safety aspects of the structures. As-built drawings of the project are required upon completion of construction.

4.14 Non-structural measures

Non-structural approaches to flood management comprise those activities which are planned to eliminate or mitigate adverse effects of flooding without involving the construction of flow-modifying structures.

Non-structural measures should always be considered conjunctively in the planning and use of structural measures such as dikes may or may not be used conjunctively with non-structural approaches, but are not a prerequisite to the use of non-structural measures because of the potential for synergistic enhancement of their effectiveness.

4.15 Land acquisition

To ensure uniformity in respect of land acquisition for flood embankments, it is suggested that the provision for land acquisition should include extra additional width beyond the toe of the embankment on the river side and additional width of beyond the toe of embankment on the Country side. The additional widths have to be determined depending on site-specific conditions, alignment of the embankment, lateral erosion of the river, need for construction of drainage channels, etc.

4.16 Borrow areas

Generally the borrow area will be on the river side of the embankments. However, in un-avoidable circumstances, when the earth is to be borrowed from the country side the borrow pits shall not be closer than 10 m from the country side toe of the embankments. In certain cases when the depth of the borrow pit is limited to 0.3 meters the borrow pit may be closer to the embankment but in no case the distance between the toe of the embankment and the edge of the borrow pit shall be less than 5 meters. In order to obviate development of flow parallel to the embankment, 5 to 6 meter wide cross bars spaced at 50 to 60 meters centre to centre shall be left in the borrow pits.

4.17 Legislation and regulatory controls

Regulatory controls on construction of new flood protection works, changes and work within the existing dikes, and related maintenance that are fundamentally within the purview of the line agencies shall be also taken into account when preparing dike design.

Construction of dikes and maintenance may also be subject to other provincial and district legislation and regulations, as well as local bylaws and zoning.

In general, floodplain policies and regulations attempt to keep road/ highway embankments entirely out of floodplains. Where this is not feasible, regulations and practice required most road embankments to be sufficiently elevated to avoid overtopping by a flood with a stated percent chance of being exceeded in any given year.

This requirement does not imply an embankment provides an additional flood control role. Instead, the intent is to prevent loss of the embankment as a result of overtopping flows associated with smaller floods. Floodplain regulations require road and highway project design flows to consider potential effects on any existing flood control channels, levees, and retention areas.

4.18 Performance and monitoring of flood embankments

Embankments can become less effective over a period of time for a number of reasons including:

- They experience greater loading than they have been designed for or have historically managed to withstand;
- The required standard of service or some other functional requirement has changed;
- They have deteriorated from their intended condition - as constructed or maintained.

The tendency for the performance of earth embankments to deteriorate with time is of particular concern when considering the increasing loading that will continue to be placed on defences as a result of climate change, and the increasing rate of occurrence of extreme events.

The factors that affect the performance of flood embankments, and their potential failure under extreme events, can be complex. They may be built on low strength, permeable or compressible foundations; the strength and water tightness of material in the body of the embankment may be inherently weak or affected by animal burrows or soil deterioration.

Common hazards or causes of failure are (a) zones of weak or highly permeable material causing slippage or seepage; (b) reduction of crest level and standard of protection due to settlement or the crest being worn away in places causing overtopping; and (c) local seepage paths at junctions with other structures. Because flood embankments are rarely subject to their full loading, these "weakest links in the chain" can go undetected unless there is good monitoring and condition assessment.

The geotechnical characteristics and behaviour of the embankment and its foundations are key factors affecting performance.

CHAPTER 5

REFERENCES

5 REFERENCES

Best Practice Guidelines for Flood Risk Assessment in the Lower Mekong Basin. FMMP-C2, 2009

California Department of water Resources. Division of Safety of Dams. Guidelines for the design of small dam embankments. 1993

CIRIA. Standards for the repair of buildings following flooding C623 London, 2005

CUR, The Rock Manual The use of rock in hydraulic engineering (2nd edition), 2008

DEFRA, Department of Environmental and Food Affairs. Management of Flood Embankments. A Good Practice Review. 2007

Dike Design Guidelines – Best Management Practices for British Columbia, Province of British Columbia Ministry of Water, Land and Air Protection July 2003

FEMA. Non-Residential Flood proofing — Requirements and Certification for Buildings Located in Special Flood Hazard Areas. Technical Bulletin 3-93

