Nile Basin Initiative

Eastern Nile Subsidiary Action Program (ENSAP)

Eastern Nile Technical Regional Office (ENTRO)

Flood Preparedness and Early Warning Project

Flood Embankment Design, Operation and Maintenance Manual in Ethiopia

April 2010

Prepared By:
Ephrem Tamiru (Consultant)
Email: tamiruephrem@yahoo.com

Revised by:
Ephrem Tamiru & ENTRO

P.O. Box 27173-1000, Addis Ababa, Ethiopia Tel. +251 (11) 646 1130/646 1132, fax: +251 (11) 645 9407
Email: entro@nilebasin.org or entro@ethionet.et Web site: www.nilebasin.org/entro

FLOOD PROTECTION DESIGN MANUAL
PART I: GENERAL INFORMATION

PERFACE

This production of this manual was initiated and sponsored by Nile Basin Initiative (NBI), Eastern Nile Technical Regional Office (ENTRO) under close oversee of the Ministry of Water Resources of the Federal Democratic Republic of Ethiopia and paramount contributions of other stakeholders in the country and regions. The purpose of the manual is to give guidance to practicing engineers and associated connoisseur in the engineering planning, study, design and construction of flood protection works, and other affiliated water resource professionals. It can also be used as a baseline for further future update and revision.

The manual represents an advice of good practice and therefore takes the form of recommendations. Compliance with it does not confer immunity from relevant statutory and legal requirements as there are a lot of decisions, judgements and site specific conditions to be optimized by the investigator, designer or construction supervisor.

In preparing this manual short theoretical background followed by illustrative figures, tables, charts and worked examples were included for beginners. The comments given by ENTRO professionals, World Bank Expert Team, and the interaction during the brainstorming workshop together with participants' remarks were very useful and carefully included.

I extremely thankful to Dr. Babikar Abdalka, Regional FPEW Coordinator, and all ENTRO staffs for the cooperation made during the realization of this manual. Their involvement is not only limited on the smooth flow of consultancy services but also largely on their invaluable technical inputs at all stages of the preparation of the manuscript.

I would like also to extend my appreciation for the cooperation received from Ministry of Water Resources, specially W/o Semunesh, Hydrology Department Head and Ato Muluken from Amhara National Regional State Water Resource Bureau.

Let me also pay my sincere gratitude for Eng. Mekuria Tessema (Civil and Geotechnical Engineer), Ato Million Gebreyes (Sociologist & Development Consultant) and Ato Getahun Worku (Environmental Consultant) for their dedicated and significant contributions made for the different parts of the manual.

My thanks are also to various other persons who have directly or indirectly helped me in the accomplishment of this manual.

April 2010

Ephrem Tamiru

Consultant

Table of Contents

1. INTRODUCTION

1.1 General Background

Major irrigation development potential areas in Ethiopia are located along the major river banks and valley bottoms. Those along the river banks would be flooded during storm or high discharges in the rivers which require protection against flooding from the rivers by flood protection structure dykes.

Rainfall runoff from high grounds to the valley bottom and plain areas will cause flooding to infrastructures and settlement areas. Such flood event will cause large casualties and damage to property which in turn have economic effects on a national level. One of such recent flood damage example in the country is the August 2006 Dire Dawa Flood event which caused high casualties. In such a case, flood containing structures and flood diverting structures are required for the protection of settlement areas and infrastructures.

With this basic background, Eastern Nile Technical Regional Office (ENTRO) has sponsored the preparation of flood protection design manual which includes the design, operation and maintenance of flood protection embankments and associated structures.

The manual has been prepared by a Water Resource Consultant Mr. Ephrem Tamiru funded by Eastern Nile Technical Regional Office (ENTRO) under Flood Preparedness and Early Warning Project (FPEW-1).

The Flood Preparedness and Early Warning Project is a sub-project of the Eastern Nile group of countries under the Nile Basin Initiative. This sub-project focuses on the regional aspects of flood management including strengthening of national capacity. As part of the implementation programme and national capacity building, it is an important first step in establishing a guideline for the design, operation and maintenance of flood protection embankments.

Embankments can be built to provide protection of flooding for a design period. However, there is a possibility that flood embankments will be overtopped by floods that are bigger than the design flood. Therefore, embankments are structural measure to confine flood with in allowable flood corridor.

Hence, the objective of the manual is to bring a measure of consistency to the design of flood protection structures in Ethiopia to ensure that minimum standards of the design are achieved.

The manual is aimed to cover most aspects of flood protection works, starting from the project planning aspect up to the construction aspect. It includes project planning, data requirements like topographic survey, geotechnical investigation and hydrological investigation and method of analysis. It also addressed socioeconomic and environmental impact considerations as a result of flood as well as structural measures.

The manual identifies a set of design standards and procedures to be used for the design of flood protection structures primarily associated with irrigation and land development projects.

1.2 The Need for Flood Protection Design Manual

Most irrigation schemes in Ethiopia, particularly those with gravity command, located close to the river, are prone to flooding. Most existing schemes have flood protection dykes, however for several reasons the existing flood protection might not work to the required level of protection.

It is apparent that in some cases dykes are breaching not only through overtopping but because of structural failure, usually as a result of poor construction or inadequate maintenance. Causes of embankment failures and their mitigation measures have been discussed in the relevant part of the manual.

In some cases river beds are aggrading, the water level during floods is higher and thus the embankment top needs to be raised regularly in order to maintain the same level of protection.

Therefore, efforts have been made to explore lessons and experiences of design of flood protection works in Ethiopia mainly from Awash Flood Control and Watershed Management Project.

1.3 Purpose of the Flood Protection Design Manual

The purpose of this manual is to present basic principles used in the design and construction of flood protection embankments and associated structures in Ethiopia. The manual applies to all engineers and institutions having responsibility for designing and constructing flood protection works.

The manual identifies a set of design standards and procedures to be used for the planning, design, construction and maintenance of flood protection embankment mainly associated with irrigation and land development projects in flood plain areas.

The manual is general in nature and not intended as step by step design procedures and needs the judgment of the design engineer on a particular project

1.4 Scope of the Manual

Structural methods for protection of floodwater can be done through several methods like: storage, diversion, enhancing channel capacity, and constriction of flood wall within the flood corridors. The applicability of one or more of these methods may depend on the specific local condition. However, the scope and discussion of this manual is limited more of on design of flood protection embankments. Therefore, further updating of the manual is future sited to include several structural flood protection measures and practical examples.

As the design criteria and technology evolve the Manual will require revisions and improvement so as to create the dynamic of the manual.

Comments and suggestions concerning the content and format are welcomed from the users of the manual to the address specified in the cover page.

1.5 Structure of the Manual

The manual contains 10 major parts including this **Part 1: General Information**. A general structure of the manual has been presented in this part of the manual as well as at the beginning of the manual to facilitate easy use of the manual. Detailed table of content has been provided at the beginning of each major Part. Reference has been provided at the end of each chapter for further references. The consultant has visualized the recommendation made from various stakeholders and rearranged the general structure of the manual as it is shown below. The structure of the manual is rearranged to follow implementation procedures in flood protection works.

- Part -1 General Information
- Part -2 Project Planning
- Part -3 Socio Economic and Environmental Criteria

- Part -4 River Geomorphology & Geotechnical Investigation
- Part -5 Topographic Survey
- Part -6 Flood Hydrology
- Part -7 Hydraulic Channels Design
- Part -8 Flood Protection Structures
- Part -9 Construction Aspect
- Part -10 Inspection and Maintenance
- Appendix Useful Data, Conversion factors and References.

FLOOD PROTECTION DESIGN MANUAL
PART 2: PROJECT PLANNING

Table of Contents

CONTENTS OF OTHER PARTS

1 INTRODUCTION

1.1 General Background

Flood is unavoidable natural phenomenon occurring in our daily life. Nowadays where there are environmental changes created by global warming, the occurrence of flood frequency and calamities to human life and properties are very dreadful.

To cope up the flood disasters and early preparedness for the problems, all concerned governmental and non-governmental bodies need to have a foresight and plan of action. Therefore, to effect this, the Nile Basin Initiative, the Eastern Nile Technical Regional Office (ENTRO), is taking responsible for providing administrative, financial management, and logistical support in the implementation and management of the Eastern Nile Subsidiary Action Program (ENSAP). In general, ENTRO's core functions are: ENSAP coordination and integration; project preparation; financial management; communications and outreach; training; monitoring and evaluation; information exchange; and serving as the secretariat for ENSAP organizations.

Thus the preparation of this manual is initiated by ENTRO as a first action plan for flood preparedness and early warning project in Ethiopia. The manual incorporates 10-parts where project planning is one of the manuals that have paramount importance for water resource engineers and associated professionals.

1.2 Flood Disasters and Associated Problems

I. Disasters in General

There are many disaster occurrences on earth which are sudden, accidental event that causes many calamities and injuries. Most disasters also result in significant property damage. Common natural causes of disasters include earthquakes, floods, hurricanes and typhoons, and tornadoes. Tsunamis (popularly, but incorrectly, known as tidal waves), volcanic eruptions, wildfires, landslides and avalanches rank among the other natural forces that sometimes create disasters.

Not all disasters are produced by the forces of nature. Many modern-day disasters involve accidents aboard passenger-carrying airplanes, ships, or railroads. Other “man-made” disasters can be traced to the collapse of buildings, bridges, tunnels, and mines, as well as to explosions and fires unintentionally triggered by humans.

Although acts of war and terrorism also inflict death and destruction, these events are intentional rather than accidental, and therefore are not considered disasters in the context of this article.

II. Flood Disasters

During raining or snowing, some of the water is retained by the soil, some is absorbed by vegetation, some evaporates, and the remainder, which reaches stream channels, is called runoff. Floods occur when soil and vegetation cannot absorb all the water; water then runs off the land in quantities that cannot be carried in stream channels or retained in natural ponds and constructed reservoirs.

Floods not only damage property and endanger the lives of humans and animals, but have other effects as well. Rapid and uncontrolled runoff causes soil erosion as well as sediment deposition problems downstream. Spawning grounds for fish and other wildlife habitat are often destroyed. High-velocity currents increase flood damage; prolonged high floods inundates farmland, town, and impair traffic movement. It interferes with drainage and economic use of lands and water logging. It causes epidemic diseases, rise of prices and lose of work. Bridge abutments, bank lines, sewer outfalls, and other structures within floodways are damaged, and navigation and hydroelectric power are often impaired. Financial losses due to floods are commonly millions of dollars each year.

III. Causes of Floods

The flowing are the main causes of floods:

(1) Intensity of Rainfall in Catchment Area. The intensity of rainfall in the catchment area is the main cause of a flood. If the rainfall intensity is normal and the storm duration is short, the surface run-off will flow down smoothly through the tributaries and rivers will not create any trouble to the downstream side. But if the rainfall is very heavy and the storm duration is longer, then the surface run-off will be increased unexpectedly and it may exceed the normal carrying capacity of the channel and hence overtopping of the river bank may occur and the surrounding area may get submerged.

(2) Topography of the Catchment. A catchment area with steep slope increases the run-off and increases sediment inflow due to the high velocity of the water. While a catchment area with flatter slope reduces the run-off and sediments inflow due to low velocity of the flowing run-off.

(3) Sedimentation of rivers. If the tributaries of a river carry heavy sediment load the river bed goes on silting up gradually every year. Thus, the carrying capacity of the river goes on reducing every year. Ultimately the cross-section of the river will be shallow and it will not be able to carry the high flood discharge.

(4) Obstructions on the River Flow. In hilly areas, sometimes it may happened that debris or a landslide may form an obstruction in the river valley like a dam, and thus a reservoir may be formed on the upstream side. Due to heavy rainfall when the water pressure reaches a maximum value, then suddenly that obstruction may be removed and a high column of water may rush downstream with destroying effects.

(5) Inadequate Drainage Works. Drainage works such as culverts, aqueducts, bridges, etc passes water below them. The structures which are constructed for the smooth running of the flood water may be inadequate for high flood discharge. Thus the water level may rise on the upstream side and submerge the surrounding areas.

(6) Urbanization. Previously small town may get enlarged due to population influx and construction of various infrastructures. The town may be unplanned and the older parts of the enlarged city have residences near rivers or dry gullies. As the city gets modernized many asphalt streets, buildings, pavements, etc cover major parts of the area and this reduce the rainfall infiltration rate resulting in high run-off coefficient. Thus during heavy rain the downstream settlers will suffer from high floods causing damages.

1.3 Flood Disaster Management

Flood disaster management can be effected in two general ways. The first one is using structural measures such as dykes, groynes, retaining walls; etc which are described in the different parts of this manual; whereas the second method is the non-structural way such as flood forecasting and early warning for evacuation of people and properties.

i. Flood Forecasting

One way of flood disaster preparedness is to predict the danger (short-time or long-period) and prepare or warn the possible affected people or localities. The other alternative way of flood disaster preparedness, for repeatedly affected areas, is to construct permanent flood protection physical structures upstream of the danger area. This needs historical records of past flood events with their recurrence intervals.

Two main types of floods impact society and the environment: river floods and flash floods. Large-scale river flooding occurs over a period of weeks or months and can result in the inundation of hundreds or thousands of square kilometers. Flash floods occur over a period of hours as the result of extremely heavy rainfall and can affect very small areas. Flash floods also pose great risks to human life.

In a natural disaster a flood or other calamity minutes and even seconds of warning can create a difference between life and death. Because of this, concerned offices and government officials need to work to use the latest technological advances to predict when and where disasters will happen. It is also necessary how to study and how best to analyze and communicate this information once it is obtained. The goal is to put technology to effective use in saving lives and property when nature unleashes its power with devastating results.

Flood prediction involves consideration of many factors. These include the characteristics of a river basin (such as soil type, ability to hold water, and slope), how water behaves in the river, and meteorology. Another important factor in flood prediction is how people respond to warnings.

For flash floods it is important to observe rainfall rates and the flow of water in a river or stream. Increasingly, nowadays it is possible to track rainfall by using advanced radar systems that can measure where the waterfalls and with what intensity. Furthermore monitoring river and stream levels using low-tech gauges positioned at key locations along a channel are the conventional method. This information is put into computer models that calculate the future volume and height of a river at particular locations. As with many forecasts, there is an element of uncertainty in flood forecasts and the uncertainty is even higher so for flash floods since these occur over very short time periods. Emergency officials frequently warn people to seek high ground immediately if a flash flood warning is issued for their location. Details of flood forecasting methods and magnitude estimations are presented in Part-6 of the Flood Hydrological component.

ii. Flood Protection Works

Flood control works are physical structure constructed for protections of properties, human life, natural resources, etc. The physical structures could be gully afforestation, check dams, flood reservoirs, retaining walls, soil bunds, levees, dykes, diversion channels, etc. They are permanent structures which are supposed to implement on flood disaster prone areas (see Part-5: Flood Protection Structures component of this manual). The flood protection works are preferred when the non-structural options are less effective.

iii. Flood Rescue and Management

We are always living with disaster. Every year many lives are lost due to calamities which some of them can be saved by following disaster management principles. The most important activity after disaster occurrence is rescue. A properly managed rescue operation can result in saving of lives.

Therefore it is important to note that rescue operation involves rescue workers, rescue plan, safety aspects, causality management, etc. Disaster Management consists of four phases of cycle as shown in Fig. 1.1 below.

- **(a)** Mitigation: Mitigation efforts prevent hazards from developing into disasters or to reduce the effects of disasters when they occur. Example of mitigations, for flood disasters, are flood retaining works or river training works, creating awareness in public about flood disaster through media and schools so as they should know what to do during flooding.

- **(b)** Preparedness: In the preparedness phase, emergency managers develop plans of action for when disaster strikes. It includes the proper training programs of rescue workers, maintenance and checking of equipment and emergency services, evacuation plans and control supplies, etc. It also includes developing volunteers from common people to work during rescue operations

(c) Response: The response phase is the reflection of country's readiness to cope quickly and effectively during and after disaster. It includes the mobilization of necessary emergency services like police, volunteers, NGOs, etc. A well practiced plan developed in preparedness phase can result in effective rescue efforts. Rehearsals of rescue plans is essential to achieve optimum output in the response phase, medical facilities should be checked and enhanced from time to time. The response phase starts with rescue phase. For example: to facilitate emergency operations, the flood prone regions should be equipped with a number of emergency centers. The role of such centers should be maintained stock of food, building materials, etc for emergencies. They should also do the task of survey, assessment of damages and teaching of evacuation measures to common people.

(d) Recovery: The aim of recovery phase is to restore the affected area. Recovery phase is mainly concerned with rebuilding destroyed property, re-employment, repair of essential infrastructures and services. The recovery phase starts when the immediate threat to human life has reduced. In the long term disasters, which may last up to many months, the recovery phase starts early. The recovery phase is generally long and may take up to five years. The recovery can also be categorized as restoration, rehabilitation and reconstruction.

In disaster management, although it is beyond the scope of this manual, the important steps, procedures and essential elements need to be recognized. For instance the concepts of psychology of rescue, rescue workers, rescue plans, rescue equipment, safety in rescue operations, etc are some of the vital elements in disaster rescue.

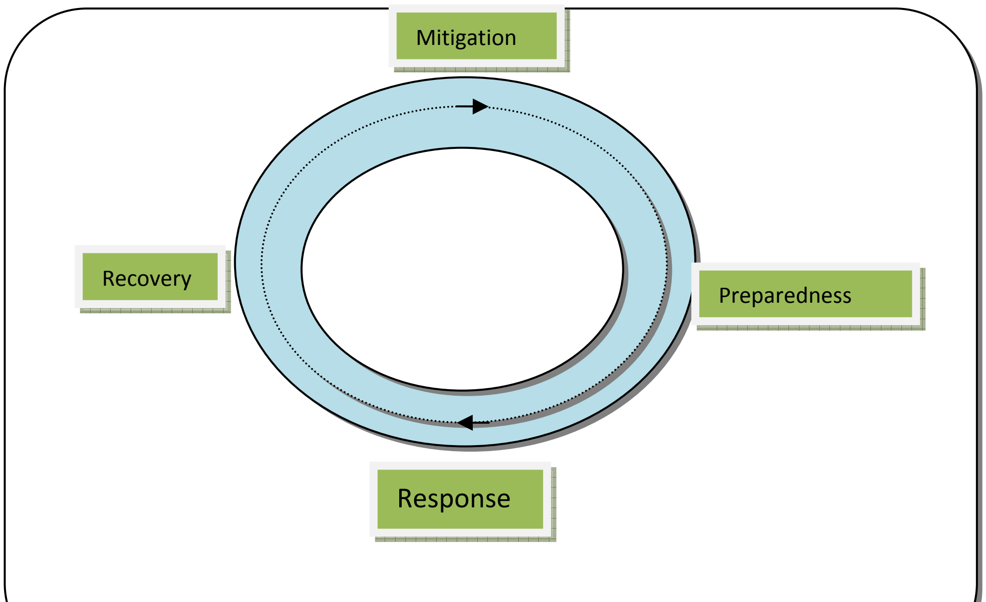


Fig. 1-1 A Graphic Representation of Four Phases of Disaster Management

2 PROJECT AND PROJECT MANAGEMENT PRINCIPLES

2.1 General

Project can be conducted by individual people or organizations. Individual peoples may have projects in their day to day life or for their wishes to be accomplished in a certain period of time. Similarly institutions and organizations may also have projects to be carried out to fulfill the purpose of their establishment.

Project can be any type from simple time based activity to high-tech and sophisticated proposals. For example, for simple case, building one's own residence, buy car, have plan to tour or plan for marriage. Other types which may involve different organizations, individuals, countries, resources, etc could be high tech or high concern of many. As an illustration, for instance; constructing roads to connect to regions or countries, planning new educational curriculum, computer software development, constructing flood protection works against X-area, etc.

Project can be successful, partially doing well or may fail. Additionally project may take time more than the planned period or finished according to the schedule or ahead of the plan. Some planned activities may consume large money or lose their original purpose. Therefore, projects are dynamic events which could not be planned straight forward on one shot but encounter them different phenomena through their accomplishment schedule.

2.2 Project Concepts

Project is a Latin word came from projectum means throw forward. Although project have been defined by different professionals in a slightly varying ways, however, for the sake of smooth flow of this manual and common understanding, the general concept of project is described as follows. Project is a planned activity having a beginning and ending timeframe, encompassing many interlinked action, with definite objectives and goals, and to be undertaken with limited resources (money and human) and in quality comportment.

Project sometimes misunderstood. Nor permanent work or routine activities without timeframe should be considered as project. Moreover, when a project ends its planned schedule and activities (rollout), i.e., when it starts functioning its planned goal, it ceases to be termed as project. However, after the end of a project if anything is required to be changed it shall be handled under change management.

2.3 Project Management Principles

Similar to the definition of the term 'project' mentioned above, management may also have been defined in various approaches but with the same general concept. Management in general is an activity process, composed of some basic functions, for getting the objectives of any entity or enterprise accomplished through and with the efforts of its personnel. Or in other terms management is a science of arranging various activities and group of people in an organization to achieve a common goal. Wherever and whenever objectives are to be achieved through organized and cooperative endeavor, management becomes essential for directing and unifying efforts towards a common purpose.

Projects improperly managed may result in delay of the completion time, excessive cost beyond the budget, creates problems on consecutive activities, deficit or excess on project staffs, etc. Project management generally includes the following:

- manage and control the plans;
- lead team;
- follow standards;
- organize and coordinate;
- resolve disputes and conflicts;
- administration;
- cooperate with client and donors;
- learn as you go process;
- represent the project, etc.

2.4 Managerial Skills and Levels

In a project or a company there are three distinct levels of management along with their respective functions (Fig.2-2).

Top level management: Top or first level management of a project or company is constituted by its board of directors and the executive. Major responsibilities of top level management are

- to make an outline of planning through the formations of basic objectives and policies of the project;
- to determine the basic pattern of the project's organizational structure;
- to arrange for effective coordination of all activities;
- to make staffing of departmental and other important executives;
- to prepare overall budget and programs for short-term and long-term operations;
- to exercise overall control in respect of all operations and activities;
- to ensure continuity of the project according to the plan;
- to maintain public relations with all outside parties for improving the project image and protecting its interest, etc.

Middle level management: Between top management and supervisor level, there is an allocation of another level of management known as middle level management. Middle level management in large projects is bifurcated into two parts: upper-middle or intermediate management and lower middle management. Middle management is constituted by division, departmental and sectional managers and its main functions include

- to develop derivative objectives and policies;
- to prepare departmental budget and programs in the context of overall planning;
- to execute plans through orders, instructions and advice;
- to exercise control in different areas through the application of quality standards and cost standards;
- to effect coordination between top management and supervisory management, etc

Supervisory management: It is the lowest level of management and it is constituted by superintendants, foremen and inspectors. This level of management has the following main functions

- to supervise the actual works and operations through guidance, checking and overseeing;
- to translate the plan into actions through the provisions of facilities and resources and the creation of favorable working environment;
- to exercise control over the work-in-progress through applying quantity standards and time standards;
- to send information and progress reports to higher management;
- to motivate personnel for improving productivity of the project;
- to put all managerial orders, instructions, policies and programs into action and to make higher managerial accomplishments.

When a manager becomes top manager of a project or an organization his/hers conceptual and design skill will be more essential than technical skills. It is generally concerns on the visions of the project and on every big decision like on finance, etc. If this top managers just after being promoted still continues to involve himself or herself more on day today shop level or technical problems, then his/her valuable time is exhausted in solving those problems. Here authority or responsibilities of down level managers are reduced/curtailed and in a course of time channel of command will be disturbed.

Middle mangers' prime focuses on execution of plan through orders and exercise control in different areas through applications of quality standards and cost standards. Top managers who try to do this job make the middle mangers non-functional. Supervisors' level needs more of technical skills and on day today activities and problems. Fig. 2-2 shows the pyramidal managerial skills and organizational hierarchy.

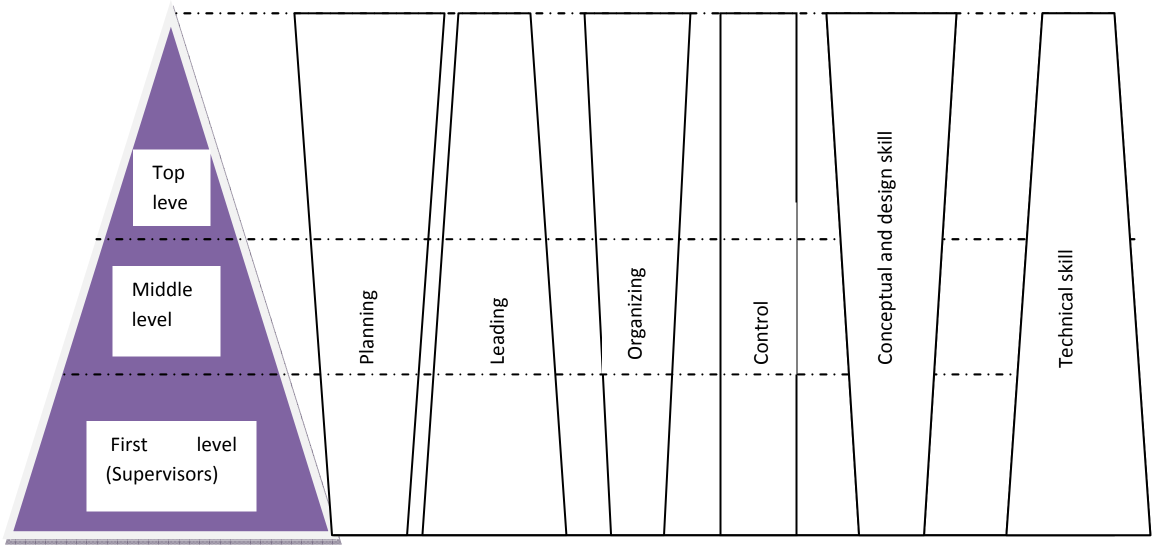


Fig. 2-2 Managerial Skills and Organizational Hierarchy

2.5 Project Management Angles

Classical project management concerns mostly were concentrating on important three points (angles); these are, managing time, cost and quality (see Fig. 2-3). Actually these are the three strategic body and principal indicators of project management.

However, nowadays project management knowledge areas are increased from three to nine angles. These are

- goal management;
- time management;
- cost management;
- quality management;
- risk management;
- human resource management;
- communication management;
- procurement management and
- Integration management.

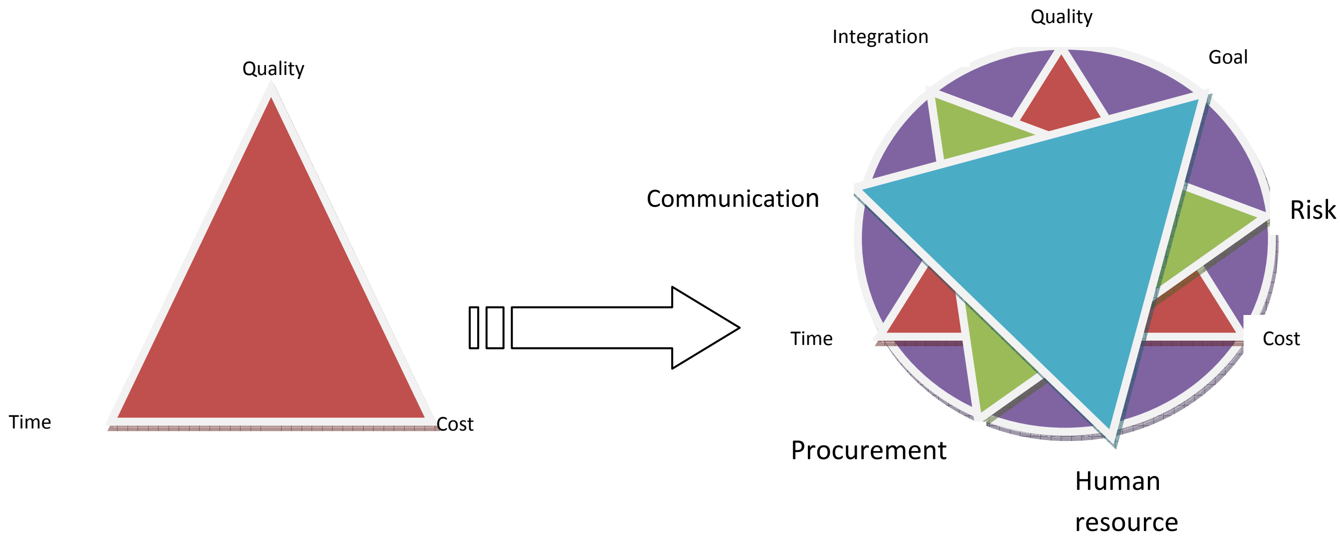


Fig. 2-3 Project Management from Triangle to Nonagon

3 FUNDAMENTALS OF PLANNING

3.1 General

Management started from early antiquity where human is in the crudest mode of life. Now it has gone long time to evolve as social science and current management theory classifies management roles into five managerial functions:

Planning, Organizing, Staffing, Leading and Controlling.

Therefore, planning is one of the managerial roles, and it is the rational and orderly thinking about ways and means for the realization of certain goals. It involves thought and decision pertaining to the future course of action. It anticipates and precedes action rather than a reflective thinking about the past events. Absence of planning before doing implies rashes, in-prudent or short-sightedness in the performance of work. Before undertaking any work, it is worthwhile to ascertain *what the work is, how, when and where the work to be done*.

Other functions of management, as aforementioned earlier, including planning, are a subject of discipline and specialization. Here it is introduced only for the sake of completeness and interested readers are advised to refer on management books.

3.2 Nature and Characteristics of Planning

There are several ways in which a particular thing can be done. Planning involves the choosing of the course of action from all available alternatives for accomplishing the desired results with the greatest economy and certainty. Planning sketches a complete mental picture of things yet to happen in the project or company through a process of looking ahead. The proposed course of action is charted out in greater details with the help of a complex chain of plans like policies, procedures, programs and budget focused on the activities of the project or enterprise.

Planning has a number of characteristics:

i. Primacy

Planning is an important function that usually precedes other functions. Obviously, without setting the goals to be reached and the lines of actions to be followed, there is nothing to organize, to direct, or control in the project. But this should not lead us to think that planning is isolated from other managerial functions or have no impact on it.

ii. Continuity

Planning is a continuous and never-ending activity of a manger to keep the project as a going concern. One plan begets another plan to be followed by a series of other plans in quick successions. Actually hierarchy of plans operates in a project at any time. Planning gets used up where tomorrow becomes today and calls for further planning day in a day out. Again, the incessant changes make planning a continuous necessity.

iii. Flexibility

Planning leads to the adoption of at specific course of action and the rejection of other possibilities. This confinement to one course takes away flexibility. But if further assumptions upon which planning are based prove wrong, the course of action is to be adapted to the altered situations for avoiding any deadlock. Accordingly when the future cannot be molded to conform to the course of action, the flexibility is to be ingrained in planning by way of adapting the course of action to demands of current situations.

iv. Unity

Planning is made by different managers at different time. Maintenance of consistency or the unity of planning is one of its essential requirements. Objectives provide the common focus for unifying managerial action in planning. Moreover, policies and procedures introduce a consistency of executive behavior and actions in matters planning.

v. Precision

Planning must be precise as to its meaning, scope and nature. As guides to action, planning is to be framed in intelligible and meaningful terms by way of pinpointing the expected results. According to the capacity and facilities of the project, planning must be realistic in scope rather than dreams indicating pious desires. As planning errors are far more serious than mistakes in other functions of management and cannot be offset by effective organizing or controlling, i.e., the planning precision is of utmost importance.

vi. Pervasiveness

Planning shall be pervasive covering the entire project with all its segments and every level of management. Planning is not the exclusive responsibility of top management only, but it extends to middle and lower management as well. Although top managers are mostly preoccupied with planning works because of their wide jurisdiction of operation and decision making, planning is of equal necessity to every manager. Further, there is nothing to choose between a large project and a small project so far as the planning process is concerned.

3.3 Benefits and Limitations of Planning

Planning has many benefits and some limitations. For instance the merits of planning are:

- planning leads to more effective and faster achievements in any organizations and projects;
- planning gives a competitive edge to companies and projects that forecast and plan;
- planning secures unity of purpose, direction and effort by focusing attention on objectives;
- planning minimizes costs of performance;
- planning has unique contributions towards efficacy of other managerial functions and

- Planning provides the basis for control in a project or organization.

Some of the limitations of planning are:

- if reliable information and dependable data are not used planning is sure to lose much of its values;
- by limiting individual freedom, planning may stifle individual initiatives and personal development, for instance, rigidity may appear from managers' failure to revise the plan, particularly policies and procedures and
- planning may take time and expensive process.

3.4 Elements of Planning

Planning consists of several individual plans or component parts that are bound together in a consistent structure of operations. Any complete and thorough planning must have the following elements:

i. Objectives and Strategies

Objectives are basic plans which determine goals or end results of the projected actions of an enterprise or a project. By setting goals, objectives provide the foundation upon which the structure of plans can be built. To guide and unify efforts towards the desired end, objectives stand steadfastly over all operation in a company or a project. An effective management nowadays implies that all planning work must spell out the objectives to be realized from proposed action. Objectives are elaborated by strategies that become the action commitments through which the mission of the project is fulfilled. Strategies determine the basic long-term goals of a project and help to adopt a course of action along with the allocation of resources required for carrying out such goals.

ii. Policies

Policies and further guides to thinking and action of subordinates in a project, even though they must lie within the boundaries fixed by objectives. Policies are written statements or oral understandings in some general terms for governing actions in repetitive situations. Realization of objectives is made easy with the help of policies, as policies provide standing solutions or answers to similar problems. As general guides to thinking and action, policies are however open to interpretation by different managers, particularly at the upper level, thereby permitting managers to exercise imitative, judgment and discretion in all cases. Policies are plans in the sense that they also aid in mapping out a course of action to be followed for the achievement of end results. Briefly, policies are supposed to pre-decide issues, to avoid recurring analysis and to give a unified structure to other types of plans.

iii. Procedures

Procedures indicate the exact manner in which a certain activity is to be performed. They prescribe definite tasks with their chronological sequences to be assumed by a number of employees within the limits chalked out by the policy. Procedures outline a fixed path through the defined area of policy. In contrast to policies which provide general guides to thinking and action procedures are more definite and specific guides to action only for the fulfillment of objectives. Procedures may be durable like policies, but they are not as flexible as policies are.

iv. Programs

Programs weld together different plans for implementing them into a complete and orderly course of action. All individual plans in the form of policies and procedures are assembled in such a fashion as to put them into a workable form for achieving predetermined goals or objectives of a project. Piecemeal plans are transformed into a master plan at this stage of program planning. Programs are necessary for both repetitive and non-repetitive courses of action. Programs for repetitive action are referred to as *routine planning*, while programs for non-repetitive action go by the name of *creative planning.*

v. Budgets

Budgets are plans containing statements of expected results in numerical terms, i.e., Birr, man-hours, product-units and so forth. A cash budget projects the expected flow of cash for a period in advance, a revenue and expense budget shows anticipated revenues and expenses, or a capital expenditure budget reflects the estimated capital outlay over a period. All these budgets prove beyond doubt that budgeting is planning. But budgets are primarily used as control devices and they fail to serve this purpose unless budgets are developed into plans for utilizing them as definite standards of work performance.

To summarized, budgets and programs are closely interrelated. While many programs are implemented by means of some budgets, the budgets in themselves are very often utilized as the entire planning program in many business enterprises. As a matter of fact, budgets are adopted as the main instrument of planning in many companies through integrating different departmental budgets into one master budget for business as a whole.

3.5 Classification of Plans

Planning provides a future course of action. Such a course of action may involve either repetitive or non-repetitive operation as well as short-range or long-range operation. According to the nature and use of planning, different plans can be divided into three groups-basic plans, standing plans and master plans.

i. Basic plans

Objectives and strategies are basic plans which are necessary for all types of planning and operations. The entire planning activity is geared into action through the formulation of objectives and strategies. Objectives and strategies have predominant importance not only in planning, but they have a unique role in other managerial work of organizing, directing and controlling also.

ii. Standing plans

Standing plans which include policies, procedures and methods have application only in repetitive action, whether it involves long-range or short-range operations. To ensure a consistency of executive behavior and managerial action, standing plans provides a ready guide to action for solving recurring problems. Standing plans have no use unless similar problems appear in the organization again and again. For non-repetitive operations, special problems are faced in each case and they are to solve in a different way. However, standing plans limit the individual freedom and restrict managerial authority for ensuring integrated and co-operative actions. In other words, they constitute constraints for managerial operations.

iii. Master plans

Programs are master plans which indicate the complete course of action along with timing and strategy considerations. All plans are meshed together in an orderly way under programming for mapping out the course of action. Programs are applicable to both repetitive and non-repetitive operations. As non-repetitive operation does not require standing plans, some special considerations arise in programming for such operations. Like the program, budgets as a kind of plans perform the same functions and have the same characteristics and application.

Plans can, however, be classified in various other ways. In terms of jurisdictions, plans may be company-wide (overall) or subsidiary (derivative) in nature. In terms of scope, plans may be either detailed or broad in character. In terms of enterprise functions, plans may relate to production, sales or other activities,

4 PROJECT CYCLE AND THE PLANNING PHASE

4.1 Major Activities in Flood Protection Planning Phases

In flood protection or other development projects, there are three main project development consecutive phases. These are project planning, project implementation and project operation & maintenance. The planning phase is also further divided into three main investigation stages, such as reconnaissance, feasibility and detail design stages (see Figs. 4-1 & 4-2).

Understanding new development and redevelopment in the context of the project life cycle is an important concept for proper selection and implementation a project. The concept, planning and design phases of a project may be spread over a period of months to many years. The same is true for implementation and operation phases. The last two phases (implementation/construction and operation & maintenance) are dealt separately in Part-9 & Part-10, respectively whereas the planning phase is discussed in this chapter.

The extent of the studies required for the varies components of a flood protection project of a given dimension varies greatly from site to site depending upon the problem at hand and degree of disaster and also may not be adequately visualized in advance. The studies generally proceed in stages, the details of each stage growing out of the one before. It normally follows a learn-as-you-go procedure in which characteristics of the flood and protections required are developed in progressively greater detail as the investigation proceeds. The three above-mentioned main investigation stages are, therefore, interlinked as presented below.

4.1.1 Reconnaissance Study Stage

Objectives:

Reconnaissance studies are normally the first step of project investigations. They are preliminary in nature and the main objectives for flood protection works may be listed as follows:

- To identify and quantify the type and magnitude of flood anticipated to the area in question;
- To discover the social, economic and environmental impact magnitude of the flood on the area;
- To identify suitable sites for flood protection works and their appurtenant structures based on regional and local geology, topographic expression and anticipated depth to sound foundation and any other safe diversion location;
- To investigate apparent alternative solutions for inclusion in the plan or rejection;
- To investigate and study the various sites and project alternatives to the confidence level required;
- To compare the candidate sites and project alternatives and formulate the project best suited for the stated purpose;
- To record lower ranked sites and project alternatives for future reference;
- To provide preliminary cost figures and implementation schedules for the selected site and project.

Activities:

The main reconnaissance activities involved at this stage may be summarized as follows, according to their order of accomplishment:

- **Data and information collection**. These include literature and map surveys such as geological and topographic maps to indicate general geology, assessment of watershed conditions, hydrologic conditions and possible pollutants to the concerned area (sediments, coliform bacteria, solid/liquid wastes, etc). Aerial photos to study drainage patterns, vegetation covers, landslides, etc. Information like infrastructures and properties that are affected by the flood, previous use of the proposed site(s) locations, existing aggradations, drainage conditions after mitigation, etc are necessary data at this stage.

- **Field visit or inspection**. Walk over surveys at the flood prone or disaster area, draining catchments, diversion locations, development areas, flood prone areas, downstream drainage areas, specific locations of the flood protection site alternatives, availability of sound foundation, availability of construction materials, etc shall be made.

  Foundation conditions are often revealed by or can be inferred from visual erosion features, rock-outcrops, and manmade excavation such as road cuts, building excavations, soil pits and rock quarries. Additional examination on the availability and suitability of construction materials on the vicinity of the sites such as for embankment core or lining, shell, riprap, rock fill, aggregates, filter and transition materials. The field visit of a reconnaissance study is carried out, depending upon the magnitude of the work, by respective professionals such as engineers, geologist, socio-economist, environmentalist, etc.

- **Office analysis.** Small amounts of office analyses on the information gathered and field inspected are required to eliminate all but a few of the project alternatives. Preliminary design which is illustrated by necessary drawings, showing the general layout, main sections and salient features, elements and structures shall be made. As part of the office analyses, a preliminary cost estimate is prepared. The objective of the estimate is not only the construction cost but also to make a survey of the total cost involved in developing and implementing the project under study such as filed investigation, engineering, environmental mitigation costs, management and other owner’s overhead costs. Attention must be given to access, transport, construction methods and materials, site arrangements and facilities, communications, etc. At this level of investigation only the main project components are known. Allowance for physical contingencies must therefore be made. The cost parameter of the project is a multidisciplinary where each disciplines provide cost input for further investigations and implementations.

- **Reporting.** The report often starts with executive summary or synopsis. The stated purpose of the project is referred and documented. All main project data and information are also referred and documentation provided on the elements and parameters used. Chosen layout and the main project features and elements shall be described and illustrated by drawings including discarded solutions and alternatives. Cost parameters shall also be included. The report shall conclude with firm statement on the technical, economic and with environmental feasibility of the project.

4.1.2 Feasibility Study Stage

Objectives:

The second organized step in flood protection project planning and investigation is called the feasibility study. In this stage, one or more identified projects are brought one step further in the planning process (Fig.4-2). The general purposed of this investigation from all discipline aspects, are:

- Further reviews of the reconnaissance study and screen by discarding un-potential sites or project alternatives as the feasibility study goes;
- Estimation of cost of development and construction;
- Estimation of net economic values to be produced;
- Estimation of cost of operation, maintenance and replacement;
- Assessment of the impact of implantation of the project that will have on the environs and the cost of the effects;
- To compare benefit to mitigating cost ratios for seeking decision on appropriation of funds or negotiation of loans;
- To prepare database for the next stage of investigations.

Activities:

The activities that are needed here are multidisciplinary (as mentioned in the reconnaissance study) and highlighted below:

- *Collection and evaluation of existing data*. On aerial photos, hydrological records, geological records, sediment and erosion records, seismic records, environmental aspects, socio-economic aspects, etc.
- *Field investigation*. Depending on the size of the project, the following geotechnical investigation may be carried out (a) exploration by test pits, trenches and visual inspection; (b) exploration by geophysical methods; (c) exploration by hand and power driven augers, and investigation by coring and nor-coring drilling methods; (d) field tests; (e) laboratory tests, etc.

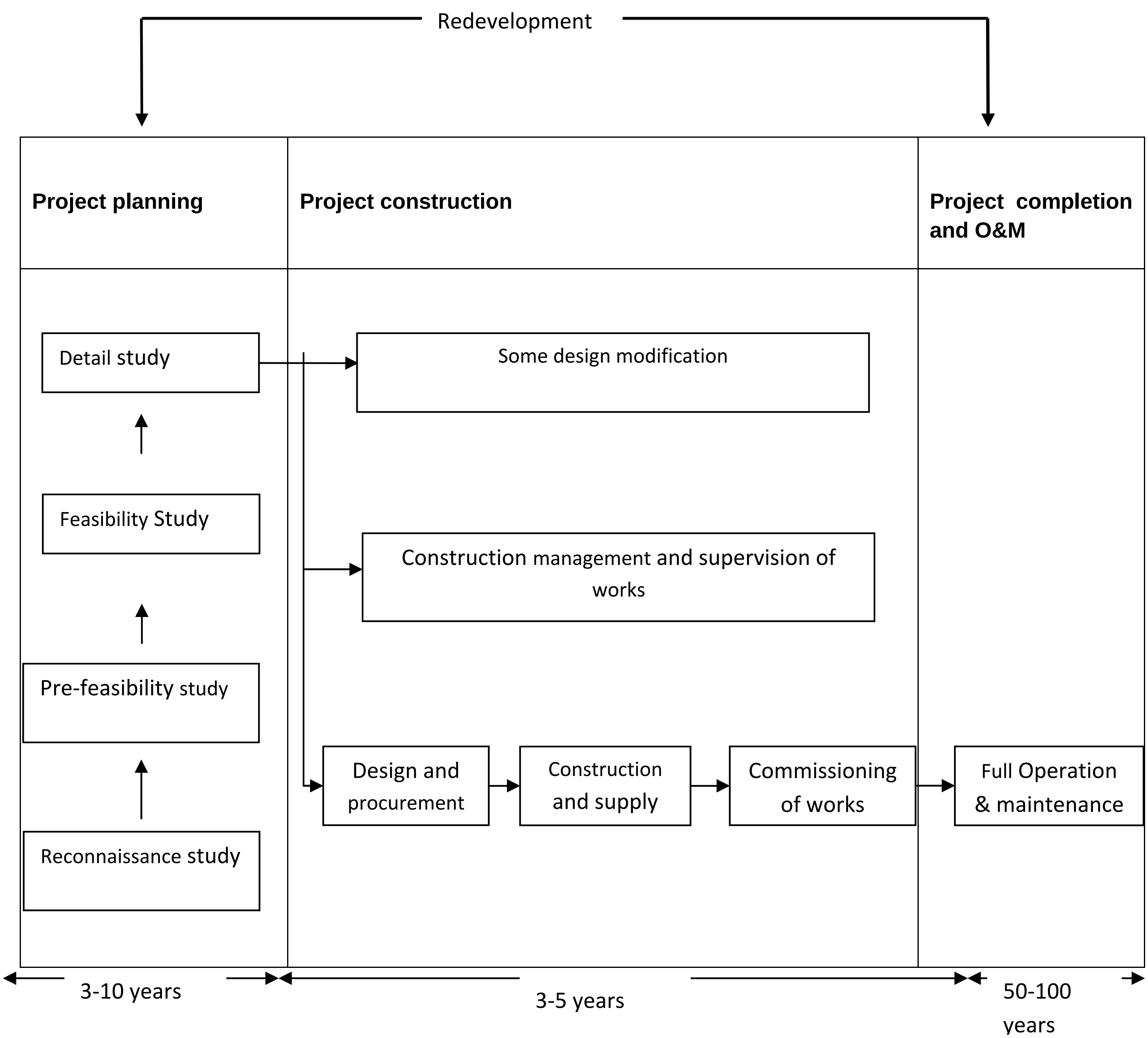


Fig. 4-1 Flood Project Development Cycle

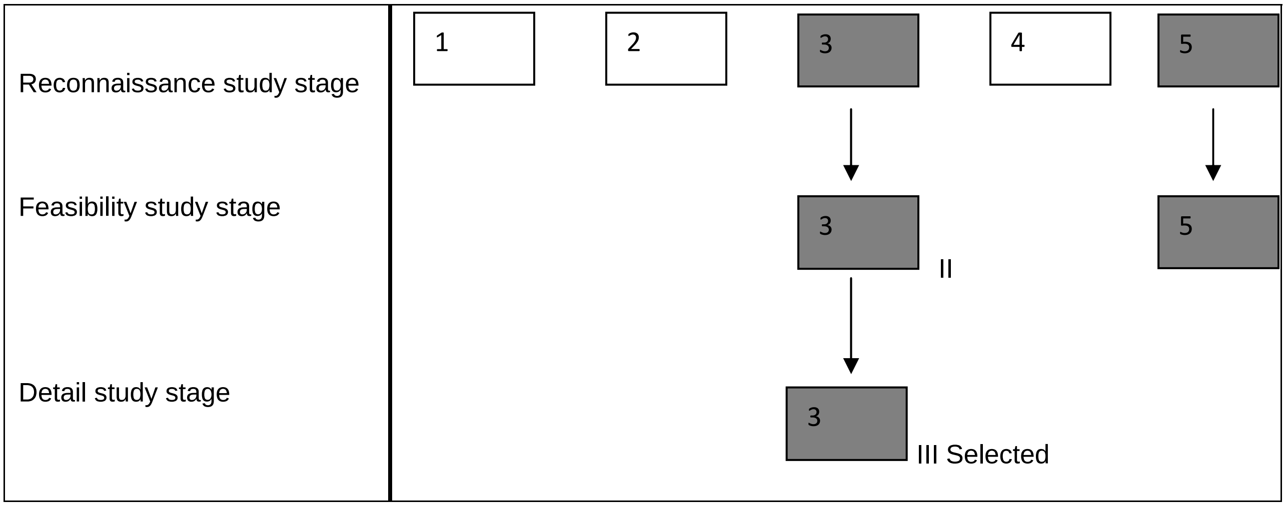


Fig. 4-2 Flood Project Development Selection Options

- *Office work*. Analyses, designs and cost estimates will be made in detail design phase than the reconnaissance stage. Relatively detailed layouts and designs on flood protection works and ancillary structures, and assessment of construction materials site will be provided.