FEMA. Riverine Erosion Hazard Areas, Mapping Feasibility Study. 1999

FEMA. Wet Flood proofing Requirements for Structures Located in Special Flood Hazard Areas. Technical Bulletin 7-93

Gary E. Freeman and J. Craig Fischenich. Gabions for Streambank Erosion Control. May 2000

Gerrit J. Schiereck. Introduction to bed, bank and shore protection, Delft University Press. 2000

GJ. C.M. Hoffmans and HJ. Verheij. Scour Manual. AA Balkema, 1997

Government of Bangladesh Ministry of Water Resources. Guidelines and Design Manual for Standardized Bank Protection Structures. Bank Protection Pilot Project FAP 21. 2001

HASKONING, Delft Hydraulics. Meghna River Bank Protection Study, Bangladesh. 1994

Highway Embankments versus Levees and other Flood Control Structures. U.S. Department of Transportation Federal Highway Administration. September 2008

Hossain, M.Z. and Sakai, T. Severity of Flood Embankments in Bangladesh and Its Remedial Approach. Department of Agricultural and Civil Engineering Graduate School of Bioresources, Mie University, Tsu, Japan.

K. W. Pilarczyk. Design of Revetments, Dutch Public Works Department (RWS), Hydraulic Engineering Division. 1997

Kathleen H. Frizell, James F. Ruff, and Subhendu Mishra. Simplified Design Guidelines for Riprap subject to overtopping

Krystian W. Pilarczyk. Geosynthetics and Geosystems in Hydraulic and Coastal Engineering Rijkswaterstaat, Delft, Netherlands. A.A. BALKEMA. 2000

Maccaferri. River Training Works, Problems and Solutions. General Brochure

Margaret S Peterson, River Engineering. Prentice Hall. 1986

Michelle Cramer, Ken Bates, Dale E. Miller. Integrated Streambank Protection GuidelinesS

Minnesota Department of Natural Resources. Floodplain: Elevation Requirements

NEDECO-SPAN-WDC. Mekong River Bank Erosion Study, 1995

P Ph Jansen. Principles of River Engineering The non-tidal alluvial river. 1979

Pham Bach Viet, Lam Dao Nguyen and Ho Dinh Duan. Using remotely sensed data to detect changes of riverbank in Mekong River, Vietnam.

PIANC. The Design of armoured Slopes Report of Working Group No. 22. of the Permanent Technical Committee II. 1997

R. van der Meij. Design of optimal flood protection embankment combining different forms of artificial intelligence. Deltares, Delft, The Netherlands

Texas Department of Transportation. Hydraulic Design Manual Revised March 2009

US Army Corps of Engineers. Philip J. Soar and Colin R. Thorne Channel Restoration Design for Meandering Rivers September 2001

US Army Corps of Engineers. River Engineering Hydraulic and Channel Stabilization Surveys, EM 1110-2-1003. January 2002

US Army Corps of Engineers. Slope Stability. EM110-2-1902. 2003

V. Thuy, I. Sobey and P. Truong. Canal and River Bank Stabilization for Protection aginst Flash Flood and Sea Water Intrusion in Central Vietnam. Department of Agriculture and Rural Development, Quang Ngai, Vietnam.

West Virginia University. Disaster and Emergency Management Resources Flood proofing Measures. Section 3.7

World Meteorological Organization. The Role of Land Use Planning in Flood Management. A Tool for Integrated Flood Management. 2007

APPENDICES

APPENDIX 1 GLOSSARY

N.B. Words in capitals are also listed separately.

Abutment	That part of the valley side against which the dam is constructed, or the approach embankment in case of bridges which may intrude some distance into the water-way.
Accretion	Build up of material solely by the action of the forces of nature through the deposition of waterborne or airborne material.
Aggradation	A build up or rising of the channel bed due to sediment deposition.
Alluvial	Describing the' genesis of sediments by flow of rivers.
Alongshore	See LONGSHORE.
Angle of approach	Local angle between direction of approaching flow and bankline.
Apron	Layer of stone, concrete or other material to protect the toe of a structure against scour.
Armour layer	Protective layer on rubble mound breakwater composed of armour units.
Armour unit	Large quarry stone or special concrete shape used as primary (wave) protection.
Axis of stream	Line joining the mid points of the surface of the stream at successive cross-sections.
Back rush	The seaward return of the water following wave RUN-UP.
Backwater curve	The longitudinal profile of the water surface in an open channel where the depth of flow has been increased by an obstruction such as a WEIR or DAM across the channel, by increase in channel roughness, by decrease in channel width or by a decrease of the bed gradient.
Barrage	A barrage built across a river, comprising a series of gates which when fully open allow the flood to pass without appreciably increasing the flood level upstream of the barrage.
Barrier	The function of a barrier is to control the water level. It consists of a combination of a concrete or a steel structure with or without adjacent ROCKFILL DAMS.
Bathymetry	Topography of sea/ estuary/ lake bed.
Beach	By common usage the zone of BEACH MATERIAL that extends landward from the lowest water line to the place beyond the high water line where there is a marked change in material or physiographic form, or to the line of permanent vegetation.

Beach material	Granular sediments usually sand or shingle moved by the sea.
Bed load	The quantity of sediment moving along the bed by rolling, jumping or sliding with at least intermittent contact.
Bed protection	A (rock) structure on the sea bed or the bed of a river or estuary in order to protect the underlying bed against erosion due to current and/or wave action.
Bend scour	EROSION in (the outer part of) a river bank.
Berm	1) Relative small mound to support or key-in an ARMOUR LAYER, 2) A horizontal step in the sloping profile of an EMBANKMENT.
Bifurcation	Location where a river separates in two or more reaches or branches.
Bio-engineering	The sole use of vegetation for protection against erosion (also known as soft engineering).
Biotechnical engineering	A combination of vegetation and structural units for bank erosion protection.
Blanket	A layer or layers of graded fine stones underlaying a breakwater, GROYNE or rock EMBANKMENT to prevent the natural bed material being washed away.
Braided river	A river type with multiple channels separated by shoals, bars and islands.
Braiding belt	Area extending on both sides along a BRAIDING RIVER out to the extreme historic alignments of the river banks.
Canal	A large artificial channel, generally of trapezoidal cross-section, designed for low velocity flow.
Catchment area	The area which drains naturally to a particular point on a river, thus contributing to its natural discharge.
Channel	A general term for any natural or artificial bed for running water having a free surface.
Coast protection	Works to protect land against EROSION or encroachment by the sea.
Cofferdam	A temporary structure enclosing all or part of the construction area so that construction can proceed in the dry.
Conceptual design	Design stage concerned with the evaluation of the erosion problem, selection of the strategy to control erosion and choice of suitable kind of revetment.
Confluence scour	Erosion at the CONFLUENCE of rivers.

Confluence	The junction of two or more river reaches or branches.
Cover layer	The outer layer used in a revetment system as protection against external hydraulic loads.
Crest	Highest part of an embankment, breakwater sea wall, SILL or DAM.
Dam	Structure built in rivers of estuaries, basically to separate water at both sides and/or to retain water at one side.
Deep water	Water so deep that waves are little affected by the bed. Generally, water deeper than one half the surface wave lengths is considered to be deep water.
Degradation or erosion	A lowering of the channel bed due to SCOUR.
Design storm	Sea walls and embankments will often be designed to withstand wave attack by the extreme DESIGN STORM. The severity of the storm (i.e. RETURN PERIOD) is chosen in view of the acceptable level of risk of damage or failure.
Detailed design	Design stage that involves the detailed design and specification of the engineering revetment systems, including filters.
Dike	A long, low EMBANKMENT with a height usually less than four to five metres and a length more than ten or fifteen times the maximum height. Usually applied to DAMS built to protect land from flooding.
Discontinuity	Any actual or incipient fracture plane in a rock mass including bedding planes, laminations, foliation planes, joints and fault planes.
Diversion channel	A WATERWAY used to divert water from its natural course. The term is generally applied to a temporary arrangement e.g. to by-pass water round a DAM site during construction.
Dredging	Removal of any soil by bank-sided or floating equipment below water level, irrespective of the method employed.
Durability	The ability of a rock to retain its physical and mechanical properties (i.e. resist DEGRADATION) in engineering service.
Dynamic equilibrium	Short term morphological changes that do not affect the MORP-HOLOGY over a long period.
Eddy	A vortex-type motion of fluid flowing partly opposite to the main current.
Embankment	Fill material, usually earth or rock, placed with sloping sides and with a length greater than its height. An embankment is generally higher than a DIKE.