DESIGN PROCESS

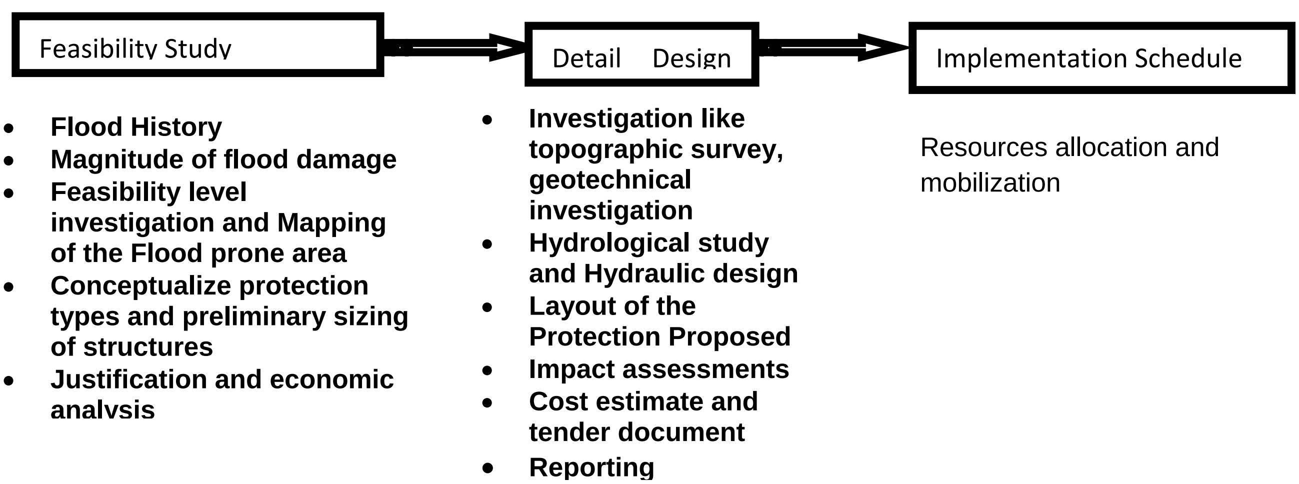


Fig. 4-3 Design Process in Flood Protection Works

- *Reporting.* A comprehensive report shall be prepared covering all aspects mentioned above. Statements shall be made in respect to technical, economic and environmental feasibility of the projects. Recommendations on further investigations and detailing will be made for the next stage of investigation. The owner also decides here whether to continue or terminate with the selected project.

4.1.3 Detail Study Stage

Objectives:

The detail investigations or the specification stage is the final one with only one project alternative and geared for construction or implementation phase. All the data required are for detail designs and preparation of construction drawings.

The main aims may be summarized as follows:

- For revision and finalization of feasibility studies and recommendations;
- To conduct additional investigations on field and laboratory for missed data, final confirming information and definite plan preparations;
- To prepare definite plans, find designs and tender documents;
- To enable the owner to engage with competent Consultant, Contractor and Suppliers;
- To enable to estimate relatively accurate cost of the project;
- To prepare construction schedule, etc.

Activities:

The major activities are:

- Review of the feasibility study;
- Detailed field investigation and inclusion of additional data by supplementary pitting, drilling, field and laboratory tests, etc;
- Finalizing layouts, detail drawings, specifications, bill of quantities, cost estimates, etc.;
- Preparation of the final contract documents;
- Reporting which includes the above mentioned objectives and activities in detail.

4.2 Tender and Contract Documents Preparation and Award of Contract

In the planning phase, after getting approval of the final design works, the subsequent work of the planner is to go ahead with the preparation of well-organized tender document for competent bidder selection and preparation of a standard contract document so that the winning contractor will sign immediately after the award of the contract.

4.2.1 Tender Documents

Tender proposal or bid proposal is a written offer submitted by the contractor in pursuant to the notifications given to execute certain work or supply of some specified articles or transport of materials at certain rates with the terms and conditions laid down in the tender documents. The form in which it is to be submitted is supplied by the owner/employer to eligible contractors on usually payment bases. The tender duly filled in place in the tender boxes and signed will be returned to the service procuring entity. Tenderer or bidder is the competitive service giving organization, i.e., contractor.

Tender form is a printed standard form of contract giving standard conditions of contract, general rules and directions for guidance of the contractor. There is also a memorandum for giving (i) general description of work, (ii) time allowed for construction period and (iii) other several forms attached in the tender documents such as bid guarantee, agreement, performance bond, insurance, etc.

Tender documents The various terms and conditions of the contract which are to be formulated while inviting tender for flood protection works are:

- invitations for bid;
- instructions for bidders;
- bidding data sheet;
- evaluation criteria;
- eligible countries;
- general conditions of contract;
- special conditions of contract;
- specifications;
- forms of bids (qualification information, letter of acceptance, security forms, agreement, etc);
- Bill of Quantities;
- drawings;
- guidance on environmental protections; etc.

All the above documents are signed by contractors page by page, necessary entries are made and a forwarding letter on letter head of a contractor with bank draft or other form of earnest money are put in closed cover. The cover is then sealed and dropped in the tender box within the time limit for the tender. The name of the work and the name of the contractor are super scribed on the cover.

Tender notice is an inviting tender paper or notice which is advertised through media or particularly addressed to bidder(s) looking for competitive contractors. The tender notice has its own format and includes very important data where the tederers base their offer and also used as one of the evaluation criteria.

Tender notice sent to press to publicity organizations has to stay sometimes to be reached by all competitors. Depending upon the urgency and size of the work it shall float from 10-day to three weeks depending upon the tender law of the country. Global (international) tenders may float longer taking into account the time required to response the notice. A tender notice might reach to bidders as closed tender (to specific specialized bidders or due to justifiable urgency of the works) or open tender (to all qualified and legal bidders) according to the service procurement law of the country.

4.2.2 Contract Document

Contract is an agreement which is enforceable by law. Contract is invariably follows a proposal from one party and its acceptance by another. In absence of any of the above elements of a contract it becomes void, i.e., without a legal effect and mutual agreement or benefit leads to rejection. Generally contract is a written undertaking for execution of works or supply of materials or for the performance of any service connected therewith duly accepted and registered by the competent authority on the behalf of government or organization or individuals.

i. Essentials of Contracts

Essentials of contracts are the following particulars by which all agreements must be made in order to constitute a valid contract:

- the contract shall be made by parties competent to contract;
- the contract shall be made by free consent of the parties;
- there shall be a definite purpose and its acceptance;
- there must be mutual benefits;
- the contract shall be made so that the considerations and objects are lawful and
- the meaning shall be certain.

A person is competent to contract provided (a) s/he is of age majority according to the law of the country. A person who is a minor or doddering age cannot sign an agreement. Further no contract shall be made by a subordinate authority who has not been directed or authorized to do so; (b) s/he is of sound mind: a person is said to be sound mind for the purpose of making contract provided s/he is capable of understanding it and of forming a rational judgment as to its effect upon his interest at the time when s/he performs the contract, and (c) s/he is not disqualified from contracting by any law to which s/he is subject.

Consent is said free when (a) it is not caused under influence, (b) it is not caused by committing or threatening to commit any act of forbidden by the national code, (c) it is not caused by fraud, (d) it is not caused by misrepresentation, and (e) it is not caused by mistake.

A definite proposal and acceptance means that the terms of the contract must be precise and definite and there must be no room for ambiguity or misconstruction therein.

A consideration or object of an agreement is said to be unlawful if forbidden by law or fraudulent or of such nature that, if permitted it will be defeat the provisions of any law or involves or implies injury to the person property of another or opposed to public policy or regarded as immoral by the court.

Furthermore, for a contract to be valid the meaning of the agreement of which shall be made must be certain or capable of being made certain.

ii. Types of Contracts

The common construction contracts are as follows:

- unit price or item rate contract;
- percentage rate contract;
- lump-sum contract;
- labor contract;
- material supply contract;
- piece work agreement;
- cost plus percentage contract;
- cost plus fixed fee contract;
- cost plus sliding or fluctuating fee-scale contract;
- target contract; and turnkey (combined engineering & construction) contract, etc

The aforementioned contracts have their peculiar applications and suitability to the specific job type. To select the type of contract to the situation at hand one has to clearly understand its application, advantages and disadvantages of the contract and conducting proper write up of the contract which will not lead to disputes and litigations.

iii. Contract documents

The following documents usually considered to be part of a contract agreement and included or referred to be binding in the law. According to the order of importance (in case of discrepancies) the main documents are:

- The signed agreement form;
- the letter of acceptance;
- the agreed Bill of Quantities;
- the final drawings;
- the Addendum to the Contract;
- the Special Conditions of Contract;
- the General Specifications and Method of Measurement;
- the General Conditions of Contract;
- the Schedule of the Work, Equipment & Labor,
- the Tender Proposal of the Contractor and

- any other appendices.

4.2.3 Evaluation and Award of Contract

Based on the tender notice and tender document bid submission regulations, the sealed pre-qualifications of the contractors (if any) and the financial statements are opened in the presence of the contractors or their legal representatives, the employer, the consultant and any other members of the tender committee. The officer opening the tender has to show all the members the sealed bid box, tear the sealing paper, and count the number of the submittals and any documents inserted in the box.

To avoid biasness, the pre-qualifications and financial statements have to be submitted in a separate envelope. First the pre-qualifications have to be opened reading aloud what is submitted and written in the envelope. In the subsequent days the consultant and client shall give grades or assign to each bidder either by weight or responsiveness degrees stipulated in the evaluation criteria. Before the opening of the financial offers those bidders which are not satisfying the stated competition requirements shall be announced to the bid opening members. The inadequacies of the unsuccessful bidders in terms of their company experiences, manpower, machinery, etc stated in the instruction to bidders shall be told to the tenders and members. Those bidders which are responsive according to the distributed evaluation criteria, their financial offers will be similarly opened in the presence of the parties and tender committee.

Following the opening arithmetic checks shall be made and financial grades need to be given where the total of both pre-qualification and finance will determine the winning contractor. Prior to the expiration of the proposals validity, the Client shall notify the successful contractor that has obtained the highest score in writing by registered letter or fax message and invite the contractor for negotiation, if any.

Negotiation may include a discussion of the submitted pre-qualification, the schedules, fiancé, manpower, machineries, etc. After finalizing the negotiation and corrected arithmetic errors and further revised schedules, staffs, equipment, etc a letter of acceptance shall be written with the employer. Following the letter of acceptance the contractor and the client shall sign immediately the contract for the execution of the work based on the negotiated conclusions and revised bid proposals.

FLOOD PROTECTION DESIGN MANUAL
PART 3: SOCIAL AND ENVIRONMENTAL CONSIDERATION

Table of Contents

CONTENTS OF OTHER PARTS

1. INTRODUCTION

1.1 General

The objective of the manual is to bring a measure of consistency to the design of flood protection structures in Ethiopia to ensure that minimum standards of the design are achieved.

This part of the manual will enable planners, engineers and operators to better understand what social and environmental consideration should be addressed during planning, design and operation of flood protection works.

Chapter 2 discusses the social considerations which mainly focus on issues related to land/property ownership, displacement and relocation, dispossession of property, compensation. It also deals with public participation on flood embankment construction; which will have strong implication on cost recovery, operation and maintenance and ultimately on sustainability of flood protection structures.

Chapter 3 discusses the environmental considerations which mainly focus about preventing unwanted repercussion on physical, biological natural resources. Besides the guideline will consider all mitigation measures to minimize and contain any environmentally adverse effects on floodplain ecosystem, community goods and properties at wetland areas, water sources, social service infrastructures and other community properties.

1.2 Rationale of Social Consideration

This manual is prepared following the recognition that setting parameters for physical measures does not necessarily guarantee the flood control in a sustainable way. It is therefore quite mandatory to include non structural measures as a major complementary to the structural measures. The parameters for setting parameters/guideline may deal with the construction of such flood controlling physical measures as dykes, levee, etc. While the non structural flood controlling measures may include interventions starting from awareness creation on the issue up to improving land and water management practices through enacting various institutional bylaws at the very grassroots level. It is in recognition of the role of these non structural/physical elements that the present manual tries to include the social and environmental issues as part of the binding guideline in designing the flood control measures in the targeted area. With regard to the social issues, the main rationales for their inclusion in the manual include the following:

The need for having adequate awareness on the flood problem and sensitize the public on the issue. This is a steeping stone before embarking on any flood controlling physical measures. Apart from their traditional knowledge on the issue, the public need to have adequate and well organized information on the flood: its causes, pattern, demerit, merit (some times the local people do recognize the merit, such as flood retreat cultivation) etc.

Promoting stakeholder participation (local frontline communities and other relevant actors/bodies) is very much required. Thus, the flood mitigation measures should be designed and implemented in a participatory way. The manual, therefore, should include provisions on how to ensure the public participation together with the timing and level of participation.

The social/socio economic elements should be addressed due to the need for maximizing net benefits from floods and flood protection measures to all the flood prone households. There is a need to exhaust all information about the pluses and minuses of the flood risk and the proposed flood controlling measures. As mentioned above, in some cases there is a strong recognition for the merit of flood in terms of providing moisture to the soil during the dry spell period and thereby increasing cropping intensity. This recognition has a high value in terms of increasing food availability. In proposing a flood mitigation measures, therefore, ensuring net socio economic benefit is essential through the consideration of with and without project situation.

In minimizing loss of life and property due to flood, the need for putting in place early warning system, emergency planning, and post-catastrophe aid should also major element in the manual. Even though flood mitigation measures can reduce the potential hazardous risk, there may be instances where lives and properties could be damaged due to unconfined flood level by the installed structures. Thus, this state of affair should always be checked and preparation of early warning system together with the contingency plan is required.

1.3 Rationale of Environmental Consideration

Most poverty alleviation measures for development and improvement of livelihoods and human security like; industrialization, agriculture, flood control and protection works, etc., are drivers of environmental and ecosystem degradation. Accordingly, flood management policies and practices have to be viewed within the overall matrix of drivers of environmental enhancement on the one hand, while on the other hand they are also drivers of environmental degradation. It is, therefore, extremely important to balance development imperatives: flood risks control, their relation to social and economic vulnerability and sustainable development vis-àvis the preservation of ecosystems.

Ecosystems provide enormous benefits to humans. Human security and well-being are closely related to maintaining ecosystems and avoiding environmental degradation. Human inventiveness is presented with a dilemma between development imperatives and environmental degradation. Issues of sustainable development, ecosystem services in the development process, human security and the related development imperatives are discussed, from an embankment dyke flood control perspective, in this chapter.

The guidelines proposed under this section presents brief description of backgrounds of impacts related to flooding and flood control measures; and outlines environmental mitigation measures proposed for possible impacts due to embankment dyke project implementation.

The Constitution of Federal Democratic Republic of Ethiopia (FDRE) and the national economic policy and strategies do recognize environmental sustainability as a key prerequisite for lasting successes.

In the past, water sector developments in Ethiopia had suffered from lack of coherent policy directions and strategies to guide development initiatives. Several large scale dams and irrigation schemes have failed to meet their objectives and goals, due to inadequate planning, technical deficiencies and negligence to environmental conservation considerations.

These deficiencies have been well recognized since then, and the principle of Comprehensive Integrated Water Resource Management (CIWRM) framework was adopted and has been promoted. This has led to the preparation and endorsement of the Water Resource Policy and strategies of Ethiopia in the early years of 2000.

The water resource policy of Ethiopia stems on the following guiding principles;

- ✓ Social equity;

- ✓ Economic efficiency;
- ✓ Systems and technical reliability and
- ✓ Environmental sustainability.

Hence, attempts made in water sector developments should endeavour to follow and address those guiding principles.

The Environmental Policy of Ethiopia (EPE), on the other hand requires water sector development projects and management practices to undertake Environmental Impacts Assessment (EIA) process, and to include the costs and benefits of protecting watershed forests, wetlands and other key ecosystems in the economic analyses of such projects.

Ecosystems provide enormous benefits to humans. Human security and well-being are closely related to maintaining ecosystems and avoiding environmental degradation. Human inventiveness is presented with a dilemma between development imperatives and environmental degradation. Issues of sustainable development, ecosystem services in the development process, human security and the related development imperatives are discussed, from an embankment dyke flood control perspective, in this chapter.

The guidelines proposed under this section present brief description of backgrounds of impacts related to flooding and also impacts due to implementation of flood control measures; and outlines environmental mitigation measures proposed for possible impacts due to embankment dyke project implementation.

2. SOCIO-ECONOMIC CONSIDERATION

2.1 Design Stage

2.1.1 Early Consultation

Early consultation is required at the design stage to enshrine public participation at the beginning and ensure the appropriateness of the proposed flood mitigation measures. Activities include: discussion on the need for the mitigation measures with the Woreda and Kebele level officials that include relevant Woreda administration, representative from relevant (agriculture and natural resource)Woreda sector offices , Development Workers at the Kebele level, and Kebele administration.

Discussion with representatives of the targeted community is the second consultation. Here the discussion gives due consideration to the opinion of the public. At this point, common understanding and agreement on the need for the flood mitigation measures should be reached between the technocrats and the local people. Discussion issues include: the appropriateness of the potential site for the flood mitigation measure, property ownership on the site, type of technologies to be installed, merit and demerit of the mitigation measures, etc.

2.1.2 Exhausting Structural and Non Structural Measures

Before fully endorsing the structural flood mitigation measures (as it happens in many cases), other non structural measures should also be explored with the local people. The non structural measures may include traditional land cultivating and conservation practices that have a bearing in the flood problem. At this point, all of these traditional practices should be explored and communities need to be given the utmost opportunity to rank and prioritize causes and effects of the flood problem. They should forward which traditional practice exacerbate flood, which ones minimize; which ones to be promoted and which ones to be abandoned. This discussion should be guided with structured checklists and recording format so that the final design would take into account the concerns of the local people and ultimately helps in forwarding the appropriate flood mitigation structure.

The traditional skills that need to be exhausted and built upon in the consultation process include flood event alarm notification, emergency evacuation, rehabilitation and reinstating of affected livelihood and properties, etc. Besides, how communities organize themselves for such events should also be explored. Issue of partnership with other actors in addressing flood problems must be discussed. All these discussions should be guided with structured checklists and recording format so that the final design would take into account the concerns of the local people and ultimately helps in forwarding the appropriate flood mitigation structure.

2.1.3 Consideration of Property Dispossession

The construction of flood mitigation structure does in many cases entail the dispossession of communal or private properties. If this is unavoidable, what is required is to minimize the impact on properties to the minimum possible level. Consultation with local people (as indicated above) does help in minimizing the impact on properties through selecting proper sites for the structures. At this point, participatory land and property inventory should be done. Cadastral survey is required to identify properties together with the owners, size and type.

Assessing the potential impact of the flood mitigation measure on the existing social facilities is important to know the magnitude and resource requirement to relocate/reinstate the services. Inventory of the existing services (including the water points, access road, and animal movement corridor) should be done. Besides, assessing the potential impact on any sites of social and historical significance (religious sites, cemeteries, etc) deserves special considerations. In such cases other option should be explored further.

The inventory and impact assessment exercise should be culminated with the preparation of an action plan that shows the extent of displacement, relocation of services and resource requirement to compensate the would-be affected people and reinstate their social amenities.

2.1.4 Choosing Optimal Structural and Non structural Measures

Once exploration of the impacts of the various measures is completed, those flood mitigation measures with optimum result that reconcile the local need and technical parameters should be selected in a participatory way. Perhaps this may not be a one time decision; rather it could be an iterative exercise that requires continuous deliberations before reaching the final stage. At this stage agreement could be reached on the site, rout alignment for the flood mitigation structure, kind of technology to be used, etc

2.1.5 Assessment of Local Capability

Assessing the local capability in maintaining and sustaining the proposed mitigation measures is very important at the designing stage. The capability of the frontline community and local partners (mainly Woreda level relevant government office) is the main factor in ensuring the sustainable use of the flood mitigation intervention. The local capacity has a bearing on the kind of flood mitigation technologies to be proposed. As much as possible technologies that can be operated and maintained within the local means and capacity should be envisaged and designed to contain the flood problem. A technology that can be scaled up and be nurtured by the local input and raw material (like earth, etc) should be opted.

2.2 Construction

Prior to engagement in full construction, the early stage of this phase started with clearing all compensation and rehabilitation issues that ultimately will have strong bearing in the entire process. Priority should be given to local people for any job opportunities. Supervision on the construction activities should be done in close alliance with local administration and community representatives to correct any potential problems. Problems to be monitored include: any trespassing on communal or individual properties, grievances on displacement, compensation and rehabilitation issues, etc

2.3 Operation and Maintenance

Once the flood mitigation interventions chosen, operational and maintenance matters (e.g. responsibilities, funding, etc) must be thought about, agreed and resolved before construction starts as part of the planning and design stage investigations. Requirements during the operation and maintenance stages stated as follows.

2.3.1 Defining the Expected Role of Frontline Community

In operating and maintaining flood protection measures communities are at the forefront, though their effort always requires a technical backup from all other primary stakeholders in the locality. In defining their role, the following should be given attention: consideration of community's yearly working calendar, i.e. their busy and slack time; preferred contribution (cash, labour, material, etc) in operating the flood mitigation measure ; and defining the role of various sub community structures, like villages or group of households.

2.3.2 Defining the Role of Other Stakeholders

Operating and sustaining the service of the flood mitigation measure requires a consortium effort, with clear mandates, by all stakeholders. Apart from the local communities who face the brunt of the flood problem, the role of other formal institutions should also be stipulated during the design stage. This helps the technical people in proposing the right technical parameters and standards (like routine earth moving activities) for operating and marinating the flood mitigation structure. Such standards or guideline for routine activities can only proposed after assessing the capacity of formal and informal institutions like the Woreda level relevant government offices and other community based groupings.

2.3.3 Local Level Capacity Requirements

Social feasibility of projects, including the flood mitigation measures, is partly a function of local capability. In operating the flood mitigation interventions at community level, a capacity requirement assessment should be done at the designing stage. Capacity could be in the form of availing labour, local raw materials, etc. Proper designing of the flood mitigation measure should also take into account the capacity of other stakeholders at the grass root level (Kebele and Woreda level functionaries) in extending their support to the community during the operation and maintenance stages.

2.3.4 Resource Requirements

Community based development projects do require resource that can be raised in kind and cash forms. Assessing the required community level required resources is therefore essential at the designing stage. The community's labour man days requirement during the construction, operation and maintenance stages need to be estimated at the outset. This helps in preparing good participatory action plan that stipulates the contribution of different partners, mainly communities. Other resource requirements should also be estimated and apportioned for all stakeholders. For instance, resource requirement to conduct sessions on Farmer's Training Centre on flood mitigation measures study and operation and maintenance issues need to be considered.

2.3.5 Monitoring and Evaluation

Monitoring and evaluating activities to conduct supervision and timely performance assessment of the flood mitigation measures should be instated so as to enhance effectiveness and sustainability of the interventions. The right monitoring indicators should be developed in the designing stage. Motoring indicators include: the size of land (crop or grazing) saved from the flood, number of families affected or saved, etc. Detail indicators need to be developed so that performances of the flood mitigation measures will be gauged.

As part of the monitoring exercise, early warning data should also be collected and analysed. This may not be feasible at the community level. Nevertheless, the relevant government body at zonal or regional level need to provide the data and based on this the contingency plan to cope with any uncontained flood should be prepared whenever needed. Simple monitoring format is always recommended so as to encourage the participation of community members as well as grass root level development workers. The following format can be taken as a starting point in the monitoring exercise around public participation and sense of ownership of any flood mitigation measures at the community level. It should be noted that this is an indicative of the monitoring exercise and need to be further contextualized as required.

Table 2-1: Preliminary Suggestion on monitoring Public Participation in Flood Mitigation and Early Warning

Indicators	**Data Source**
Public consultation conducted	Records by the development committee or community members
Engaged community members in reconnaissance and designing activities of flood mitigation measures	Records from the Woreda sector offices and DA offices
Partnership established between the community and others actors	Number on the engagement of others in the flood mitigation support (data from local administration and sector offices)
Resources availed and commitment by the community (labour, cash, time...)	Records of the community level committee in charge of the coordinating plus records by a DA
Established committee and working groups at community level	Records by the coordination committee and DA
Engagement of most vulnerable ones (poor, female headed households , etc)	Records from the DA and coordinating committee
Response to early warning and signs of flood problem	Records from the partners(NGOs etc) and other woreda and zonal level actors
People supported and saved in case of uncontained floods	Records from the NGOs and government offices involved in the support
Saved properties and damages, e.g. on land not affected (may be as performance evaluation indicator)	Records from the Woreda level sector offices e.g. agriculture

2.4 Capacity Building and Stakeholder Analysis

2.4.1 Capacity Building

The ultimate aim of the participatory process in the flood mitigation measures are three fold to raise awareness on the flood issue, to promote sense of ownership, and finally to build local capacity through additional skills as well as promote the existing traditional flood problem coping mechanisms. In building the capacity and transferring knowledge, the local Framer's Training Centres (FTC) at Keble level need to be the hub in promoting two way communications between the technocrat and local community members. Similar to the other agricultural extension lessons, well framed and regular skill transfer sessions on the operation of flood mitigation measures need to be organized within these FTC,s. One of the DAs who serve within the FTC can also be nominated as a focal person on the flood study and operation issue with regular back up by the Woreda level relevant sector offices

2.4.2 Stakeholder Analysis

Community level projects usually doomed to fail due to gap seen in mandate and responsibility. Who is doing what should be cleared at the very beginning and design stage of the flood mitigation measures. For this, brief stakeholder analysis that stipulates the responsibilities and obligations of each stakeholder need to be done during the designing stage. A simple matrix (row and column) can serve the purposes. The important thing is defining the mandates of each stakeholder together with the schedule and resource requirement. The following table can be taken as a sample in this exercise but should be contextualize to serve the local purpose.

Table 2-2: Sample Stakeholder Matrix

Stakeholder	Involvement	Level of participation
Local/targeted community	During in all stages of study up to monitoring and evaluation the reconnaissance mission	Suggesting all the possible flood mitigation measures, outlining their expected contribution and their engagement in the operation and maintenance activities
Technocrats from the Woreda/ Zonal or regional level	Forwarding the various technical options and compatibility	In the reconnaissance, study, design and construction phases
Woreda level sector offices	Coordinating the over all operation	In all stages
Development Agents working at the grass root levels	Liaison between the community and others as well as giving regular support	In all stages

Other partners (NGOs, etc)	Participate in reconnaissance, study, construction, and operation and maintenance	In all stages
All actors in tandem	This includes not only for flood mitigation but also for early warning and also during any rehabilitation and relief support activities	Especially in the relief and rehabilitation operation in case of uncontained flood problem.

It should be borne in mind that the above table is meant is to give broad indication for the need to stipulate the mandates and level of involvement by different stakeholders at the very early stage. But depends on each circumstance, such table need to come out with the exact details of the stakeholders (which NGO, which government office, which grass root level workers, which village within the broad community, etc). Thus, the table by no means should be taken as a final.

Finally summary of expected activities in different stages of flood protection projects have been highlighted in the following table. The table gives some detail activity lists at different stages of flood protection projects which relates to the concerned parties and their expected outputs which can further be developed in the future. The project stages can generally be adopted for development works in Ethiopia.

Table 2-3 : Summary of Expected Activities in Different Stages

Stage	Activity	Participant	Output
Project Identification (Flood Mitigation Measure)	Reconnaissance visit to conduct community consultation on the flood problem and identification of problem areas	Community Representatives, local administration, Professionals (Regional/Zonal and Woreda levels) perhaps with hired consultant	Inception report showing communities understanding and awareness on the issue and intended activities –a way forward.
Feasibility Study	-Conduct the social feasibility together with other sector studies Exploring structural, non structural and traditional options -Also conduct baseline survey which will also help for monitoring and evaluating the flood mitigation interventions -Conduct and weigh the benefit with and without flood situation -Stakeholder analysis	-Professionals ,perhaps with hired consultant -Communities in exploring all flood mitigation measures.	-Feasibility report showing all the options and the best opted one(s) -determining the responsibilities of various actors -
Design stage	-Determining the best socially friendly flood mitigation structure(s) -Community consultation and giving feed back on the selected structure design -Preparation of relocation and compensation action plan for the would be affected properties and households -Formation of flood mitigation users /care takers group	-Professionals with consultation and fee back to the communities	- A flood mitigation design as agreed by all primary stakeholders. -Establishment of a committee (flood mitigation measure users/caretakers group from the targeted community -Action plan for relocation and compensation of properties
Construction	-Prior to mobilization stage, payment of compensation for any affected properties.	-Contractor -Communities -Flood mitigation users group/ care takers	-Socially friendly flood mitigation structure

	-Constructing the agreed flood mitigation structure		
Monitoring and evaluation	-Routine monitoring of the structure for its technical and social compliance -Evaluating the performances in mitigating the flood; assessing positive and negative impacts on the livelihood of the communities	-Woreda level professionals drawn from the relevant sector offices -Flood mitigation users group/care takers	-regular reports to relevant stakeholders (to community, Woreda...) -Evaluation reports showing the performance and impacts of the flood mitigation measure
Early warning an Contingency Plan	-routine data collection on the flood behaviour and hydrology to forecast flood	-Relevant sector office(s) from the Woreda , including the disaster preparedness office.	Early warning and contingency plan in case of flood that may not be contained by the already constructed flood mitigation structure.

3. ENVIRONMENTAL CONSIDERATION

3.1 Policies, Legislatives and Institutional Frame Works

There are several policy and legal frame works and requirements as related to developments and environmental protection. Some of the relevant policies and legislative frame works include;

Water Resource Policy: The Ministry of Water Resources has formulated the Federal Water Resource Policy for a comprehensive and integrated water resource management. The overall goal of the water resources policy is to enhance and promote all national efforts towards the efficient and optimum utilization of the available water resources for socio-economic development on sustainable bases. The policies are to establish and institutionalize environment conservation and protection requirements as integral parts of water resources planning and project development.

Ethiopian Water resource proclamation, Proclamation No. 197/2000: The purpose of the proclamation is described as to ensure that water resources of the country are protected and utilized for the highest social and economic benefits of the people of Ethiopia, to follow up and supervises that they are dully conserved; ensure that harmful effects of water are prevented, and that the management of the water resources is curried out properly.

Rural Land Administration and Land use, Proclamation No. 456/2005. The proclamation discusses on the need for sustainably conserve and develop natural resources and pass over to the coming generation through the development and implementation of a sustainable rural land use planning based on the different agro-ecological zones of the country.

The Environmental Policy of Ethiopia (EPE): endorsed in April 1999. The EPE's overall policy goal is summarized in terms of the improvement and enhancement of the health and quality of life of all Ethiopians, and the promotion of sustainable social and economic development through the adoption of sound environmental management principles.

Conservation Strategy of Ethiopia (CSE): Paramount amongst these was CSE, approved by the council of ministers, which provided a strategic framework for integrating environmental planning into new and existing policies, programs and projects.

Wildlife Policy: The policy is developed by the Ministry of Agriculture. The overall goal of the policy is described as; "to preserve, develop, manage and sustainable utilise Ethiopia's wildlife resources for social and economic development and for the integrity of the biosphere".

Forest Policies and Strategies: The Proclamation on Conservation, Development and Utilization of Forests was issued in 1994 (Proclamation No. 94/1994) to provide for the Conservation, Development and Utilization of Forests.

The Federal Constitution of 1995 sets out important articles related to Development and Environmental rights; Article 43 discusses the right to development. The Constitution under Article 44 highlights about environmental rights as follows: all persons have the right to a clean environment. All persons who have been displaced or whose livelihoods have been adversely affected as a result of state programs have the right to commensurate monetary or alternative means of compensation, including relocation with adequate state assistance.

Environmental impact assessment Proclamation No. 299/2002 is promulgate in

December 2002. The primary objectives of this proclamation is to make EIA mandatory for defined Categories of activities undertaken either by the public of private sector.

Proclamation No. 9/1995 establishes the Federal Environmental Protection Authority (EPA). EPA prepares environmental protection proclamations and does the federal government approve it. Environmental Protection Organs Establishment proclamation No. 295/ 2002 was provided in October 2002.

These policy and legal frame works have to be considered for planning & implementation of programs and projects alike. The flood protection embankment will refer to and incorporate the needs and requirements of the relevant policy and legal objectives and consult the mandated institutions for environmental resources protection and management.

3.2 Environmental Impacts of Flooding

Flooding poses significant impacts to sustainability and well being of environmental resources and properties when in excess. Social infrastructure, economic development, human life all suffer from adverse impacts of flooding. On the other hand, flooding plays a key role in determining the level of biological productivity and diversity of flood plains. It contributes to soil fertility, to recharges of water sources, to habitat formation and turnover, to exchange of nutrients and organisms, etc.

3.2.1 Beneficial Impacts of Flooding

The following are some of the benefits of flooding that are suspected to be compromised by flood protection dykes;

- ✓ Recharging of soil moisture and replenishment of alluvial soils;
- ✓ Recharge of water resources;
- ✓ Replenishment of wetland areas; and
- ✓ Nutrient recycling and enrichment of farmlands, grazing grounds, and aquatic flora and fauna resources.

Recharging water sources:

Floods are natural hydrologic processes and provide variable river flows and are an intermittent source of freshwater supply, filling natural depressions and recharging groundwater. Inundation of the flood plains helps recharge the groundwater, which is an important source of drinking water and is essential for agriculture. They are an important source for restocking local man-made water sources such as ponds, reservoirs, dams and irrigation channels, meeting round-the-year demand.

Agriculture: Recharging of soil moisture and replenishment of alluvial soils

Floodwaters carry nutrients and sediments, which are deposited on flood plains, enriching the soil. Rice paddies are flooded deliberately to take advantage of this natural fertilization process.

Fishery development:

A river basin is an ecological unit interconnecting upstream spawning habitats with downstream rearing habitats for a variety of species and other aquatic systems. Seasonal habitats on the flood plain, created by variable flow regimes, are essential for various stages of the life cycle of species. Floods provide an ecological trigger for both the spawning and migration of certain species. Some species spawn on the flood plain itself, whereas others migrate upstream to spawn in the river channel, providing an abundant supply of fish and alternative income sources at the household level.

Rejuvenation of the river ecosystem:

The river ecosystem is a critical habitat for the biota: fish, wildlife and waterfowl. Seasonal variability and variable sediment and flow regimes help maintain ecological biodiversity in rivers and flood plains. Wetlands or swamps located in flood plains serve as natural buffer zones for excessive flood flows and play host to many birds, fish and plants. Supplementary livelihoods in the form of recreational and eco-tourism activities can be made possible by the presence of the rich river ecosystem, bestowed with abundant flora and fauna. Surface runoff and flooding can help wash down pollutants and contaminants deposited on land caused by the intensive use of pesticides and fertilizers. They also flush out accumulated organic substances brought by untreated drainage water from farmlands, stockyards; factories and domestic use and restore the ecological health of stagnant rivers and streams by diluting them and providing clean water.

3.2.2 Adverse Impacts of Flooding

The death toll and economic losses posed by flood calamities are annually increasing all over the world, mainly related to the climate change and other anthropogenic consequences. In Ethiopia the recent flood hazards of year 2006 has risked lives of number of farmers in the rural areas. Damage to infrastructure and property is also significant.

Flooding inundates fertile plain lands that are productive agricultural and grazing grounds and renders unfit for production.

Floods can create environmental problems and create risks if precautions are not taken to minimize the spread of pollution due to flooding. Overflows of onsite sanitation facilities, leachates from unmanaged solid wastes, sewage backs up into basements and untreated sewage disposals, bypass from treatment plants located in flooded areas can cause health hazards during flooding. This issue is of great concern in urban areas in developing countries, where untreated sewage is disposed into open drainage systems. In addition, wastewater contains microbial pathogens, suspended solids, toxics, nutrients, trash and other pollutants and can contaminate drinking water sources.

Flood waters have potential to spread industrial as well as domestic chemicals. The location of dangerous chemicals should be carefully planned on the basis of knowledge of the flood plain and flooding mechanisms. Chemical stores or factories on flood plains may suffer leaks or damage to installations, thereby resulting in the spread of these chemicals and oil spills through flood waters. Such contamination can have long-term impacts on the health and habitability of the flooded area. It takes concerted effort, time and financial resources to clean up such environmental pollution. During the clean-up process, disposal of chemicals is an important decision, and should be taken with great care, as this has the potential to affect recipient ecosystems.

Major adverse impacts of over flooding include;

- ✓ Damages to properties, farmlands, livestock grazing grounds,
- ✓ Damage to social infrastructures and blockage of accesses.
- ✓ Damages to natural environment and ecological resources,
- ✓ Inundation of productive land,
- ✓ introduction of invasive species, resulting in loss of indigenous flora and fauna species,

- ✓ Creates Obstacles to wildlife and livestock passages
- ✓ Public health risks and vector infestation; epidemic waterborne diseases like diarrhea, dysentery, cholera, typhoid, pneumonia, etc.
- ✓ Spread of pollutants including chemicals, domestic and industrial pollutants.

Measures taken for flood protection, therefore, try to curb all those challenges and to ensure sustainability of resources, safety of infrastructure and human life. Construction of flood protection by structural flood control measure (like embankment dykes) on the other hand, has adverse impacts by rendering unfavourable situation and interfering with the benefits that flood offers to the natural environment and to socio-economic development of the downstream users.

3.3 Past Experiences of Flood Damages in Ethiopia

The effect of flooding in Ethiopia is felt mainly during the heavy rain seasons, which lasts from June to end of September when about 80% of the rains are received. Rivers spill over their normal channel and inundate significant plots of low-lying plain land which are suitable for; agriculture, animal grazing, and wildlife habitation, recreational play grounds and human settlement etc. Such plots of land are left idle or damaged frequently and annually by flooding effects. Vegetation cover and wildlife habitats, crops, infrastructure and properties found on those plots of land are exposed to sever losses and damages.

Torrential downpours are common in most parts of the country. Large scale flooding is rare and limited to the lowland areas where major rivers cross to neighbouring countries. However, intense rainfall in the highlands causes flooding of settlements close to any stretch of river courses. According to a study reported by Kefyalew Achamyeleh, the most serious flood problems are found in the Awash River basin. Irrigation development in this basin is quite advanced and is located in the flood plains on either side of the river with close to 70% of the country's large-scale irrigated agriculture; thus, high economic damage occurs during flooding. It is estimated that in the Awash Valley almost all of the area delineated for irrigation development is subject to floods; this amounts to an inundated surface of some 200,000-250,000 ha during high flows. The other rivers where significant floods occur are the Wabi-Shebelle River in south eastern Ethiopia near the Somali border and Baro-Akobo/Sobat River in western Ethiopia, near the Sudanese border. In the Baro-Akobo Plain an area of about 300,000-350,000 ha is prone to annual flooding and in the Wabi-Shebelle Basin some 100,000 ha may be inundated (kefyalew Achamyeleh, Ethiopia: Integrated Flood Management).

The level of the waters of two main lakes (Awassa and Besseka) has been gradually increasing from time to time causing damage to infrastructure in a number of areas. Both of these lakes are situated in the Rift Valley. Several small streams originating in the mountain range traverse townships and cities including metropolitan Addis Ababa. Torrential rains, common during the rainy season, cause sudden rise in the flow of these streams, which bring about flood damages to settlements along their banks. A similar situation had affected the town of Dire Dawa in 2006.

In urban areas additional factors contribute to flooding; incomplete and poor sewerage and drainage systems cause significant floods; housing developments and paved road surfaces hamper infiltration of rain water into the local ground aquifer, while increasing runoff water. Roads also interfere with drainage, concentrate flows to specific locations and cause damages to receiving downstream lands and properties. Some streets and residential areas with no storm drains all quickly become small rivers after even fairly modest rainfall.

Figure 3-1: Impacts of flooding in Dire Dawa during 2006

Source: UNICEF – News letter, **By Jane O'Brien ,** 29 August 2006

Figure 3-2: Flood Affected and Stranded Local Man in Omo Southern Ethiopia August 2006

Source: UNICEF – News letter, By Jane O'Brien, 29 August 2006. Thousands were stranded & Hundreds have died in floods across Ethiopia in 2006.

For Ethiopia, the most important river basin in terms of existing developments and associated flood management is the Awash River. Uncontrolled soil erosion and land degradation resulting in heavy sediment transport in streams and rivers has caused significant reduction of the capacity of the Koka reservoir, which serves as the only impounding reservoir for Awash flows. Water supply for irrigation and hydropower generation downstream depends on releases from this reservoir. The reservoir also serves as means of flood retention to protect downstream developments.

- Flooding in urban settlements, especially in Addis Ababa, annually causes damages to Property along streams coming down from the nearby hills. In most cases such damages occur on illegal settlement at the banks of the streams. Proper zoning and protection of riverbanks from obstructive structures to allow flood passage can curtail unnecessary damages of property due to floods.
- The Regional Flood Preparedness and Early Warning System for the Eastern Nile countries under the Nile Basin Initiative, mainly to benefit Sudan, Ethiopia and Egypt, is to be started soon. Successful completion of this project will enable these countries to improve flood management and reduce flood

damages to life, property and infrastructure, as well as to the environment. This is in sharp contrast to the previous situation of non-existence of exchange of advance information for early warning of flood occurrence.

- The experience in the Awash Basin related to silting-up of the Koka dam should also serve as a undeniable demonstration for what may be in store for the planned large dams on the Nile tributary rivers from Ethiopia.

In one of the recent rainy season flooding phenomena the whole 10,000 ha farm of Amibara Project has been inundated with crops standing in the farm land. the Amibara Irrigation Project, financed with contributions from EC, WB and AfDB, a flood protection scheme of dikes was constructed around the project to protect the development from flood damage.

In the Lower Plains, the Dubti area plantation and most of Dubti town have been inundated with flood, mainly coming from Logiya and Mille rivers. Such floods have also caused damages to the Afar pastoralists in the area between Dubti and Assaita isolating them and their livestock.

Emergency food and clothing assistance have often been supplied by military helicopters with the sponsorship of the Disaster Prevention and Preparedness Commission (DPPC).

These flood damages being witnessed both at the Middle and Lower Awash are a result of weakening of the previously existing entities established to ensure with proper management of the flood dikes and river courses in these areas so that such flood occurrences of disastrous proportions should not take effect.

3.4 Flood Control Measures and Methodologies

3.4.1 General

Flooding is a natural phenomenon resulting from excessive runoff created by rainfall events and also due to man's interference with the flow regime and flow patterns of storm /river water and rain water infiltration processes.

Flood and floodplain management play important roles in protecting people and socio-economic development. Until recently, flood control and protection have been engineering-centred, with little or no consideration being given to the social, cultural and environmental effects of the selected strategy, nor to long-term economic concerns. They have largely relied on structural solutions (e.g. embankments, bypass channels, dams and reservoirs, etc.), which have unfortunately changed flow regimes, fixed river shape or have separated river channels from their flood plains, resulting in loss of habitats, biological diversity and productivity. During the past half century, flood control and protection have slowly moved from an emphasis on structures towards incorporating complementary non-structural measures such as flood forecasting and land use regulations.

The adverse impacts of some of these structural measures and the growing concern for sustainable development have highlighted the need to address the negative consequences of flood control and protection measures on the environment. Increasing environmental concern for sustainable development has facilitated a shift from "flood control" towards "flood management". It is now recognized that floods are a natural phenomenon, which determine the natural regime of a river; and that any structural interventions have impacts on the natural environment, which can cause environmental degradation and impair services provided by ecosystems.

The need for the paradigm shift from flood control to flood management is enshrined in the concept of Integrated Flood Management (IFM): a process promoting an integrated approach to flood management. Integrated Flood Management aims at maximizing the net benefits from flood plains and minimizing loss of life from flooding. Integrated Flood Management addresses issues concerning human security and sustainable development from the perspective of flood management, within the framework of Integrated Water Resources Management (IWRM).

The essential elements of integrated flood management are:

- ✓ Adopting a basin approach to flood management;
- ✓ Bringing a multidisciplinary approach in flood management;
- ✓ Reducing vulnerability and risks due to flooding;
- ✓ Enabling community involvement; and
- ✓ Preserving ecosystems.