Erosion	The wearing away of material by the action of natural forces.
Facing	A coating of a different material, masonry or brick, for architectural or protection purposes e.g. stonework facing, brickwork facing (concrete dam) or an impervious coating on the upstream slope of the DAM or waterside of an embankment.
Falling apron	Toe protection of granular material, such as concrete blocks or boulders, placed directly on the existing subsoil or river bed (i.e. without filter).
Fetch (length)	Relative to a particular point (on the sea), the area of sea over which the wind can blow to generate waves at the point. The fetch length depends on the shape and dimensions of the fetch area, and upon the relative wind direction.
Filter	Intermediate layer, preventing fine materials of an underlayer from being washed through the voids of an upper layer.
Floodplain	The area within the flood EMBANKMENTS.
Flood routing	The attenuating effect of storage on a flood passing through a valley, a CHANNEL or RESERVOIR by reason of a feature acting as a control e.g. a reservoir with a spillway capacity less than the flood inflow or the widening or narrowing of a valley.
Flow regime	Combinations of river discharge and corresponding water levels and their respective (yearly or seasonally) averaged values and characteristic fluctuations around these values.
Freeboard	The height of a structure above STILL WATER LEVEL.
Gabions	Mattresses and rectangular baskets made from protected steel wire mesh and filled with loose material such as boulders, bricks etc.
Geotextile	A synthetic fabric which may be woven or non-woven used as a FILTER or separation layer.
Gradings	Distribution, with regard to size or weight, of individual stones within a bulk volume. Heavy, light and fine gradings are distinguished.
Granular filter	A band of granular material which is incorporated in an EMBANKMENT dam and is graded so as to allow SEEPAGE to flow across or down the filter zone without causing the migration of the material from zones adjacent to the FILTER.
Grouting	Way of improving stability of revetments by filling joints or gaps with cement or bitumen mortars.
Groyne	A structure generally perpendicular to the shoreline built to control the movement of BEACH MATERIAL.

Hardpoint	Local non-erosive bankline either natural or artificial (massive, stable structure).
Head	End of groyne or spur dike.
Headwater level	The level of the water in the RESERVOIR.
HWL	High Water Level. Water level for a return period 1: 100 year.
Hydraulics	Science of water motion/ flow/ mass behaviour.
Hydrology	Science of the hydrological cycle (including precipitation, run-off, fluvial flooding).
Igneous rocks	Formed by the crystallization and solidification of a molten silicate magma.
Integrity	The degree of wholeness of a rock block as reflected by the degree to which its strength against impacts is reduced by the presence of flaws.
Levee	Natural or Flood EMBANKMENT less than one meter in height.
Life Time	Total time for which the structure is designed to remain in function.
Lining	A coating of asphaltic concrete, concrete, reinforced concrete to provide water tightness, to prevent EROSION or to reduce friction of a canal, tunnel or shaft.
Longshore	Along the shore.
LWL	Low Water Level. Water level for a return period of 1: 100 year.
Maintenance	Repair or replacement of components of a structure whose life is less than that of the overall structure, or of a localized area which has failed.
Mattress	A blanket of brush, poles, plastic, fibres or other material lashed together to protect the EMBANKMENT or river channel from EROSION.
Maximum water level	The maximum water level, including flood surcharge, which the embankment has been designed to withstand.
Mean	The average value of a parameter.
Meandering	A single channel having a pattern of successive deviations in alignment which result in a more or less sinusoidal course.
Metamorphic rocks	Formed by the effect of heat and pressure on IGNEOUS or SEDIMENTARY rocks for geological periods of time with the consequent development of new minerals and textures within the

	pre-existing rock.
Morphology	The transport of sediment and the consequential changes with time of the river or sea bed and river banks.
Numerical model	A description of the reality by means of mathematical equations which allow predicting the behaviour of flows, sediment and structures.
One-dimensional (1-D) model	A NUMERICAL MODEL in which all the flow parameters are assumed to be constant over the cross-section normal to the flow. There is only a velocity gradient in the flow direction.
Overtopping	Water passing over the top of the embankment or flood control structure.
Physical model	See SCALE MODEL.
Pitching	Squared masonry or precast blocks or embedded stones laid in regular fashion with dry or filled joints on the upstream slope of an EMBANKMENT waterside or on the sides of a channel as a protection against wave.
Pore pressure	The interstitial pressure of fluid (air or water) within a mass of soil, rock or concrete.
Porosity	Laboratory measured property of the rock indicating its ability to retain fluids or gasses.
Porous	In terms of REVETMENTS and ARMOUR, cladding that allows rapid movement of water through it such as during wave action (many GEOTEXTILES and sand asphalt can be non-porous under the action of waves but porous in soil mechanics terms).
Prototype	The actual structure or condition being simulated in a model.
Protrusion scour	Scour immediately upstream from a local structure or obstruction due to local acceleration of the flow.
Quarry run	Waste of generally small size material, in a QUARRY, left after selection of larger GRADINGS.
Quarry	Site where natural rock stone is mined.
Quasi three-dimensional (3-D) model	A NUMERICAL MODEL in which the flow parameters vary in two dimensions, but which allows to determine the flow parameter in the third dimension.
Reach	Part of a river channel in longitudinal direction.
Refurbishment, renovation	Restoring the embankment to its original function and level of protection.
Regime equations	Empirical formulae based on typical relations between channel

	dimensions (incl. slope and roughness) and river discharge.
Regime theory	Empirical method for predicting river characteristics.
Regulating/ retention reservoir	A RESERVOIR from which water is released so as to regulate the flow in the river.
Rehabilitation	Renovation or upgrading.
Replacement	Process of demolition and reconstruction.
Return period	In statistical analysis an event with a return period of N years is likely, on average, to be exceeded only once every N years.
Return Period	Recurrence time, average time interval between subsequent events in which conditions are exceeded. When designing a structure, the return period is usually larger than the projected lifetime, because, for instance, if both would equal 50 years, the structure would have a 64% probability of failure during its lifetime.
Revetment	A cladding of stone, concrete or other material used to protect the sloping surface of an EMBANKMENT, natural coast or shoreline against EROSION.
Revetments	Layered systems of cover intermediate and filter layers placed on a sloping surface as protection against hydraulic forces and scouring
Rip-rap	Wide graded quarry stone normally used as a protective layer to prevent EROSION of the sea and/or river bed, river banks or other slopes (possibly including the ad-joining crest) due to current and/or wave action. Also: layer of loose stones acting as cover layer in an embankment revetment, a bed protection or a falling apron.
River regime	Combinations of river discharge and water levels, characteristic for a prescribed period (usually a year or a season) and determining for the overall MORPHOLOGY of the river.
River training structure	Any configuration constructed in a stream or placed on, adjacent to or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce SCOUR, or in some other way alter the flow and sediment REGIMES of a river.
Rock degradation model (armour stone)	A model under research and development, which attempts to predict yearly weight losses from the ARMOUR, taking account of rock properties and site conditions.
Rock weathering	Physical and mineralogical decay processes in rock brought about by exposure to climatic conditions either at the present time or in the geological past.
Run-up, run down	The upper and lower levels reached by a wave on a structure,

	expressed relative to still water level.
Scale or physical model	Simulation of a structure and/or its (hydraulic) environment in usually much smaller dimensions in order to predict the consequences of future changes. The model can be built with a fixed bed or a movable bed.
Scour protection	Protection against EROSION of the river banks and bed in front of the TOE.
Scour	Washing away of the bed/ bank material under the action of current and wave.
Sediment load	The sediment carried through a CHANNEL by streamflow.
Sedimentary rocks	Formed by the sedimentation and subsequent lithification of mineral grains, either under water or more rarely on an ancient land surface.
Seepage	The interstitial movement of water that may take place through an embankment or revetment.
Shallow water	Commonly water of such depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than half the surface wave length as shallow water.
Shoulder	Horizontal transition to layer of larger size stones which is placed at higher elevation.
SHWL	Standard High Water Level. Water level exceeded during 5% of the time.
Significant wave height	The average height of the highest of one third of the waves in a given sea state.
Significant wave period	An arbitrary period generally taken as the period of one third of the highest waves within a given sea state.
Slope protection	The protection of EMBANKMENT slope against wave action or EROSION.
Slope	The inclined face of a cutting or canal or EMBANKMENT.
SLWL	Standard Low Water Level. Water level exceeded during 95% of the time.
Spillway	A structure over or through which flood flows are discharged.
Spur (-dike) or Groyne	A structure extending from a bank into a channel that is designed usually to protect the banks or to provide enough water depth for navigation purposes.
Stationary process	A process in which the mean statistical properties do not vary with time.