In line with this principle, it is essential to integrate the embankment dyke design and construction with none structural flood control measures (as discussed below), so as to ensure sustainability and safety of flood mitigation measures. Relevant institutions, experts and the community have to be involved and consulted during planning and decision making for measures to be adopted and included for flood protection.

3.4.2 Structural Flood Control

Physical structures are constructed to protect flooding of areas suspected to be damaged by the effects of excessive water flow. Structural flood control facilities include; Dams and reservoirs, embankment dykes, canalization, ponds and storage basins, bypass and diversion structures etc.

The structures are meant to re-direct the excess flow away from areas to be protected; like farmlands, settlement sites, infrastructure development sites etc. falling in the flood plains. However, such redirecting of flows and disruption of the flood inundation in the flood plains, adversely affects the flood plain ecology and natural resources adapted to the flooding phenomenon if not properly managed. On the other hand, Structural measures can not completely eliminate the risk of flooding. Nevertheless, because of their physical presence, they have the potential to create a false sense of security, leading to inappropriate land use in the protected areas.

3.4.3 None Structural Flood Control/Flood Management

Non-structural measures play an important role in reducing not only the catastrophic consequences of residual risks, but also adverse impacts on the environment. some of the non-structural measures relevant to reducing negative impacts on the environment include; Flood forecasting and warning, Land use regulations, Flood proofing, Emergency preparedness, response and recovery, Living with floods.

3.4.4 Mitigation of Adverse Impacts due to Structural Flood Protection Measures

In order to mitigate adverse environmental impacts caused by structural measures of flood Management and thus maintain river ecosystem health, the following issues need to be addressed:

- ✓ It essential to be aware that many of the flood management measures have the potential to cause consequential hydrological, morphological and environmental impacts, with further significant impacts on socio-economic development.

- ✓ Non-structural flood management measures such as land use regulations; flood forecasting and warning; and disaster prevention, preparedness and response mechanisms; have limited environmental consequences and should be actively considered as viable options, both as independent or complementary measures to structural flood control measures/ embankment dykes in this case.

- ✓ The ecological health of a river corridor depends not just on the water quality, or on the percentage of the total flows released, but also on the naturally variable quantity and timing of flows throughout the year.

- ✓ While designing embankments, the effects of lateral disconnection should be kept to a minimum by properly spacing the embankments and adequately assessing and balancing the economic and environmental consequences. Removal and setting back of embankments, where feasible, should be undertaken after a comprehensive study.
- ✓ Chanalization should be avoided as far as possible as an option for flood mitigation. However, if adopted, use of soil bioengineering and soft revetments should be considered, so that flood management objectives are not compromised while mitigating impacts on the environment.

3.5 Environmental Considerations for Embankment Dyke Development

Embankment dykes are constructed mainly from compacted earth material and is used to control flooding. Since early times, embankments have played a vital role in protecting people on flood plains against frequent flooding and continue to be the most favoured flood management option. From environmental conservation points of view, the following aspects of embankment dyke construction needs special attention.

3.5.1 Potential Impacts of Embankment Dykes

Structural flood control measures taken to limit the inundation of the flood plains undermine almost all of the beneficial impacts mentioned under section 3-2-1 above. Moreover, the flow in rivers that are confined between embankments has greater power because the velocities are higher than if the river were allowed to spread out onto the flood plain. This stream power can result in destructive bank erosion and channel changes that are more severe than if the river was able to overflow its banks.

The ability of a river to overflow onto its flood plain during high flow events prevents building up of this destructive stream power. Flood plains as such serve as a kind of “pressure release valve”. The flood plains store the spilled water and attenuate the flood peaks downstream while recharging the groundwater.

Some of the major adverse impacts of embankment dyke (structural flood control) are;

- ✓ Flood peaks increased downstream and Interference with downstream water uses; damage to social service infrastructures etc.
- ✓ Interference with the natural flood route and ground water recharge in the floodplain (Surface water flow modification, Ground water flow modification /change in water table);
- ✓ Impoverishing flood plains and riverine fisheries adapted to the natural flood cycle;
- ✓ Embankment construction can interfere with the periodical recharge and replenishment of marshlands and wetlands that harbor fish population. The connectivity between the river channel and the floodplain is broken by the embankment dyke, leading to impoverishment of the fish habitat. All floodplain

structures, processes and species needing frequent inundation are affected; loss of exchange of nutrients and carbon with flood plain, loss of floodplain refuges and spawning areas for river species, loss of floodplain forests, no more silt deposition on flood plain, no more habitat creation on the flood plain;

- ✓ Obstacles to wildlife and livestock passages;
- ✓ Creation of stagnant water (on the land side of embankment), favorable breading ground for disease causing vectors;
- ✓ Increased risk of life and property due to Structural failure and flood levels being higher than capacity of control structures;
- ✓ Displacement of people from temporarily inundated reservoir area;
- ✓ Loss of wild lands and wild life habitat;
- ✓ Increased soil erosion possible (both local scour and overall degradation); and
- ✓ Possible sedimentation of material eroded in embanked reach at downstream side.
- ✓ Introduction of invasive species, resulting in loss of indigenous flora and fauna species at the downstream discharge side.

<table>
<tr><th>Environmental parameters</th><th>Impacts</th><th>Possible mitigation measures</th></tr>
<tr><td>Flow regime</td><td>Higher water stages and velocities at above bank full flows
• Flood peaks increased downstream</td><td rowspan="4">Embankments should be planned in conjunction
with other structural measures such as dams
and detention basins, as well as non-structural
measures
• Spacing of embankments should allow for the
morphological lateral movement of the river
• Embankment designs should minimize the
disruption in lateral connectivity by setting
balanced standards of protection based on
economic and environmental criteria
• Setting embankments farther back from river
channel depending on land use conditions
• Removal of embankments separating flood plain</td></tr>
<tr><td>Sediment/
channel
structure</td><td>• Loss of connectivity between river and flood plain
• Loss of pool and riffle patterns and other heterogeneities
in channel form
• Increased erosion possible (both local scour and
overall degradation)
• Possible sedimentation downstream, of material
eroded in embanked reach</td></tr>
<tr><td>Water quality</td><td>Loss of exchange of nutrients and carbon with flood
Plain</td></tr>
<tr><td>Habitat/
biodiversity/
natural
resources</td><td>• Loss of floodplain refuges and spawning areas
for river species
• Loss of floodplain forests (timber, fruits, medicines)</td></tr>
</table>

	• All floodplain structures, processes and species needing frequent inundation are affected • No more silt deposition on flood plain • No more habitat creation on the flood plain	from river in combination with land use planning, if the flood plains are not occupied by human development

Table 3-1: Impacts of embankment dyke on various river corridor processes

3.5.2 Need for Lateral Connectivity

By containing flows within embankments, impeding seasonal floodplain inundation, the floodplain area exposed to inundation is restricted. This disrupts the lateral hydrological connectivity along the river corridor, with various environmental effects on both the ecology of the channel and its flood plain.

Further, embankments that are too close to the main channel decrease the natural heterogeneity of the flood plain, and impede the creation of new side channels and wetland areas. This reduction in habitat heterogeneity can dramatically impact fish populations, as many backwaters that were periodically connected to the main watercourse during the river floods no longer receive seasonal flows. These backwaters can be critical breeding and feeding areas for fish.

Lack of floodplain inundation reduces transmission loss and groundwater recharge, thereby severely affecting the groundwater resources and their associated ecological and economic benefits. This has consequences on base flow-groundwater interactions, and degrades riverine habitats.

Flood water spreading onto the flood plains improves soil fertility by depositing silt, exchanging nutrients and carbon between flood plain and channel, creating new habitats, and reinstating floodplain refuges and spawning areas for river species. Embankments reduce floodplain fertility because sediments and their nutrients are no longer deposited and exchanged.

Since embankments cannot guarantee absolute flood prevention, they can be designed to only provide a moderate level of protection. The degree of protection is generally driven by economic considerations. For example, it may be appropriate to protect agricultural lands against floods of a one-in-ten-year return period and allow them to be inundated during higher floods, thereby still maintaining the natural benefits of flooding (e.g. delivery of nutrient and organic rich sediments).

Embankments that are designed for protecting urban and industrial areas need to be combined with bypass/diversion channels and/or detention/retention basins.

Hence, it is should be understood that there is a need to give due weight to the environmental impacts of construction of embankments, while making these design decisions.

3.5.3 Need for Adequate Embankment Spacing

When designing the alignment for new embankments, one should keep their likely adverse impacts in view. Particularly, efforts should be made to include floodplain water bodies such as ponds, wetlands, etc., within the embankments, setting them as far apart and as far away from the main channel as feasible.

Typically, embankments result in steep-sided, trapezoidal, single channel cross-sections, rather than the more natural multiple channels with gentle bank slopes and flat-lying floodplain surfaces. By reducing the area that can be flooded, and by maintaining a larger proportion of the flow in the main channel with lower roughness, embankments decrease the travel times and increase flood peaks downstream. The high depth-to-width ratio of embanked channels makes them inherently unstable during high flows, requiring continuous maintenance. Otherwise, sever adverse impacts and hazards can result, with higher magnitude and severity compared to situations without he dyke.

Removing or setting back the embankments, in parts of the flood plains that are not intensively used for human development, can result in lower water levels and flow velocities, leading to larger in-channel storage and reducing flood peaks downstream. In certain situations where flood plains are used extensively for economic activities, this may not be a viable option. In such cases, a possible option for partly restoring the river-floodplain interaction is to set back the embankments, farther from the main channel thereby partially re-establishing the lateral connection with floodplain wetlands and backwaters and restoring the river's ability to move about. This also reduces the velocity of the stream, results in lower flood stages, and restores, in part, the natural functions of the flood plain, including temporary flood storage.

A river corridor is an enormously complex system, which cannot be fabricated. A comprehensive integrated approach is, therefore, required to undertake removal of embankments including land use planning. Magnitude, frequency and characteristics of floods, geographical setting and socio-economic background of the region have to be taken into account in any given situation.

3.6 Principles of Environmental Management

3.6.1 General:

The issue to be addressed here is how environmental damage can be avoided or reduced so as to ensure that development initiatives and their benefits are sustainable. The directive of environmental management should be to achieve the greatest benefit presently possible for the use of natural resources without reducing their potential to meet future needs and the carrying capacity of the environment.

Environmental management plan specifies mitigation and monitoring actions with time frames, specific responsibilities assigned and follow-up actions defined. Major negative impacts and proposed mitigation measures for embankment dyke project have been out-lined in the above sections. Implementations of these measures have to be carried out at different stages of the embankment dyke project; planning, design, construction & operation phases.

3.6.2 Guideline for project planning stage Environmental Management

During the project planning, the principal issue for environmental consideration is to give inputs for embankment dyke route selection, to avoid sensitive habitats or resource areas. Accordingly, environmental management activities during the planning phase include the following;

- ✓ Describing and classifying the project; Project description includes location of the project works and activities to be undertaken, ancillary works like material production, access roads, technology and labour requirement etc. Classifying the project on the other hand includes screening process to find whether the project is environmentally critical or none critical and to assess the sensitivity of the environment.

- ✓ Identifying alternatives; Two types of alternative identification has to be carried out; 1) to compare alternative flood protection options (either structural or none structural methods), 2) Alternative route to the proposed flood route corridor
- ✓ Environmental specialist participates in the initial site inspection visits and coordinate with the project team
- ✓ Conduct environmental screening to identify salient and clearly visible environmental parameters of the proposed flood protection project; and assess the sensitivity of the receiving environment.
- ✓ Identifying means to control secondary development in the influence area of the project.
- ✓ Conduct consultation with the public
- ✓ Define the scope of the environmental study(type of EA, geographical boundary,
- ✓ Prepare terms Of reference (TOR) for the environmental assessment.
- ✓ Embankments should be planned in conjunction with other structural measures such as dams and detention basins, as well as non-structural measures as discussed under section 3.4.2 above.

3.6.3 Guideline for Design phase Environmental Management

The design stage activities should assess all aspects of the local condition of the project area including engineering, environmental, social and economic situations that can have influence on the project implementation. The designer has to consult and involve stakeholders including the local community, the relevant institutions (water bureaus, agriculture & rural development bureau, environmental Protection Bureau etc), NGOs, experts, academic and research institutions.

The design should assess the benefits (both socio-economic and environmental benefits) being accorded by the periodic flooding of the flood plain and try to device design options that can maintain or enhances the positive impacts of flooding.

- ✓ Embankments should be planned in conjunction with other structural measures such as dams and detention basins, as well as non-structural measures as discussed under section 3.4.2 above.
- ✓ Provide design options to maintain flood plain connectivity (lateral connectivity as described above) with the river channel or storm flows; this can create opportunities for the recharge of water sources and the marshlands in the flood plains, which are habitats for diverse flora and fauna species. Embankment designs should minimize the disruption in lateral connectivity by setting balanced standards of protection based on economic and environmental criteria
- ✓ The design phase studies should identify sensitive ecological sites suspected to be adversely affected as a result of embankment dyke project implementation. Detailed EIA study may be required based on the project scale and sensitivity of the site.
- ✓ The design should give due considerations to the EIA study proposals and consider options to avoid or mitigate impacts on sensitive habitat sites like fish spawning grounds and bird nesting areas.
- ✓ Spacing of embankments should allow for the morphological lateral movement of the river.
- ✓ Setting embankments farther back from river channel depending on land use conditions.

3.6.4 Environmental Management during Construction and Maintenance of Dykes

3.6.4 .1 Guidelines for Vegetation Management on Flood Protection Works

- ✓ Retain vegetation cover to maintain or enhance environmental values on the side slopes of the over width dykes. Vegetation (including roots and canopy) can improve both dyke safety and habitat through soil conservation and erosion control. For example, setback strips, over bank and vegetation between flood protection works and the watercourse are recognized for their dyke safety, environmental and aesthetic values.
- ✓ Vegetation management guidelines for flood protection dykes are determined by the public safety need for visibility for inspection, access for efficient operation and maintenance, and minimization of detrimental effects to dyke fills and bank protection.
- ✓ Vegetation management should, where possible, include efforts to preserve and enhance fish and wildlife habitat in the overall stream/river corridor.
- ✓ Vegetation management in environmentally sensitive bird nesting areas shall be scheduled annually out side of the nesting seasons for birds in the area.
- ✓ Care should be taken to minimize disturbance to stems and roots of vegetation where possible during site access for maintenance activities.
- ✓ Dyke Crests shall be kept clear of vegetation other than trimmed grass, and accessible with due regard for inspection sight lines.
- ✓ The landside side slope of dyke fills (Dyke side slopes) shall be kept clear of vegetation other than trimmed grass, including a minimum 3 metre strip beyond the landside toe. (Increases in the landside strip may be considered where necessary, consistent with minimum needs for efficient operation of moving equipment.)
- ✓ The waterside side slope of riverside and setback dyke fills shall be kept clear of vegetation other than trimmed grass to the toe of dyke fill as determined by the dyke height. Large vegetation (greater than 300 mm diameter trunk/stem) shall be removed from an additional 2 meter strip measured horizontally. Consideration may be given to increasing the riverside strip consistent with minimum needs for efficient operation of mowing equipment;
- ✓ Preserving streamside vegetation adjacent to a setback dyke;
- ✓ Provided dyke safety is not affected, trees may be retained on the side slopes of over width dykes without bank protection, so long as they are spaced and pruned. Trees should be thinned, topped or removed (especially if higher than 15 meters) and the lower 1.5 meters of trees should be regularly pruned of branches to maintain inspection sight-lines. To facilitate possible emergency works, trees are not recommended in the freeboard range (0.6 meters vertical).
- ✓ Vegetation on the dyke fill side slopes should consist of closely trimmed grass or low ground cover. Depending on local growth rates, dyke slopes should be cut at least once annually. Trees and brush growth should be removed.

3-6-4-2 Maintenance of Fish Habitat

Avoid oil spillages, refuelling of vehicles and machinery at riverbanks & marshlands; do not deploy fuel leaking & defective equipments, machinery and vehicles at the riverbanks and marshlands frequented by fish population. Chemical and fuel products damage the fish population, and can also affect the health of population feeding on those fishes through he food chain; provided that the fishes survive he pollution impact.

Avoid disposal or deposits of a deleterious substance of any type in water frequented by fish or in any place under any conditions along the river banks and wetlands.

Avoid construction material, spoils and debris disposal in wetlands and river water while construction activity is undertaken. It is important to note that sediment laden water is considered a deleterious substance due to it's direct and indirect impacts on fish and fish habitat.

Avoid vegetation clearance in areas inhabited by fish. Marshlands and riparian vegetation along the river edges provide refuge areas for various fish species and protect them from the faster currents found mid-river. They also supply food and nutrients to the estuarine food web. On smaller river systems, riparian vegetation can shade the water, providing water temperature regulation through the summer and winter months. Riparian vegetation also intercepts surface water, filtering it and preventing some sediments and pollutants from reaching the river. Sometimes, spawning areas are located immediately adjacent to dykes. In these situations additional care should be taken to prevent disturbance to spawning fishes and incubating eggs.

3-6-4-3 Maintain Flood Plain Connectivity with the River Channel

Flooding frequency is the major driving force behind the community composition of flora and fauna resources in a floodplain. Species diversity of plants and wildlife can diminish due to disruption of flood plain connectivity from the river channels by embankments. Hence it can be expected that disrupting floodplains from their river channel may have a serious impact on biodiversity. Sample studies have indicated loss of forest plant species in flood plains disconnected from their river channels, degradation of fauna resources previously inhabiting flood plain due to flood recession by embankment dykes. So attention has to be paid to the effects of flood disruption on species richness and species composition of floodplain and try to maintain the connectivity of flood plains by providing allowances for intermittent flow to the flood plain.

3-6-4-4 Avoid Water Quality Degradation and Siltation during Construction

Material brought for construction should be stockpiled away from river courses, wetlands / marshland areas

Avoid dumping of debris and spoils into water courses and marshlands

Avoid storage of chemicals and fuel products in the water reaches

FLOOD PROTECTION DESIGN MANUAL
PART 4: RIVER GEOMORPHOLOGY AND GEOTECHNICAL INVESTIGATIONS

Table of Contents

CONTENTS OF OTHER PARTS

1. INTRODUCTION

1.1 General Background

Water:

Water is a chemical composition of hydrogen and oxygen atoms (H2O). Water is the most important natural resources not only of a state or of a country, but for the entire living things including humankind. The prosperity of a nation depends primarily up on the exploitation of this resource. Thus, it can be stated that the primary wealth of a nation is water. Water can exist in this planet as (i) rivers, streams, oceans and lakes (called surface water), (ii) groundwater which percolates from precipitation of rain, river or subsurface flow of other water bodies, and (iii) atmospheric water existing as vapor or snow.

Water is the best friend of human life. But, if water, especially flowing water as river/stream or flood or tidal waves, is not properly managed it may cause extreme misery to human life by changing its positions (from river course or oceanic boundary) and devastating vast areas; destroying bridges, power supplies, and other human utilities.

Rivers:

When small streams and brooks are naturally collected together by their water courses and concentrated to form large stream they are called river. Sometimes rivers and streams are interchangeably used but mostly river is preferably used to denote a main stream into which several tributaries flow.

The importance of rivers has been recognized since time immemorial, and rivers have occupied a very prominent place in every stage of human development. With the incoming of modernization and mechanization of human life, water and hence rivers are becoming more and more important. They provide us water for industrial uses and are a source of energy. Rivers have always been satisfying our domestic, irrigation, and other demands, and that is why most of cities were established in the vicinity of the natural rivers.

In the primitive times, there was absolutely no control on these natural rivers or rivers augmented by floods, and hence, they used to cause tremendous devastation and troubles to human beings. But with the development of science and technology, and in his bid to control nature, man has devised and is devising means and ways to control the mighty rivers and their floods.

Floods:

When it rains or snows, some of the water is retained by the soil, some is absorbed by vegetation, some evaporates, and the remainder, which reaches stream channels, is called runoff. When soil and vegetation cannot absorb all the water; water then runs off the land in quantities that cannot be carried in stream channels or retained in natural ponds and constructed reservoirs is called flood. About 30 percent of all precipitation is runoff, and this amount may be increased by melting snow masses. Periodic floods occur naturally on many rivers, forming an area known as the flood plain. These river floods often result from heavy rain, sometimes combined with melting snow, which causes the rivers to overflow their banks. A flood that rises and falls rapidly with little or no advance warning is called a flash flood. Flash floods usually result from intense rainfall over a relatively small area. Coastal areas are occasionally flooded by unusually high tides induced by severe winds over ocean surfaces, or by tsunamis caused by undersea earthquakes.

Floods, like rivers, not only damage property and endanger the lives of humans and animals, but have other effects as well. Rapid runoff causes soil erosion as well as sediment deposition problems downstream. Spawning grounds for fish and other wildlife habitat are often destroyed. High-velocity currents increase flood damage; prolonged high floods delay traffic and interfere with drainage and economic use of lands. Bridge abutments, bank lines, sewer outfalls, and other structures within flood ways are damaged, and navigation and hydroelectric power are often impaired. Financial losses due to floods are commonly millions of dollars each year.

Flood controls and river training works:

Flood controls and river training works, in its wider aspects, covers all those engineering works which are constructed on a flood or river course so as to guide and confine the flow to the channel and to control and regulate the flood or river bed configuration, thus ensuring safe and effective disposal of the floods and sediment loads.

The activities of flood control and river training sometimes are very difficult to distinguish between them and overlap, and may be represent the same thing. But it is generally to be understood that flood control works are those protections done for normal or flush floods and whereas stabilizing and training of the river along a certain alignment with a suitable waterway is called river training.

The basic methods of flood control have been practiced since ancient times. These methods include reforestation and the construction of artificial structures such as dams, levees, reservoirs, and flood ways.

Through the centuries people have created a flood problem by cutting down trees and digging up the vegetable cover of the soil, thus increasing soil erosion. Cultivation decreases the ability of the soil to retain water and increases runoff. Vast land areas along the headwaters of rivers throughout the world have been laid waste by intensive cultivation and subsequent erosion. Flood control in these areas has been directed to restoring vegetation and instituting efficient methods of soil management, such as crop rotation and contour plowing.

Although dams have been used for many centuries, their primary purposes were to build up water reservoirs for irrigation and other domestic uses and to create power. Only recently have they been constructed specifically for flood control. An effective method of controlling floodwaters is to construct coordinated groups of dams and reservoirs on the headwaters of the streams that lead into the main rivers, so that water can be stored during periods of heavy runoff and released gradually during dry seasons

Another method of flood control is the construction of levees/dykes, groynes, spurs, floodway protections, etc on the lower reaches of floods/rivers to confine or divert the floodwaters. The floods/rivers are widened at certain points and allowed to overflow. Inundation of certain confined areas prevents the flooding of other areas.

1.2 River Geomorphology and Geotechnical Engineering

Geomorphology is a wide subject. In short geomorphology is the scientific study of landforms and landscape. It usually applies to the origin and dynamic transformation of the earth’s land surfaces, and the morphology of the sea floor. To understand geomorphology, geologic time scale is needed to record the evolution of landforms which is in general the gradual transformation by the action of different agents.

Based on the agent of land transformation mechanisms and type of changes, geomorphology can be classified as:

i. static geomorphology (actual landforms without any further changes);
ii. dynamic geomorphology (short term changes that cause land transformation);
iii. generic geomorphology (long term changes that cause landforms), and
iv. environmental geomorphology (landscape ecologic links between geomorphology and neighboring disciplines).

Therefore, river geomorphology is the dynamic geomorphology type by the action of rivers and also influenced by actions of ecologic links of the earth surface. Rivers and floods are one of the main earth's land surface transforming agents and the study of their behavior is vital in controlling and mitigations measures.

Geotechnical engineering is a recently evolved branch of civil engineering that is the study of the interactions of superstructures and subsurface materials and the arts of designing for the superstructure's foundations or substructures. Geotechnical engineering encompasses soil engineering, rock engineering, foundation engineering and so many others that are linked with underground structures like tunneling, grouting, geo-membrane, etc. In flood protection works geotechnical investigations and designs are necessary as in many other civil works. In geotechnical engineering considerable knowledge geology is required and also there is more than thirty percent overlap between engineering geology and geotechnical engineering. Moreover, after studying the behavior of rivers and flood, appropriate measure of protections are partly effected through geotechnical analysis and designs.

1.3 Objectives and Scope

Objectives of this manual are to give guidance to flood protection engineers and associated professionals on the knowledge of works of rivers and floods on the transformation of the earth's surface, and also on the methods, the rationale behind and the precautions required in the geotechnical investigations of surface and sub-surface investigations for flood protection works.

The scope of the manual is limited to flood protection works. Topics which require higher and separate specializations in geomorphology and geotechnical engineering are left or slightly touched only from the general importance and their contribution on the study. Laboratory testing and procedures for soils and rocks are not included in this manual.

2. RIVER GEOMORPHOLOGY

2.1 General

One of the agents of the dynamic geomorphology on the earth's crust is the actions of flowing waters as rivers, streams, brooks and floods. Additionally winds, glaciers-rivers and floods change the earth's surface forming gullies, big valleys, mountains, river and flood deltas, etc.

To understand and then to take proper measures against their adverse effects, it is important to give abridged summaries on the general weathering processes and then the works of streams or equivalently floods on the transformations of the earth's surface. Therefore, this chapter deals with river geomorphology which is understood includes surface runoffs encompassing flood geomorphology.

2.2 Weathering

The rocks of the earth surface break and undergo decay under the influence of the actions of the atmospheric agencies like wind, sun, water and organisms, and produce soil. This phenomenon is called weathering. Weathering and erosion together lowers the mountains and produces sediments. The weathering may be defined as a process which tends to break and decompose rocks in place. It includes two processes: (i) disintegration or physical breaking, and (ii) decomposition or chemical decay. In nature these two processes work simultaneously.

Erosion: erosion is a process which includes the destruction of existing rocks and removal of the product from the site of destruction. Transport is the important aspect of erosion. It is usually done by water, wind or ice.

Denudation: the combined effect of weathering and erosion is called denudation. It involves the general wearing down of the earth's surface.

Exfoliation: it is mainly a physical weathering process in which large sheets of rock peel off from an outcrop. In exfoliation, reduction in pressure due to removal of the overlying rock plays an important part. As each slab breaks off, it releases weight from the underlying mass. As a result its outer layers expand and separate from the rock mass. Exfoliation is commonly seen in homogeneous rocks like granite.

Spheroidal weathering: if an outcrop of jointed rock is subjected to chemical weathering, rounded boulders are produced. This process is known as spheroidal weathering. The spheroidal weathering resembles the exfoliation except that it takes place on a much smaller scale. In spheroidal weathering, the hydration which is accompanied by increase in volume plays an important part. As the minerals in the rock weather to clay, they increase in size because of the addition of water into their structure. This increase in volume exerts an outward force due to which the concentric layers of rocks break loose and fall off.

2.2.1 Physical Weathering

Physical weathering or disintegration involves applications of mechanical forces. In physical weathering a rock breaks into smaller pieces without any chemical change. Thus by breaking rocks into smaller pieces, physical weathering increases the amount of surface area for chemical attack. The principal agents of physical weathering are frost, heating and cooling, and organisms.

Frost:

When water freezes it expands about 9% of its volume. In nature water enters into cracks of rocks. Up on freezing it expands and exerts great pressure on the walls of cracks. As a result, the rock breaks into pieces. This process is called frost wedging. By this process angular fragments of rocks are broken off from the high mountain ranges. The rock fragments roll down the hill slope and accumulate at the base to form talus deposits.

Heating and cooling:

It is believed that daily cycle of temperature change weakens rocks, particularly in hot dry regions. Heating a rock causes it to expand and cooling causes it to contract. The repeated expansion and contraction tend to develop crack in rocks. Furthermore, the rock minerals in a rock mass have different coefficients of expansion and contraction which as a result creates differential stress inducing cracks on the rock. In desert areas the coarse grained rocks like granite, disintegrate soon into their constituent crystals and become desert sands as a result of temperature variations.

Organisms:

Plants, burrowing animals and men play an important part in the disintegration of rocks. Plant roots grow into rock joints. As the roots grow thicker, they exert mechanical pressure and wedge the rock apart. The burrowing animals, such as earthworms, ants, termites and rodents also contribute the disintegration of rocks. Men also break rocks by making road cuts, tunneling, quarrying, mining and cultivating the land.

2.2.2 Chemical Weathering

Chemical weathering or decomposition is a process in which rocks are broken down by the chemical decay of minerals. During chemical weathering, a set of chemicals reactions act on rocks which changes their minerals to more stable forms. The principal agents of chemical weathering are water and organisms.

Water:

Water is the main agent of chemical weathering. Although water in pure form is inactive, it is becomes a powerful chemical agent when a small amount of oxygen and carbon dioxide are dissolved in it. Rain water usually contains these gases. The main reactions involved in the chemical weathering are oxidation, hydration, carbonation and solution.

Organisms:

Many dead organisms produce acids as they decay. These acids increase the solvent power of water. For example, the solubility of silica, alumina and iron is much greater in the presence of organic acids.

2.2.3 Rate of Weathering

The rate of weathering of rocks depends on the following factors:

- ✓ The particle size influences the rate of weathering. The smaller the particle, the greater the surface area for chemical attach;
- ✓ The material composition of a rock is also very important. The slate which is mainly composed of clay minerals is very resistant to weathering while this is not true for marble;
- ✓ the order in which the silicate minerals weather is the same as their order of crystallization. From Bowen reaction series it is clear that olivine crystallize first and it is the least to resist weathering, while quartz which crystallize last, is the most resistant;

- ✓ the rate of weathering also affected by the climatic factors. Chemical weathering is most active is in warm humid regions and least active in cold or arid regions.

2.3 Works of Streams

2.3.1 Stream Erosion

The term “stream” includes the channelized flow of any size, from the smallest brook to a very large river like Nile. Although the term “river” and “stream” are used synonymously, the term “river” is preferably used to denote a main stream into which several tributaries flow. The geological work of streams is to “erode” the valleys, “transport" the material thus eroded, and “deposit” the same in the lower reaches at favorable sites.

The streams cause erosion in four ways: (i) chemical action, (ii) hydraulic action, (iii) abrasion, and (iv) attrition.

Chemical action: it includes the solvent and chemical action of water on country rocks. The chemical decay works along joints and cracks and thus helps in breaking the bedrocks.

Hydraulic action: the swiftly flowing water hammers the uneven faces of jointed rocks exposed along its channel and remove the jointed blocks. This process of erosion is called “hydraulic action”. At the bottom of waterfalls, the channels are eroded at an enormously rapid rate by the hydraulic action.

Abrasion: the flowing water uses rock fragments such as pebbles, gravels, and sands as a tool for scratching and grinding the sides and floor of the valley. This process of erosion is called “abrasion”.

Attrition: it is the breaking of the transported materials themselves due to mutual collision. The attrition causes the rock fragments to become more rounded and smaller in size.

In addition to the above, the streams acquire their load by many other means. Much of the material carried by a stream is contributed by underground water, overland flow and mass wasting.

2.3.2 Stream Transportation

The amount of solid material transported by a stream is called its “load”. The streams transport its load in three ways: (i) in solution (dissolved load), (ii) in suspension (suspended load), and (iii) along the bottom (bed load).

Dissolved load: the dissolved load is brought to the stream by groundwater. Some amount of it is also acquired directly from soluble rocks which occur along the stream’s course.

Suspended load: suspended load forms the major portion of the load carried by streams. Usually only smaller particles such as clay and silt travel in suspension, but during floods much larger particles are carried this way.

Bed load: the forward force of moving water acts more directly on the larger grains at the bottom, pushing, rolling and sliding them along. The particles moved in this way constitute the “bed load” of stream. Locally the medium size material may travel partly by rolling as bed load and partly in suspension. This process of intermittent jumping is called “saltation”. In saltation, the heavy particles are lifted occasionally for a few seconds by a swift eddy.

The velocity of a stream is affected by a number of factors, including gradient, channel size and shape, load and discharge. The increase of velocity increases the transporting power of a river as much as the 6th power of the velocity.

$$\text{Transporting Power} \propto V^6$$

It means that during floods the transporting power of a stream suddenly rises very much and it becomes capable of moving big boulders which would otherwise remain quite immovable.

2.3.3 Deposition

The loose rock materials transported by a stream downstream are deposited where the velocity of flowing water is reduced. The sorting of the material takes place automatically as the large and heavier particles settle quickly while the smaller and lighter ones continue their journey further ahead. The material which a stream deposits as sediment is called “alluvium” or “alluvial deposits”.

2.4 Features of Stream Erosion

Pot Holes: “pot holes” are the circular and deep holes, cut into solid rocks by sand grains and pebbles, swirling in fast eddy. They are commonly found on the channel floor.

Waterfalls: the falling of stream water from a height is called a “water-fall”. Waterfalls occur at places where the stream profile makes a vertical drop. Such a situation is usually found where gently inclined, erosion resistant beds overlie the non-resistant beds. The softer rock is eroded fast while the harder one offers resistance and forms a ledge at a height, from which the stream’s water falls down deep into the gorge.

When the water falls over the ledge, it erodes the less resistant beds of the cliff. Due to this undercutting a portion of the upper resistant bed breaks off and the waterfall retains its vertical cliff while it gradually moves upstream.

Gorges: narrow and deep river valleys which develop in hard rocks are called “gorges”.

Meanders: the symmetrical S-shaped loops found in the course of a river, are called “meanders”. Meanders develop in mature rivers. Mature rivers are those which have cut down to an approximately graded profile. In such rivers side cutting becomes very prominent which results in the development of meanders. The meanders grow due to deposition of sediment along the slip off side and erosion at the undercut side (Fig. 2-1).

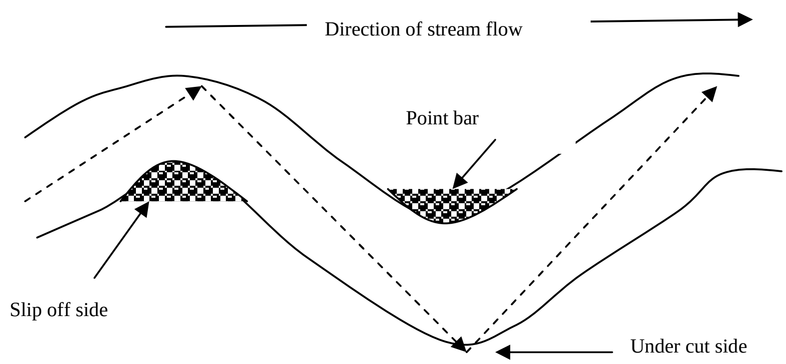


Figure 2-1: Stream Meander

Meanders continually change their position. They move both downstream and to the side. The sideways movement occurs because at bends the swiftest currents shift toward the outside bank causing erosion at the outside of the curve and deposition on inside of the curve. In this way a stream migrates sideways and slightly downstream by eroding its outer bank and depositing a sand bar at the inner bank.

Oxbow lake: the meanders grow by eroding its outer bank and depositing sediments at the inner bank. During this process the sharpness of the river bends increases progressively and the neck of meander becomes narrow and narrow. Finally a stage comes when the river cuts through the neck and starts flowing straight leaving behind its roundabout course. Such left out old meanders which remain filled with stagnant water, are called "oxbow lakes: (Fig. 2-2).

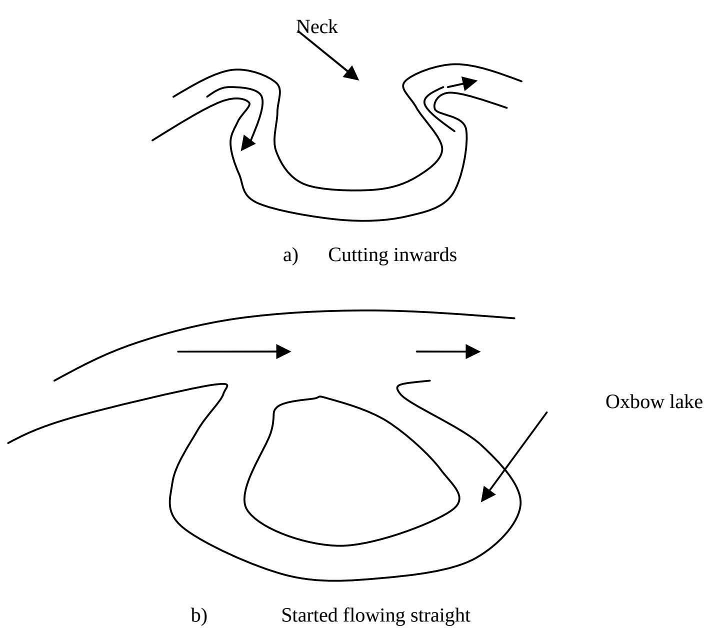


Figure 2-2: Formation of an Oxbow Lake

Entrenched meanders: on many occasions, the land is uplifted. The uplifting of a mature stream would cause it to give up lateral erosion and revert to down cutting. River of this type are said to be “rejuvenated”. When a meandering river is rejuvenated, it starts down cutting again. As a result the meandering channel is deepened and the old meanders get entrenched into the bedrock. Such meanders are called “entrenched meanders”.

2.5 Depositional Land Forms

Alluvial fans: the alluvial material which flows down form mountains accumulates at foot hills where the stream enters a plain. The deposition occurs due to abrupt change in the gradient of river valley. Such deposits spread out in the shape of flat fans and are called “alluvial fans”. Usually the coarse material is dropped near the base of the slope while finer material is carried further out on the plain. Alluvial fans from many adjacent streams along a mountain may merge to form a long wedge of sediment called “alluvial aprons”.

Flood Plains: during floods a river overflows its bank and sub-merges the adjacent low lying areas where deposition of alluvial material takes place. A wide belt of alluvial plain formed in this way on either side of a stream, is called “flood plain”. Its name is appropriate, because the flood plain gets submerged only when a river overflows its bank at flood stages.

Natural Levees: “natural levees” are the low ridges which are formed on both sides of a river channel by the accumulation of sediment. They tend to confine the flow of river water into its channel between flood stages. The natural levees occur in rivers which have broad flood plains. During floods the river overflows its bank and its velocity decreases rapidly. As a result most of the coarse sediment is deposited along the area bordering the river channel and finer sediments are deposited more widely over the flood plain. In this way, successive floods build up ridges on both sides of a river channel, which are called “natural levees”.

The areas behind the levees are poorly drained as water cannot flow up the levees to join the river. The marshes thus formed, are called “back swamps”. A tributary stream often has to flow parallel to the main stream until it can breach the levee. Such streams are called: “Yazoo tributaries”.

Point bars: in meandering rivers, sediment deposits occur as point bars. The “point bars” are the crescent shaped deposits which occur at inside bends of a river channel (Fig. 2-1).

Deltas: “deltas” are deposits built at the mouths of streams. The deltas are usually triangular in shape with their apex pointed upstream. When a stream enters an ocean or lake, the currents of the flowing water dissipate quickly. This results in the deposition of the series of sedimentary layers which make up the delta. The material of most details is well sorted and many deltas are uniformly graded. Delta consists of three sets of beds: (i) bottomset beds, (ii) foreset beds, and (iii) top-set beds

(i) Bottomset beds: the thin horizontal beds which overlie the ocean bottom are called “bottomset beds”. They are mainly composed of fine grained sediment, such as silts and clays.
(ii) Foreset beds: foreset beds begin to form prior to the accumulation of bottomset beds. These beds are composed of coarse sediment which is dropped almost immediately when a river enters a lake or ocean. The foreset beds appear similar to cross bedding and their angle of slope varies from 120 to 320 depending on the grain size the material.

(iii) Topset beds: foreset beds are covered by thin nearly horizontal topset beds. These beds occupy the upper surface of the delta. They are composed of a mixture of coarse and fine material.

2.6 Base Level and Graded Streams

Longitudinal Profile: plot of the relative elevation of a stream bed from headwaters to mouth is called a "longitudinal profile". The longitudinal profile of a stream is generally concave upward which is in accordance with the steady downstream decrease in slope.

Base Level: the level which controls the depth of stream erosion is called a "base level". As base level is the lower limit of the longitudinal profile, streams cannot cut below this level. There are two types of base level: (i) ultimate base level, and (ii) local base level. The "ultimate base level" represents the lowest level to which a stream can erode its valley. It is therefore, the level at which the mouth of a stream enters a lake or ocean. Resistant rock beds, waterfalls, lakes or artificial dams which lie along a river course form "local base levels". They act as limiting levels for the stretches that exist immediately upstream. Thus act as limiting levels for the stretches that exist immediately upstream. Thus local base levels are temporary obstructions to down cutting encountered by a stream.

Any change in base level causes a stream to change its characteristics. Lowering of base level increases the stream's gradient. As a result the velocities increase and down cutting is accelerated. The erosion first starts from near the mouth and then works upstream until the stream profile is adjusted along its full length. Thus the bedrock channel is deepened and parts of the old valley floor are left as a terrace along the walls of the new valley. Such steps-like features are called "river terraces" (Fig 2-3)

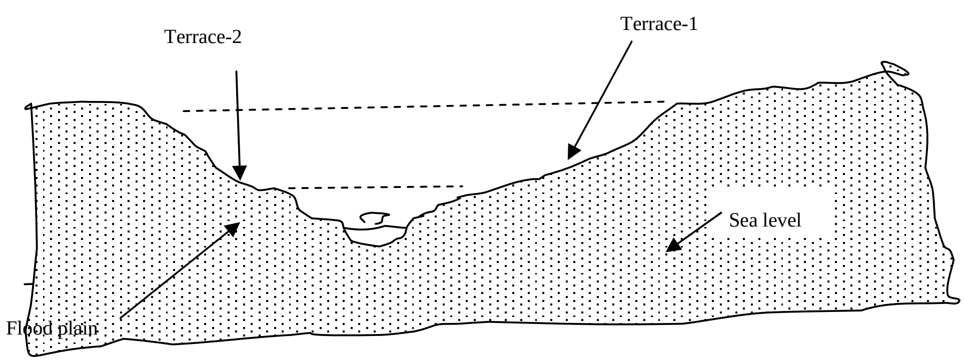


Figure 2-3: River Terraces

A rise in base level reduces the stream's sediment transporting capacity. As a result the stream deposits sediment thereby building up its channel. Sometimes the capacity of a loaded stream is lowered to such an extent that heavy aggradations takes place. Now the single river channel can no longer carry its load. In such circumstances the individual channel subdivides itself into a series of smaller channels. These channels are separated by many low islands which are the product of heavy deposition. Such a stream is called "braided stream."

Profile of equilibrium: streams have tendency to cut and remove the material from high gradient regions and deposit it in the low gradient areas. In due course of time when a uniform longitudinal slope is developed throughout the stream course, the process of erosion and deposition ceases. Such a slope is called the "profile of equilibrium" or "graded profile" of a stream. Thus a graded stream will not erode or deposit material but will simply transport it. As a matter of fact this stage is never reached due to many disturbances.

2.7 Drainage System

Drainage basin: a land area surrounded by divides which contributes water to a river is called "drainage basin".

Divide: it is a ridge of high ground which separates two drainage basins. Divides range in size from a ridge separating two small gullies to "continental divides" which divide continents into large drainage basins.

River capture: "river capture" is a process by which a part of the drainage of one river is diverted into another. Due to greater discharge, slope or other factors, the stream on one side of a divide may erode its valley more actively than the stream on the opposite side [Fig. 2-3 (b)]. While lengthening its valley head ward, it may reach the less active river by breaking down the divide between the two and divert a part or all of its drainage into its own channel. The river capture commonly occurs in the youth stage. It becomes less common as streams grow larger and is rare in large rivers.

2.7.1 Drainage Pattern

Although all drainage networks branch in the same way, the shape of their patterns varies greatly from one kind of terrain to another, depending upon the rock type or structure. The chief drainage patterns are: (i) dendritic, (ii) trellis, (iii) rectangular, (iv) radial, and (v) parallel.

(i) Dendritic Pattern:

In "dendritic pattern" the streams show a branching tree like arrangement [Fig 2-4(a)]. This pattern develops in terrains covered with uniform rock types, such as horizontal sedimentary rocks or massive igneous or metamorphic rocks.

(ii) Trellis pattern:

A "trellis drainage pattern" is one in which major streams are parallel and short tributaries join the main stream at nearly right angles [Fig 2-4(b)]. This type of drainage pattern develops in regions containing folded or tilted strata. Here the main stream develops in the strike valleys cut into the soft rocks, while tributaries flow down the resistant ridges.

(iii) Rectangular pattern:

Differential weathering of faults or joint systems in bedrocks localizes the stream flow producing a more ordered rectangular drainage. In a "rectangular drainage pattern" angular deflection of stream course are apparent [Fig 2-4(c)].

(iv) Radial patterns:

In a "radial drainage pattern" streams flow outwards in different directions from a central high point. This pattern commonly develops on an elevated structure such as a volcano or dome [Fig 2-4(d)].

(v) Parallel pattern:

This type of drainage pattern develops in a terrain containing tilted rock beds and parallel faults. The major stream occupies the fault while tributaries which are parallel, meet the stream approximately at the same angle [Fig 2-4(e)].

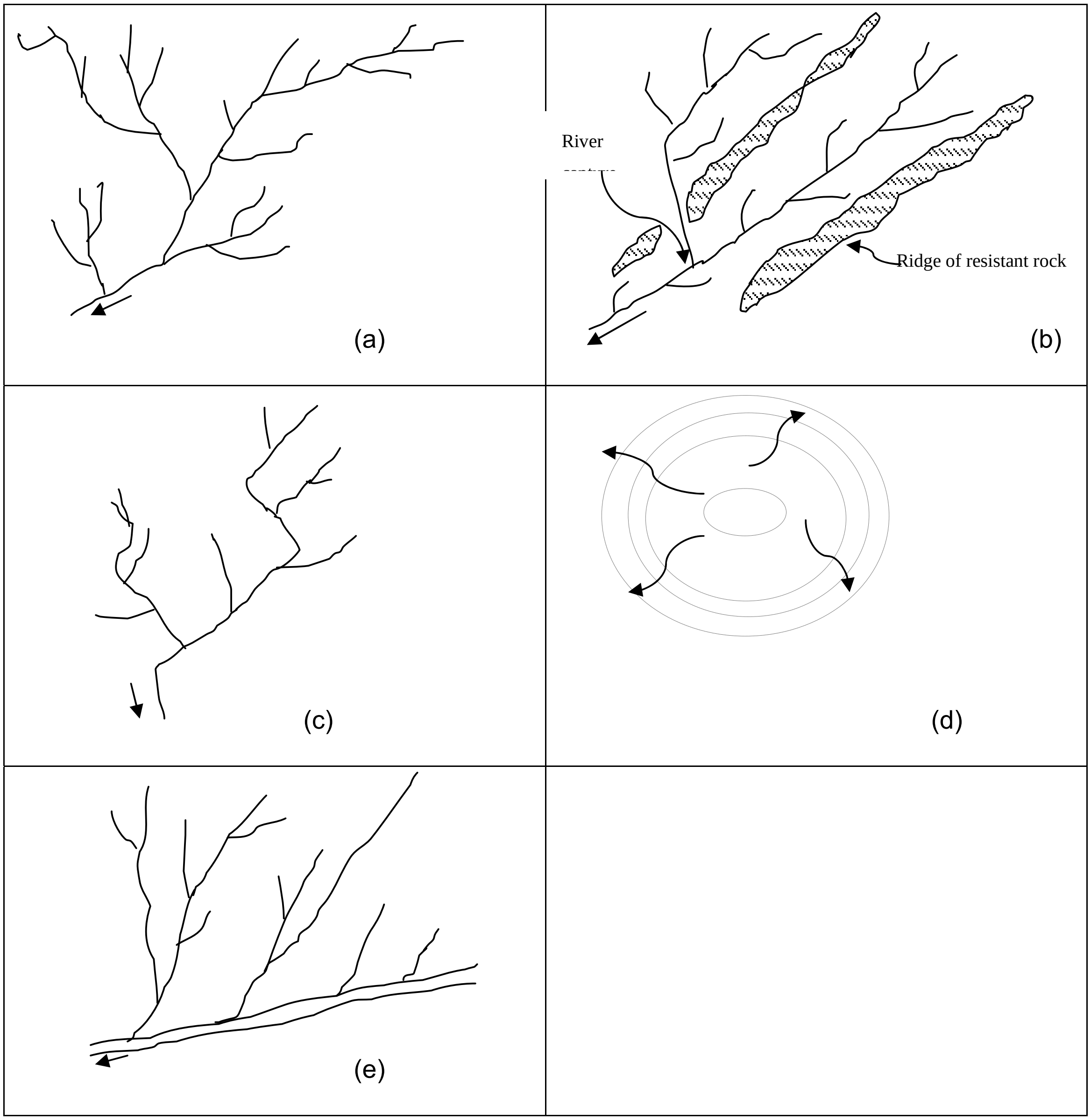


Figure 2-4: Drainage Patterns

2.8 Type of Streams

On the bases of development and origin, the streams have been classified into four groups: (i) consequent streams (ii) subsequent streams, (iii) antecedent streams, and (iv) super-posed streams.

(i) Consequent streams: consequent streams are those which follow the slope of the initial land surface.

(ii) Subsequent streams: these are tributary streams which develop on the sloping sides of a stream valley. Subsequent streams generally take their course along the weak and easily erodible zones, such as rock boundaries, fault zones, joints, etc. These weak zones are discovered and eroded after the development of the consequent streams.

(iii) Antecedent streams: antecedent streams are those which are able to maintain their original course across the area of uplift.

(iv) Superposed streams: geologic events may strongly control the course of a stream. Streams flowing in a dendritic pattern on horizontally bedded younger formation may erode through it to expose the underlying, strongly folded and faulted older rocks of varying harness. Over the older rocks, the stream courses will not easily adjust to form a wholly new drainage pattern appropriate to the structure of rocks. Such streams are called “superposed streams”. Superposed streams do not show any relation to the structure of the underlying rocks.

2.9 Stages of Valley Development

The development of stream valleys takes place in an orderly fashion. A valley passes through three stages during its evolution. The three stages are: (i) youth stage, (ii) mature stage, (iii) old stage.

(i) **Youth stage**: a stream is said to be in the youth stage where cuts its valley downward to establish a graded condition with its base level.

- (a) Position: the youth stage is commonly found in mountainous regions from where a stream starts its journey;
- (b) Erosion: down cutting is dominant;
- (c) Valley: narrow V-shaped valley;
- (d) Longitudinal profile: longitudinal profile is ungraded. The gradient is steep and waterfalls and rapids are common;
- (e) Valley floor: the stream occupies most of the width of the valley floors as a result there is little or no flood deltas;
- (f) Stream pattern: the stream course is angular and little meanders. Tributaries are short and few.

(ii) **Mature Stage**: a stream is said to be in a mature stage downward erosion diminishes and lateral erosion dominates.

- (a) Position: mature stage is found in the plains lying adjacent to the mountain region;
- (b) Erosion: down cutting is slight and side cutting becomes dominant.
- (c) Valley: broad and trough shaped;

(d) Longitudinal profile: waterfalls and rapids are absent. Valley bottom is graded so that the longitudinal profile exhibits a relatively smooth curve. The gradient is moderate;
(e) Valley floor: the stream swings in meanders. The flood plains are narrow and sandbars are present;
(f) Stream pattern: the stream moves in meanders. The tributaries are many.

(iii) **Old stage: i**n old stage, the flood plain of a stream becomes several times wider than its meander belt.
(a) Position: the old stage is found near the month of streams;
(b) Erosion: in the old stage, the stream ceases to enlarge the flood plain. The main work of a stream is rework the unconsolidated sediment of the flood plain;
(c) Valley: valleys become wide and open with low boundaries which may be indistinct;
(d) Longitudinal profile: the gradient becomes very low. The stream approaches base level and it aggrades strongly;
(e) Valley floor: oxbow lakes are common. Natural levees are also present. They are accompanied by back swamps and yazoo tributaries. The meander belt is narrower than the valley floor;
(f) Stream pattern: the stream pattern is meandering with oxbow lakes. The tributaries are few and large.

In the above discussion it is assumed that the base level of a stream remains constant as a river progresses from youth to old stage. On many occasions, however, the land is uplifted. The effect of uplifting on a mature stream is to abandon lateral erosion and revert to down cutting. Such rivers are said to be “rejuvenated”. Mature streams readjust to uplift by cutting a new flood plain at a level below the old one. This produces step-like features in the river valley which are called “river terraces”.

2.10 Cycle of Erosion

To understand the cycle of evolution of a landscape, let us start with a relatively flat upland area in a humid region.

I. During the youth stage, the area remains more or less flat. The stream valleys are generally narrow.
II. In mature stage, the relief of the land increases and the landscape is changed into one consisting of hills and valleys. The “relief” of an area may be defined as the maximum difference in the elevation.
III. In old stage, the streams will approach base level and the land will be reduced to a peneplain. The ‘peneplain’ is an undulating plain which lies nearly at base level. Some mounds or hillocks of hard rocks still persist. These hillocks are called “monadsnocks”.

2.11 River Bank Stability

River bank slide possibility can be identified or recognized from the ground conditions such as cliffs, undercut of a river bed, seepage zones, contact zones, tension cracks, etc. These ground conditions can be explored from field visits on the site and for large valleys may be from aerial photo interpretations.

Although at the time of river bank investigation all conditions appear normal but the flank sliding can be triggered by earthquakes, heavy rainfall, river flow submergence, blasting, etc. The sliding can be in the form of falls, topples slides or flows. If the above river bank sliding possibilities are identified or analyzed in the project site, mitigation measures shall be sought before the occurrence of the problem. These remedial measures include:

- Changing the geometry of the sliding mass. For example, reduce the driving soil mass from the bank, i.e., reduce the driving moment;
- Drainage arrangement. Water reduces the shearing strength that can be developed between the grain particles on the failure plane. If inclined drilled boreholes are bored along the drainage slope or surface trench drainage are provided on a susceptible landmass, the effective stress along the assumed failure plane will be increased thereby increasing the resisting shear force;
- Toe support. For small land sliding mass, a retaining structure at toe can be provided by anchoring below the assumed failure plane, and
- Slope reinforcement by bolting, anchoring, etc.

Therefore, appropriate technique could be adopted based on the type of instability, the seriousness of the problem and the availability of funds.

2.12 Behaviors of Rivers and Suggested Bank Protections

The behavior of rivers are generally governed by the longitudinal slope (or velocity), width of the waterway, the sediment load amount and type, magnitude of the flood, the stage of the river, etc. As discussed in the works of streams (rivers) in the aforementioned chapters, the general activities and effects on the channel are well treated, however, in this section; it is worthwhile to make some important remarks on bank protections methods depending upon the river behaviors on the various reaches.

River protection works depend mainly upon the type of river, its regime and upon the characteristics of its flow. There are no two regions or places of rivers with the same hydrology and there are no two floods/rivers with the same characteristics. The protection works, therefore, vary from place to place and from time to time and also depend up on the finance availability at that time. A particular method of river protection, which has been successful in a particular reach on a particular river, may not be the best solution for other rivers or of for other reaches of the same river.

Hence it is important to note that no particular method of river protection can be stated to be the best. It all depends upon the exigencies arising out of a particular problem. However, in general, it can be stated that no river protection method is of immense use in isolation unless; it is supplemented by other methods. Having the aforesaid precautions, Table 2-1 may be of help and guide to practitioners.

Table 2-1: River Reaches, Behavior and Suggested Protection Matrix.

S. No	River reach or type	Behavior	Suggested bank protection methods	Some design parameters, if any
1	Straight reach	High velocity in the middle, lower water level at the middle and higher water level at the edges with eddy eroding currents.	Guide banks on both sides of the river (gabions, stone pitching, etc). If the river bed is erodible submerged banks (or sills) shall be placed across the bed at reasonable distance between them with their top levels at or slightly below the designed bed level.	It is a preferable place where weirs, barrages and bridges to be placed. Length of guide banks for upstream portion, $L_{u/s}$, is variable with design flood and width of the river (W), according to Gales (Garg S. K, 1989). $L_{u/s} = 1.25W$ for $Q \leq 20{,}000m^3$ $L_{u/s} = 1.5W$ for Q□ $20{,}000m^3$. For downstream parts $L_{d/s} = 0.25W$
2	Bends	Bending portions of a river are characterized by scouring on the concave side and silting on the convex side. The scouring and silting in a bend may continue due to the action of the centrifugal force exerted on upon the flowing water. Moreover, at the bottom, the velocities are much less than at the top, and enough centrifugal force is not available to counteract the tendency water at the top to move inwards. Hence the water dives in, from the top at the concave end (causing scouring), and moves at the bottom towards the concave end causing deposition.	Avoid putting any possible engineering structures (bridges, culverts, weirs, etc) on river bends because the waterway is mostly unstable and in a more dynamic processes. Depending upon the energy of the flood, the waterway width and space availability for protections structures methods like downstream aligned deflecting groynes, artificial cut-offs on the convex parts of meandering river (shortening the course of the meandering), guide banks, pitching of banks or combination of the above methods may be used.	Too long groynes on erodible rivers may easily be damaged. In such cases successive groynes with increasing length may be adopted. The spacing of groynes (S) depends on • The length of the groynes (i.e., the protection width); • The bank type (larger spacing is required for groynes located on convex banks, S = L, and smaller one for concave banks, S 2 to 2.5L); • The width of the river (wider for wide rivers and smaller spacing for narrow river channels); • The type of groynes Banks of bending rivers can be protected (depending upon the size and the energy of the river) by stone pitching or concrete blocks or brick lining or by growing vegetable covers.
3	Deltas and flood plains	They are the lower reaches of rivers and mostly characterized by depositional land forms.	There is no single hard-and-fast solution for this reaches, however, the possible flood protections measures are forming river banks from the available deposits, regular maintenance and dredging, although expense, to keep flow of the water on the same channel.	Any engineering structures on this reach shall be resistant to abrasion of the sediment load and top free draining (for example, ford type of crossing the river).

3. GEOTECHNICAL SURVEYS AND EXPLORATIONS

3.1 Purpose of the Geotechnical Investigations

A natural stratum is quite unlike any other material of construction to man. Most of the commonly used materials of construction such as wood, steel, concrete, or reinforced concrete are capable of proper structural analysis once a few simple and well chosen physical and mechanical parameters are known. One can select the material which best meets the prevailing conditions and then determine the allowable stresses or other design parameters for that material. On the other hand, no choices of stratum is normally available to a geotechnical engineer because it is already on the construction site and also the engineering properties of the stratum is very complex which are attributed by a number of factors including mineralogy, structures, compositions and other geological facts.

Although, the degree of a geotechnical investigation mainly depends on the nature and size of the superstructure and/or substructure, and on the complexity of the substrata, the following objectives are behind ever geotechnical explorations:

- Surface data (topography, landforms, land covers, etc),
- Subsurface information with the help of pits, trenches, adits, shafts, boring, drilling, geophysical methods, field and lab tests,
- Sources, quantities and qualities of construction materials (rocks, gravel, sand, earth, etc.).

These surface and subsurface data enable the designer to develop and design sub or superstructures which satisfy safety, serviceability and economy.

3.2 Review of Previous Studies

Surveys and analysis of any pervious works such as maps, aerial photos, core archives or other relevant documents at or near the particular site are very important during the desk work of the reconnaissance and feasibility stages.

3.2.1 Maps

Maps are means for source of information for engineering or other purposes. They are representation from top views with some supplementary details of symbols or notes to indicate the three dimensional state. These maps can be topographic, geologic, agricultural, etc., with their specific purposes and methods of presentation.

(i) Topographic maps.

Topographic maps are divided by parallel latitudes and longitudes with contours and other explanatory legends. Roads, big river, towns, historic and other important features can be shown on the topographic maps. These maps can be obtained from Ethiopian Maps Agency (EMA) with various scales. For engineering purposes, scales of 1:50,000, 1:100,000 and 1:250,000 are available. For feasibility and detail studies, a separate site surveys shall be done to the site of anticipated structures with suitable contour intervals.

Topographic maps are of considerable value in the exploration of foundations and construction materials for flood hydraulic structures. The locations and elevations of exploratory holes, outcrops, and erosion features can be placed on the map, and the landforms portrayed by the contours indicate to some degree the type of soil and subsurface geologic conditions. Furthermore, on the purchased topographic maps, the alignment and rough designs of flood protection works and appurtenant structures can be placed for cost estimates of the reconnaissance or prefeasibility stages.

(ii) Geological maps

Geological maps show the rock units, formations, important lineaments and other geological features. The maps use letter symbols, graphics, colors and other legends to depict the formations, lithology and the discontinuities. One or more geologic sections can be found in a geologic map for indicating depth wise interpretations.

Considerable useful engineering information is obtained from previous geologic maps. These maps indicate the soil type, rock type and depositional materials on the project area. Experienced engineering geologist in association with the geotechnical engineer can predict the engineering parameters for subsequent studies and designs.

For feasibility and detail designs, a separate engineering geological study must be made for the site. In area of complex geology, several types of geologic maps may be required such as surficial geologic maps, structural geologic maps, hydrological maps, geomorphologic maps, etc.

(iii) Agricultural soil maps

The agricultural soil maps are surficial extending to a depth not greater than 2m, and consist of classifying soil according to color, structure, texture, physical constitution, chemical composition, biological characteristics, morphology and their suitability for various crops.

In order to apply agricultural soil maps to explorations of foundations and construction materials, knowledge of pedagogical system of classification is necessary. The three distinct horizons (A, B & C) and their causes (leaching) must be understood. The textural classification of ‘A’ horizon, agricultural soil terminologies and their equivalent engineering soil names and other important data on the maps can give quantitative information for engineering use.

3.2.2 Aerial Photos

An aerial photograph is a picture of the earth surface taken from the air. Area of a certain region can be photographed from the top with the help of moving plane in a strip of overlapping portion. The photographs used for most engineering purposes are vertical photographs in which the axis of the camera is perpendicular to the surface of the earth or nearly so, however, sometimes oblique aerial photos may be used to identify complex geologic structures. These aerial photos can similarly be purchased from EMA.

Aerial photos interpretation of earth materials and geologic features requires experience. The diagnostic aspects in aerial photo interpretation include terrain position, topography, drainage pattern, erosional features, color tones and vegetative covers. Interpretation is limited mainly to surface and near surface conditions.

Unless the area is covered by dense vegetation, the aerial photographs can reveal manmade and natural surficial features. Important engineering geological characteristics such as vegetation cover, river channels, ridges and valleys, joint systems, landslides, fault zones, folds, dykes and other geologic structural features can easily and quickly be identified from the photos. For engineering and geological interpretation of surface material, a scale of 1:20,000 has been found satisfactory. Stereoscopic examination and identification of the above features are used for detailed engineering works.

3.2.3 Other Documents

In the review of previous studies of reconnaissance or feasibility stage, data collection and information gathering are necessary to enrich the available design parameters of the geotechnical work at hand. Some of the geotechnical information sources are (a) groundwater depth, logging and permeability characteristics of the information from nearby boreholes for water supply or other purposes; (b) core archives of the site or near the site of any prior studies and (c) earlier geotechnical studies for roads, building and bridges.

3.3 Surface Investigations

In addition to the information obtained in the office work of the reconnaissance stage, subsequent field trip is necessary in locating sources of construction materials and in making a general foundation condition assessment. The field visit also helps to confirm the knowledge obtained by aerial photos and maps of the deskwork. Elementary types of field-testing can also be conducted.

3.3.1 Visual Observations

In visual observations, long radial walkover visits and understanding of the geological processes in the formation of soil deposits are very important for correct interpretations and extraction of the necessary data. Such geological phenomena include weathering, transportation of soils by different agents (water, wind and gravity) and the transformation of the deposits.

The visual observations concentrate in finding the construction materials in the different reaches of transportation agents and exploration of the foundation conditions from exposed surface as follows.

(i) Alluvial fans and cones

When streams or flood gullies gradient abruptly change from steep to flat at the juncture of valleys or plains, deposition will take place. Coarse materials are deposited first at the head of the fan while the fines materials at the outer edge forming alluvial cones and fans over large area. Therefore, sands and gravels can be obtained for concrete, shell and transition materials for flood protection structures. But the common drawbacks are the presence of boulders in the upper reaches and being skip graded for the lower edges. Sieving and blending may give satisfactory construction materials. As foundations of flood protection works, they shall be looked precariously because they induce seepage and settlement problems. The sands and gravel on these areas also entail low shear strength and un-reworked.

(ii) Valley fills

In a braided stream (a stream with interwoven channels, constantly shifting through islands of alluvium and sandbanks and with a wider and shallower cross-section), sands and gravels of good qualities can be obtained. Whereas in meandering streams of broad valleys, fine-grained soils such as silts and clays can be extracted (see Chapter-2).

The presence of a high water table is a major difficulty in the use of these deposits, especially as a source of impervious material because the water can wash-down during borrowing. As a foundation base, these deposits are usually questionable and thorough investigations are required.

(iii) Lakebeds

Old lakes' beds but with no water currently or partly dried can produce impervious cores for earth fill embankments and for impervious linings. Lake sediments or lacustrine deposits are the result of sedimentation in still water. Except near the edges of the deposits where alluvial influences are important, the materials are very likely to be fine-grained silt and clay. They are recognizable by their flat surfaces and surrounded by high ground.

These deposits are mostly considered good foundation with regard to their imperviousness and poor foundation in terms of their compressibility. For small and flexible structures, they can be founded on these deposits with precautions and high allowance of settlement factors.

(iv) Residual soils

Residuals soils are those which have developed and remained in place over the rocks from which they are formed. They can be recognized by a profile showing a gradual transition from totally decomposed soil through partly decomposed rock, fractured and fissured rock to bedrock. They are formed by sever tropical weathering, secondary chemically reaction and leaching.

Clay minerals like montmorillonite, kaolinite, halloysite, allophone, etc., are present in residual soils. They are partly derived from the parent rock and partly due to the severe weathering.

The engineering properties of these soils depend mainly on mineralogy, texture and the percentage of each clay mineral. For instance, residual soils with considerable percentage of montmoillonite are poor for foundation or as construction materials and those soils having kaolinite minerals such as lateritic soils are satisfactory for foundation, core and lining purposes.

(v) Eolian Deposits

Soils deposited by wind are known as eolian deposits. They are two types; sand dunes and loess. Sand dunes are recognizable as low elongated or crescent shaped sand hills with flat slope windward and steep slope leeward of the prevailing winds. Usually with little vegetative cover, rich in quartz, poorly graded, no cohesive strength, moderately high permeability and compressibility. Whereas loessial deposits are windblown dust covering extensive area. They consist of clay, silt and fine sand contributing as a binding agent. Both types of eolian deposits can be used with high precautions for construction materials and with thorough investigations and suspicion for foundation of structures.

(vi) Outcrops, natural and manmade cuts

Outcrop refers to the total area over which a particular rock until exists at the surface whether visibly exposed or not. In visual observations, outcropped rocks around the construction sites are sought to study their suitability and quantity for construction materials of the proposed structures and to check the suitability of the site as foundation base.

Naturally cuts (walls of gullies, land sliding surfaces, fault surfaces, etc.,) and manmade cuts (roads, pits, previous foundation excavations, etc.) also depict depth-wise stratification and lithology of the ground. These exposures help to estimate the engineering properties for design purposes.

On outcrops, natural and manmade cuts; geological structures with their orientation like fissures, joint systems, discontinuities and unconformities must be examined so that they will enable the investigator to predict the permeability and bearing capacity of the foundation.

3.3.2 Preliminary Sampling and Testing

In preliminary sampling and simple testing, qualitative data can be obtained in the early stage of the investigation phase. Experience and simple hand tools or materials are used for soils/rocks identification and hence prediction of the engineering properties. Tools or materials like hammer, knife, pick, jar, hand augur, water, acid, our hands and human feelings are used for describing and identifying construction materials and foundation characteristics.

(i) Rock hardness

Rock hardness classes can be estimated in the field as shown in Table 3.1. These simple surface sampling and testing are conducted on outcrops and exposed riverbed rocks.

During field sampling and testing, it is customary to describe the rocks encountered by origin; color, grain size and composition (see Table 6-1).

Table 3-1: Hardness Classification of Intact Rock

Class	Hardness	Field test	Approx. range of uniaxial compressive strength [Kg/cm2]
I	*Extremely hard*	*Many blows with geologic hammer are required to break intact specimen*	*>2000*
II	*Very hard*	*Hand held specimen breaks with hammer and of pick under more than one blows*	*2000 -1000*
III	*Hard*	*Cannot be scraped or peeled with knife. Hand held specimen can be broken with single moderate blow with pick*	*1000 -500*
IV	*Soft*	*Can just be scraped or peeled with knife. Indentations 1mm to 3mm show in specimen with moderate blow with pick*	*500 -250*
V	*Very soft*	*Material crumbles under moderate blow with sharp end of pick and can be peeled with a knife, but is too hard to hand-trim for triaxial test specimen*	*250-10*

(ii) Acid test

The acid test is used for identification of rocks of lime stone and joint filling in rocks by calcite. In soils, the acid can also be used for checking whether the cementing materials are calcium carbonate or not. If calcium carbonate is present in the above substances, an effervescence reaction will take place indicating the presence of the mineral. Dilute hydrochloric acid, HCL (1:3, i.e., one part concentrated HCL and three parts of distilled water) is used for this kind of preliminary testing. Care should be taken in using HCL and rinse with tap water if it comes in contact with hand.

(iii) Identification for coarse-grained soils

Soils containing more than 50% individually visible particles are coarse-grained soils. Soils containing less than 50% individually visible particles are fine-grained soils. Table 3-2 shows the classification and average particle diameters of soils. Coarse and fine-grained soils can be identified using different techniques as shown below.

a) Using settlement techniques:

Place a sample in a glass of jar or a medicine bottle with straight edges. Put in some water about 10 cm above the top of the sample in the jar. Shake vigorously and then stand to allow the soil to settle. Gravel and coarse sand will settle immediately, fine sand and coarse silt more slowly, taking about 30 seconds. Any clay and fine silt fraction will remain in suspension for some hours before settling.

The approximate quantities of each size (coarse, medium and fine) can be visually seen as layers in the jar. The depth of each layer can be measured and percentage can be estimated proportionally to the depths.

b) Vibration test (alternative method):

Place a dry and pulverized soil sample on a board or piece of stiff card. Hold the card at a gentle slope and tap lightly with a pencil or stick. The finer material with move up the slope or remain in place, the coarser will move down the slope. Then a rough estimate of each percentage can be made to classify the soil at hand.

Table 3-2: Classification and Particle Diameters of Soils

Soil	**Stone**		**Gravel**		**Sand**			**Silt**	**Clay**
	Boulders	*Cobbles*	*Coarse*	*Fine*	*Coarse*	*Medium*	*Fine*		
Diameter (mm)	*200*	*76*	*19*	*5*	*2*	*0.4*	*0.074*	*< 0.002*	*< 0.002*
			Coarse					**Fine**	

(iv) Identification of fine-grained soils

Preliminary field identifications of fine-grained soils are to be performed on particles less than 0.4mm (fine sand and lesser diameters). In this classification, screening is not intended and simply removed by hand the coarser particles that interfere with the test. The following simple field tests can be performed on fine-grained soil for further classification of the fine soils.

a) Dry strength (crushing characteristics):

After removing particles larger than 0.4mm sieve size, mold a pat of soil to the consistency of putty, add water if necessary. Allow the pat to dry completely by oven, sun or air and then test its strength by breaking and crumbling between the fingers or simply find lumps of dry sample and break between fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

Dry strength is the characteristics of clays of high plasticity (CH). Typical inorganic silts have only a slight dry strength. Silty fine sands and silts have practically the same low dry strength but can be distinguished from each other by their feeling during powdering of the dried sample.

b) Dilatancy (reaction to shaking):

After removing particles larger than 0.4mm size, prepare a pat of moist soil with a volume of about 5cc. Add enough water to nearly saturate the sample. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. Squeeze the pat between fingers. The appearance and disappearance of the water with shaking and squeezing is referred as a reaction.

This reaction is called quick if water appears and disappears rapidly; slow, if water appears and disappears slowly, and no reaction, if the water condition does not appear to change.

Therefore, very fine clean sands give the quickest and most distinct reaction whereas, plastic clay has no reaction. Inorganic silts show a moderately quick reaction.

c) Toughness (consistency near plastic limit):

The soil sample used in the dilatancy test is dried by working and molding until it reaches the consistency of putty. The time required to dry the sample is indicative of its plasticity. Further, the moisture content is reduced by rolling and rerolling on a smooth surface or between the palms into 3mm-dimaeter thread till it reaches plastic limit. The resistance to molding at the plastic limit is called toughness. After the thread, crumbles, the pieces are lumped together and slight kneading action is continued until the lump also crumbles. The tighter the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays which occur below the A-line. Highly organic clays have a very weak and spongy fell at the plastic limit.

d) Organic content and color:

Fresh, wet organic soils usually have a distinctive odor of decomposed organic matter. This odor can be made more noticeable by heating the wet sample. Another indication of the organic material is the dark color.

When the field identification is supplemented with laboratory classification criteria, the soil can be well described and the engineering properties can be well ascertained. The Unified Soil Classification System, USCS (Fig. 3-1) gives a best aid in soil identification.

The USCS is a self-explanatory tabular presentation for identifying and classifying soils in engineering uses. It classifies into two major groups, namely coarse and fine-grained soils. The classification is based on field and laboratory identification procedures. It consists of typical names and group symbols with a prefix or a suffix as necessary (Table 3-3).

Table 3-3: Soil Nomenclatures

Soil type	Prefix	Sub-group	Suffix
Gravel	*G*	*Well graded*	*W*
Sand	*S*	*Poorly graded*	*P*
Silt	*M*	*Silty*	*M*
Clay	*C*	*Clayey*	*C*
Organic	*O*	*W1<50%*	*L*
Peat	*P*	*W1>50%*	*H*

The sub-groups are well identified by laboratory results from gradation and consistency tests. Specially for fine grained soils, the subgroups are identified from the plasticity chart (a graph of plasticity index against liquid limit). The A-line (an equation $I_p=0.73(W_l-20)$ separates the more clay materials from those that are silty and also organic from the inorganic. Silt, clay and organic fractions are further subdivided into low (L) or high (H) plasticity when the liquid limit (Wl) is less than 50%

and more than 50% respectively.

Soils consisting the characteristics of more than one group can be termed as boundary soils and hence are assigned dual group symbols, e.g., GW-GC meaning well graded gravel with some clay fines. If the plot of liquid limit versus plasticity index falls on the A-line or the line Wl=50 shall be assigned the appropriate borderline classification. Soils which plot above the A-line or practically on it and which have a plasticity index between 4 & 7 are classified as CL-ML.

(v) Visual and manual soils identification reporting format

After visual observations and simple manual identification of the soils, the borrow materials or foundation soils can be reported with the format shown by Table 3-4.

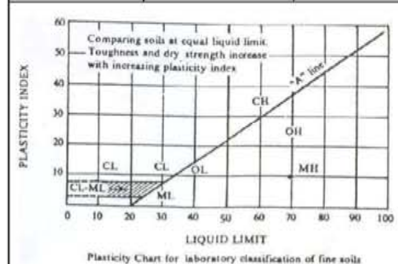
Table 3-4: Data Form for Visual and Manual Soil Identification

Project: ____________________ Table No. _____ Sheet: __ of __
Prepared by: ________________ Date __________________

Identification				Gradation (Estimation)				Color (wet state)	Soil Description	Group Symbol
Sample No.	Hole No.	Location or station	Depth (m)	Max. size (mm)	Gravel (% plus 5mm)	Sand (%mm to 0.074 mm)	Fine (% minus 0.074mm)			

Figure 3-1: Unified Soil Classifications Including Identification and Description (R. Gopal et al, 1991)

Figure 3.1 Unified Soil Classification Including Identification and Description

				Field Identification Procedure (Excluding larger than 3 sq-inches and basing fractions on estimated weights)			Group Symbols	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
COARSE-GRAINED SOILS More than half of material is larger than No. 200 sieve size[2]	(The No. 200 sieve size is about the smallest particle visible to the naked eye)	Gravels More than half of coarse fraction is larger than No. 4 sieve size	Clean gravels (Little or no fines)	Wide range in grain size and substantial amount of all intermediate particle size			GW	Well graded gravel-sand mixtures, little or no fines	Give typical name: indicate approximate percentages of sand and gravel, max. Size, angularity, surface condition and hardness of the coarse grains, local or geologic name and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. Example: Silt sand, gravelly, about 20% hard angular gravel particles ½-in maximum size, rounded and sub angular sand grain coarse to fine, about 15% non plastic fines with low dry strength , well compacted and moist in place, alluvial sand (SM)	Use grain size curve in identifying the fractions as given under field identification	Determine percentage of gravel and from grain size curve depending on percentage of fines (fraction smaller than NO. 200 sieve size). Coarse grained soils are classified as follows: Less than 5% GW, GP,SW SP; More than 12% GM, GC, SM, SC; 5% to 12% Borderline cases requirement use of dual symbols	$Cu = D_{60}/D_{10}$ greater than 4 Cc = between 1 & 3	
				Predominantly one size or a range of size with some intermediate sizes missing			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines				Not meeting all gradation requirement for GW	
			Gravels with fines (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below			GM	Silt gravels, poorly graded gravel-sand-silt mixture				Atterberg limits below "A" line or ωp greater than 4	Above "A" line with ωp between 4 and 7, are border line cases requiring use of dual symbols
				Plastic fines (for identification procedures see CL below)			GC	Clayey gravels, Poorly graded grave-sand-clay mixture				Atterberg limits below "A" line or ωp greater than 7	
		Sands More than half of coarse fraction is larger than No. 4 sieve size	Clean sands (Little or no fines	Wide range in grain size and substantial amount of all intermediate particle size			SW	Well graded sands, gravelly sands, little or no fines					
				Predominantly one size or a range of size with some intermediate sizes missing			SP	Poorly graded sands, gravelly sands, little or no fines				Not meeting all gradation requirement for SW	
			Sands with fines (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)			SM	Silty sands, poorly graded sand-silt mixtures				Atterberg limits below "A" line or ωp less than 4	Above "A" line with between 4 & 7, are border line cases requiring use of dual symbols
				Plastic fines (for identification procedures see CL below)			SC	Clayey sands, poorly graded sand-clay mixtures				Atterberg limits below "A" line or ωp less than 7	
		Identification Procedures of Fraction Smaller than No. 40 Sieve Size											
FINE-GRAINED SOILS More than half of material is smaller than No. 200 sieve size[2]				Dry strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near plastic limit)							
		Silts and clays Liquid limit less than 50		None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. Example: Clayey silt, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes; firm and dry in place loess (ML).				
				Medium to high	None to very slow	Medium	CI	Inorganic clays of low to medium plasticity, clays, sandy clays, silty clays, lean clays					
				Slight to medium	Slow	Slight	OL	Organic silt sand, organic silt-clays of low plasticity					
		Silts and clays Liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts					
				High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays					
				Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity					
		Highly organic soils		Reedley identified by color, odor, spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils					

Plasticity Chart for laboratory classification of fine soils

1 Boundary classifications: Soils possessing characteristics of two groups are designed by combination of group symbols, for example, GW-GC, well graded gravel-sand mixture with clay binder.
2 All sieves on this chart are U.S. Standard which is the same also for Ethiopia

3.4 Subsurface Investigations Methods

3.4.1 Excavations

The common subsurface investigations using manual or machine hand excavation methods are test pitting, trenching, auguring, adits, etc. The detail methods of execution of the investigations are as described below.

3-4.1.1 Test Pits

Test pitting is an effective means of exploring and sampling earth foundations and construction materials. It exposes the surface for visual examinations, and helps in sampling and testing. The depth of test pitting is generally governed by the investigation requirement but the nature of the soil, groundwater table depth and the equipment and other facilities used during pitting are also have high impacts on extending the depth. Maximum and economical hand-dug depth is about 5m. Deeper test pits shall be done by mechanical equipment. In deep test pits, possibility of side collapse and dead air problem prevention shall be considered (i.e., cribbing and ventilation, respectively). The minimum top width is about 1mx1.5m, but depending upon the depth of excavation, the dimension can be increased to 1.5mx2m and even higher. Whereas the spacing depends upon the uniformity of the material, types of the superstructure and appurtenances.

When water is encountered in a pit, a suitable dewatering system may be required for further progress. Small and portable pump can be used for this purpose.

During excavation, the bottom of the pit shall be kept fairly level and of full section so that each lifts may represent the corresponding portion of the deposit in quality and quantity. The excavated material shall be placed round the pits, preferably clockwise, as stockpiles separated when significantly different materials are encountered and marked stakes shall be driven into the stockpiles to indicate the depth from which the materials were excavated in order to facilitate logging and sampling later on.

Preliminary sampling and field testing, sampling and logging of the soil shall be done for test pitting. Test pit-logging format of Fig 3-2 & Fig 3-3 may be used for stratification representations. The rocks and soils can be described as indicated in indicated in Tables 3-5 to 3-8.

Test pits left open for inspection shall be provided with covers or barricades for safety. Pits and trenches shall be suitably fenced. They shall be filled back properly when explorations are completed.

Minimum types of tools for manual pitting in each group (usually three persons) required are 1 shovel, 1 crowbar, 1bucket, 1 pick, a suitable length rope, pulley arrangements, dewatering system (if any) and materials and tools for sampling.

Table 3-5: Weathering Classification of Rocks

Grade	Class	Diagnostic features
I	*Fresh*	*No visible sign of decomposition or discoloration. Rings under hammer impact*
II	*Slightly weathered*	*Slight discoloration inward from open structure, otherwise similar to fresh.*
III	*Moderately weathered*	*Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength obtained is less than fresh rock but cores cannot be broken by hand or scraped by knife. Texture preserved.*
IV	*Highly weathered*	*Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in less than half mass. Texture becoming indistinct but fabric preserved.*
V	*Completely Weathered*	*Mineral decomposed to soil but fabric and structure preserved. Specimens easily crumbled or penetrated.*
VI	*Residual Soil*	*Advanced stat of decomposition resulting in plastic soils, rock fabric and structural completely destroyed.*

Table 3-6: Grain Size Description of Rocks

Grain Size Description	**Rock Type**	
	Igneous and metamorphic rocks, grain diameter (mm)*	**Sedimentary rocks, grain diameter (mm)***
Very coarse-grained	*>60*	*>60*
Coarse-grained	*2 to 60*	*2 to 60*
Medium-grained	*0.06 to 2*	*0.06 to 2*
Fine- grained	*0.002 to 0.06*	*0.002 to 0.06*
Aphanitic (very fine for sedimentary rock)	*Grain size is too small to be perceived by unaided eye.*	*<0.002*
Glassy	*No grain form can be distinguished*	

Table 3-7: Joint Spacing Description of Rocks

Joint description	**Spacing of joints (the average distance between the individual joint measured perpendicularly)**	**Fracture Index (FI) = No. of fracture per meter of core**
Very close, VC	*<50mm*	*>20*
Close joint, C	*50 to 300 mm*	*4 to 20*
Moderately close joint, MC	*300 to 1000mm*	*1 to 3*
Wide Joint, W	*1000 to 3000 mm*	-
Very wide joint, VW	*>3000 mm*	-

a) Lithologic descriptions of rocks:

The lithologic descriptions encompass:

- color;
- weathering Condition (Table 3-5);
- preliminary strength (Table 3-1);
- grain size (Table 3-6);
- laboratory and field strengths (uniaxial compressive strength, point load strength, velocity index, RQD, etc),
- basic geological rock type, local name, etc.

b) Discontinuities in rocks:

The discontinuities shall be described as joints, fractures, fissures, shears, veins, seams, bedding, foliation of drilled induced. Other like:

- spatial orientation (dip and strike);
- extent (percentage of discontinuity coverage over rock surface);
- spacing (Table 3-7);
- opening (the separation distance of the joint between the rock faces can be presented as maximum and minimum opening);
- filling (breccias, gouge, calcite, etc);
- roughness (very rough, rough, smooth, polished, etc).

c) Soil description:

The description of a soil in a logging may include:

- typical name (as gravel, sand, silt, clay and combination);
- approximate percentage of each soil;
- max. size of gravel;
- shape of coarse-grained soil (as angular, sub-angular, sub-rounded and rounded);
- hardness of the coarse grained soil;
- color (use basic color and the presence of molting and banding);
- odor (as none, earthy or organic);
- moisture content (as dry, moist, wet and saturated);
- plasticity (as none, low, medium and high);
- structure (as stratified, lensed, heterogeneous, etc);
- cementation (as weak, moderate and stone. Note the reaction to HCL);
- local or geologic name;
- group symbol (Fig 3-1).

Table 3-8: Composite Soil Types and Its Description

Term	**Composition of the coarse fraction**
Slightly sandy gravel	*Up to 5% sand.*
Sandy gravel	*5% to 20% sand.*
Very sandy gravel	*Over 20% sand.*
Gravel/sand	*About equal proportions of sand and gravel.*
Very gravely sand	*Over 20% gravel*
Gravely sand	*20% to 5% gravel*
Slightly gravely sand	*Up to 5% gravel*

Fig 3-2: Typical Log Format for Pit/Trench

Log of Test Pits/Trenches

Project:…………………………………………………

Purpose:………………………………………………..

Location:………………………………………………..

Coordinates: …………………………………………

Surface elevation: ………………………………….

Date:…........(started)………………(completed)

Test pit no. …………..

Pit dimension: ………

GWT ……………………

Weather condition:……………………………….

………………………………….

Graphic log	Depth (cm)	Samples: DS = Disturbed UD = Undisturbed	Soil Description	Remarks

Digging crew: ……………………………………

Supervisor: ………………………………………..

Logged by: ………………………………………..

Approved by: ……………………………………

Client: …………………………………………

Consultant: ……………………………………

Contractor: ……………………………………..

3.4.1.2 Trenches

Purpose wise, trenches are similar to test pits in all respects except that they are continuous over a length and provide a continuous exposure of the surface along a desired line or section. They are best suited for exploration on sloppy ground and also in studying following some geological structures.

The fieldwork consists of excavation and open trench form the top to the bottom of the slope to reach representative undisturbed material. Either a single slot trench down the face of the slope or a series of short trenches spaced at appropriate intervals along the slope may be excavated. Depending upon the extent of investigation required, use may be made by manual excavation tools, bulldozers, backhoe or dragline.

Similarly; logging, sampling, preliminary testing, etc., on the soil can be done as that of on test pits (Fig 3-2).

3.4.1.3 Drifts/Tunnels

Drifts or tunnels are normally employed to explore at depth the continuity or character of surface formation. They are most frequently used for the investigation of fault or shear zone, buried channels and suspected places of weakness in foundation, abutments and beneath steep slopes or back of cliff like faces to determine extent of weathering, shear zone and bed rock configuration in areas of fossil valley. A rectangular section with minimum clear dimension of 1.5m width and 2m height is adopted in hard rock. In soft rocks, side and roof supports are required.

They are also used for taking undisturbed samples of rock for tests in the laboratory and for performing in situ tests to determine the modules of elasticity and deformation of rock formations and shear, etc, required to study the properties of the rock.

Logging and sampling of exploratory drift shall proceed concurrently with excavation operation. They should be mapped giving direction of the dip, fault zones, shear zones and seams, etc. Level location and piezometric heads of seepage flows, if any, occur shall be recorded date-wise.

3.4.2 Auguring

3-4.2.1 General

Auger borings often provide the simplest method of soil investigation and sampling. They may be used for any purpose where disturbed samples are satisfactory and are valuable in advancing the holes to depths at which undisturbed sampling or penetration testing or permeability testing is required. Depths of auger investigations are, however, limited to by the ground water table and by the amount and maximum size of gravels, cobbles and boulders as compared to the size of equipment used.

In auguring, boring is done by turning the auger to the desired distance in the soil, withdrawing it, and removing the soil for examination and sampling. The auger is inserted into the hole again and the process is repeated. Auguring is entirely done for soils of soft or hard in nature.

3-4.2.2 Hand Augers

Hand operated augers may be used for boring holes to a depth of 6m in soft soils which can stand unsupported. Depending upon the shape of the cutting edges and sample holding arrangements, hand augers can be helical, Iwan type, Fenn type, etc.

3-4.2.3 Power Driven Augers

They are used for greater boring depths (say about 100m) or where hard of stiff soil strata are encountered. These machine-driven augers can be helical type or disc type or bucket type depending upon the arrangement of the cutting edges and sampling mechanisms. They are available in different diameter sizes.

3.4.3 Boring

3-4.3.1 Core Drilling

Rotary drilling is useful when the soil is highly resistant to auguring and when deep and detailed subsurface information with relatively undisturbed samples and in situ tests are required. In a rotary drilling when the cores cut by the bits are stored and returned in core barrels, the rotary drilling is known as core drilling while alternatives of rotary drilling like mud rotary and shot rotary drilling are also available. The accuracy and dependability of the records furnished by core drilling depend largely on the size of the core in relation to the kind of materials drilled, the percentage of core recovery, the behavior during drilling and the experience of the drilling crew. For instance, a rock that will core well in a large diameter bit may break up badly in a small hole and hence it is important to use the largest practicable size diameter hole and core barrel. Recovery of rock is much more important than making rapid progress in drilling the holes. Portions of a core that are lost will probably represent shattered or soft, incompetent rock, whereas the recovered portions represent the best rock from which an overoptimistic evaluation of the foundation likely will be made. A reasonably large percentage of core recovery, on the other hand, will provide a more continuous section of the materials passed through. The cores provide information on the character and composition of the different information and lithology with evidence of the spacing and tightness of joints, seams, fissures and other structural details.

In drilling in soft materials, the circulation water must be reduced or stopped entirely even though a marked delay in operation may result. While drilling in hard rocks, sufficient amount of water can be used to facilitate the rapid advancement of the drilling operation. Furthermore, telescopic drilling can be used in advancing the depth of drilling, i.e., smaller and smaller diameters of holes are used progressively as the drilling operation goes downward on hard rocks. Besides the above points, in rotary drilling color changes and loss of the circulation water shall prudently be attended with depth. The changes are an indication of formation variations and geological structure, respectively.

In rotary drilling important knowledge are required with regard to:

- drilling equipment;
- core barrels;
- bits;
- accessories, etc.

3-4.3.2 Wash Boring

Wash Boring is none sampling boring where the hole is advanced by raising, rotating and dripping the bit into the soil at the bottom of the hole. To prevent cave in, the soil is supported by a casing. In wash boring, the cuttings are removed with continuous flow of water through the annular space between the casing and drill rod carrying suspended soil particles, and overflows at the top of the casing through a T-connection into a container, from which the effluent is re-circulated back with the help of pump through the drill rod. The drop hammer is commonly supported on a tripod and pulley.

The change in soil strata can be surmised from the rate of progress and the change in color of the slurry flowing out. Samples recovered from wash boring are of no value. Sampling with suitable samplers and field testing (e.g. SPT) shall be done after the borehole has been cleaned. Care must be taken to consider the effects of the top moisten soils on sampling or testing. The method is applicable to all types of soils except rock.

3-4.3.3 Percussion Boring

Percussion or churn boring is also a none sampling boring where the drilling is carried out by breaking up the formation by repeated blows of a heavy bit or a chisel inside a casing pipe. Water is added and the cuttings form slurry which is removed intermittently by sand pumps or bailing. The method is suitable in boulder and gravely strata.

3.4.4 Geophysical Methods

A geophysical exploration may be included in a site investigation for an important big flood protection project in order to provide subsurface information from measurements taken on the surface. It can be conducted over large area at reasonable cost. The information obtained may help to eliminate less favorable alternative sites, may aid the location of test holes in critical areas and may prevent unnecessary repetitive drilling (if any) in a fairly uniform ground. Boreholes provide information about the strata where they sunk but tell nothing about the ground in between. Nonetheless boreholes are an essential part of any geophysical survey which aid in interpretation and correlation of geophysical measurements.

Geophysical methods are used to determine the geological sequence, bearing and structure of subsurface rocks by the measurements of certain physical properties or forces. The properties which are made most use of geophysical exploration are electrical conductivity/resistivity, reflection and refraction of shock waves, density, magnetic susceptibility and gravitational attraction of the underlying materials. Out of the different geophysical investigation techniques, seismic and electrical resistivity is very common for engineering uses.

In flood protection projects, seismic or electrical resistivity method may be employed before drilling operation (if any) of for foundation sites depending upon the complexity of the geology and the sizes of the project and the ancillary structures.

Geophysical methods of exploration can be employed mainly after the reconnaissance stage, and in between when reconfirmations are needed. The results of the geophysical methods can be presented as sections (geo-electrical section for vertical electrical sounding, VES), plan maps, pseudo sections, etc. The cost of performing geophysical surveys is very often less expensive than the cost of drilling; therefore, a judicious use of both methods can produce the desired information at an overall lesser cost. Although the geophysical exploration is relatively inexpensive, it needs special training for interpretation of the field readings.

3.5 Seismic Assessment

Earthquakes are caused by strain accumulated along fault planes due to the plate tectonic effect (which is the main source) or by volcanic eruptions, local ground subsidence, bombs, etc. Therefore, earthquake in the proposed flood protection project needs due considerations during the design of the flood hydraulic and other associated structures. However, the associated engineering concerns with earthquake are:

- additional forces or stresses created on structure;
- land displacement, lurching, ground cracking, etc and
- liquefaction needs assessment.

3.5.1 Additional Stresses Due to Earthquake

The horizontal and vertical acceleration of the foundation ground causes both hydrodynamic pressure and horizontal inertia force on the dam sections. The former force can easily be computed from the dam geometry whereas the computation of the latter type needs data from the country earthquake zoning and historic information from geophysical observatories.

The Ethiopian Building Codes of Standard entitled Design of Structures for Earthquake Resistance (EBCS-8, 1995) can be used for small flood protection works where the code gives earthquake zones for a recurrence interval of hundred years. For the different parts of the country the earthquake zone gives, α_h, horizontal bed rock acceleration and multiplying this factor with, I, the structural importance factor, gives the design horizontal bed rock acceleration ration with respect to gravity, g.

EBCS-8 is standardized for 100-yrs return period (life span of most structural buildings); however, big flood protection dams and other huge structures are more serious and highly hazardous than buildings when the problem occurs. Therefore, for more safety reasons higher recurrence intervals need to be assessed, for instance, 200 to 1000-yrs. This in general will shift EBCS's earthquake zoning and the magnitude of bed rock accelerations. Thus, in this case other methods need to be followed or the Addis Ababa University Geophysical Observatory shall be consulted.

3.5.2 Land Displacement due to Earthquake

Reservoir water and heavy rainfall generally reduces the shearing resistance of masses of land in the vicinity of the storage area and hence the earthquake will insinuate large landslide which could cause overtopping of the water above the dam bringing damages to the structure and downstream human life. Therefore, the slope type, any visible tension cracks, big boulders, and any hanging or unstable land masses must be carefully examined specially for flood protection dams.

3.5.3 Liquefaction

Loose saturated sand deposits may lose a part of or all of their shear strength when subjected to a sudden shock or other dynamic loads causing sliding cracks and generally failure of flood reservoirs. Thick sand or sandy soil usually imposes liquefaction or boiling which results in failure of dams called liquefaction.

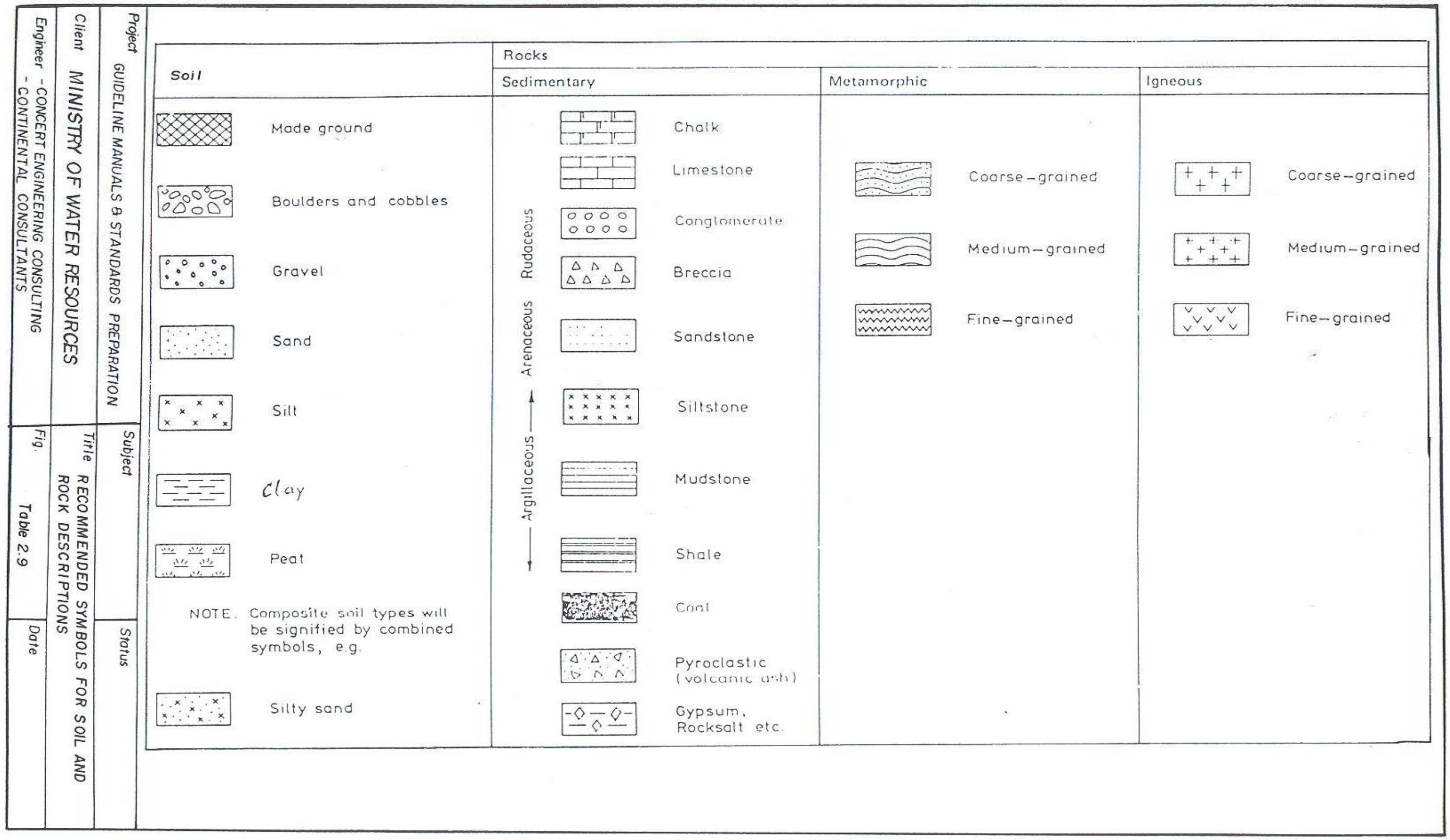


Figure 3-3: Recommended Symbols for Soils and Rocks Descriptions

4. GEOTECHNICAL FIELD TESTS

4.1 Exploratory Hole Locations for Flood Protection Works

In sub-surface investigation, various kinds of exploration methods are used as described in Chapter 3. In a particular site one of a combination of these methods can be employed at a point or several points on a line or many points on an area in parallel or staggered arrangements. The location, number and depth of exploratory holes depend mainly on the following major factors:

- type of the structure;
- size or scale of the project;
- size and type of appurtenances;
- sub-surface conditions;
- type of investigation;
- stage of investigation.

The recommended locations and depth of geotechnical investigation for different types of structures and appurtenances are variable. The investigator what s/he must bear in mind here is that the insinuated proposals are only suggestions. The decision shall be made by the investigator based on engineering judgment to that specific project and geologic complexity. As preliminary suggestions Table 4-1 might be adopted.

4.2 Permeability Tests

A foundation or body of hydraulic structures specially flood protection reservoirs consists of a soil and/or rock mass with void spaces between discrete soils grains or discontinuities in rock mass. These spaces are connected to make a highly complex and intricate network of irregular tubes. These tortuous interconnected tubes are the very concern of the geotechnical engineer because the seepage water creates instability, piping and boggy downstream conditions.

The ease, with which water flows through the soil or rock medium, permeability, is very important in may engineering structures. The perviousness of a stratum can easily be expressed in terms of its permeability coefficient, K, which can be determined in lab, fields or by empirical approaches. The principal advantage that can be gained by field-testing is the attainment of the in situ nature in lieu of the disturbed qualities in lab.

Field permeability test can be further categorized as pumping-out test (or simply pumping text), pumping-in test and observation of the velocity of dye in a moving water. Although pumping-out test is more accurate than any other methods, in water engineering, the pumping-in types (i.e., measured inflow of water accepted by the ground through the open bottom of the pipe or through uncased section of hole) are favored. The pumped-in or the flowing fluid by its weight is mostly water. The tests are invalid and may be grossly misleading unless the water is clean to prevent the plugging of the fissures. Furthermore, the temperature of the water shall be higher than the temperature of the ground so as to preclude the creation of bubbles in the ground which greatly reduce the acceptance of water.

Table 4-1: Proposed Location, Methods, Spacing and Depths of Exploratory Holes for Flood Protection Structures

S. no	Type of structure or material	Locations	Methods of exploration	Spacing of exploration	Depth of exploration	Remarks
1	Borrow fill area	Over the area	Mostly manual excavations (test pits, trenches or augers)	100- 200m	Limited to end of suitable depth or sufficient quantity	For rock masonry source, very limited investigations or visual examinations may be sufficed. For sand and gravel also visual inspection and sampling depth-wise and along the trenches
2	Embankment or dykes	Length-wise over the foundation on the axis	Pitting, trenching, auguring or may be including drilling or in combination.	100-200m	≤H, where H is the height of the embankment	Parameters to be sought here are foundation strength, permeability or piping due to hidden buried channels. If the structure is big geophysical data are very helpful in setting the locations, depths and spacing of exploratory holes.
3	Retaining structures (masonry, concrete, gabions, etc)	Length-wise over the foundation on the axis	Pitting, trenching, auguring or may be including drilling or in combination	200-500m	≥ 1.5B, Where B is the width of the foundation.	
4	Pipes	Along the pipe center line alignment	Pitting, trenching or auguring.	200-500m	≥ 1.5D, Where D is the diameter of the pipe	
5	Flood protection reservoir	Dam axis, foundation area, reservoir area, abutment, spillway, spillway alignment, inlet/outlet , dissipation pool, tunnel, etc if any	Pitting, trenching, auguring and including drilling and geophysical methods or in combination of the above	Variable according to the location	Variable according to the location	Geophysical data are very helpful in setting the locations, depths and spacing of the exploratory holes.

In pumping-in tests, a variety of approaches can be used with special advantages for each arrangement. These are constant head tests, variable head tests and packer tests and infiltration test. The degree of impermeability can be expressed as shown in

Table 4-2 below. The different methods of field permeability or conductivity coefficient determination can be referred as required.

Table 4-2: Expression of Degree of Permeability

Degree of permeability	Coefficient, K(cm/sec)
High	$\geq 10^{-1}$
Medium	10^{-1} to 10^{-3}
Low	10^{-3} to 10^{-5}
Very Low	10^{-5} to 10^{-7}
Practically impermeable	$\leq 10^{-7}$

4.3 Sounding Tests

4.3.1 General

Sounding tests are field tests that are carried out in a bore hole to determine underground information for foundation design purposes. The tests can be effected by static or dynamic pernetrometer for measuring skin friction and point resistance, respectively. In static penetrometer, the rod is pushed into the soil with a constant speed and the corresponding point and surface resistance are measure, while in dynamic penetrometer (where the standard penetration test, SPT, is one), the sounding device is made to penetrate the ground with an impact load.

4.3.2 Standard Penetration Test (SPT)

Out of the many available dynamic penetrometer testing, SPT is the one, which most widely used and it is an economical means to obtain subsurface information. The penetrometer essentially consists of a metal rod with a wedge-shaped tip having a cross sectional area of 10cm2. The metal rod is encased in a metal pipe with specific dimension.

The penetration resistance is expressed as the number of blows of a 63.5kg hammer freely dropping from 76.2cm height to force the sampler 30cm in the soil. SPT shall be done at 1.5 to 3m intervals and at each change of stratum.

(i) Precautions needed:

In conduction SPT, the following major precautions are needed:

- to clean the bottom of hole before test;
- the fall must be the right height (for non automatic trip) and free from friction;
- the sampler must be good, that means, without wear and tear (both the outer and inner walls including cutting edges), shall be clean and slightly coated with oil at the beginning of each test;
- the casing, if any, shall be short of the level of testing;
- the drill rod must be standard and without bends;
- the first 15cm blows (seating drive) are disregarded and the rest 30cm blows is counted for N values. This N values shall be corrected for overburden pressure, dilatency, energy ratios, etc;
- the test shall be halted and the boring log shows refusal if (a) 50 blows are required for any 15cm increment, (b) 100 blows, for the 30cm, are obtained and (c) 10 successive blows produce no advance. The penetration resistance shall be recorded as (No. of blows)/d, where d is the actual depth of penetration during refusal.

(ii) Corrections of N values:

It can easily be observed that N values performed in adjacent boreholes by different equipment cannot give similar values. This suggests standardization of SPT to some energy ration to have reproducible N values for all equipment that are manufactured in different geographical locations. Furthermore, this N value also depends on the depth of borehole because the value is influenced by the overburden pressure. Therefore, the number of blows, N, shall be corrected as N' for a number of factors before using for design values. These influencing elements are:

- overburden pressure;
- sampler, rod length and borehole diameter corrections, and
- some other factors.

One of the most common methods for combined correction is the one which amend for rig accessories at the standard energy ration of 70% and to over burden pressure (Bowles, 1988). The standard corrected blow count can be computed from the measured N values as follows:

$N'_{70} = C_n * N * n_1 * n_2 * n_3 * n_4$ .... (Eqn. 4-1)

Where N'_{70} = Adjusted N for standard energy ratio

C_n = Adjusted for overburden pressure computed as $C_n = [P''_o / P'_o]^{1/2}$

P''_o = 95.76KPa (or 1kg/cm2) and P''_o is the overburden pressure to the same unit

n_i = Adjustment factors from the rig accessories.

(iii) SPT correlation

Geotechnical engineers are often expected to give predictions of strata behaviors even when little or no relevant test results are available. Thus, SPT has been used in correlating for unit weights, relative density, angle of internal friction, undrained compressive strength, allowable bearing capacity, modulus of compressibility, etc.

Different literatures give correlation in the form of tabular presentation, empirical equations, charts, etc. However, putting all these relationships into this manual is not only redundant but also may cause confusion and misuse. Moreover almost all these correlations were established for transported soils in that particular country which may not be adopted for all geographical regions. Therefore, care must be taken in correlating design parameters from the relations established in this manual or other references. In defining a single design parameter, one must able to use different approaches (correlations) and must adjust the estimated value based on engineering judgment from field observations and by averaging or comparing and elimination techniques.

For undrained compressive strength, q_u, in KPa, is correlated with N as follows (Bowles, 1988):

$q_u = KN$ …………….. (Eqn. 4-2)

Where K = A coefficient which depends on the site, however, a value of K = 11.97 has been used.

Based on corrected N values, Bowles gave a relationship for relative density (Dr) as follows:

$N'_{70}/D_r = 32 + 0.288\, P'_0$ …………… (Eqn.4-3)

Where P'_0 is the overburden pressure in KPa.

For internal angle of friction, different authors give the following several approaches (Alemayehu, 1992):

Osaki: $= \sqrt{20N} + 15$ …… **(Eqn. 4-4)**

Sowers: $= 28 + N/4$ ……… **(Eqn. 4-5)**

4.4 Groundwater Table Measurement

Groundwater affects many elements of foundation design and construction. High ground-water table produces low bearing capacity, lateral pressure on retaining walls, construction hindrance, corrosive action, etc. Therefore, groundwater table (GWT) shall be established as accurately as possible if it is within the probable construction zone.

Some groundwater situations may have an important bearing on the choice of the type of flood protection dams to be constructed and on the estimates of costs of foundations. During borehole drilling or excavation it is important to determine:

- GWT and its variation with season of the year;
- the location of perched aquifer;
- the presence of artesian aquifer;
- flow direction;
- water loss during drilling due to cavities;
- groundwater quality (Chemical analysis);
- pore pressure measurements, etc.

Depending upon the type of the stratum, measurement of GWT shall be made after suitable time lapse of drilling operation; for example, for sand stratum, in 30 minutes after drilling, for silt and clay after 24 hrs. These time lapses are set depending upon the stabilization of GWT in the strata. Commonly 12 to 24hrs, after drilling, is a practicable time stabilization of GWT in boreholes. An efficient method of measuring water table in a borehole is by water level indicator of both visual and sonic type.

4.5 In situ Unit Weight Determination

The weight of a unit volume of soil is the most easily determined property. Consequently, it has become the basic parameter to which all other performance characteristics are related.

The in situ density of a soil is used in the stability analysis, provides information on the natural state of compaction and as an input in many soil mechanics analysis.

The methods most commonly used for the determination of in situ unit weight of a natural soils deposit or compacted earth fill are:

- Core-cutter method,
- Sand replacement method, and
- Water displacement method

4.5.1 Core Cutter Method

The core-cutter method consists of driving a core-cutter of known volume into the soil after placing it on a cleaned soil surface. The driving of the core-cutter is usually done by hitting or power-driven (see Fig 4-1). Then the cutter filled with the soil is removed and the excess soil is trimmed off. The cutter with the soil is weighed. The volume of the cutter is calculated from the dimensions and the in situ unit weight is determined by dividing the weight of the soil in the cutter by the volume of the cutter. If the water content of the soil, W, in the cutter is determined in the laboratory, the dry unit weight of the soil can also be computed by dividing the in situ unit weight by 1+W.

Figure 4-1: Undisturbed Sampling by Core Cutter (Photo Showing during Core Cutter Retrieval: Axum Water Supply Project)

ILLUSTRATION 4-1:

A 1000cubic centimeter core cutter weighing 946.80g was used to check the compacted unit of weight of an earthen flood protection dyke. The weight of the core cutter filled with soil measured at the site was noted to be 2770.60g and the water content was also measured from sample in an oven was 10.45%. If the specification of the earthwork is to satisfy 95% relative density and the laboratory value of the maximum dry density using Modified Standard Proctor test on the soil is found to be 1.70g/cc; then find the max.dry compacted unit weight and check the satisfactory of the compaction against the specification.

Solution:

Wt of soil in the cutter = *2770.60g-948.80g = 1823.60g*

Compacted unit wt, γ = *W/V = 1823.8/1000 = 1.823g/cc*

Max.dry unit wt, γ_d = $\gamma/(1+\omega) = 1.823/(1+0.1045) = 1.65$ *g/cc*

Checking the relative compaction= $(\gamma_{d\ field}/\gamma_{d\ max\ in\ lab}) \times 100 = 1.65/1.75 = 97.1\% \geq 95\%$, *it is ok!*

4.5.2 Sand Replacement Method

This method can advantageously be used for hard and gravely soils because it is very difficult or impossible to drive core-cutter in these layers.

The sand replacement consists of making a hole into the ground. The excavated soil is weighed. The volume of the hole is determined by replacement with calibrated uniformly graded and clean air-dry sand. Knowing the weight of excavated soil (after properly collecting and weighing) and the volume of the hole (from volume of replaced sand), then the in situ unit weight or dry unit weight can be similarly determined as that of core-cutter method. There is a sand cone device and template arrangement for the sand replacement method.

ILLUSTRATION 4-2:

During geotechnical investigation for foundation bearing capacity determination of big flood protection retaining wall, a sand replacement method was used to measure the in situ unit weight with the following observations. Follow the procedure and compute the natural (in situ) unit weight of the subsurface material.

Wt of excavated soil	*= 761.25g*
Wt of sand + cylinder (W1)	*= 10,500g*
Wt of sand + cylinder after pouring in the excavated hole	
and cone (W2)	*= 9,450g*
Wt of sand + cylinder after re-pouring for the cone only (W3)	*= 9005g*
Volume of a calibrated container	*= 1000cc*
Wt of the sand in filling the calibrated container	*= 1550g*

Solution:

Wt of sand filling the excavated hole and the cone	*= W1-W2*
	= 10,500-9,450
	= 1,050g
Wt of sand filling cone only	*= W2-W3*
	= 9450-9005
	= 445g
Wt of sand filling hole only	*= 1050-445*
	= 605g
Unit wt of sand	*= 1550/1000*
	= 1.55g/cc
Volume of the hole	*= 605/1.55*
	= 390.32cc
In situ unit wt	*= 761.25/390.32*
	<u>= 1.95g/cc</u>

4.5.3 Water Displacement Method

This method is suitable for cohesive soils only where it is possible to have a lump sample. A small sample is trimmed to a regular shape from a large sample prepared in the field. The sample is weighed, W1. The sample is then coated with a thin layer of paraffin wax and weighted, W2. A metal container with an overflow arrangement is prepared. The coated sample is then gradually lowered into the water filled container and the overflow water is collected in a measuring jar. The measured volume of overflowed water, V2, is the volume of the displaced water or volume of the coated soils specimen, If V is the volume of the uncoated specimen:

$V = V_w - \{[W_2 - W_1]/□_p\}$ ………….. (Eqn. 4-6)

Where, $□_p$ = the unit weight of paraffin wax,

Thus bulk unit weight of the soil, $□_b$, is:

$□_b = W_1/V$ ………………. (Eqn. 4-7)

The dry unit weight of the soil, □d, is:

$□_d = □_b/(1+\omega)$ ……………(Eqn.4-8)

Where ω = moisture content in decimal.

4.6 Strength Tests

4.6.1 General

The strength tests can be of different types such as shear strength, bearing and settlement strengths, torsional strength, etc. In geotechnical engineering, the first two items are very important in relation to water work structures.

4.6.2 Shear Strength

The shear strength of a substratum is the resistance of a sliding soil or rock mass along its internal surface to counteract the action of external forces. This resistance is contributed by one or a combination of the following forces: friction (sliding, rolling or interlocking) and attraction between individual particles called cohesion.

The source of cohesion in soil particles is more of geologic and chemical aspects of the soil grains, i.e., the existence of cementation, charges, van Der Waal force of attraction, electrostatic force, electromagnetic force, etc, among the soil particles.

In the field, shear strength can be measured by:

- Field Vane test,
- The boreholes shear test, etc.

(i) Field Vane Shear Test:

The method is used to measure the in situ strength of cohesive soils those which are too soft or sensitive for sampling. The procedure is that the vane (Fig 4-2) is pushed gently into the soil up to the required depth or at the bottom of a borehole and torque (T) is applied gradually (1 to 6^0 per minute) to the upper end of the torque rod until the soil fails in shear due to the rotation of the vane. The torque is measured by noting the angle of twist. Then the unit undrained shearing resistance can be estimated from Eqn.4-9 as follows:

$$T = C_u \pi \left(\frac{d^2 h}{2} + \frac{d^3}{6} \right) \quad \text{……………. (.Eqn. 4-9)}$$

Where

- T = applied torque;
- D = diameter of vane blade;
- H = height of the vane blade;
- C_u = undrained strength, equivalent to C_u values in the lab.

The test shall be made in 1.5 to 3m or appropriate intervals and corrections shall be made depending up on the apparatus constants.

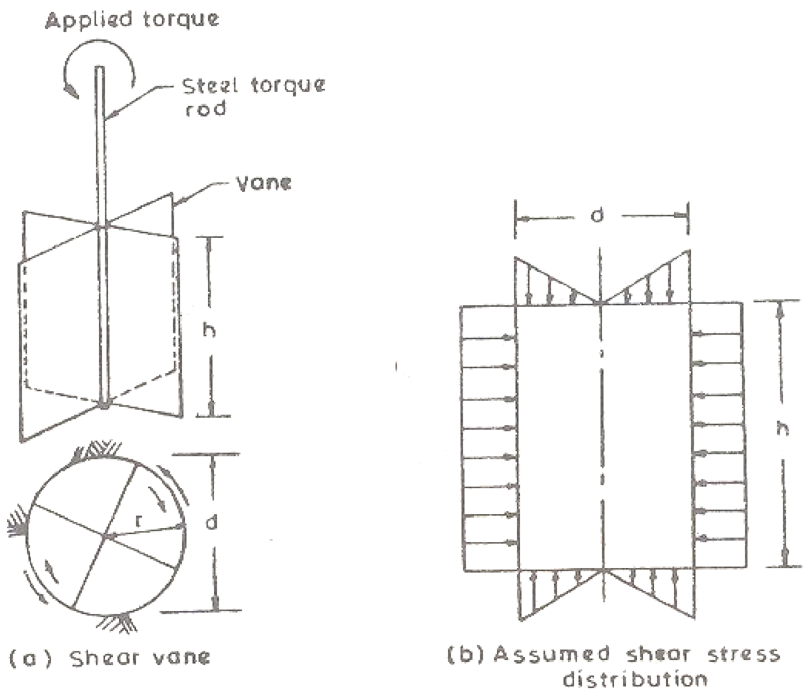


Figure 4-2: The Vane Shear Test

ILLUSTRATION 4-3:

An in situ vane shear test was conducted in a clay soil of at the bottom of test pit. A torque of 15Nm was required to shear the soil. Determine the undrained shear strength of the clay if the vane is 100mm in diameter and 150mm long.

Solution:

$$T = C_u \pi \left(\frac{d^2 h}{2} + \frac{d^3}{6} \right)$$

$$15000 = C_u \pi \left(\frac{100^2 x 150}{2} + \frac{100^3}{6} \right)$$

C_u = 0.0052N/mm^2 = 5.2KN/m^2

(ii) Borehole shear test:

The test consists of expanding a serrated cylinder halves in a drilled borehole by applying pressure from the surface through a piping system. Next the cylinder is pulled with the pulling load and displacements recorded. The expansion pressure is σ_n and the pulling load can be converted to shear strength S to make a plot as in direct shear test (S versus σ_n) to obtain in situ strength parameters Φ and C.

The shear strength parameters are drained values if the test is performed in the displacement range of about 0.5mm/min or less. Or pore pressure measurement arrangement can be made for fast test to obtain both total and effective stress parameters. It is applicable for all fine-grained soils.

4.6.3 Bearing Capacity

The substrata upon which a flood hydraulic structure lies must be capable of carrying the loads from the superstructure without a shear failure and with the resulting settlements being tolerable for that structure. The safe intensity of pressure, which shall be carried by the formations without excessive settlement and shear failure, is the allowable bearing capacity.

The bearing capacity of a foundation material of a flood structure can similarly be computed as that of building or other structures. These are using:

- Bearing capacity equations (Tezaghi, Meyerhof, Hansen and Vesics equations),
- SPT correlation,
- Plate loading test,
- Codes or presumptive values, etc.

Therefore, when need arises, consultation to books, texts and codes shall be made. However, for preliminary designs or when elaborate investigation is not justified, Table 4-3 may be used as presumptive bearing capacity value for different sub-surface materials subjected to vertical static loading.

Table 4-3: Presumptive Bearing Values* under Vertical Loading (ESCP 3, 1983).

Supporting ground type	Description	Compactness** Or consistency***	Presumptive bearing value (KPa)	Remarks
Rocks	Massive crystalline igneous and metamorphic rocks (granite, basalt, gneiss)	Hard and sound	4,000	
	Foliated metamorphic rock (slate, schist)	Medium hard & sound	2,000	
	Sedimentary rock (hard. Shale, siltstone, limestone)	Medium hard & sound	2,000	These values are based on the assumptions that the foundations are carried down to the unweathered rock
	Weathered or broken-rock (soft limestone)	Soft	1,000	
	Soft shale	Soft	600	
Non-cohesive soils	Gravel, sand and gravel	Dense Medium dense Loose	400 300 200	Width of foundation (B) not less than 1m
	Sand	Dense Medium dense Loose	300 200 100	Ground water level assumed to be a depth not less than B below the base of the foundation
Cohesive soils	Silt	Hard Stiff Medium stiff Soft	200 150 100 50	
	Clay	Hard Stiff Medium stiff Soft	300 200 100 50	
Peat and organic soils				Are highly compressible and settleable. They are unsuitable to carry the loads from important structures
Made-up ground or fill				They have to be treated with suspect unless they are properly compacted.

* The given bearing values don't include the effect of the depth of embedment of the foundation

** Compactness: dense N □ 30

medium dense N = 10 to 30

loose N □ 10, where N = standard penetration value

*** Consistency hard: q_u □ 400 KPa

stiff: q_u = 100 to 200KPa

medium stiff: q_u 50 to 100KPa

soft: q_u = 25 to 50KPa

where q_u = unconfined compressive strength

5. FIELD SAMPLES FOR LABORATORY TESTS

5.1 General

It is a common saying that representative sampling with prudent attention in testing of laboratory samples is half of the design of the project. For instance, a sample which has been disturbed but submitted to the laboratory for testing as an undisturbed (relative term) is of less value than no sample at all because the result of the test may lead to erroneous conclusions and faulty design.

Due to the nature of the subsoil or due to the requirement of the work piece, some laboratory tests are mandatory in flood structure investigations. The most important laboratory tests required for geotechnical designs are strength, compressibility and permeability. Additionally; index properties, chemical behaviors, etc, of the substratum may be sought.

5.2 Disturbed Sampling

It is a representative sampling but the natural conditions are disturbed during cutting or in sample handling or testing. Disturbed sampling does not mean a wrong sampling, however, rather the method of determining the laboratory geotechnical properties of the formation does not require the in situ conditions.

Disturbed sampling can be done in:

(i) Accessible excavations by manual means such as in pits, trenches, exposed cuts, etc. (Fig 5-1);
(ii) Non-accessible excavations such as from hand augers, power driven auger, rotary drilling, etc.

Disturbed sampling is done for

- classification and description;
- Remolded soil properties determination;
- Chemical tests;
- Petrography/mineralogical analysis, etc.

5.3 Undisturbed Sampling

It is nearly impossible to obtain a truly undisturbed sample due to:

- relief of the confining pressure;
- volume displacement of a driven sampler;
- side frictions of the sample collection device;
- possible changes in water content;
- loss of hydrostatic pressure;
- suffering in handling and transportation of the samples;
- quality and attitude of drilling crew, laboratory technicians and supervising engineer;
- bad working environment, etc.

However, in general usage, the term undisturbed means a sample where the possible precautions have been taken to minimize disturbance or remoulding effects.

Undisturbed sampling can also be done:

(i) In accessible excavations by manual methods such as in pits, trenches, exposed cuts, etc, (Fig 5-2, 5-3 & 5-4),

(ii) In non- accessible excavations such as drill hole using double-tube soil samples, thin-wall open drive samplers, piston samplers and core samplers. Appropriate references shall be made before embarking on these activities.

Undisturbed sampling is required in flood protection works mainly for:

- shear strength tests (C &));
- crushing strength (for rocks);
- consolidation tests (Cc, Cr, CQ, etc);
- permeability test, K;
- natural density, moisture, etc (see Chapter 4).

5.4 Sample Sizes

The quantity of soil/rock submitted for laboratory tests is dependent on the type and combination of tests to be performed. Therefore, adequate sample size, number and quantities shall be known and planned a head of sampling operation. These help to avoid later additional sample ordering (which has high effect on cost!) or information gap. Some soil index tests can be done from a single sample if prudent planning is used, e.g., Atterberg limits, sp. gravity, gradation, organic content, etc., can be conducted from a single sample.

The size of undisturbed samples usually depends upon the dimensions (length and area) of the sampling device, the number of duplicate test required to obtain a particular parameter, testing apparatus type and the laboratory procedures to be followed. Tables 5.1 & 5.2 may precariously be followed.

5.5 Handling and Storage of Samples

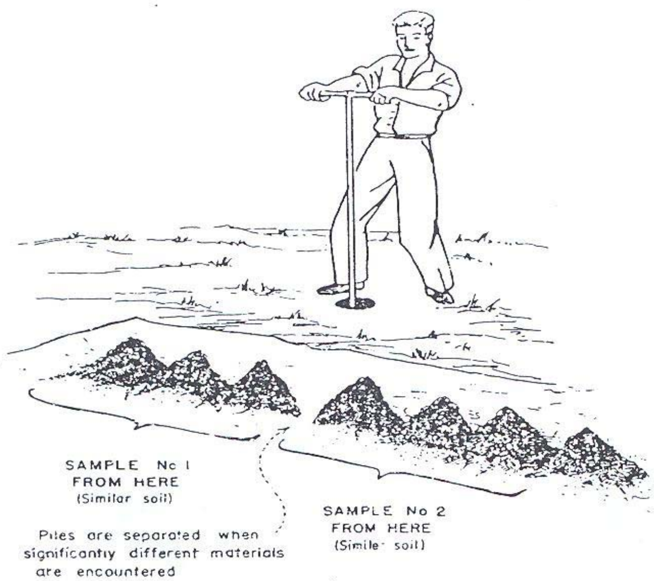
To get dependable and reproducible results from laboratory tests, a sample must be properly handled and stored starting taking from the in situ till it enters to testing apparatus.

Proper handling includes:

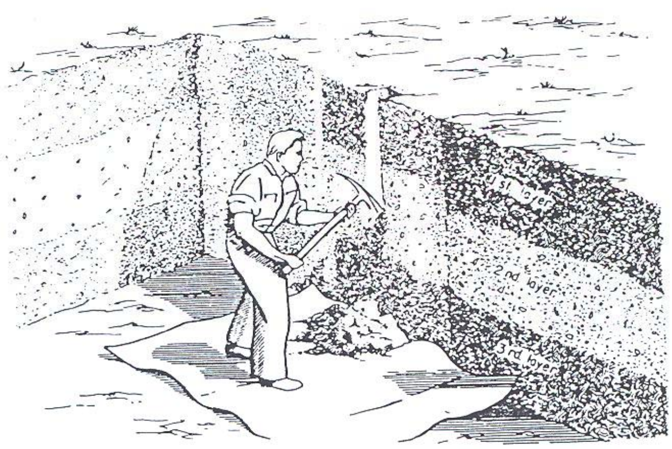
- sealing undisturbed samples by wax and paraffin;
- carefully lifting and placing of samples in a sample container,
- cautions in storing, transporting and handover;
- properly mixing, quartering, trimming, extruding and placing (in the apparatus) of samples, etc. During storage, single or packed samples shall be placed in a shade. Special precaution shall also be taken for extreme weather conditions, e.g., hot climate or rain.

5.6 Sample Identifications

Samples shall immediately be labeled after being taken from exploratory holes using identification tags. Tags shall contain pertinent information about the samples and shall also be placed one inside the samples and the other on the outside. Tags shall be covered properly by plastics to prevent damping due to moistures. Cans may be marked on the exterior or labels securely fastened to the outside surface; but not on the lids, as lids may be interchanged. Fig 5-5 shows a typical sample tag.



a) Auger Sampling.

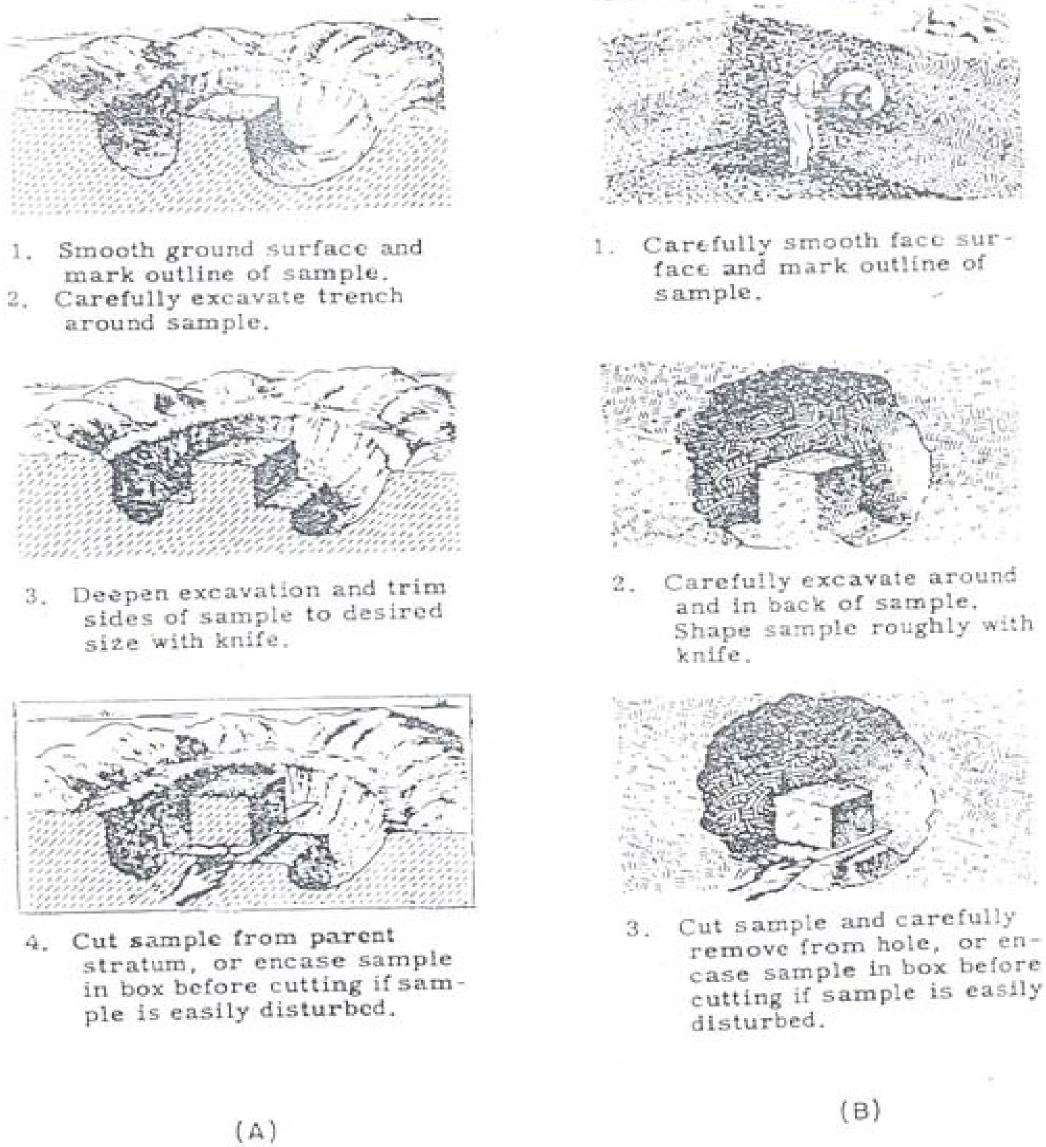


Individual samples are taken from each layer of soil.

Composite samples are taken from two or more layers of soil.

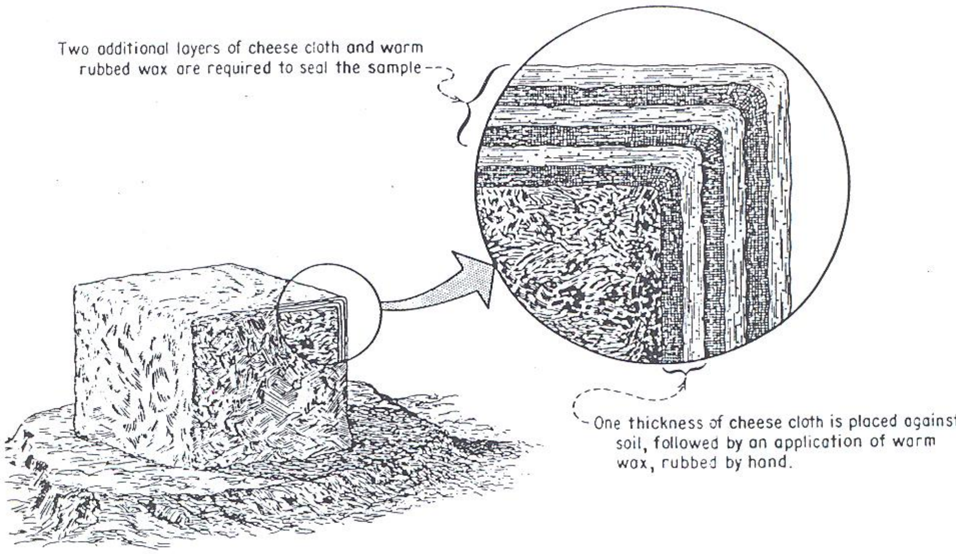
b) Sampling in Trench

Project GUIDELINE MANUALS & STANDARDS PREPARATION	Subject	Status
Client MINISTRY OF WATER RESOURCES	Title REPRESENTATIVE DISTURBED SAMPLING	
Engineer - CONCERT ENGINEERING CONSULTING - CONTINENTAL CONSULTANTS	Fig. 4.1	Date

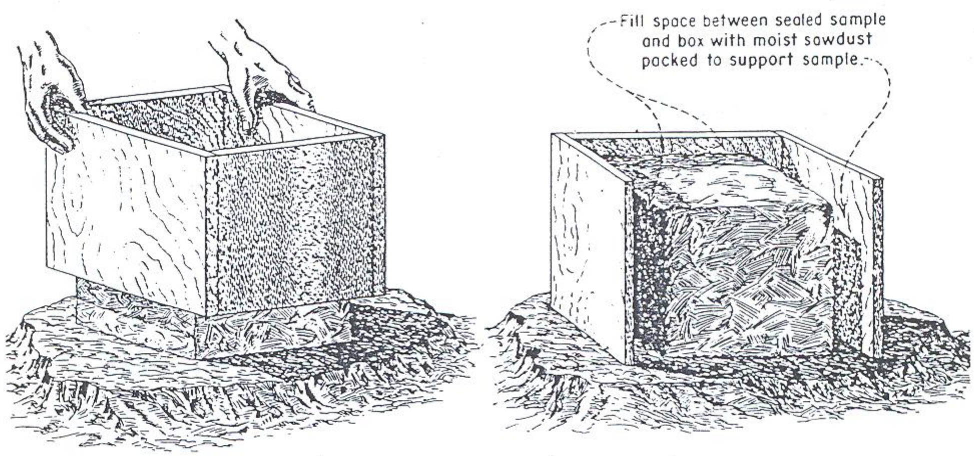


Initial Steps to Obtain a Hand-Cut Undisturbed Block Sample: (A) Bottom of Test Pit or Level Surface. (B) Cut Bank or Side of Test Pit.

Figure 5-2: **Representative Undisturbed Hand Cut Block Sampling (USBR, 1990)**



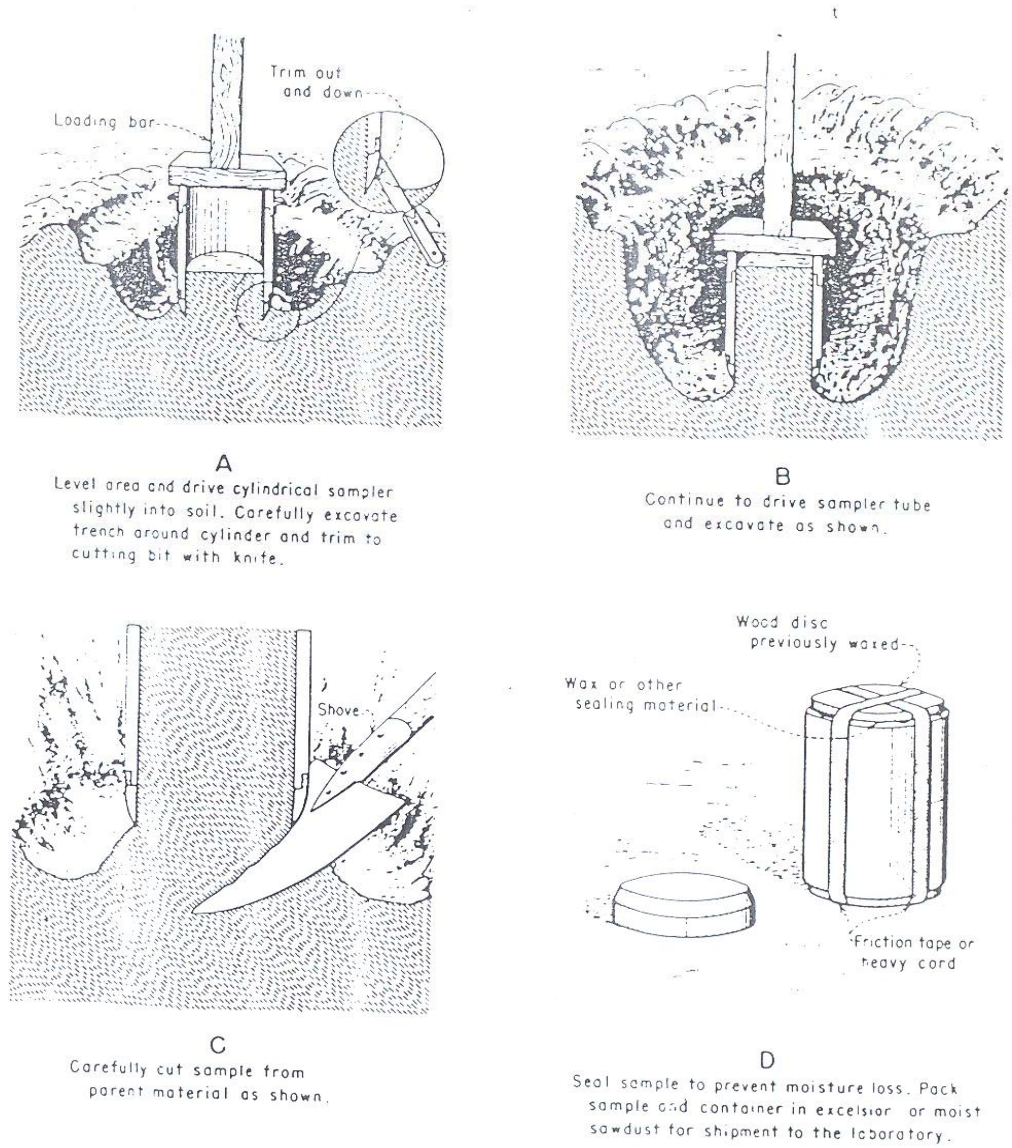
(A.) METHOD FOR SEALING HAND-CUT UNDISTURBED SAMPLES



(B.) ENCASE EASILY DISTURBED SAMPLES IN BOX PRIOR TO CUTTING

Final Steps to Obtain a Hand-Cut Undisturbed Block Sample.

Project GUIDELINE MANUALS & STANDARDS PREPARATION	Subject	Status
Client MINISTRY OF WATER RESOURCES	Title Contd.....	
Engineer - CONCERT ENGINEERING CONSULTING - CONTINENTAL CONSULTANTS	Fig. 4.2	Date



Project: GUIDELINE MANUALS & STANDARDS PREPARATION	Subject	Status
Client: MINISTRY OF WATER RESOURCES	Title: METHOD OF OBTAINING A HAND-CUT UNDISTURBED CYLINDRICAL SAMPLE	
Engineer: - CONCERT ENGINEERING CONSULTING - CONTINENTAL CONSULTANTS	Fig. 4.3	Date

Figure 5-5: Photo Sowing Method of Obtaining a Hand Cut Undisturbed Cylindrical Sampling (Shefe Dam Site, SNNPRS, Ethiopia).

Sample Information

Project: __________________	Sample ID. No.: _________________
Location: _________________	Lading no.: ___________________
Purpose: _________________	Surface elevation: _______________
Date: ___________________	Depth to sample:
	Top ______ bottom _____________

Tests required:
a.____________________
b. ___________________
c. __________________

NB: See letter of transmittal

Figure 5-6: Typical Labeling Format or Tag of Samples

Table 5-1: Representative Disturbed Sample Sizes for Laboratory Tests

Purpose of sample	Soil Type	Sample Size
Soil classification, gradation, Atterberg limits and chemical tests	*Cohesive Soils*	*10 to 30kg*
	Cohesionless soils	*30 to 50 kg*
Compaction tests	*Cohesive Soils*	*50 to 100kg*
	Cohesionless soils	*50 to 100 kg*
Permeability, remoulded	*Cohesive Soils*	*50 to 100 kg*
	Cohesionless Soils	*50 to 100 kg*
Relative density	*Cohesionless soils*	*50 to 100 kg*

Table 5-2: Representative Undisturbed Sample Sizes for Laboratory Tests

Purpose of sample	Soil type*	Test and Sample length**
Shear strength	*Fine-grained hard to firm cohesive or cemented cohesionless*	• *Unconfined compression: two 15 cm length cores required.* • *Triaxial shear: four 15 cm length cores required or* o *15 to 30cm cube block sample or* o *15 to 20cm diameter and 15 to 30cm long cylindrical sample*
Coefficient of permeability	,,	*One 15cm length core required*
Load consolidation and permeability	,,	*One 15cm length core required*
In place density and moisture	,,	*One 15cm length core required*
Sliding factor, between soil and concrete	,,	*One 15cm length core required*

Note: **For rock strength, one 15cm length core sample may be used for unconfined compressive or three 15cm cores for triaxial tests. Petrographic, durability, sp. Gravity, porosity and absorption tests can also be done.*

***For routine index tests in undisturbed samples, lower values of Table 5-1 may be used.*

An identification of samples, the identification of holes shall be followed. That is, holes are normally numbered in the order of excavation and the series shall be continuous throughout the various stages and reports of the work. The hole number can be prefixed with a one or two-letter designation to describe the type of exploration. For example, for borehole: BH, for test pt: TP, for hand and power auger: HA & PA respectively, etc., can be used.

In a single exploratory hole, several samples may be taken at various depths. To differentiate the samples depth-wise, effectively, increasing subscripts may be used. For example, in a borehole which is numbered as 5 and if three consecutive samples are going to be taken, then they can be differentiated, respectively, as BH5/1, BH5/2 BH5/3, to designate samples taken at depths 1,2 & 3.

5.7 Sample Shipment

Samples in small containers to be shipped by common carrier shall be packed in strong boxes, preferably wood. Great care shall be taken for undisturbed samples by providing in the package for reducing vibration. Further, the box container shall be placed on skids or on sand field sacks.

On the boxes, important notes shall be written on it including about the samples information and precautions, such as: “THIS SIDE UP”, “DON’T DROP IT”, “PROTECT FROM SUNLIGHT OR OVER-HEATING”, etc. Every sample shipment to a laboratory shall be accompanied by a letter of transmittal (Table 5-4).

5.8 Samples from Pits, Trenches and Exposed Surfaces

Fig. 5-1, 5-2, 5-3 & 5-4 show disturbed and undisturbed sampling in pits, trenches and exposed surfaces. During the excavation of pits, the bottom of the hole shall always be kept fairly level to the full size the hole so that each lift may be corresponding portion of the deposit in quality and quantity.

In homogenous formation, samples may be taken at 2m meter interval and when there is stratification at every layers. If the stratification is thin lenses, and difficult to take from each layer, then representative samples shall be taken from the hole, inside walls of the excavated depth or every fifth or tenth of bucketful. The dividing line between strata is generally determined based on visual examination of color, texture, etc.

5.9 Samples from Augers

Samples collected from augers are disturbed. Accurate logging as pit or as trench is difficult because the entire profile cannot be seen. Hand or power driven auger samples taken from the hole shall be placed in individual piles in an orderly depth sequence. New piles shall always be started when significant different materials encountered (see Fig. 5-1a). Sufficient sample size can be obtained by combining from similar consecutive piles separately or a composite sample by combining and quartering from each piles.

5.10 Important Laboratory Tests for Flood Protection Works

A number of laboratory tests can be carried out on a single hydraulic structure component, but this is unwise not only due to redundant tests but also uneconomical and needs longer time of completion.

Table 5-3 shows the major important laboratory tests required in water works. However, these test types can be increased or decreased depending upon the scale of the project. Furthermore, the number of tests that shall be conducted on each location shall also depend of the total number of samples taken from each hole or in other words on the homogeneity and depth of the exploratory hole required.

During laboratory sample shipment and on the letter of transmittal, the supervising geotechnical engineer shall carefully consider and notify the peculiar characteristics of residual soils and the special laboratory handling procedures of some of the tests. Current researches depict that the tropical residual soils need special attention than that of transported temperate soils. It goes to the extent of the development of Residual Soil Mechanics. Some of the laboratory tests which need special attention and modification are index tests, Proctor tests, consolidation tests, permeability tests, shear strength tests and some others (Blight, 1997).

5.11 Field Laboratory Arrangement

In remote areas and when the exploration deems numerous laboratory tests, arrangement of field laboratory service is necessary. This mobile laboratory program improves the quality of the data by decreasing the disturbances that could be created by shipment vibration, moisture loss, etc.

Table 5-3: Important Laboratory Tests for Flood Protection Structures and Materials of Construction

S. No	Structure/material	Sample location	Purpose	Types of tests	Remarks
1	Retaining walls	Along the alignment	Foundation investigation	• index tests(sp.gr, Atterberg limits, gradation, etc); • free swell; • bulk density and natural moisture content; • shear strength, consolidation., rock strength, etc	Samples may depend on the size of the structure and ease of estimation from visual observation. All are undisturbed samples except for index, free swell and chemical analyses, if any.
2	Diversion canal	Along the alignment	Foundation investigation	Same as above including permeability	There are some tests that could be eliminated
3	Pipe line	Same as '1' above	Same as '1' above	Same as '1' above	Same as '1' above
4	Borrow materials	Clay source	As core, lining, blanket, etc	• index tests; • free swell; • compaction; • permeability; • shear strengths; • consolidation; etc	Both from disturbed and undisturbed samples
		Embankment fills	As shell and selected excavated fills	Same as above depending upon the material type. Some tests on fine portion	
		Sand	Concrete fine aggregate, transition materials, bedding, etc	• sp.gr, &sieve; • soundness; etc	
		Gravel	Concrete fine aggregate, transition materials, bedding, road surfacing, etc	Same as above including abrasion tests, if required	
		Rock rocks	As riprap, rock-toe, rock fill, masonry, etc	Visual examination, compressive strength, & petrographic analysis, if required	
5	Flood protection reservoir	Dam axis	Foundation investigation	Several tests	Dam engineering texts and codes shall be consulted

Table 5-4: Letter of Transmittal

[......................] Project

Letter of Transmittal for Soil/rock Tests

To : [………………………………….] Enterprise
Material Testing Department

From: [………………………………………….]

Date: [………………………………………….]

Project Location: [………………………………]

Purpose: Geotechnical Investigations

Lading No: [………………]

Dear Sir,

These samples were taken from test pits (*or drilled holes, as appropriate*) for foundation and materials investigations. Therefore, we kindly request your office to perform the tests listed in Table 1 shown below according to known standards. For these handed-over samples: attachment of methods of testing (i.e., designation number), lab raw data, tables, graphs, charts, sought design parameters and interpretation of test results are necessary. Furthermore, it is our great hope that the same necessary sample handling precautions will be taken as that of the care taken in the field and shipment.

Thank you for your thoughtful considerations.

Table 1 Summary of Samples Handed over and Tests Required

Sample Identification no.	Tests Required									Others	Remarks
	Gradation (combined)	Gradation (sieve only)	Index tests (L_l, W_l & W_p)	Sp. gr	Moisture content	Bulk unit wt. $\square_b$	Shear strength				
							CU	UU	q_u		
BH1/1	√		√	√	√	√					
BH3/5	√		√	√		√					
TP1/1		√	√	√	√	√	√	√			
HA2/3	√		√	√	√	√					
PA1/1	√		√	√	√	√					
TR1/1		√	√	√	√	√	√	√			
Total samples	**4**	**2**	**6**	**6**	**5**	**6**	**2**	**2**	**0**		

An identification of samples, the identification of holes shall be followed. That is, holes are normally numbered in the order of excavation and the series shall be continuous throughout the various stages and reports of the work. The hole number can be prefixed with a one or two-letter designation to describe the type of exploration. For example, for borehole: BH, for test pt: TP, for hand and power auger: HA & PA respectively, etc., can be used.

In a single exploratory hole, several samples may be taken at various depths. To differentiate the samples depth-wise, effectively, increasing subscripts may be used. For example, in a borehole which is numbered as 5 and if three consecutive samples are going to be taken, then they can be differentiated, respectively, as BH5/1, BH5/2 BH5/3, to designate samples taken at depths 1,2 & 3.

6. CONSTRUCTION MATERIAL INVESTIGATIONS

6.1 General

In geotechnical investigations construction materials explorations comprise visual, field and laboratory assessments of locally available soil and rock materials. The assessments include impervious fills, embankment fills, transitions and drains, concrete ingredients, embankment protections, etc., of flood protection structures or other ancillary works. Elecrtro-mechnical equipment, instrumentation and some sub- and super-structural components are dealt separately out of this manual.

6.2 Impervious Materials

6.2.1 Properties Required

Impervious materials are used in many parts of flood hydraulic structures such as in embankments, flood protection reservoirs, etc. These impervious materials include clay soils, stabilized soils, concrete and asphalt.

In clay soils, there are desirable and non-desirable properties which the investigation has to look for use as impervious material for core, blanket, lining, etc. These objectionable properties, after compaction, include:

- high shrinkage and swelling;
- perviousness;
- compressibility;
- erodibility;
- disperseability;
- non-workability as construction materials;
- low shear strength, etc.

These non-desirable properties can also be reduced by additives, blending, high quantities, etc., but always the economics have to be there. Lateritic clays, silty clay, sandy clay, silty sandy gravely clay or the types which give low permeability are the preferred impervious materials.

6.2.2 Clay Core and Blanket

The impervious clay core in reservoirs shall extend to full impervious depth or partially depending upon the extent of tolerable seepage allowed and other supplementary seepage prevention provided. Imperviousness, low shrinkage and swelling, good workability and low compressibility are highly needed.

As blanket, clays provide reduction of seepage loss, decrease in hydraulic gradient and uplifts to the structure. Materials for blankets on reservoirs do not need to be high quality as in case of core.

Peat, organic clay and inorganic clays of high plasticity shall as much as possible be avoided for core and blanket uses. Sources are from nearby excavations for appurtenance structures or borrow area of minimum distance from the structure. Exploration method, field and laboratory tests required for clay sources as discussed in Chapter 4 & 5.

6.2.3 Canal Lining

On flood diversion canal construction, linings are used to reduce unnecessary water leakage and prevent water logging of adjacent lands. Commonly used canal linings are clay, concrete, masonry, brick, etc. However, unless high degree of seepage control is required, clay lining is the most commonly used and locally available economic material.

For clay canal lining, high degree of impermeability, low shrinkage and swelling potential, erosion resistance and good workability are required. To achieve these properties thick linings, blending of good clays with silt, sand and gravel, and protective covers are needed. For this purpose, the sources are from canal excavation or nearby borrow area. Undesirable stones and plant growths shall be removed from the stock piles.

Before placing the lining materials on the bed and slopes of the canals, field and laboratory explorations (Chapter 4 and 5) and foundation preparations (excavation, scarifying and moistening) are required. Then blending (if necessary) and compaction in lifts at optimum moisture content are needed.

6.3 Previous Materials

6.3.1 Properties Required

Previous or semi-previous materials are required for flood protection dam body, body of dykes, canal embankment fill, road surfacing and as transitions and drain materials. The properties entailed vary depending upon the placement of the materials as shown below.

6.3.2 Shell Materials for Flood Protection Embankments and Dams

Permeable or semi-permeable type materials are used in rolled earth embankments and dams to provide an outer shell of high strength to support the impervious core, to secure favorable hydraulic conditions of drainage and to act as filters between materials of greatly differing grain sizes. After compaction and placement, the shell mass must meet:

- homogeneity and free from voids;
- free from compressibility;
- high shear strength, etc.

As cheap embankment filler materials like alluvial deposits, weathered rocks and other previous loose deposits can be used. These sources can be found from foundation excavations for dams and appurtenant structures and suitable nearby borrow areas. Methods of exploration, field and laboratory tests required could be referred in Chapters 4 and 5.

6.3.3 Diversion Canal Fill

Depending upon the design requirements, earth embankments for diversion canals may consist of impervious or pervious soils placed loose, partially compacted by equipment or well compacted by rollers, or a combination of these.

The selection criteria as earth embankment fill are not as such critical as that of linings, cores or blankets. Suitable material from canal excavation or borrow areas can be used. Usually cut-and-fill method with good compaction and lining is used. Exploration methods, field and laboratory tests required could also be referred in Chapter 4 and 5.

6.3.4 Transition Filters and Drains

Transition filters and drains are required at different locations in the flood protection body of a dam. These are between the core and shell, between the foundation material and shell, between the rock-toe and shell, in foundation relief wells and drain trenches, etc. They can be horizontal filters, chimney drains, toe filter, relief wells, trench drains, filter around pipes, etc.

Every filter has to satisfy two main criteria: On the one hand, the migration of soil particles (piping) shall be prevented and on the other hand, the seepage water has to freely drain to reduce the development of uplift pore water pressure. The following criteria must meet to satisfy the above requirements:

I. [D_{15} of filter]/[D_{15} of protected (base) materials] = 5 to 40 provided that the filter does not contain more than 5% of material finer than 0.074mm (No 200 sieve);
II. [D_{15} of filter]/[D_{85} of protected (base) materials] = 5 or less;
III. [D_{85} of filter]/[Max. Opening of pipe drain = 2 or more;
IV. The grain size of the curve of the filter shall be roughly parallel to that of the base material.

Although the quantity requirements of pervious materials for filters and drains are usually small, quality requirements are high. Often a single layer material will be inadequate and multi-layer shall be designed. Fine sand, silt or clay in the previous material is objectionable, and processing by washing or screening is required to produce acceptable material from most natural deposits. Although grading requirements will be different, materials for filters are commonly secured economically from sources acceptable for concrete aggregate or from crushed aggregate sources.

6.3.5 Bedding or Blanket under Riprap

Ripraps are usually used for dams, embankments, canals and stilling basins. To prevent the erosive action of water waves that penetrate and remove the underlying embankment materials, riprap bedding or blanket are provided.

Sand and gravel, crushed rocks or weathered rocks are mostly used under riprap. The characteristic requirements of bedding under riprap are similar to filters and drains, however, material found in most gravel deposits, rock fines in quarrying operation and weathered rocks with fewer amounts of fines are commonly employed without detail analysis. But deposits like ancient gravel beds that have been deteriorated by weathering and talus or slope wash deposits where water action has been insufficient to remove the fines and soft rocks are unsuitable for this purpose. Furthermore, smooth gravels from a river source may also have low interlocking action for the upper riprap protection.

6.4 Concrete Aggregates

Aggregates are those chemically inert materials which when bonded with cement paste form concrete. They constitute the bulk of the total volume of concrete and hence they influence the strength of the concrete structures to great extent. The aggregates shall be hard, strong, dense, durable, and free from injurious amounts of clay, loam, vegetable and other foreign matters. Depending upon the size, the aggregates are classified as fine aggregate (sand) and coarse aggregate (gravel).

6.4.1 Sand

The grading requirement of sand for concrete flood protection works is similar with other similar or ordinary concrete civil works. Sand may be obtained from river, lake, sea and pits. The sum of all types of deleterious materials in fine aggregate shall not exceed 5%. River sands usually suitable for concrete but mostly contaminated with mud and thus advisable to wash such sands before use. Sea sand is also contaminated with salts from sea water which are liable to attach the steel reinforcement. Pit sand also generally suitable, but it is liable to contain silt or other organic matter. The other sand source is by crushing suitable hard rock to the required grading. There is a better control and qualities from this source. Therefore, the economic comparison must be made between the naturally available sands in the locality and from crushing before deciding the source.

The characteristic requirements of sand for flood hydraulic structures are more or less similar to other concrete works. Generally, more exacting investigations are needed than for other purposes.

Angular sand has good interlocking property which results in a strong mortar while rounded grained sand does not afford sufficient interlock in the matrix. Soundness values of less than 8% in five reaction cycles are assumed to be satisfactory. The method of exploration and field and laboratory tests for sand can be referred in chapter 4 & 5.

6.4.2 Gravel

Similarly the grading requirement of gravel for concrete flood protection works is similar with other similar or ordinary concrete civil works. Gravels can be obtained from alluvial deposits or from crushed hard stones. The sum of the percentage of all type of deleterious substances in gravel shall not also, as sand, exceed 5%. As far as possible, flaky and elongated gravel shall be avoided. Soundness result (sodium sulphate test) shall be < 10% for five cycles.

Gravels are usually obtained from crushed basalt, granite, gneiss and other suitable hard, dense and durable rocks. Gravels shall be rough or angular and approximately rounded. Flaky and elongated gravels shall be avoided or within the recommended proportion.

In most cases of gravel, the material brought at site may be single sized aggregates (i.e., ungraded) of nominal sizes 63m, 40mm, 20mm, 16mm, 12.5mm and 10mm or it may be in the form of graded aggregates of nominal sizes 40mm, 20mm, 16mm and 12.5mm. The other is an all in-aggregate (i.e., mixture of sand and gravel).

The size of the gravel used depends upon the nature of the work. The maximum size for mass concrete (as in dams) is 20mm and 63mm for plain concrete. For reinforced concrete, gravels having a nominal size of 20mm are generally considered satisfactory.

6.5 Rock Sources

6.5.1 Properties Required

Rocks for construction may be required as massive rock fill for flood protection dams, rip-rap for dams' stilling basins and canals, rock-toe in dams, drainage blankets and as masonry works in flood hydraulic structures. Rock sources shall satisfy two main criteria: first the source shall able to produce rock fragments in suitable sizes, i.e., moderate to slightly weathering and joints must exist for production and excavatability; secondly, the rock material shall be hard, dense and durable to with stand destructive forces that encounter during placing or due to wave action or due to normal weathering. The common sources are bed-rock, river boulders, talus or surface deposits classified as basaltic, granitic, gneiss, hard limestone, etc depending the use of the location and place of use. Economic comparison must be made when different sources are available on a site.

During field examination it is customary to describe the rock source by origin, color, grain size and composition as described in Chapter 3.

6.5.2 Rip-raps for Dams and Canals

(i) Rip-rap for earth embankments and dams:

Hand-placed or dumped rip-rap is mostly provided on upstream faces of earth embankments and dams to break the running waves and to withstand the destructive wave actions on the embankment materials.

Angular rock fragments are mostly suitable as riprap because they can easily break running waves and at the same time interlocking effects for the stability of individual fragments on slopes under wave action. Sizes of the rock fragments are usually between 0.4m3 to 0.8m3. For medium and large size flood protection reservoirs, the upper range, with some combination of smaller sizes are recommended, but for small ones, the lower ranges are sufficed. Excessive amount of fines must be avoided.

(ii) Rip-rap for stilling basins:

Riprap blankets are commonly employed as energy dissipaters at spillways, outlet works, weir, d/s aprons and other structures where high velocity turbulent flows are available. The quantities required are usually comparatively small, but quality requirements may exceed those of reservoir ripraps. Larger sizes of the dam riprap specifications may be used depending upon design requirements and space availability.

(iii) Rip-rap for canals:

Another use of riprap is in canal construction where severe erosion of the channel would otherwise occur. These locations are usually in short reaches of canals below structures, adjacent to bridge piers and at sharp turns. Depending upon the size of canal, 30 to 90cm thick riprap can be used in canals. The size of the riprap fragments can also vary from smallest gravel size to 0.8m3 rocks. Reasonably well-graded with minimum compaction can be used as riprap in canals.

6.5.3 Rock-toe

Rock fragments together with layers of fine, coarse sands and gravels are used as toe filter for earth dams. Material requirements are as described in Sec. 6-5.1 and 6-5.2, but sizes are from 15 to 20cm diameter.

6.5.4 Rock Fill

Properly placed rubble or dumped and slightly compacted rocks are required for the body of rock fill embankments and dams. These massive rock fragments help for stability purpose of the structure.

Quality requirements and tests needed are as described in the above sections. Whereas, for size requirement, it depends on the construction procedure. Up to 3 m diameter may be used for this purpose.

6.5.5 Stone Masonry Work

Rocks for masonry purposes are required in different parts of flood hydraulic structures. These are masonry walls for camp housing, different thick walls in weir, canal falls, canal stone linings, retaining walls, etc.

In addition to strength requirements of the stones, shapeability and dresseability of the rock fragments are necessary. Commonly ignimbrite rock is preferable but granite, basalt, trychte, etc., can also be used.

6.6 Water

Large amount of water is usually required in every construction site. Water is required for concrete mixing, curing, compaction for embankments, foundation moistening before, placement, etc. Water for the above purposes can be obtained from river, spring, sea, lake, pond, groundwater, rain, tap, etc.

Water to be used for the construction shall have the following properties:

- it shall be free from injurious amounts of oils, acids alkalis, organic and inorganic impurities, and
- it shall be free from mud, iron, vegetable matter or any substance which is likely to have adverse effect on concrete, masonry or reinforcement.

PH values between 6 & 9 are usually don’t need special precaution, but out of these ranges special protective measures such as increasing the cement proportion in the mix, increasing the dimensions of the section or corrosive resisting cement types shall be used. Similarly, sulphate attack (SO_3) is small if the water contains a concentration of sulphate less than 300mg/l, otherwise similar remedial measures have to be taken on the concrete works.

FLOOD PROTECTION DESIGN MANUAL
PART 5: TOPOGRAPHIC SURVEY & MAPPING

Table of Contents

CONTENTS OF OTHER PARTS

1 INTRODUCTION

1-1 General

Topographical surveys are usually required at the feasibility study and design stage of projects. In case of flood protection projects topographic survey is required along the existing or proposed alignment of flood protection embankments for producing topographic maps, long section and cross sections.

This part of the manual therefore, describes the various topographic survey procedures and presentation for design, construction and approvals required for flood protection project. Project Engineers and field survey crews should familiarize themselves with this manual as well as the basic topographic procedures prior to start up of any field survey activities.

1-2 Topographic Survey

The design survey that establishes the alignment on the ground and obtains the data necessary to prepare the design as well as the construction plans is the basic task in the process of the any engineering projects which mainly be discussed in this Part of the manual.

1-3 Drawings and Mapping

1-3.1 General

The over all objective of the design process is to present the design concept in to readable maps and drawings. The development plans shall be presented in a neat, standard and professional manner. The reader is advised further reading on drafting standards. However, this sub-section is giving highlights on the requirements of mapping and drawings.

1-3.2 Drawing Album

The final design of the project should be presented in a Drawing Album which consists of the following major sections:

I. General Layout Drawings;
II. Plan and Profiles Drawings;
III. Typical Structures Detail Drawings; and
IV. Standard Drawings.

General layout drawings include the location maps of the project and detail layouts of proposed mitigation measures. The following major information's are to be included in location maps and layout maps:

a) Exact location of the work site with respect to the nearby known town, latitude/longitude;
b) Major rivers and water bodies including names;
c) Major roadway and access to the nearest village;
d) Access route from the nearest village to the project site. If multiple turns along project length, please provide Lat. and Long. coordinates for each turn;

e) Drawing scale and north arrow;
f) Latitude and Longitude coordinates for the point of beginning and point of ending of the project;
g) Centre line and boundary of the proposed work with respect to the river bank including right of way; and
h) Location of structures.

Plan and Profiles Drawings includes detailed plan view of the proposed protection embankment generally shown on the same sheet as the profile with the profile shown at the bottom and the plan view on top. The following minimum information appears to be included in both plan view and long section of the drawings:

Plan View:

a) The location of center line referenced to the river bank and existing features;
b) A beginning and ending note and stations shall be indicated on the title sheet;
c) Location of bench marks relative to the proposed centre line or river banks;
d) Stations or Chainage shall be shown at regular interval and at location where there is changes in section and direction;
e) Location and orientation of the cross section;
f) Drawing scale if necessary, separate horizontal and vertical scales;
g) Exact location of the work site with respect to the nearby known town;
h) Construction right of way;
i) Permanent right of way

Profiles view:

j) Existing ground profile, top embankment and design flood levels;
k) Distance and Invert elevations shall be shown at regular intervals and whenever there is a change in section
l) Maximum possible dimensions showing length, width, and height or depth;
m) Location of proposed and existing structures;
n) Boring indicating depth and type of soil encountered;
o) Maximum possible volumes;

Cross Section: Typical cross-sections are needed at random locations to show the proposed mitigation measures with respect to the river cross section including property boundary,

2 GENERAL MAPS AND SURVEYING BACKGROUND

2-1 Topographic Maps and Features

In most cases 1: 50,000 and 1: 250,000 scale topographic maps are available for all most all parts of the country from the Ethiopian Mapping Agency. The 1: 50,000 scale topographic maps are having most features of the ground of the project area including 20m contour, rivers and tributaries, land use and land cover, wetland areas, etc. Therefore, these topographic maps will allow first hand information and a good understanding of the general features of flood plain areas in concern.

In the case of flood mitigation project, these maps are useful to map and delineate the flood plain area, delineate drainage/watershed areas, and for determination of all catchment characteristics data's. Catchment characteristics like slope, stream length, and maximum and minimum elevation are useful data's for the processing of hydrologic data's.

Large scale maps are only available for specific project areas where development projects have been initiated for feasibility and detail level of study. Therefore, one of the needs for topographic survey requirement is to develop large scale topographic maps for further level of study and design. The detail layout of flood protection dykes will then be mapped on large scale topographic maps, indicating dyke centre line relative to the river banks.

2-2 Aerial Photos and Satellite Imagery

Satellite imagery and the aerial photographs are useful to identify permanent topographical features such as rivers, streams and drainage courses, and also to distinguish between wet lands, grazing lands and arable lands.

It will also be of use in assessing land-use in the zones potentially flooded areas and in scoping of dyke alignments and selection of zones for detailed topographic survey and geotechnical investigations. The identification of land-use in various sub-catchment areas will also allow determination of SCS curve numbers for estimating design flood and flood levels for dykes as well as for cross drainage culverts.

It will also be useful in the preparation of the survey plans for the project, as well as designing the embankment layouts.

2-3 Digital Maps

Digital Terrain Model (DTM) combined with aerial photos and satellite imagery will allow rapid calculation of existing ground levels, determination of critical sections, along the various river and drainage sections. It will also allow rapid sizing of flood mitigation structures for feasibility level study.

In addition to the above usage, DTM will also help in scoping of topographic survey plans. Supplemented with topographic survey, it will be used for layout design of flood protection embankment.

3 TOPOGRAPGIC SURVEYS AND DATA PROCESSING

3-1 General

The study and design procedures have been discussed in **Part 1: Project Planning** of this manual. Topographic maps and topographic data's are the basis for the design of any engineering structures. This Chapter has discussed the topographic survey data requirements and procedures for flood protection works.

3-2 Conventional Topographic Survey Equipment

The following equipments are required to carry out topographical survey:

- Total Station: including reflector prism for data collection and
- Hand held GPS for locating specific locations of the project area and also for tracking important route;
- Traverse Survey: Theodolite capable of measuring to 1", staves with leveling bubbles, steel tape.
- Site Survey: Theodolite capable of measuring 20" or automatic level with angular circle for flat areas staves with leveling bubbles, steel tape.
- Leveling: Automatic level, staves with leveling bubbles, steel tape.

The equipment needed can be summarized as:

- Total station including prism reflector or 1 second theodolite;
- 20 second theodolite;
- Automatic level with angular scale;
- Staves with leveling bubbles;
- 50 m or 100 m steel tape;
- 30 m linen tape;
- Base plates for staves;
- Ranging poles;
- Ranging rods;
- Concrete benchmarks;
- Wooden and bamboo pages and nails;
- Sledge hammer;
- Survey books; and
- Paint and brushes

3-3 Information Required

The following data shall be obtained by the survey crew in the field: topographic map of the flood corridor including river layout, existing and proposed flood embankment alignment, bench marks, profiles and cross-sections, high water marks, direction of flow, stream bed profile and stream cross sections. Those data's are the minimum topographic survey data requirement for the study and design of flood protection works. Detail of the requirement is discussed in the following sub sections.

3-3.1 Topography map

Topography data extending 200 meters or maximum flood marks on left and right of the river banks for the critical areas of the river should be recorded. The map should indicate the general features of the river including river banks, existing and/or proposed alignments of protection works, flood water marks, direction of river flow and other major features in the project area.

Project control and bench marks should be established during the survey and shown on the maps. Alignment control points should be placed far enough back from the structure to perpetuate the alignment for construction staking.

The topographic map survey report should give the location, elevation and description of the bench mark used in the survey referenced to the National Datum. Bench marks should be set to meet third-order accuracy.

3-3.2 Streambed Profile and Cross-Sections

Obtain elevations in the bottom of the streambed at 100 meter interval and additional at slope changing locations and develop long section of the river bed. Profile of both river banks and top of existing dykes should also be plotted in one long map so as to show the relative level to each other. The long section of the river bed should be plotted

The river cross-sections at 50 meter intervals should extend 1 meter above the 100 year flood plain elevation on both sides of the river banks in case where there is no existing flood protection work. This gives coverage distance far enough to cover the high water marks or any proposed height increase of the flood wall and flexibility on the design of the alignment of protection work. Sufficient stations should be taken to show the design engineer the shape of any fill around the structure.

The possible scope of topographic work required is summarised in the following table.

	Activities	**Type**	**Scope**	**Instrument**
1	Layout maps	Steep	0.25 m contours	Total station and/or Theodolite
		Plain	0.50 – 1 m contours	
2	Long Sections	Small Rivers	1:500 horizontal scale 1: 100 vertical scale	Level
		Medium Rivers	1:1000 horizontal scale 1: 100 vertical scale	
3	Cross Sections	Small Rivers	Every 1000 m 1: 100 scale	Level
		Medium Rivers	Every 1000 m 1: 100 scale	
4	Bench Marks	Steep	0.50 km spacing	Level
		Plain	1.00 km spacing	

Table 3-1: Topographical Survey Requirements

3-4 Horizontal and Vertical Control

3-4.1 General

The first step of topographic survey is establishment of baseline or controlling points with in the vicinity of the project area that can be transferred from the national datum level. The national geodetic datum levels are available on the 1:50,000 topographic maps prepared by the Ethiopian Mapping Agency to be used for the establishment of bench marks.

These controlling points shall be established using acceptable survey procedures of triangulations. These monuments shall be coordinated from the nearby national geodetic benchmarks. Once the horizontal and vertical monuments have been established in the project area of survey, all centrelines and levels shall be referenced to these monuments. These coordinated benchmarks shall be shown on the topographic maps.

3-4.2 Location References/ Datum

All controls shall be related to the benchmarks of the proposed flood mitigation project site and the national grid and their reduced level should be related to the mean sea level.

Give the location, elevation and description of the bench mark used in the survey. All elevations should be referenced to the national grid and their reduced level should be related to the mean sea level. The Global Positioning System (GPS) or topographic map of scale 1:50,000 prepared by the Ethiopian Mapping Agency whichever is available will be used for the establishment.

3-4.3 Project Control and Monuments

The project control or monuments should be properly established using stable natural features or reinforced concrete blocks

3-4.4 Benchmarks

The first step of topographic survey is establishment of a system of benchmarks along the flood corridor and along the proposed flood protection works. Benchmarks may be located on permanent features where they are stable and safely located.

These benchmarks can be transferred from the project controlling points with in the vicinity of the project area that can be transferred from the national datum level. The national geodetic datum levels are available on the 1:50,000 topographic maps and their detail descriptions may be obtained from the Ethiopian Mapping Authority (EMA). These controlling points shall be established using acceptable survey procedures of triangulations. These monuments shall be coordinated from the near by national geodetic benchmarks. Once the horizontal and vertical monuments have been established in the project area of survey, all centrelines and levels shall be referenced to these benchmarks. These coordinated benchmarks shall be shown on the topographic maps.

Sketches with clear descriptions of the location of each benchmark should be prepared as is shown in Figure 4-1.

PROJECT NAME **Jimma Masonry Storm Drainage Project** PROJECT TYPE: NEW DESIGNE APPROXIMATE CHAINAGE: ○ L ● R CONTROL POINT ID: **HD 12**	MONUMENT *Driven iron*	STAMPED
	ESTABLISHED ……………………………. DATE: JULY,2009	
	RECOVERED …………………………... SET BY:	

…………………..SYSTEM OF COORDINATE … UTM …ZONE……37……………

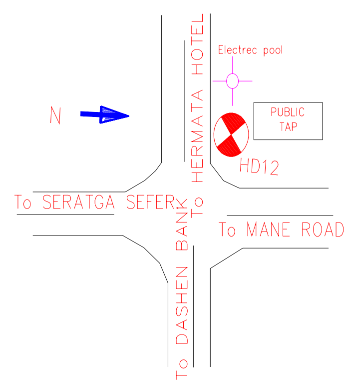
DESIRABLE HORIZONTAL ACCURACY:

LATITUDE:	NORTHING(Y) = **848039.344**
LONGITUDE:	EASTING(X) = **260766.227**
MAPPING ANGLE : ° ‘ “	ELEVATION= **1728.396**

SCALE FACTOR: **1.00000000**	HEIGHT FACTOR: **1.00000000**	COMBINED FACTOR: **1.00000000**

COORDINATE SYSTEM	DATUM	ELLIPSOID	ACTUAL ACCURACY
UTM	**ADINDAN**	**WGS 84**	

POINT MONOGRAPH



PHOTOGRAPH

GPS survey General Information	Device Type	Oper. Mode	Rec. Per.	Unit No.
	”			
	Last Observation date	**Min. Observation Period**	**Operator Code**	

POINT DESCRIPTION

Note: *-Elevation of BM is set by direct leveling*

HD 12 is located at the right side of the existing asphalt road from Mentena Kebele a head Hermata Hotel, at the front there is a public tap. The point is marked at the edge of the existing asphalt road; it's a driven iron in to the ground.

Figure 3-1: Sample Benchmark Monograph

3-5 Topographic Map Survey

Topographic Map surveying and mapping is the one of major physical activities to be undertaken during flood embankment design.

The survey crew apart from the route of the river should record the performance of existing structures like diversion weir or dam, cross drainage culverts, bridges and all other utility lines. Information on existing structures may be obtained from local residents and the District Bureau of Operations in addition to the physical field survey. This information should be obtained for the adjacent structures upstream and downstream as well as the structure at the site. Recover or re-establish the alignment points controlling the centreline of the existing roadway or existing flood embankment. Alignment control points should be placed far enough back from the structure to perpetuate the alignment for construction staking.

For structures, Topography 500 feet (150 meters) left and right of the structures centreline for distance of 600 feet (200 meters) from the centre of the any river structure should be recorded.

The survey should also include: river and stream crossing structures, top and bottom of river bank, dyke alignment, vertical control, profiles and cross-sections, crossing data, topography, high water marks, direction of flow, stream bed profile and stream cross sections.

3-5.1 Strip Survey

Topographic surveys along river with spot levels at 50m centres, or more frequently for sudden drops in elevation, showing benchmarks and physical features including embankments, buildings, retaining walls side drains, road crossings, rocky outcrops, surface stones, or boulders ,electric pole and other infrastructure.

The centreline of the proposed flood embankments should be aligned from the starting point (like headwork site) all the way up to the end point where the embankment to be anchored in to high ground level. Collection of data and the survey extends the full width of the river and up to 100m or high ground level on either side of the banks.

Spot levels will be taken where there is a level change of the ground, and to be taken in 20-40m grid depending up on the ground condition. The collected data will be plotted to generate Strip topographic map of the area and other features that are existing

Strip topographic map of the flood corridor will be prepared at a scale of 1:1000 or 1:2000 with contour interval of 0.5meter to be prepared for the actual layout of the drain.

3-5.2 Long and Cross Sections

Longitudinal profile survey along the centreline of the river and existing as well as proposed flood embankments (both left and right bank) to be conducted for the design of the flood mitigation infrastructure

Cross Section Surveys of both left and right banks of the river showing bank height & slope at 250m centres for smaller reach and at 5 to 10km for very long reach depending up on the section and slope of the river.

3-6 Field Procedures and Data Collection

3-6.1 Field procedures

Transferring a known national grid bench mark from the nearest known points to establishing bench marks along the traverse line

Dividing over all area into different block areas will be helpful in order to survey by different Surveyor crews simultaneously. Each block area will be connected to each other by close traverse method

Number of surveying crews is assigned such that it is possible to complete the survey work with in specified time.

3-6.2 Data Collection

Data will be collected for different sites and types of work according to the standard requirement or scale.

Every data of each traverse should be checked by forming of common base line

Collection of data, which offsets 100 meter, will be made and a topographic map of the route will be prepared at a scale of 1:1000 or 1:2000 with contour interval not more than 1 meter to be prepared for the actual planning and locating of the flood mitigation infrastructure.

The extent of data collection to the left and right from the proposed centre line of the River or natural drain depends up on the width and height required to contain the flood and the features required to be indicated on the Topographic map.

3-7 Office Works

The objective of topographic survey is to generate proper maps with required scale for the use of office works or design works. Therefore, the main task in the office is data processing and produce maps, profile, layouts and boundary marks using appropriate software (like LisCAD, and LandCAD, Surfer or Civil designer Software).

The best practice in office work starts to maintain the original data for any reference during the project execution and starting the data processing maintaining proper records and filing system.

The office work should start from quality check of the collected data. For so many reasons direct survey data may be poor in quality for example outliers may be inserted.

3-8 Production of Survey Results

The result or the output of surveying is achieved through by producing different types of maps and drawing such as Topographic maps, Profiles (Longitudinal, Section)

3-8.1 Topographic Maps

Topographic maps showing benchmarks and physical features including embankments, buildings, retaining walls, side drains, road crossings, rocky outcrops, surface stones, or boulders ,electric pole and other infrastructure.

Finally the map is aimed to produce planning and detail design of infrastructures. The map enables structures design to take account of the actual ground conditions.

3-8.2 Profiles (Long and Cross Sections)

Longitudinal profile showing the original ground level along the river bed, proposed centreline of the proposed embankment, interceptor drainage for the design of flood mitigation infrastructure

Cross Section drawing showing both left and right banks of the river , bank height & slope, other physical features like embankment or dyke ,retaining walls if present (top of wall and bed level), etc,

Cross-sections are needed at 25 meter intervals and should extend left and right of the centreline a distance far enough to cover the high water marks or any proposed height increase of the flood mitigation. Sufficient cross sections (possibly at section changes) should be taken to show the design engineer the shape of river.

The cross-sections should extend 1 meter above the 100 year flood plain elevation, be normal to stream flow and other locations that may affect the design of proposed flood mitigation structures. Due to the meandering tendency of most of our stream systems, it is extremely difficult to locate a representative section that is at right angles to both the low flow channel and the flood plain.

Proper sectioning of these locations requires surveying across the flood plain at right angles, then pivoting at the channel bank to shoot at right angles across the channel, and then pivoting again by swinging the angle necessary to continue across the flood plain normal to the direction of flow. An alternate procedure is to shoot across the flood plain and channel on a straight line properly recording the skew of the flood plain and channel so that the designer can make the necessary dimensional adjustments in the office.

Profiles and/or cross-sections may be needed by the designer for quantity computation at locations like culverts, bridge approaches, etc.

3-8.3 Layouts (Plan)

Layout plan overlay on topographic map gives design Information, the designer needs some basic information to use in developing overlay construction plans.

4 CONSTRUCTION SURVEY

4-1 Pre-Construction Survey

The pre-construction survey includes checking plans, elevations of completed design work of flood embankment or associated structures mostly during handover stage.

Plans shall be checked to verify that site conditions have not changed between location surveys and construction, and also between location (continued) survey and preliminary plan completion. Meander migration, bank caving, aggradations, head cutting or other natural or man-induced changes in the channel may have occurred which would require that the designer reconsider decisions made on the basis of conditions

Such survey is different from those which exist at the planning or design stage. This is best accomplished with a joint field inspection by design, construction, and maintenance personnel. Additional objectives of the Pre-Construction Survey are to assure accuracy and to ascertain if the designer has properly visualized existing situations and designed accordingly.

4-2 Construction Control Survey

The Construction control survey of changed conditions may require river control works, revisions to locations and orientation, rearrangement, or other modifications of the design to accommodate the changes that have occurred. Plan changes required because of differences between location surveys and construction field inspections shall be made in consultation with the designer. Some changes could significantly affect either the height or the length of the flood embankment or associated system.

4-3 Final Completion Survey

The Final Completion Survey following completion of the project should document any deviations from the original plans as well as an initial assessment of the hydraulic performance. Changes must be incorporated into "as-built" plans for future reference.

This type of survey helps to produce the final plans and drawings “As Built” plans and drawings which serve many functions related to documentation of the final location of all elements of the flood mitigation system and related facilities, any changes that were made in the design during the construction process (i.e., size of facilities, materials used, addition or elimination of facilities), and any variation between the original plans and specifications and the final installed facilities.

The levels of the Bench Marks located on the flood embankment and the adjacent original ground are taken as monuments which are concrete types in which the 25mm diameter steel rod are embedded. The levels are registered are plotted in relation to the original design (construction) levels and flood embankment levels. The result will be used to indicate the vertical and horizontal displacements of the flood embankment.

4-4 Borrow Area Survey

Borrow area Survey to acquire a complete documentation of geological conditions at the proposed site(s) selected detailed topographic map of scale 1:1000, or 1:2000 with 2.5 or 5 m contour intervals is of primary importance, if borrow area is along river valley, cross sections along axis of valley at 100 m interval is required.

This type of Survey will enable to find out the size of the borrow area. Having determined the average depth of excavation from the borrow area, the total volume material can be estimated for planning the design.

FLOOD PROTECTION DESIGN MANUAL
PART 6: FLOOD HYDROLOGY

Table of Contents

CONTENTS OF OTHER PARTS

1 INTRODUCTION

1.1 Purpose and Scope of the Flood Hydrology

Information in this part of the manual is intended for use by hydrologic engineers in the preparation of flood hydrology studies necessary for the design of flood protection works and associated features. It provides general guidance for estimating both the magnitude and frequency of floods.

Hydrology for flood protection works includes the information and procedures for estimating flow peaks, volumes, and time distributions of storm water runoff for specified catchment area or watershed. The analysis of these parameters is fundamental to the design of flood protection works, such as channels and dykes for containing flood, culverts/crossings for safe passage of flood water and control structures for diverting part of flood.

In the hydrologic analysis of a development site, there are a number of variable factors that affect the nature of storm water runoff from the site. Some of the factors that must be considered include:

- Rainfall amount and storm distribution;
- Drainage area size, shape, slope and geology;
- Land use and cover, soil type, surface infiltration and storage;
- Slopes of terrain and stream channel(s);
- Antecedent moisture condition;
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.);
- Watershed development potential; and
- Characteristics of the local drainage system.

The typical hydrologic processes of interest in flood protection works hydrology are related to:

- Rainfall analysis;
- Determination of peak flow rate;
- Determination of total runoff volume;
- Runoff hydrograph (flow vs. time);and
- Stream channel hydrograph routing and combining of flows.

1.2 Hydrologic Data and Methods

Design floods are best based on actual flood peak data. It would have been good if actually recorded flood peaks are available and make analysis to properly understand in the context of known rainfall characteristics, e.g. heavier daily rainfalls or maximum flood discharge of a stream.

However, in most cases recorded flood data and flood peaks might not be available for most of the watersheds in Ethiopia. Moreover, the required data is often not available. Therefore, the practice of hydrology for flood protection works is an estimation process based on rainfall runoff analysis. Hence, the process of rainfall runoff analysis has been dealt in detail in this manual. Some worked examples have also been included in this final version of the manual.

Hydrologic data include records of runoff accumulated at continuous recording stream flow gauges and indirect discharge measurements, from which hydrographs may be constructed. Meteorological data include precipitation, temperature, and wind records collected at meteorological stations. Stream flow data and rainfall data are directly for the use of in the design of flood protection projects.

There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for a catchments area or watershed; the methods presented in this manual have been selected because they are widely used and accepted in the country. The widely accepted hydrological methods and analysis included in this manual:

- Rational method; is best to be used for catchment areas less than 50 hectares (0.5 km^2);
- SCS unit Hydrograph; recommended for catchment areas greater than 50 hectares
- Gumbel or Log Pearson III analysis shall preferably be used for all routine designs provided there is at least 10 years of continuous or synthesized record for 10-year discharge estimates and 25 years for 100-year discharge estimates;

These methods have been included since the applications are well-accepted in the country water resources development projects.

1.3 Design Flood

In order to design flood protection works, the peak flood in the river for the design return period of 25, 50 or 100 years is needed. Extreme flood events more than 100 years return period is required to protect urban settlement areas.

The river flood level corresponding to the design flood event will also be required. River cross sections should be surveyed upstream and downstream of the river reach requiring protection and the slope and cross section of the river obtained. The approximate river flood level for the design flood event can then be estimated using Manning's equation taking into account the surveyed cross sections, the slope of the river and an appropriate Manning's coefficient "n". Details of hydraulic procedures are presented in **Part 7: Hydraulic Channels Design**.

Where possible the river levels should be checked against flood marks left by floods, which should be noted when river cross sections are surveyed.

1.4 Definitions

The following are concepts that are important in a hydrologic analysis. These concepts will be used throughout the sections of this Part of the manual in dealing with different aspects of hydrologic studies:

Infiltration	Infiltration is the process through which precipitation enters the soil surface and moves through the upper soil profile.
Travel time (Tt) and time of concentration (Tc).	Travel time is the time it takes for water to travel from one location to another in a watershed. Tt is a component of the time of concentration, Tc, which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed.
Rainfall excess	After interception, storage, and infiltration have been satisfied, rainfall excess is the remaining water available to produce runoff.
Antecedent Moisture Conditions	The soil moisture conditions of the catchment area at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge only in the lower range of flood magnitudes – approx. below the 15-year event threshold. As floods become rarer, antecedent moisture has a rapidly decreasing influence on runoff.
Frequency	The number of times a flood of a given magnitude can be expected to occur on average over a long period of time. Frequency analysis is the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equalled or exceeded in any year.
Hydrograph and Unit hydrograph	Hydrograph is a graph of the time distribution of runoff from a catchment area. The hydrograph resulting from 1 mm of rainfall excess generated uniformly over the watershed, at a uniform rate, for a specified period of time. There are several types of unit hydrographs. The use of unit hydrographs to create direct runoff hydrographs is discussed in more detail in reference books. An example of dimensionless unit hydrograph and the relationships to the other components presented above is shown in Figure 2-1.
Peak discharge	The peak discharge (peak flow) is the maximum rate of flow of water passing a given point during or after a rainfall event
Runoff volume.	The runoff volume represents the volume of rainfall excess generated from the watershed area. The runoff volume is often expressed in watershed area-depth of rainfall or ha-m. The runoff volume for a rainfall event can also be represented by the area under the runoff portion of the hydrograph.

Table 1-1: Hydrologic Definitions

2 RAINFALL AND RAINFALL RUNOFF ANALYSIS

2.1 Introduction

Rainfall data's are available in various formats for a number of gauge stations across the country.

Therefore, the first step in any hydrologic analysis is an estimation of the rainfall that will fall on the watershed area for a given time period. The amount of rainfall can be quantified with the following characteristics:

a) Duration (hours) is the length of time over which rainfall (storm event) occurs.
b) Depth (mm) is the total amount of rainfall occurring during the rainfall duration.
c) Intensity (mm per hour). Depth divided by the duration.

In the case where runoff data are not available, rainfall frequency analysis and then rainfall-runoff analysis would be carried out to determine the design discharge for flood protection structures.

The frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of exceedence probability or return period.

a) Exceedence probability. Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically one year.
b) Return period. Average length of time between events that have the same duration and volume.

Thus, if a rainfall event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedence probability of 0.01, and a return period of 100 years.

2.2 Rainfall

The hydrologic concepts of interest in this manual are to obtain a design flood events for the design of flood protection works. One of the basic parameters for estimation of design flood magnitude is rainfall data which is focused on daily maximum rainfall data. Relationship can be developed for the rainfall amount that is causing runoff and then the design discharge for the design return period in the case of un gauged catchments.

A rainfall record is analyzed to obtain a rainfall-return period relationship. Next, the storm event corresponding to a design return period is identified as the design storm. This design storm is then used as an input to a mathematical rainfall-runoff model (i.e. Rational method, SCS method), and the resulting output is adopted as the design runoff (peak rate and/or volume).

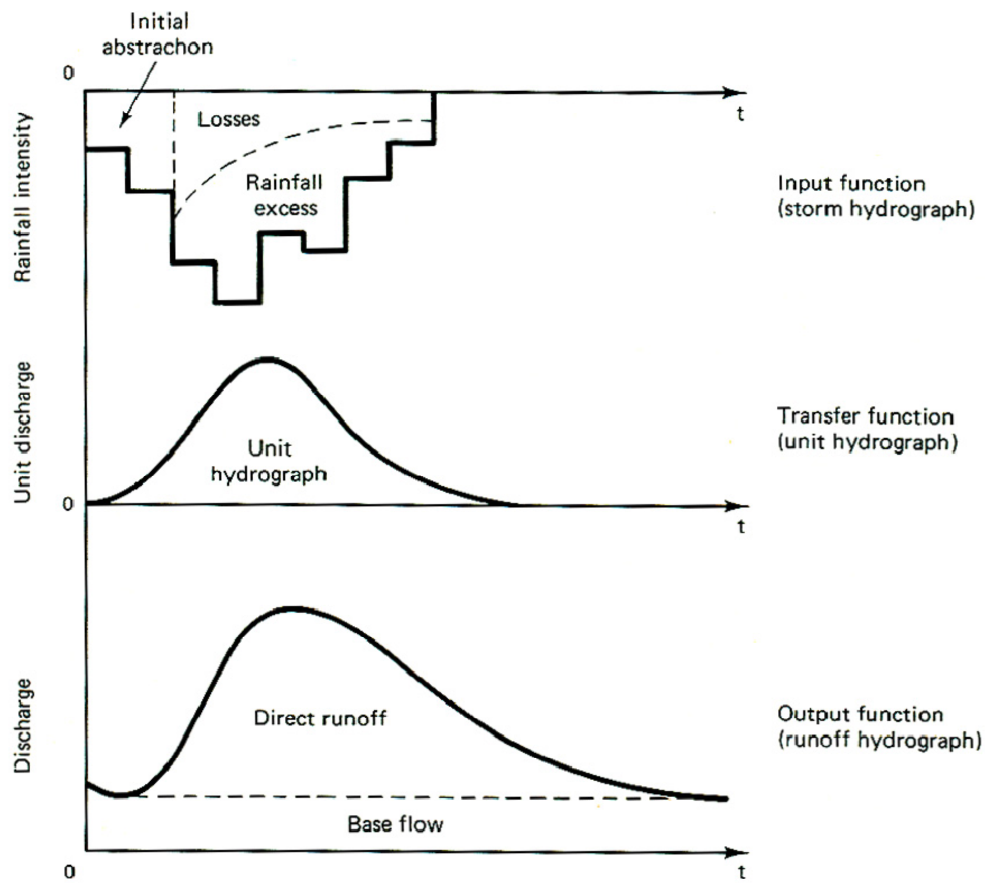


Figure 2-1: Relationship of Storm, Unit, and Direct Runoff Hydrographs

2.3 Hydrograph

A hydrograph is a graphical plot of discharge of a natural stream or river versus time. Discharge is usually expressed in m3/sec and time is expressed in hours. Discharge is plotted on Y-axis and the corresponding time is plotted on X-axis.

The shape of the storm hydrograph is related to the rainfall hyetograph (intensity vs. time) and the combined effects of the storage in the watershed or channel. The watershed and channel storage effects smooth out much of the variation evident in the storm hyetograph. The runoff hydrograph increases in magnitude shortly after the start of the rainfall event and reaches a peak after the maximum rainfall intensity has occurred.

The total runoff comprises direct runoff and base flow. For the derivation of unit hydrograph, the base flow has to be separated from the total runoff. There are several methods of which have to be referred to the standard hydrology text books.

2.4 Unit Hydrograph

A unit hydrograph can be defined as a hydrograph of direct surface runoff resulting from one millimetre of effective rainfall falling uniformly over the drainage basing in space as well as in time for a specific duration.

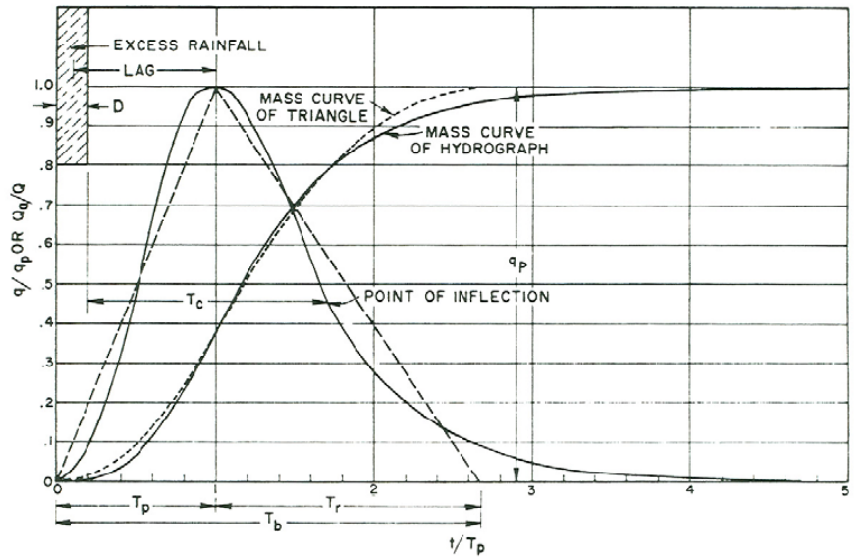


Figure 2-2: Dimensionless Unit Hydrograph and Equivalent Triangular Hydrograph

The area under a hydrograph equals the volume of direct runoff (Q), which for a UH is 1 mm. The runoff volume is related to the geometry of the triangular UH by:

Q = ½ qp(tp + tr)

where:

tp and tr = time to peak and recession time, respectively

qp = peak discharge

(Note that in the definition of the unit hydrograph, the time of concentration is defined as the time from beginning of excess rainfall to the inflection point on the recession limb of the hydrograph.)

The term for qp then becomes:

qp = Q/tp [2 /(1 + tr/tp)]

and

qp = KQ/tp (K replaces term in brackets)

2.5 Rainfall Frequency Analysis

Rainfall intensity is a parameter required from rainfall frequency analysis which will be used in the process of Rational Method. However, this parameter is usually not available in most meteorological stations in Ethiopia; therefore, empirical relationships have to be used to derive shorter duration of rainfall intervals from available maximum daily rainfall. Usually 24 hour rainfalls are available in Ethiopia.

For illustration purpose 25 years maximum daily rainfall data of Jimma have been presented in Table 2-1. The data have been subjected to statistical techniques to develop the information needed from hydrologic analyses. As a result the annual maximum daily (24 hr) rainfall is fitted to Gumbel Extreme Value Type I as shown in Figure 2-3. Using the statistical analyses, rainfall intensity-duration curves have to be developed for commonly used design frequencies like 5, 10, 25, 50 and 100 years return period. Once the intensity has been arrived it will be used in rational formula to determine the design flood.

Year	Rainfall (R)	Log (R)	Year	Rainfall (R)	Log (R)
1984	58.50	1.767	1996	48.80	1.688
1985	47.20	1.674	1997	69.00	1.839
1986	45.30	1.656	1998	62.50	1.796
1987	52.90	1.723	1999	49.90	1.698
1988	59.30	1.773	2000	50.50	1.703
1989	41.60	1.619	2001	56.20	1.750
1990	36.30	1.560	2002	42.50	1.628
1991	57.00	1.756	2003	42.60	1.629
1992	55.40	1.744	2004	46.20	1.665
1993	46.20	1.665	2005	60.90	1.785
1994	60.70	1.783	2006	44.50	1.648
1995	47.80	1.679	2007	44.60	1.649
1996	48.80	1.688	2008	62.30	1.794

Table 2-1: Jimma Daily Maximum (24 hr) Rainfall Data (Source: Consultant Private Library)

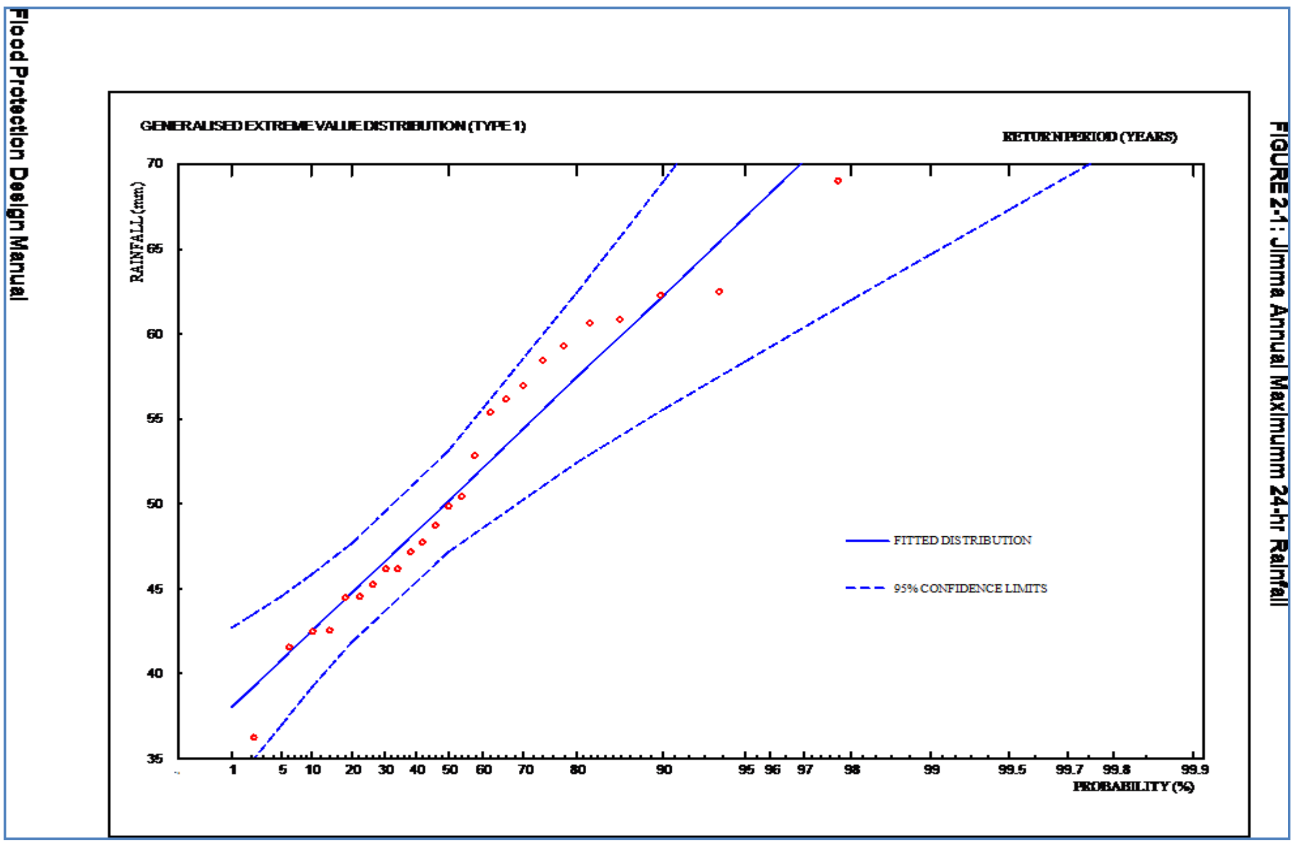


Figure 2-3 : Annual Maximum Daily Rainfall Fitted to Gumbel Extreme Value Type I

Figure 2-3 is example of statistical rainfall analysis for Jimma Town which have been used for storm drainage design. The data is best fitted to the statistical distribution which will be discussed in Chapter 3 of this Part. The aim of the rainfall analysis is to arrive at the design rainfall for the design return period. As it is shown in the figure the design return period is expressed in terms of probability on the X axis. Then the design rainfall on the Y axis can be obtained for the recommended probability.

2.6 Storm Duration Profile

As has been said earlier the available rainfall data's are registered over 24 hours. However, the actual rainfall duration in Ethiopia is normally much less than 24 hours. For this reason the selection of an appropriate time distribution for the design rainfall event must also be considered. From experience, the maximum rainfall duration of large storm is recommended to be limited to maximum 6 hours.

The design objective is to select a runoff event of a particular frequency. A particular rainfall frequency may not always produce a runoff event with an identical frequency – i.e., a smaller rainfall depth occurring in a very short period may actually produce a larger peak runoff than a larger rainfall event spread more uniformly over the event duration. As the size of the watershed decreases and the imperviousness increases, the selection of the distribution becomes critical. Larger and less impervious watersheds will often attenuate the large pulses of rainfall and smooth out the runoff hydrographs. This rainfall distribution criterion is inherent in the governing assumption in the rational method that the duration is equal to the time of concentration, and the watershed is fairly homogeneous in land use.

The SCS method is based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a 'typical" time distribution which the SCS has prepared from rainfall records. It is applicable for interior rather than the coastal regions and should be appropriate for Ethiopia. The Type II rainfall distribution will usually give a higher runoff than a Type I distribution. Figure 2-1 Type II distribution. To use this distribution it is necessary for the user to obtain

1) The 24-hour rainfall value for the frequency of the design storm desired, and then

2) Multiply this value by 24 to obtain the total 24-hour storm volume in millimetres.

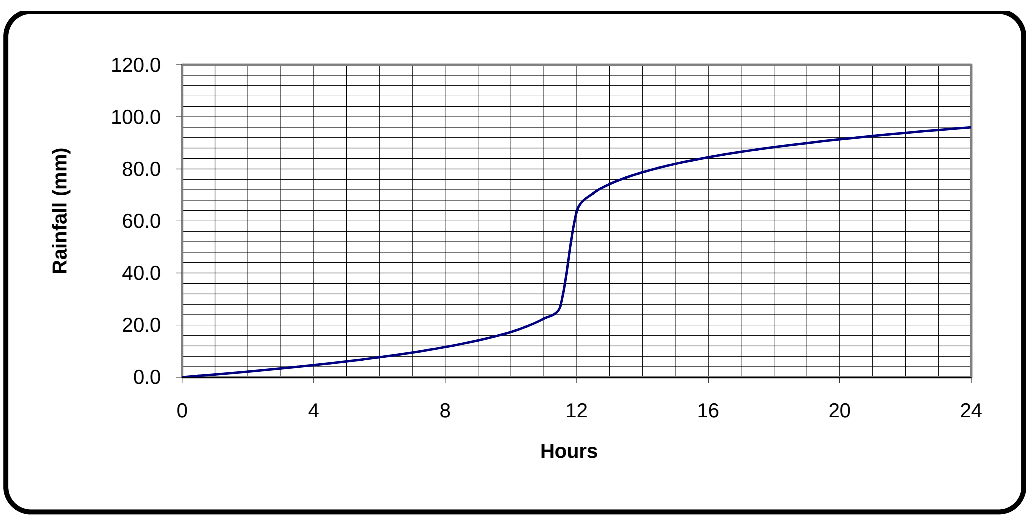


Figure 2-4: Type II Design Storm Curve

Source: ERA Drainage Design Manual 2002

2.7 Area Reduction Factor

The rainfall obtained from the rain gauge is a point rainfall. This point rainfall should be transferred to the catchment area rainfall by multiplying it by area reduction factor (ARF) as obtained in the following formula:

$$ARF = 1 - 0.044A^{0.275}$$

Where A is catchment area in Km^2

2.8 Intensity Duration and Frequency (IDF) curve

Of particular importance to the rainfall analysis is the assessment of the rainfall intensity for variations in duration and return period.

The following equation based on Gumbel Extreme value distribution and derived by TRRL (Transport and Road Research Laboratory, UK), the prediction of storm rainfall in East Africa (D. Fiddes, J.A. Forsegate, and A.O. Grigg) will be used to derive IDF curves for various durations and return periods.

$$I_t = \frac{(b+1)^n}{(b+t)^n} I_0$$

Where It = Rainfall intensity for duration 't'

I_0 = Rainfall intensity 1-hour duration

n = Constant

b = Constant

t = time in hours

Based on those the above formula the intensity duration could be produced. The detail of the procedure can be obtained in hydrology text book.

3 FLOOD FREQUENCY ANALAYSIS

3.1 General

Hydrologic systems are some times impacted by extreme events, such as sever storms, flood and droughts. The magnitude of an extreme event is inversely related to its frequency of occurrence, very severe events occurring less frequently than more moderate events. The objective of frequency analysis in the case of flood protection works is to relate the magnitude of extreme rainfall and flood events to their frequency of occurrence through the use of the probability distributions.

There are a number of frequency distributions in use in Ethiopia. Among these, the most commonly used distributions namely are Gumbel Extreme Value Type I distribution, Log Pearson and Log Normal Distributions are discussed in this manual.

3.2 Plotting Position

In order to see the fit of the distribution, the sample data has to be plotted on various probability papers. The plotting position formula is required to assign probability of exceedance or non-exceedance to a particular event. The general plotting position formula is given by

$$P(X, \geq x) = \frac{m - a}{N + 1 - 2a}$$

Where P(X >x) is the probability of exceedance, m is the rank of the event when arranged in descending order and N is the total number of observations. For the largest value m will naturally be 1, while for the lowest value it will be equal to N. Varies plotting position formula have been proposed in the literature. Given below are some of the formulas and the values of a in the formula.

Formula	Value of a
Weibull	0
Blom	3/8
Gringorten	0.44

Weibull is the most commonly used formula while Blom and Gringorten have been recommended for normal and Gumbel Distributions respectively.

3.3 Gumbel Extreme Value Distribution

The Gumbel Extreme-value distribution is the most popular distribution and has received the highest application for estimating larger events in various parts of the world. The distribution has been used for rainfall depth-duration-frequency studies [Heshefiled, 1961].

The Probability density function, P(x) for type I distribution is [Haan, 1977]:

$$Px(x) = \exp\frac{\left\{\left(\pm\frac{X-\beta}{\propto}\right) - \exp\left(\mp\frac{X-\beta}{\propto}\right)\right\}}{\propto}$$

$$-\infty < X < \infty, -\infty < \beta < \infty \ \ and \ \ \propto > 0$$

The negative and positive signs apply to maximum and minimum values (events) respectively. Here, the parameters $\propto$ and β are scale and location factors with β being the mode of the distribution.

Using a transformation, the extreme value (type I) distribution of the above equation for the cumulative distribution

$F(Y) = \exp\ (\exp(-\mathrm{y}))$ For maximum event or (extreme values)

$$Where\ y = (X - U)/\propto$$

The probability that an observation will exceed the mean values of the distribution is 1- F(Y). Regarding the estimation of parameters ($\propto$ and β). Lower and Nash (1970) concluded that the method of moments has satisfactory results as the other methods. Therefore, using this method

$$\bar{\sigma} = \frac{\sigma}{1.283} = 0.7794 * \sigma$$

And

$$\beta = \bar{\mathrm{X}} - 0.45 * \sigma$$

Where σ and X are the standard deviation and mean of the sample.

Similarly, Chow (1988) has given the type I (EV1) cumulative distribution function in the same was as follows.

$$F(X) = \exp\left(-\exp\left(-(\mathrm{X}-\mu|\alpha)\right)\right), -\infty \le \mathrm{X} \le \infty$$

Where $\propto$ and μ are the parameters and estimated from the following relationships.

$$\alpha = \frac{S_x * \sqrt{6}}{\pi} = 0.7797 * S_x \text{ and } \mu = \bar{\mathrm{x}} - 0.5772 * \alpha$$

Where μ is the mode of the distribution point of maximum probability density and "X" is the variant historically observed data.

A reduced variate "Y" can be defined as $Y = \frac{\overline{X} - \mu}{\alpha}$ and substituting Y in to cumulative distribution equation yields,

$$F(X) = \exp\left(-\exp(-\mathrm{Y})\right)$$

Simplifying and solving the above equation for Y gives,

$$Y_T = -\ln\left(\ln\left(\frac{1}{F(X)}\right)\right)$$

$$Y_T = -\ln(-\ln(1 - 1/T))$$

The value of $F(X) = \frac{T-1}{T}$

For a return period of T Year $F_T = (1+1.14KT + 1.10\ KT^2)$ ^0.5

Where $K_T = -0.78(0.57722+\ln(\ln(T/(T-1))))$

Therefore, for the EV1 distribution, X_T related to Y_1 as follows;

$$Y = \frac{X - \mu}{\alpha} \quad , \quad Y_T = \frac{X_T - \mu}{\alpha}$$

$$X_T = \mu + \alpha * Y_T$$

The method is described in the following solved example.

Example

Annual Maximum Discharge of R River is given for 24 year period. Compute flood discharge up to 1 in 50 year return period for the R River.

Year	Discharge (Q m3/sec)	Year	Discharge (Q m3/sec)
1985	652	1998	61
1986	247	1999	490
1987	156	2000	654
1988	116	2001	750
1989	482	2002	509
1990	106	2003	79
1991	NA	2004	395
1992	NA	2005	484
1993	NA	2006	397
1994	NA	2007	467
1995	149	2008	920
1996	962		
1997	174		

Table 3-1: River Awash (Only Example). 24 Year Flow Record

Step 1:

From the available river flow records, obtain the maximum discharge.

Step 2:

Compute the mean and the standard deviation of the annual maximum series.

Mean $\bar{Q} = \frac{\sum \mathrm{Qi}}{\mathrm{N}} = 412.5 \text{ m3/sec}$

Standard Deviation $\sigma = \left\{\frac{(Qi-\bar{Q})^2}{N-1}\right\}^{1/2} = \underline{276.63\, m3/sec}$

Step 3:

Compute the two parameters of the Gumbel Distribution

$\bar{\sigma} = 0.7794 * \sigma$ = 0.7794*276.63 = **<u>215.806 m3/sec</u>**

$\bar{\mu} = \bar{Q} - 0.577\bar{\sigma}$ = 412.5 – 0.577*215.806 + 0.7794*276.63 = **288 <u>m3/sec</u>**

The equation of the distribution is then

$Q_T = \bar{\mu} + \bar{\sigma} Y_T$

Q_T = 288+215.61Y_T

Where Y_T is reduced variety corresponding to T- Year return period as obtained from equation

$$Y_T = -\ln\left(\ln\left(\frac{T}{T-1}\right)\right)$$

Thus, the values of the design flood discharges for return period up to 100 year have been presented in the following table.

T Years	Y_T	K_T	F_T	$Q_{T\ (m3/sec)}$
2	0.367	0.164	1.103	367.117
3	0.903	-0.254	0.884	482.728
4	1.246	-0.522	0.839	556.720
5	1.500	-0.720	0.866	611.494
10	2.250	-1.305	1.177	773.292
20	2.970	-1.867	1.645	928.493
25	3.199	-2.045	1.808	977.725
50	3.902	-2.593	2.333	1129.385
75	4.311	-2.912	2.647	1217.535
100	4.600	-3.138	2.873	1279.925

If a graphical plot of the data is required go to Step 4

Step 4

Rank the annual maximum series in increasing order of magnitude and compute the plotting position for each event using Gringorten Formula. Probability plotting position formula, which refers to the probability assigned to each piece of data to be plotted, is used to fit the data to EVI Type distribution.

The plotting position formula for EVI distribution is using the formula developed by Gringorten (1963) (Linsley, et al 1982) and is given as:

$$\mathrm{Fm} = \frac{(m - 0.44)}{(N - 0.12)}$$

Where m = rank

N = total number of events

Return period T = 1/ (1-Fm) or 1/P

Rank	Discharge (m3/sec)	Probability (P)	Return Period T
1	61	0.0235	1.02401
2	79	0.0785	1.08515
3	106	0.1288	1.14781
4	116	0.1791	1.21814
5	149	0.2294	1.29765
6	156	0.2797	1.38827
7	174	0.3300	1.49249
8	247	0.3803	1.61364
9	395	0.4306	1.75618
10	397	0.4809	1.92636
11	467	0.5312	2.13305
12	482	0.5815	2.38942
13	484	0.6318	2.71585
14	490	0.6821	3.14557
15	509	0.7324	3.73684
16	652	0.7827	4.60185
17	654	0.8330	5.98795
18	750	0.8833	8.56897
19	920	0.9336	15.06061
20	962	0.9839	62.12500

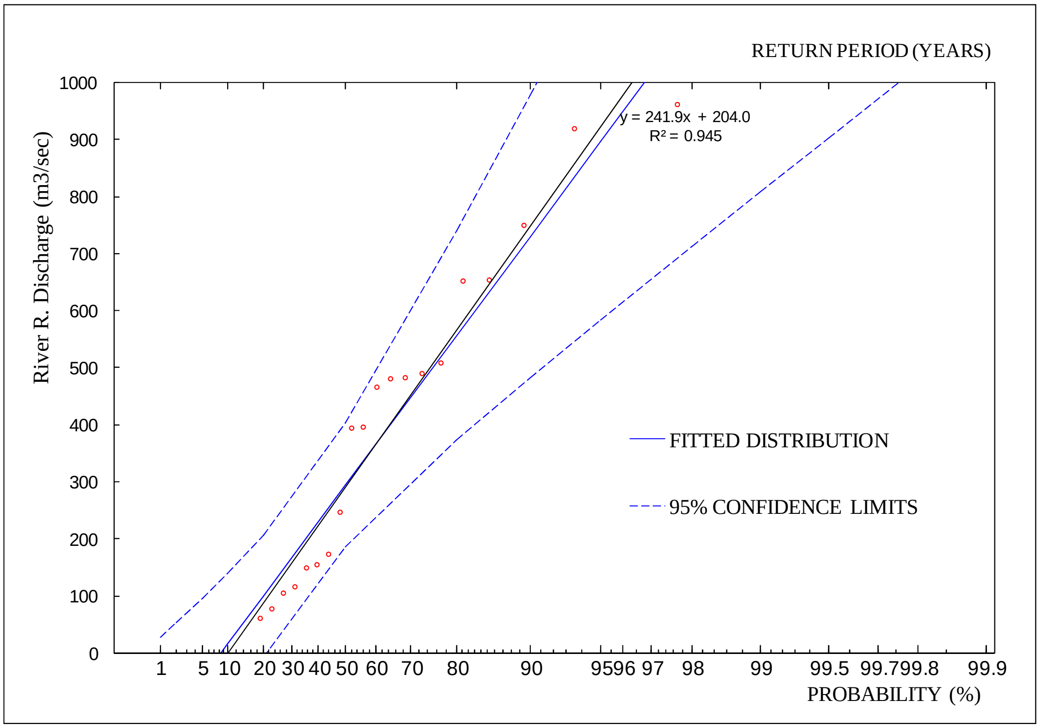


Figure 3-1: Gumbel Distribution With 95% Confidence Interval

The extreme events and estimated values using Gumbels method are confined between the upper limit and lower limit for 95% confidence interval. The plot shows that the methods can be applied for estimating extreme values.

3.4 Log Pearson Methods

Log Pearson distribution is a logarithmic transformation of the Gamma distribution.

If log X follows a Person Type III distribution, then X is said to follow a log Person distribution. Log Person distribution has got a wide application in the analysis of rainfall and discharge data's. The fit of the distribution can be checked by using the Chi-Square test or probability plotting.

Chow (1954) has given that the following general equations for any distribution from which the T-Year event magnitude can be computed.

$$X_T = \overline{X} + K_T * Sx$$

Where X is the event magnitude of the record, $\overline{X}$ and Sx are the mean and standard deviation of the sample series, K_T is the frequency factor defined by a specific distribution, and is the function of the probability level X.

3.5 Log Normal Distribution

The two parameters (the mean and standard deviation) lognormal distribution has been some times used for rainfall intensity duration analysis. If the random variable Y = Log(X) is normally distributed, then X is said to be log normally distributed. This logarithmic transformation of the normal distribution (i.e. the probability density function) is given as follows.

$$X_T = \mathrm{e}^{(\bar{Y} + \mathrm{K_T} + \mathrm{S_Y})}$$

Where

$\bar{Y}$ =mean of log (base e) transformed series

S_Y = standard deviation of log transformed series

KT = is frequency factor corresponding to probability of exceedance equal to 1/T and Cs equal to 0.0.

3.6 Estimation of T Year Flood

T year flood estimates can be obtained either graphically analytically. Graphical approach is applicable only for normal, lognormal and Gumbel EV1 distribution as for other distributions probability papers are not readily available.

3.6.1 Normal Distribution

T year flood XT is given by

$$X_T = \bar{x} + K_T * S_x$$

Where,

$\bar{x}$ = sample mean

S_x = sample standard deviation

S_x = is frequency factor corresponding to probability of exceedance = 1/T and Cs of skewness equal to 0.0. The K_T is obtained from Table of frequency factor which is available from reference materials.

3.6.2 Log Normal Distribution

$$X_T = e^{\left(\bar{Y} + K_T * S_y\right)}$$

Where,

$\bar{Y}$ = mean of log (base e) transformed series

S_y = Standard deviation of log transformed series

K_T = is frequency factor corresponding to probability of exceedance = 1/T and Cs of skewness equal to 0.0.

3.6.3 EV1 Distribution

$$X_T = u + a * Y_T$$

Where,

$$U = \bar{x} - 0.5772a$$

$$a = \left(\sqrt{6/\pi}\right) S_x$$

Y_T = reduced variety

$$= -\ln\left(-\ln\left(1 - 1/T\right)\right)$$

3.7 Goodness of Fit Tests

The validity of a probability distribution function proposed to fit the empirical frequency distribution of a given sample may be tested graphical and analytical methods. Graphical methods are usually based on comparing visually the probability density function with the corresponding empirical density function and often graphical approaches are subjective.

Even though, there are a number of analytical methods are available for testing the goodness of fit of the distribution, D-index Test is a bit better and hence discussed in this manual.

The D-Index for the comparison of the fit of various distributions in upper tail is given as

$$D - index = (1|\bar{X}) \sum_{i=1}^{6} Abs\left(x_i - \hat{X}_i\right)$$

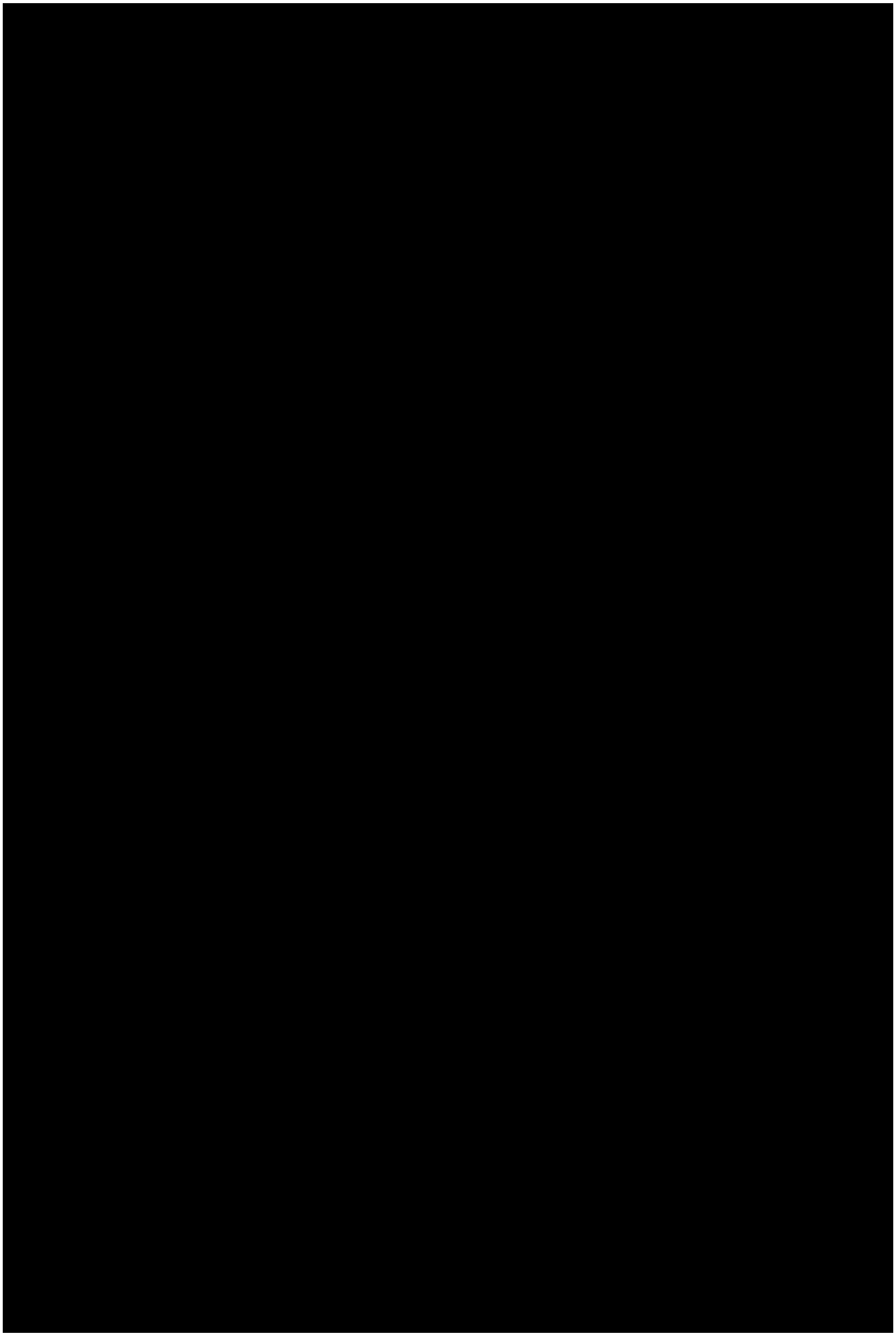
Where Xi and $\hat{X}_i$ are the ith highest observed and computed values for the distribution. The distribution giving the least D-Index is considered to be the best fit distribution.

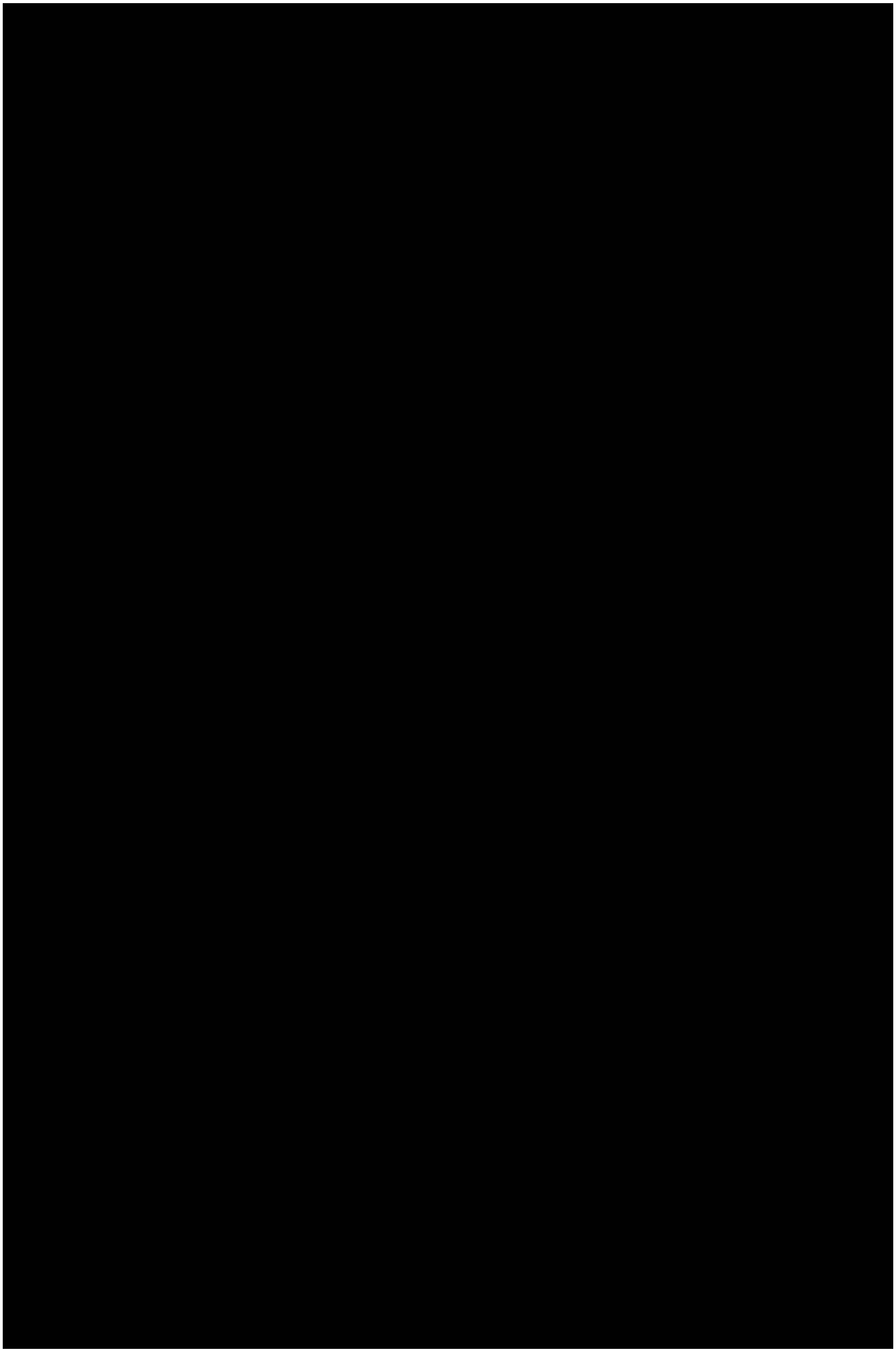
3.8 Worked Example

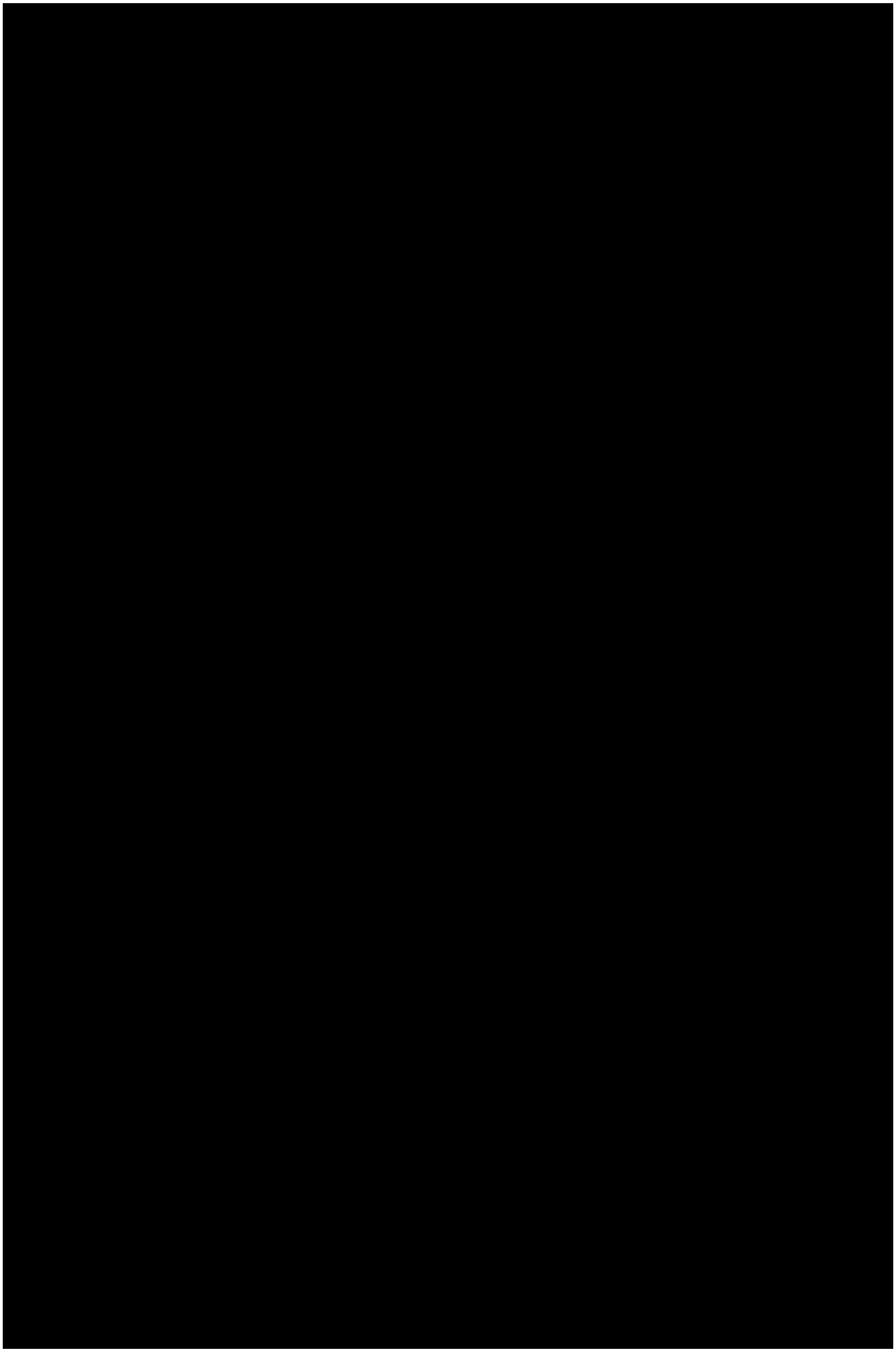
Compute 10 to 1000 Years return period floods for River Awash River at Melkasa Station assuming that the annual flood series (given in the following Table along with statistical parameters).

Year	Discharge (Q m3/sec)	Year	Discharge (Q m3/sec)
1985	652	1998	61
1986	247	1999	490
1987	156	2000	654
1988	116	2001	750
1989	482	2002	509
1990	106	2003	79
1991	204	2004	395
1992	318	2005	484
1993	426	2006	397
1994	634	2007	467
1995	149	2008	920
1996	962		
1997	174		

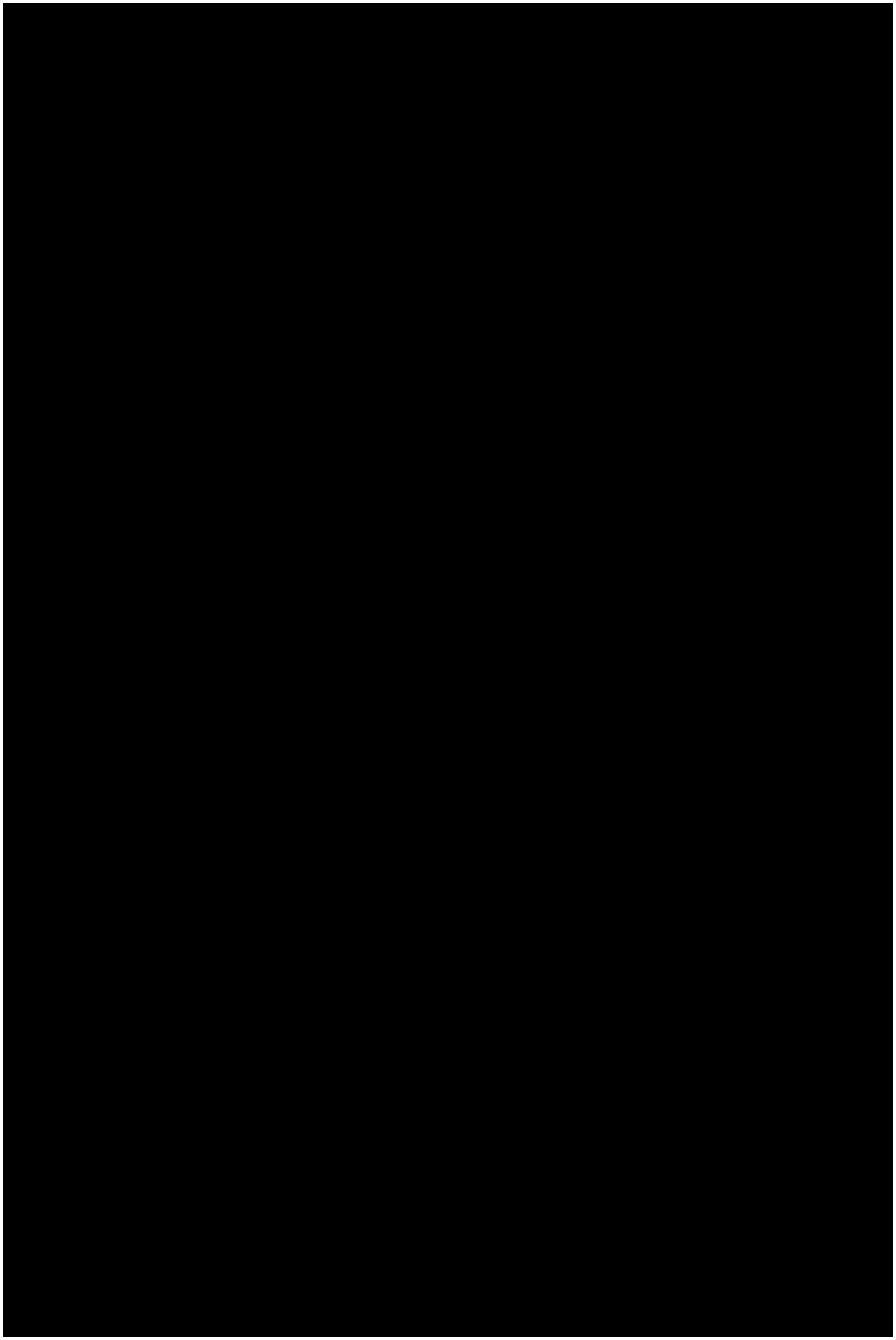
Statistical Parameters of the Data	
Mean	409.52
Standard Deviation	250.966
Coefficient of Skew ness	0.615







[redacted]



4 TIME OF CONCENTRATION

4.1 Introduction

The time of concentration (Tc) is used in numerous equations to calculate discharge, particularly with the rational method. In most watersheds, it is necessary to add the many different time of concentrations resulting from different field conditions that runoff flows through to reach the point of investigation. Water moves through a watershed as sheet flow, shallow concentrated flow, swales, open channels, street gutters, storm sewers, or some combination of these.

This section describes the many conditions and corresponding solutions that need to be considered when estimating the total time of concentration (Tc) (sum of runoff travel time).

There are also many methods utilized to estimate the time of concentration. Examples are the Kinematic Wave Method, Kirpich formula, Kerby formula, etc. However, Kirpich formula is discussed in this manual as it is widely accepted in Ethiopia.

4.2 Definition

The time of concentration is defined as the time required for water falling on the most remote point of a drainage basin to reach the outlet where remoteness relates to time of travel rather than distance. Probably a better definition is that it is the time after the beginning of rainfall excess when all portions of the drainage basin are contributing simultaneously to flow at the outlet.

Using an appropriate value for time of concentration is very important, although it is hard sometimes to judge what the correct value is.

The time of concentration is often assumed to be the sum of two travel times (Tt). The first is the initial time required for the overland flow, and the second is the travel time in the conveyance elements (open channels, streams or rivers, etc).

4.3 Factors Affecting Time of Concentration

Surface roughness: - One of the most significant effects of urban development on flow velocity is a decrease in retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development; the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns: - In small non-urban watersheds, much of the travel time results from overland flow in upstream areas.

4.4 Estimating Time of Concentration

The time of concentration, Tc, is typically defined as the time it takes for rain water for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. The time of concentration should be based upon a flow length and path that results in a time of concentration for only a portion of the area if that portion of the catchment produces a higher rate of runoff. Tc influences the shape and peak of the runoff hydrograph.

One of the most used empirical equations to calculate time of concentration for small drainage areas is the kirpich equation, given by:

$$T_c = \frac{0.0195\ (L/S^{0.5})^{0.77}}{60}$$

Where,

T_c = Time of concentration (hr)

L = the length of the catchement along the longest river channel (m)

S = the overall catchement slope in m/m = ∆H/L

∆H =The difference in elevation between the most remote point on the catchement and the outlet = H1-H2 as shown in figure where H1 and H2 are maximum and minimum elevation of the watershed

Figure 4-1: Catchment Characteristics Contributing the Runoff

Plot Longitudinal Section of the mainstream and identify different reaches having significant slope difference.

Measure the length of stream and their slopes as shown in the figure.

Find the total time of concentration using the following formula.

$TC = TC_1 + Tc_2 + TC_3 + \text{-----}\ TCn$

5 RATIONAL METHOD

5.1 Introduction

The rational method is a commonly used method for estimating flood peak of different return periods for un-gauged catchetment's. It is probably the simplest flood estimation techniques available using rainfall-runoff relationships. In spite of the many criticism regarding its over-simplification of the process of rainfall conversion into runoff, it remains possibly the most widely used method for estimating peak flood flows for a drainage system smaller than 100km^2.

The Rational Method is most accurate for estimating the design storm peak runoff for areas up to 50 hectares (0.5 km^2).

The Rational method is based upon the following formula:

$$Q = 0.278 \times C \times I \times A$$

where Q = flood peak at drainage (m3/s)

C = runoff coefficient

I = rainfall intensity from intensity-duration-frequency curve (mm/hr)

A = catchment area (km2)

The runoff coefficient (C) will be estimated on the basis of three factors - slope of catchment, permeability of soil and vegetation.

5.2 Characteristics

When using the Rational formula, an assumption is made that maximum rate of flow is produced by a constant rainfall, which is maintained for a time equal to the period of concentration of flow at the point under consideration. Theoretically, this is the time of concentration, which is the time required for the surface runoff from the most remote part of the drainage basin to reach the point being considered. However, in practice, the concentration time is an empirical value that results in acceptable peak flow estimates. There are other assumptions used in the Rational method, and thus the designer or engineer should consider how exceptions or other unusual circumstances might affect those results:

a. The recurrence interval of the peak flow rate is the same as that of the average rainfall intensity.

b. The rainfall is uniform in space over the drainage area being considered.

c. The rainfall intensity remains constant during the time period equal to the rainfall intensity averaging time.

d. The storm duration associated with the peak flow rate is equal to the rainfall intensity during the rainfall intensity averaging time to that point.

e. The runoff frequency curve is parallel to the rainfall frequency curve. This implies that the same value of the runoff coefficient is used for all recurrence intervals. In practice, the runoff coefficient is adjusted with a frequency coefficient (Cf) for the 25-year through 100-year.

5.3 Applications

Some precautions shall be considered when applying the Rational Method:

The first step in applying the Rational Method is to obtain a good topographic map and define the boundaries of the catchment area in question. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.

In determining the runoff coefficient C value for the catchment area, thought shall be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system. Also, the effects of upstream detention structures must be taken into account.

Restrictions to the natural flow such as highway crossings and dams that exist in the catchment area shall be investigated to see how they affect the design flows.

5.4 Procedures

The results of using the Rational Formula to estimate peak discharges or design flood is very sensitive to the parameters that are used. The designer must use good engineering judgment in estimating values that are used in the method.

5.4.1 Time of Concentration

The time of concentration is the time required for water to flow from the hydraulically most remote point of the catchment area to the point under investigation. Use of the Rational Method requires the time of concentration (Tc) for each design point within the catchment area. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I).

Open channel flow time can be estimated from the hydraulic properties of the channel using Manning’s Equation. An alternative way to estimate the overland flow time is to use graphical method to estimate overland flow velocity and divide the velocity into the overland travel distance. For each catchment area, the distance is determined from the starting point of the catchment to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance as detail described in **Section 4-4** of this Part of the manual. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.

5.4.2 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. Rainfall-Intensity-Duration curves for the specific catchment area can be developed as shown in **Section 2-5** of this part of the manual. The IDF curve is based for several return periods. Once the IDF curve is developed, the corresponding rainfall intensity for the design period will be obtained from the graph.

5.4.3 Runoff Coefficient

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the designer. A typical coefficient represents the integrated effects of many watershed parameters. Section 5.4.4 (Hydrological Soil Group for Ethiopia and the following paragraph discussion considers the effects of soil groups, land use, and average land slope.

Three methods for determining the runoff coefficient have been discussed and presented based on soil groups and land slope (Table 5-2), land use (Table 5-3), and a composite coefficient for complex catchment areas (Table 5-4).

Table 5-2 gives the recommended runoff coefficient (C) for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this table the C values for non-urban areas such as forest land, agricultural land, and open space can be determined.

5.4.4 Hydrological Soil Groups for Ethiopia

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Permeability and infiltration are the principal data required to classify soils into Hydrologic Soils Groups (HSG). Based on infiltration rates, the Soil Conservation Service (SCS) has divided soils into four hydrologic soil groups as follows:

Group A: Sand, loamy sand or sandy loam. Soils having a low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels.

Group B: Silt loam, or loam. Soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C: Sandy clay loam. Soils having a moderately high runoff potential due to slow infiltration rates. These soils primarily consist of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D: Clay loam, silty clay loam, sandy clay, silty clay or clay. Soils having a high runoff potential due to very slow infiltration rates, These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Soil data for Ethiopia is generally available only for soil types located near major irrigation projects and agricultural research stations. The hydrological soils groups presented in Table 5-1 are based on limited field measurements and from profile morphology and physical characteristics, as recommended in Ethiopian Road Authority Drainage Design Manual. The table is useful in project areas where such soil definitions are available so as to critically define the hydrologic soil group.

	Soil Types	Hydrologic Soil Group
Ao	Orthic Acrisols	B
Bc	Chromic Cambisols	B
Bd	Dystric Cambisols	B
Be	Eutric Cambisols	B
Bh	Humic Cambisols	C
Bk	Calcic Cambisols	B
By	Vertic Cambisols	B
Ck	Calcic Chernozems	B
E	Rendzinas	D
Hh	Haplic Phaeozems	C
H1	Luvic Phaeozems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vertic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosols	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Tm	Mollic Andosols	B
Tv	Vitric Andosols	B
Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D
Xh	Haplic Xerosols	B
Xk	Caloic Xerosols	B
Xl	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B

Table 5-1: Typical Hydrologic Soil Groups for Ethiopia

Source: Ethiopian Road Authority ERA Drainage Design Manual

Terrain Type	Soil Type			
	A	B	C	D
Flat, <2%	0.04-0.09	0.07-0.12	0.11-0.16	0.15-0.20
Rolling, 2-6%	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Mountain, 6-15%	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38
Escarpment, >15%	0.18-0.22	0.24-0.30	0.30-0.40	0.38-0.48

Table 5-2: Recommended Runoff Coefficient C for Pervious Surfaces by Selected Hydrologic Soil Group and Slope Ranges

Source: Ethiopian Road Authority ERA Drainage Design Manual

As the slope of the drainage basin increases, the selected runoff coefficient C should also increase. This is caused by the fact that as the slope of the catchment area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the catchment area.

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Multi units, detached	0.40-0.60
Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

Table 5-3: Recommended Runoff Coefficients C for Various Selected Land Use

Source: Ethiopian Road Authority ERA Drainage Design Manual

5.4.5 Composite runoff analysis

Care should be taken not to average runoff coefficients for large segments that have multiple land uses of a wide variety (i.e., business to agriculture). However, within similar land uses, it is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. The composite procedure can be applied to an entire drainage area or to typical sample blocks as a guide to selection of reasonable values of the coefficient for an entire area.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the catchment area. Composites can be made with Tables 5-2 and 5-3. At a more detailed level composites can be made with Table 5-2 and the coefficients with respect to surface type given in Table 5-4. The composite procedure can be applied to an entire catchment area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area.

Surface	Runoff Coefficients
Street: Asphalt	0.70-0.95
Concrete	0.80-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

Table 5-4: Coefficients for Composite Runoff Analysis

Source: Ethiopian Road Authority ERA Drainage Design Manual

6 SCS UNIT HYDROGRAPH METHOD

6.1 Introduction

Techniques developed by the U. S. Soil Conservation Service for calculating rates of runoff require the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or synthesised, by subtracting infiltration and other losses from the rainfall to obtain the rainfall excess.

6.2 Catchment Area

A catchment area is determined from topographic maps and field surveys. For large catchment areas it might be necessary to divide the area into sub-catchment areas to account for major land use changes, obtain analysis results at different points within the catchment area, or locate storm water drainage structures and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems shall be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the subcatchment areas.

6.3 Procedures

The general principle of the method is described below.

a) Rainfall-Runoff Relationship

The basic rainfall-runoff relationship in the SCS methodology is expressed as:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

where P = maximum potential runoff (mm)

Q = actual runoff (mm)

Ia = initial abstraction (mm)

S = maximum potential retention (mm)

The initial abstraction consists of interception, infiltration and surface storage, all of which occur before runoff begins. The empirical relation between initial abstraction and the potential maximum retention is:

$$I_a = 0.2S$$

Substitution gives:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

Which, is the rainfall-runoff relationship normally used.

S is related to the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100, and S is related to CN by:

S = 1000/CN –10

6.4 Runoff Curve Number

Different runoff coefficients for the use in the Rational Formula and Curve Number in the SCS method for various land uses, topographic conditions and soil types are recommended in the ERA Drainage Design Manual - 2002 as summarised in the tables below.

The first tables (Tables 6-1 through 6-4) give curve numbers for various land uses. These tables are based on an average antecedent moisture condition, i.e., soils that are neither very wet nor very dry when the design storm begins. Curve numbers shall be selected only after a field inspection of the catchment area and a review of cover type and soil maps. Table 6-5 gives the antecedent conditions for the three classifications.

Care shall be taken in the selection of curve numbers (CN’s). Use a representative average curve number, CN, for the catchment area. Selection of overly conservative CN’s will result in the estimation of excessively high runoff and consequently excessively costly drainage structures. Selection of conservatively high values for all runoff variables results in compounding the runoff estimation. It is better to use average values and design for a longer storm frequency. Often the runoff computed using conservative CN's for a ten year storm will greatly exceed the computed runoff for average CN's for a 25 or even 50 year storm. The hydrologic designer could consider doing both in making the most appropriate selection of design discharge.

The average percent impervious area related with Table 6-1 can also be used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

Cover Description			Curve Numbers for Hydrologic Soil Groups			
Cover Type	**Treatment**	**Hydrologic Condition**	**A**	**B**	**C**	**D**
Open space[3]	Grass cover < 50%	Poor	68	79	86	89
	Grass cover 50 - 75%	Fair	49	69	79	84
	Grass cover >75%	Good	39	61	74	80
Impervious areas	Paved parking lots, roofs, driveways, etc.	--	98	98	98	98
Streets and roads	Paved; curbs and storm drains	--	98	98	98	98
	Paved; open ditches	--	83	89	92	93
	Gravel lined	--	76	85	89	91
	Dirt	--	72	82	87	89
Urban districts	Commercial and business	--	89	92	94	95
	Industrial	--	81	88	91	93
Developing urban areas	Pervious areas only no vegetation	--	77	86	91	94
Residential areas	Town houses newly graded areas	--	77	85	90	92

1. Average runoff condition, and $I_a = 0.2S$
2. CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Table 6-1: Runoff Curve Numbers for Urban Areas[1]

Source: Ethiopian Road Authority ERA Drainage Design Manual

Curve Number Description			**Curve Numbers for Hydrologic Soil group**			
Cover Type	Treatment[2]	Hydrologic Condition[3]	A	B	C	D
Fallow	Bare Soil	-	77	86	91	94
	Crop residue	Poor	76	85	90	93
	Cover (CR)	Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Closed-seeded SR or broadcast	Poor	66	77	85	89
		Good	58	72	81	85
	Legumes or C Rotation	Poor	64	75	83	85
		Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

1. Average runoff condition, and Ia = 0.2S.
2. Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
3. Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%), and (e) degree of roughness.
   Poor: Factors impair infiltration and tend to increase runoff.
   Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 6-2: Cultivated Agricultural Land [1]

Source: Ethiopian Road Authority ERA Drainage Design Manual

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range-continuous	Poor	68	79	86	89
forage for grazing[2]	Fair	49	69	79	84
Meadow-continuous grass, protected	Good	39	61	74	80
from grazing	--	35	59	72	79
Brush-weed-grass mixture with brush	Poor	48	67	77	83
the major element[3]	Fair	35	56	70	77
Woods-grass combination[5]	Good	30	48	65	73
Woods[6]	Poor	57	73	82	86
	Fair	43	65	76	82
Farms—buildings, lanes, driveways,	Good	32	58	72	79
and surrounding lots	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
	--	59	74	82	86

1. Average runoff condition, and Ia= 0.2S
2. Poor: < 50% ground cover or heavily grazed with no mulch
   Fair: 50 to 75% ground cover and not heavily grazed
   Good: > 75% ground cover and lightly or only occasionally grazed
3. Poor: < 50% ground cover
   Fair: 50 to 75% ground cover
   Good: > 75% ground cover
4. Actual curve number is less than 30; use CN = 30 for runoff computations.
5. CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.
6. Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
   Fair: Woods grazed but not burned, and some forest litter covers the soil.
   Good: Woods protected from grazing, litter and brush adequately cover soil.

Table 6-3: Runoff Curve Numbers for Cultivated Agricultural Land

Source: Ethiopian Road Authority ERA Drainage Design Manual

Cover Description	Curve numbers for hydrologic soil group				
Cover type	**Hydrologic condition[2]**	**A[3]**	**B**	**C**	**D**
Mixture of grass, weeds, and low-growing	Poor		80	87	93
brush, with brush the minor element	Fair		71	81	89
Mountain brush mixture of small trees and	Good		62	74	85
brush	Poor		66	74	79
	Fair		48	57	63
Small trees with grass understory	Good		30	41	48
	Poor		75	85	89
Brush with grass understory	Fair		58	73	80
	Good		41	61	71
	Poor		67	80	85
Desert shrub brush	Fair		51	63	70
	Good		35	47	55
	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1. Average runoff condition, and Ia= 0.2S
2. Poor : < 30 % ground cover (litter, grass, and brush overstory)
   Fair: 30 to 70 % ground cover Good: > 70 % ground cover
3. Curve numbers for Group A have been developed only for desert shrub

Table 6-4: Arid and Semiarid Rangelands[1] Source: ERA Drainage Design Manual

CN for Average Conditions	Corresponding CN's for	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Table 6-5: Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions

FLOOD PROTECTION DESIGN MANUAL
PART 7: HYDRAULIC CHANNEL DESIGN

Table of Contents

CONTENTS OF OTHER PARTS

1. INTRODUCTION

1.1 Scope of the Manual

Part 5: Hydraulic Channel Design of the Design Manual covers design for construction, rehabilitation and upgrading of natural channels to contain and/ or accommodate flood water within the river bank and/or within the flood corridor zone while the flood spill out off the river bank and flowing on flood plain.

The importance of discussing hydraulic channel design in the case of flood protection works is to contain flood within the river bank considering rivers as natural channels.

Whether it is artificial or natural channels the basic principles is the same open channel hydraulics and hence, the application of hydraulic channel design in the case of flood protection works depends on sound knowledge of basic open channel hydraulic principles and engineering judgement. Therefore, first different methods of hydraulic channel designs will be discussed in this section of the manual considering both cases of channels of natural river channels for flood containing projects and artificial channels for diverting flood. Then application of hydraulic channel design for flood containing or for flood diversion will be discussed.

1.2 Reconnaissance Study

Detailed condition surveys of the river regime are required, to determine how much flood can be accommodated within the river bank and what additional size will be required to accommodate the design flood. The river regime and morphology have been discussed in **Chapter 2 of Part 8: Flood Protection Structures** and the reader is advised to reefer the material.

Critical section of the river regime should be determined and the layout of the flood protection with respect to the river channel layout. When deciding the layout of a river channels for flood protection the following needs to be considered during reconnaissance study:

- ✓ Topography of the flood prone area to be protected;
- ✓ Topography of the river channels;
- ✓ Topography of existing flood protection works;
- ✓ Objective of the flood protection work;
- ✓ Existing water abstraction methods;
- ✓ Land ownership;
- ✓ Design Flood Discharge for flood containing structures.

The detail of procedures for the above issues is discussed in their respective section of this manual.

During the reconnaissance study and investigation the type of flood mitigation work should be conceptualized the type of structures at least to the sketch level. Once the flood mitigation work is conceptualized during the field investigation, the next step is detail open channel hydraulic analysis principles which will be discussed in the subsequent chapters and also supported in other parts of this manual.

1.3 Design Procedures

For the hydraulic design of a channel two main parameters have to be determined once the required design discharge is known. These are:

- ✓ the bed width/flow depth (b/d) ratio; and

✓ the longitudinal slope.

Hydraulic design of natural channels for flood protection work is to decide the cross section of the flood corridor out of the river bank including the embankments layout and top level.

Uniform flow formulae such as Manning and Strickler-Manning are commonly used to determine the relationship between cross section area and longitudinal slope for both natural and artificial channels. For a stable channel section b/d value depends on the discharge as well as the soil in which the channel is made and the sediment being transported. Basically the higher the discharge the larger the b/d value, while the more tenacious (cohesive) the soil the tighter the channel section (smaller b/d value).

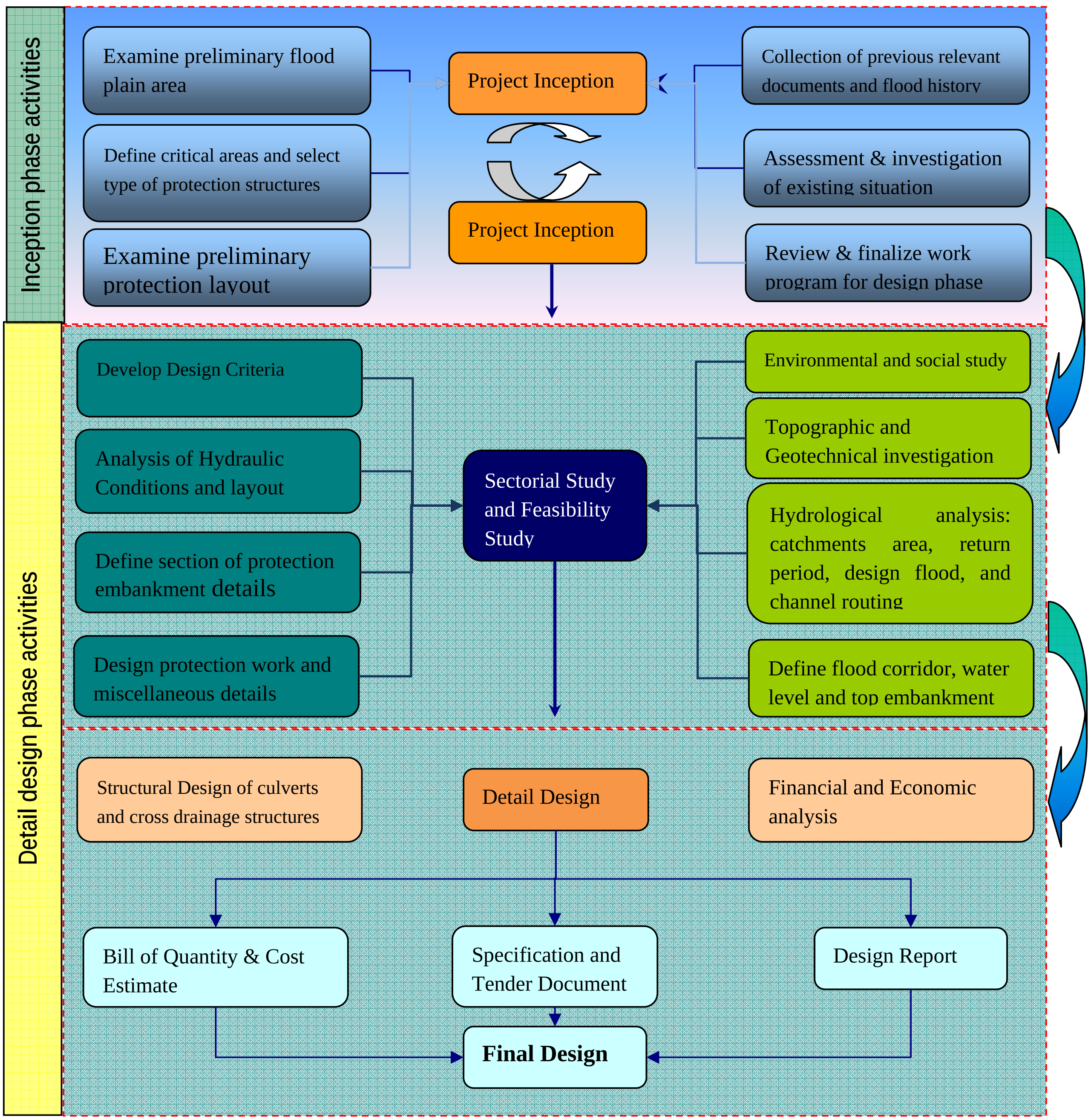


Figure 1-1: Flood Protection Design Phase Work Flow Diagram

Overall design procedures for flood protection works have been summarized as is shown in Figure 1-1 which includes all steps starting from field conceptualization up to the final design report preparation. Detail of each activity is discussed in it is relevant part of the manual. The flow diagram is to represent the overall design procedures.

The detail hydraulic analysis of the stream and the flood corridor required routing of the design flood with in the channel. For this purpose schematization of the channel is required through topographic map survey as discussed in Part 5 Topographic Map Survey Section 3-5. However, for quick reference, a very short summary of topographic survey and map requirements for flood containing structure design have been discussed in this section.

1.4 Topographic Survey and Channel Layout

When deciding the alignment or layout of flood protection dykes for flood protection the following needs to be considered during the reconnaissance and field investigation:

- ✓ Topographic map, aerial photograph and satellite imagery of the flood prone area;
- ✓ Topography of the flood prone area to be protected;
- ✓ Topography of the river channels including profile and cross sections;
- ✓ Existing water abstraction methods like intake site, diversion weir, pumping site location;
- ✓ Land ownership;
- ✓ Design Flood Discharge for flood protection structures.

In addition to maps and imagery data's the general feature of flood project is often gained by walking along the flood corridor with members of the community responsible for or concerned about, their operation and maintenance. This will add information on identification of critical areas and also to conceptualise the type of protection structures.

Major types of topographic survey required for flood protection works are:

- ✓ Strip surveys for natural channels at 1: 5,000 and 1:2,000 scale with contours at 0.5m intervals for “flat” areas and 1.0m for “hilly” areas
- ✓ Channel long sections at 1:2,000 (H) and 1:200 or 1:100 (V) scales;
- ✓ Embankment long sections (if it exists) at 1:2,000 (H) and 1:200 or 1:100 (V) scales;
- ✓ Channel cross sections extending up to existing flood embankment and high ground level at 1:200 or 1:100 (V&H) scales;
- ✓ Benchmark data including description of benchmarks

The strip survey should be aimed to produce contour map of the channel, both sides of the river boundaries top and bottom of river banks and flood plain extending up to the cliff edges or high ground level. In most cases the back side of embankment dyke is flat flood plain area and the extent of topographic survey to the flood plain area should be limited considering the scope and size of flood protection works.

After marking on contours to topographic maps, analysis will typically be carried out as follows:

- ✓ Layout of the river channel including existing flood protection structures;
- ✓ All existing features shall be marked out of flood plain, command land and village areas would be identified and shaded grey;

- ✓ Flood marks should be marked;
- ✓ Drainage lines (routes) following the low land would be marked in blue;
- ✓ Cross drainage requirements would be marked on the topographic map as well as on the profiles.

The Long section is aimed to produce the river bed slope which is one of the bases for the design data requirements and it should be able to represent nearly the river bed slope. The long section of the river bed should be plotted with the long profile of river banks (both left and right banks), long profile of flood embankment and flood mark if available (See Figure 5-4).

Cross Sections are also the basic design parameters for the design process of flood protection works (See Figure 5-3). The channel cross section survey should be able to give river bed, top and bottom of the river bank, top and bottom of the existing flood embankment and any general features like roads, pole etc.

2. OPEN CHANNEL HYDRAULICS

2.1 General

The beginning of any channel design or modification is to understand the hydraulics of the stream. The procedures for performing uniform flow calculations aid in the selection or evaluation of appropriate depths, grades and widths for natural or man-made channels. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's equation.

Open channel flow can be classified in to many types and described in various ways. (V.T. Chow, Open Channel Hydraulics). Fundamental types of flows have shortly been presented in the flowing section as discussed in Chow. Understanding the basic open channel flow principles will help in analysing the hydraulic flow of channels.

Steady Flow and Unsteady Flow: Flow in an open channel is said to be steady if the depth of flow dose not change or if it can be assumed to be constant during the time interval under consideration. The flow is unsteady if the depth changes with time. In most open channel problems it is advisable to study flow behaviour only under steady conditions. If, however, the change in flow condition with respect to time is of major concern, the flow should be treated as unsteady. In floods, for instances, which is typical example of unsteady flow, the stage of flow changes instantaneously as the wave pass by, and the time element becomes vitally important in the design of control structures.

For any flow, the discharge Q at a channel section is expressed by

$$Q = VA$$

Where V is the mean velocity and A is the flow cross-sectional area normal to the direction of the flow, since the mean velocity is defined as the discharge divided by the cross-sectional area.

In most problems of steady flow the discharge is constant throughout the reach of the channel under consideration; in the words, the flow is continuous.

$$Q = V_1A_1 = V_2A_2$$

Where the subscripts designate different channel sections. This is the continuity equation for a continuous steady flow.

Uniform Flow and Varied Flow: Space as the criterion. Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel. A uniform flow may be steady or unsteady, depending on whether or not the depth changes with time.

The classification of open channel flow is summarized as follows:

A. Steady Flow
   1. Uniform Flow
   2. Varied Flow
      a. Gradually Varied Flow
      b. Rapidly Varied Flow
B. Unsteady Flow
   1. Unsteady uniform flow (rare)
   2. Unsteady flow (i.e., unsteady varied flow)
      a. Gradually Varied unsteady flow
      b. Rapidly Varied unsteady flow

2.2 Definitions

Critical flow: The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge, when the specific energy is held constant.

Froude number: The Froude number is an important dimensionless parameter in open-channel flow. It represents the ratio of inertia forces to gravity forces. This expression for Froude number applies to any single-section channel of nonrectangular shape.

Hydraulic jump: Hydraulic jumps occur at abrupt transitions from supercritical to sub critical flow in the flow direction. There are significant changes in the depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at flood protection structures.

Kinetic energy coefficient: As the velocity distribution in a river varies from a maximum at the design portion of the channel to essentially zero along the banks, the average velocity head.

Normal depth: For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in a steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in a steady, uniform flow and is computed from Manning's equation.

Specific energy: Specific energy (E) is the energy head relative to the channel bottom. If the channel is not too steep (slope less than 10%), and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head. The kinetic energy correction coefficient is taken to have a value of one for turbulent flow in prismatic channels but may be significantly different from one in natural channels.

Steady and unsteady flow: A steady flow is when the discharge passing a given cross section is constant with respect to time. When the discharge varies with time, the flow is unsteady. The maintenance of steady flow requires that the rates of inflow and outflow be constant and equal.

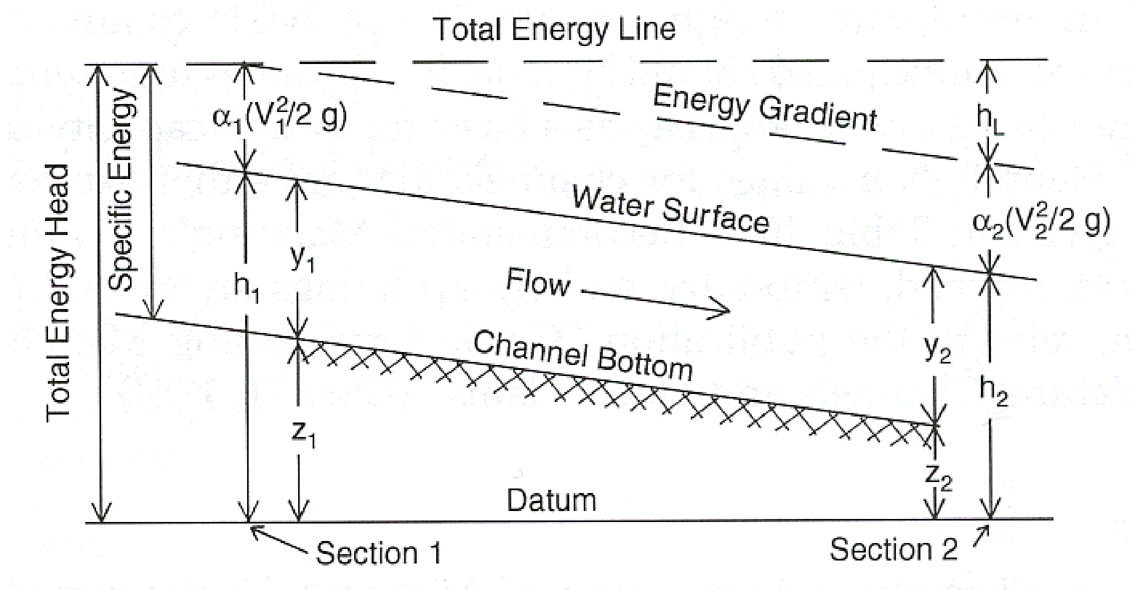
Sub-critical flow: Depths of flow greater than critical depths, resulting from relatively flat slopes. Froude number is less than one. Flow of this type is most common in flat streams.

Supercritical flow: Depths of flow less than critical depths resulting from relatively steep slopes. Froude number is greater than one. Flow of this type is most common is steep streams.

Total energy head: The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The curve of the energy head from one cross section to the next defines the energy grade line.

Uniform flow and non-uniform flow: A non-uniform flow is one in which the velocity and depth vary over distance, while they remain constant in uniform flow. Uniform flow can occur only in a channel of constant cross section, roughness, and slope in the flow direction; however, non-uniform flow can occur in such a channel or in a natural channel with variable properties.

2.3 Uniform Flow



Uniform Flow - A non-uniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the flow direction.

2.4 Uniform Critical Flow Analysis

The flow in open channels at which the energy content of the fluid is at a minimum also, that flow at which the variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called Critical Depth at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge when the specific energy is held constant.

2.5 Gradually Varying Flow

Gradually-Varied - a non-uniform flow, in which the depth and velocity change gradually in the flow direction, such that vertical accelerations can be neglected, is referred to as a gradually-varied flow.

2.6 Rapidly Varying Flow

Rapidly-Varied -A non-uniform flow, in which the depth and velocity change rapidly in the flow direction, such that vertical accelerations cannot be neglected, is considered to be rapidly-varied.

3. UNIFORM FLOW FORMULA

3.1 General

The scope of this part of the manual is limited to the design of open channels which uses the basic open channel hydraulic principles. The discussion in this section gives basic uniform flow formulas which are assumed to be very useful in the design and analysis of river channel structures including flood embankment design. If the need arises the user is expected to reefer additional reference materials for more detail of open channel hydraulics.

3.2 Manning's Equation

The normal depth is used to design artificial channels in a steady, uniform flow and is computed using Manning's equation. This formula, which was empirically derived towards the end of the 19th century, relates discharge to area of flow, the slope of the energy line, the shape of the channel and the roughness of the boundaries of the channel. It is the most widely used and recommended formula for the analysis of river channel characterization. The parameters can easily be found from field investigation.

Manning's formula is:

$$Q = A\, S_b^{0.5}\, R^{2/3} / n \qquad [m^3/s]$$

Where:

Q	=	the discharge [m^3/s]
A	=	the area of flow [m^2]
S_b	=	the slope of the energy line
R	=	the hydraulic radius = A/P [m]
P	=	the wetted perimeter of the flow [m]
n	=	Manning=s roughness coefficient

The slope of the river bed or channel bed (S_b) and river cross sections are the design parameters that can be obtained from the topographic survey in case of river channel analysis for flood protection works. Once the cross-sectional areas and river bed slope are obtained from the field investigation, the next step is to work out the depth of the design flood with in the channel so as to fined out the flood level and then decide the embankment protection level.

In addition to the channel slope and section, surface roughness exerts an influence on the value of "n", which is one of the basic parameters for the design of channels. The value of "n" for natural channels depends on the following factors which should carefully be considered in the design process:

- ✓ **Vegetation**. Weeds may be regarded as a kind of surface roughness, but they also markedly reduce the capacity of the channel and retard the flow. Its influence depends on the height, density, distribution and type of vegetation and taking likely vegetation growth into account is very important when designing channels.
- ✓ **Channel irregularity**. Variations in the size and shape of a channel along its length will tend to increase the effective value of "*n*".

- ✓ **Channel alignment**. Smooth curves with large radii will give lower values of "*n"* than sharp curves.
- ✓ **Silting and scouring**. Silting may change an irregular channel into a comparatively uniform one and decrease "*n"* though this also depends on the nature of the material deposited. The effect of scouring is generally less significant as long as the erosion of the channel bed progresses evenly and uniformly.
- ✓ **Obstructions**. The presence of log jams and the like tend to increase "n". The amount of the increase depends on the nature of the obstructions, their size, shape, number and distribution.
- ✓ **Stage and discharge**. The value of *n* in most channels decreases with increasing stage and/or discharge.
- ✓ **Seasonal changes**. The value of *n* may vary seasonally with weed growth.
- ✓ **Suspended material and bed load**. Suspended material and bed load, whether moving or not moving, consumes energy and additional head losses, increasing the effective value of "*n".*

Type of Channel and Description	Minimum	Normal	Maximum
Straight and Uniform Excavated Earth Channel in Clean Recently Completed Condition	0.016	0.018	0.020
Straight and Uniform Excavated Earth Channel in Clean but weathered Condition	0.018	0.022	0.025
Straight and Uniform Excavated Clean Gravel Channel	0.022	0.025	0.030
Straight and Uniform Excavated Channel with Short Grass and Few Weeds	0.022	0.027	0.033
Clean Straight Natural Stream on the Plain, Full Stage with no Rifts or Deep Pools	0.025	0.030	0.033
Mountain Stream with Steep Banks and No Vegetation, Bottom of Gravels, Cobbles and a few Boulders	0.030	0.040	0.050
Mountain Stream with Steep Banks and No Vegetation, Bottom of Cobbles and large Boulders	0.040	0.050	0.070

Table 3-1: Values of Manning's Roughness Coefficient, n, for Various Materials

Source: Chow, VT Open Channel Hydraulics

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. If the normal depth computed from Manning's equation is greater than critical depth, the slope is classified as a mild slope, while on a steep slope; the normal depth is less than critical depth. Thus, uniform flow is sub critical on a mild slope and supercritical on a steep slope.

Strictly speaking, uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes in river engineering, however, the Manning equation can be applied to most stream flow problems by making judicious assumptions. When the requirements for uniform flow are met, the depth (dn) and the velocity (Vn) are said to be normal and the slopes of the water surface and channel are parallel. For practical purposes, in open channel design, minor undulations in streambed or minor deviations from the mean (average) cross-section can be ignored as long as the mean slope of the channel can be represented as a straight line.

3.3 Strickler Manning Equation

The Strickler Manning's formula is:

$$Q = A\ k\ S_f^{0.5}\ R^{2/3} \qquad [m^3/s]$$

Where:

Q	=	the discharge $[m^3/s]$
A	=	the area of flow $[m^2]$
S_f	=	the slope of the energy line
R	=	the hydraulic radius = A/P [m]
P	=	the wetted perimeter of the flow [m]
k	=	Strickler=s roughness coefficient

Relation of the above two formulae gives k = 1/n.

3.4 Kennedy's Critical Velocity

RG Kennedy, working in India, deduced from observations that, for non-silting and non-scouring alluvial channels, there is a certain velocity, which he called the critical velocity (Vc). The critical velocity is a function of the depth of water in the channel (d) and the type of silt being transported. Kennedy showed that:

$$V_c = 0.84\ f\ d^{0.64}$$

Where: f = a factor depending on the nature of the silt being transported, as given in the table below.

Nature of Silt	f
Fine Silt	0.70
Light Sandy Silt	1.00
Rather Coarse Silt	1.30

Table 3-2: Silt Factor

For a range of channels, the critical velocity determined using Kennedy's equation is given in Table 3-3. For irrigation schemes abstracting water from a river and provided with a sand trap, the minimum velocity may probably be taken assuming transportation of for rather coarse silt. Where clear water is diverted, for example from groundwater, then the minimum velocity may safely be taken for fine silt.

Water Depth, d	**For fine silt such as the Indus in Sindh (m = 0.70)**	**Light Sandy Silt (m = 1.00)**	**Rather Coarse Silt (m = 1.30)**
(m)	**(m/s)**	**(m/s)**	**(m/s)**
0.3	0.2	0.2	0.3
0.6	0.3	0.4	0.5
0.9	0.4	0.5	0.7

Table 3-3: Kennedy's Critical Velocity

3.5 Maximum Permissible Velocity

The maximum permissible average flow velocity which will not cause erosion of the channel's body or river bank can be estimated only with experience and judgement. In general, old and well-seasoned channels will stand much higher velocities than new ones.

The maximum permissible velocities for different types of soils for straight channels are given in Table 3-4.

Type of Soil	**Manning's n**	**Clear Water**	**Water transporting Colloidal Silts**
		(m/s)	**(m/s)**
Fine Sand, colloidal	0.020	0.46	0.76
Sandy loam, non colloidal	0.020	0.53	0.76
Silt loam, non colloidal	0.020	0.61	0.91
Alluvial silts, non colloidal	0.020	0.61	1.07
Ordinary firm loam	0.020	0.76	1.07
Stiff Clay, very colloidal	0.025	1.14	1.52
Alluvial silts, colloidal	0.025	1.14	1.52
Shales and hardpans	0.025	1.83	1.83
Fine gravel	0.020	1.83	1.52
Graded loam to cobbles, non colloidal	0.030	1.14	1.52
Graded loam to cobbles, colloidal	0.030	1.22	1.68
Coarse gravel, non colloidal	0.025	1.22	1.83
Cobbles and shingles	0.035	1.52	1.68

Table 3-4: Maximum Permissible Velocities for Straight Channels after Aging

Source: Open Channel Hydraulics; VT Chow

4. LACEY REGIME EQUATION

4.1 Introduction

The Lacey regime equations were developed in the mid C20th in India (Pakistan) for channels flowing in alluvial soils. They may be adapted with some success for channels in colloidal (cohesive) soils and are included in this Manual to allow design of unlined channels for a fuller range of soil types.

Lacey observed that a channel is considered to be in regime if, over a hydrological cycle, neither not erosion nor deposition of material occurs.

There is only one section and only one slope at which a channel carrying a given discharge will carry a particular grade and quantity of sediment. Natural silt transporting channels have a tendency to assume a semi-elliptical section. The coarser the sediment, the flatter is the semi-ellipse (i.e. the wider the section). The finer the sediment, the more nearly does the section approximate to a semi circle.

4.2 Lacey Regime Equations

The Lacey's regime equations were adopted in 1934 by the Central Board of Irrigation, and are essentially a set of four empirical equations evolved after the study of a number of stable channels. The four equations are listed below and are discussed in following sections.

Water Surface Width (W_s)

$$W_s = 4.83\ e\ Q^{1/2} \quad \text{[metric units]}$$

Sediment factor (f)

Although the sediment factor 'f' may be related to bed material size it is considered preferable to determine its value by measuring a similar channel exhibiting regime and using the following equation:

$$f_{vr} = 2.46\ V^2/R \quad \text{(metric units)}$$

Regime Slope equation

$$S = 0.0003\ f^{5/3} / Q^{1/6} \quad \text{[metric units]}$$

Lacey Uniform Flow Formula

$$V = R^{3/4}\ S^{1/2} / N \quad \text{[metric units]}$$

$$N = 0.0225\ f^{1/4}$$

Where:

W_s = water surface width [m]

f = silt or sediment factor (may be estimated from bed sediment size, f_m, or from V & R for similar channel in regime, f_{vr})
Q = dominate discharge [m^3/s]
e = width factor
V = average velocity [m/s]
R = hydraulic radius [m]
S = hydraulic slope
N = co-efficient of rugosity.

4.3 Limitations

Lacey's regime equations (given above) have been supplemented and modified since 1934 by additional data. However there are still two obvious limitations of these equations. First they do not consider sediment load as a variable. The equations were evolved using data for stable channels in India and Pakistan carrying sediment concentrations generally less than 500mg/l by weight and may not be applicable when the sediment load is much higher. Secondly, the equations do not quantify resistance due to dunes and ripples.

4.4 Water Surface Width

As stated previously, in 1895 Kennedy found by observing stable channels that a non-scouring and non-silting velocity may be related to depth and sediment size. His formula was used together with a uniform flow equation like Manning's to design "regime" channels. The main limitation to design at this time was that where was no established relationship between bed width and depth, or for channel slope.

Lacey in 1919 established a relationship between the wetted perimeter (P) and the dominant channel discharge (Q). This first equation was subsequently converted into terms of the water surface width (Ws), and a width factor (e) was introduced. The width factor recognized that if the suspended sediment contained fine silt or clay, or the channel has been originally excavated in fine but tenacious (cohesive) material, the width could be safely reduced. Alternatively, if the soil was coarse non-cohesive alluvium (friable) and the charge (suspended load) heavy the width should be increased.

The width factor (e) generally varies from 0.7 to 1.0 for tenacious material, and from 1.0 to 1.1 for friable material. In Sindh, it was soon discovered that where Q was taken as the average mid-kharif discharge (i.e. over a 1 to 2 month period), the width factor was about 0.78.

In building a new channel it is advisable to err on the high side in selecting a value for "e", particularly if land is available. This will cause a slightly over wide channel where berms will be deposited.

4.5 Sediment Factor

The sediment factor may be considered a function of the size of the sediment transported and of the silt "charge" or concentration.

Under normal conditions, i.e. with relatively small charge admitted at the head works, various equations relating the sediment factor to sediment size have been formulated, including:

$$f_m = 1.60\ m^{2/3}$$
$$f_m = 1.76\ m^{1/2}$$

where m = average size in mm of the transported bed material admitted to the channel system.

Using the second equation (f_m = 1.76 m1/2), values for f are given in Table 3.7 for different bed material sizes.

Type of Transported Bed Material		Grain Size (mm)	Sediment Factor (f)
Silt	Very Fine	0.052	0.40
	Fine	0.081	0.50
	Fine	0.120	0.60
	Medium	0.158	0.70
	Standard	0.323	1.00
Sand	Medium	