Still water level	Water level which would exist in the absence of waves.
Stochastic	Having random variation in statistics.
Storage reservoir	A RESERVOIR which is operated with changing water level for the purpose of storing and releasing water.
Storm surge	A rise in water level in the open coast due to the action of wind stress as well as atmospheric pressure on the sea surface.
Streambed	Low water channel.
Subcritical	The flow condition above a dam by which the TAILWATER level influences the upstream head. The discharge is a function of upstream and downstream head. Also called submerged flow, submodular flow or DROWNED FLOW.
Supercritical	The flow condition above a DAM by which the upstream head is independent of the TAILWATER level. The discharge is a function of the upstream head only. Also called free flow, rapid flow or MODULAR FLOW.
Suspended load	The material moving in suspension in a fluid, kept up by the upward components of the turbulent currents or by the colloidal suspension.
Thalweg	The locus of the deepest points in a valley at successive cross-sections.
Tides	Water movements, basically due to global astronomic response of Oceans and besides, on the continental shelves and in coastal waters -and particularly estuaries and bays-strongly affected (amplified) by shallow water and coastal platforms. Typical specific definitions of associated local water levels, in decreasing order, are HAT or HHW, MHWS, MHW, MLW, MLWS, LAT or LLW.
Toe blanket	See APRON.
Toe	Lowest part of seaward and port-side breakwater slope, generally forming the transition to the sea bed.
Total load	The sum of BED LOAD and SUSPENDED LOAD in the river.
Training wall	A wall built to confine or guide the flow of water in a CHANNEL.
Turbulence intensity	Ratio of the variation of flow velocity around the mean and the mean flow velocity near the bed.
Two/ three-dimensional (2/3-D) model	A mathematical model in which the flow parameters vary in two/ three dimensions.
Underlayer	The layer underneath the cover layer that makes the transition to the underlying soil; it may consist of a granular material or a

	geotextile.
Upgrading	Improved performance against some or other criteria.
Uplift	The upward pressure in the pores of a material (interstitial pressure) or on the base of a structure.
Up-rush, down-rush	The flow of water up or down the face of a structure.
Wandering	See MEANDERING. Applied to a river showing river channels between braided and meandering.
Waterway	A navigable CHANNEL.
Weir	A low dam or wall across a stream to raise the upstream water level. Termed fixed-crest weir when uncontrolled.

APPENDIX 2 THE BEST PRACTICE GUIDELINES AND PROJECT PHASES/ STAGES

In order to manage an engineering project properly, it is normally divided in project phases. Common is a division in the following five phases:

1. Initiation
2. Planning/ Development/ Design
3. Production/ Execution
4. Monitoring/ Control
5. Closure

A project starts with an idea to solve or mitigate a problem, create a product or structure etc. In the initiation phase finances are mobilised, a project team is formed, the equipment and tools are acquired, and the idea is given its first shape. The second phase is the planning/ development/ design phase. The feasibility of the idea is tested, and, if successful, a project plan is elaborated and the design is made. In Phase 3 the plans and designs are implemented, i.e. the production takes place, the project is being executed. Monitoring during execution may reveal the necessity to correct the planning and/or design, and make adjustments in the execution. After completion of the works the project will be closed, i.e. the team will break up, the accounts will be closed, and the product or result may be handed over to a client.

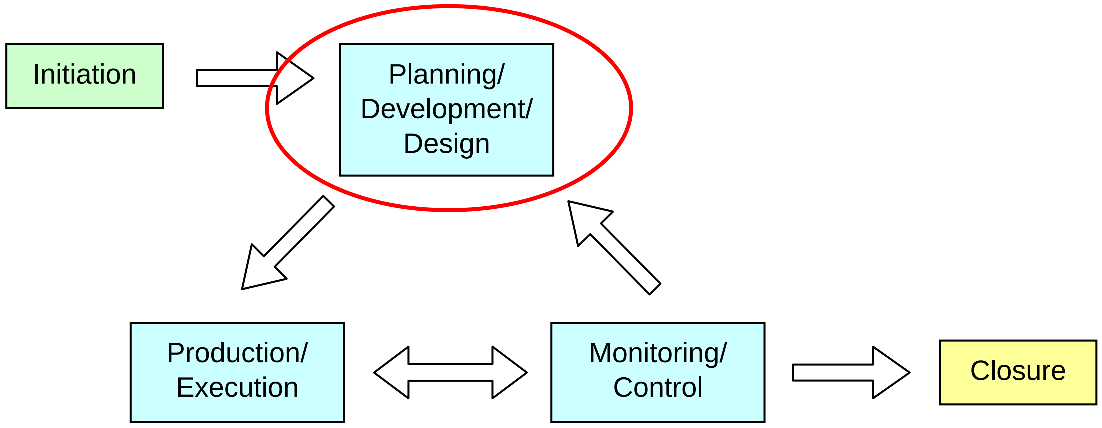


Figure **5.1** The phases of an engineering project.

The Best Practise Guidelines are almost exclusively applicable to Phase 2: Planning/ Development/ Design. This phase can be subdivided in various stages: see the list below. The number and content of the stages may differ, depending mainly on project type or country-specific preferences. The preliminary design stage for example is in engineering projects often included in the feasibility study.

a) Preliminary/ prefeasibility study
b) Feasibility study & overall planning
c) Preliminary design
d) Detailed design & detailed planning
e) Construction/ bid documents

Each section of the guidelines applies to one or more of the above stages. In the guidelines this will be indicated by displaying the above symbols in the page margin.

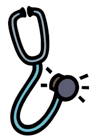
The five stages of Phase 2 contain the following:

a) Preliminary/ prefeasibility study

A prefeasibility study is the precursor to a feasibility study, design study or master plan. Its main purpose is to decide whether it is worthwhile to proceed to the feasibility study stage and to ensure there is a sound basis for undertaking a feasibility study.

A prefeasibility study generally includes:
- Definition of achievable project outcomes;
- Analysis of the development situation and constraints the project is to address, based on collected data;
- Identification of related (government and other stakeholders) policies, programs and activities;
- Preliminary assessment of the viability of alternative approaches;
- Preliminary identification of likely risks to feasibility and benefits (including risks to sustainability).

b) Feasibility study & overall planning



If a project is considered to be feasible based on the prefeasibility study, a more thorough feasibility study can start. A feasibility study defines the project and its objectives in detail, and look at various forms of feasibility:

- Technical feasibility: Can the measures technically be realised in local context?
- Operational feasibility: Will the implemented measures be manageable?
- Economic feasibility: Is the cost-benefit analysis positive?
- Social feasibility: Are the objectives and measures socially acceptable?
- Environmental feasibility: Are the environmental impacts acceptable?
- Political feasibility: Will the measures be supported by the politicians?
- Overall feasibility: Will implementation of the envisaged measures result in accomplishment of the project objectives?

Field surveys, hydrological and hydraulic analyses (in flood mitigation projects), social and environmental assessments, stakeholder meetings, costs estimates etc. are the basis for answering the above questions. If the answers are positive, the operations/ management structure and management method will be defined, and any initial planning will be detailed.

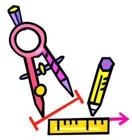
c) Preliminary design

If a project is deemed feasible, the preliminary design stage can start. This stage focuses on the technical measures and includes the following:

- Site surveys and investigations and computer modeling provide the data for preliminary design criteria;
- The design criteria are translated into the preliminary design of structures and measures in an integrated and balanced system in which the envisaged management activities are geared to one another;
- A review of the cost-benefit analysis (construction and operation) and analysis of environmental, social and political factors still show the viability of the project.

If necessary, the project planning will be adjusted based on new insights gained in this stage.

d) Detailed design & detailed planning



During the final design stage the detailed architectural and engineering drawings (the blueprints) of all physical components of the project are produced. Virtually all design problems must have been resolved before the end of the final design stage. Sufficient detail must be provided by the drawings and the report to allow reasonably accurate estimates of construction and operating costs, as well as the construction scheduling.

e) Construction documents/ bid documents

The detailed designs and construction scheduling are incorporated in construction documents and bid specifications, giving the contractors the information they need for construction.

If sections of the guidelines refer to other than the above-described phases (e.g. the construction or monitoring phase), the following symbol will be used: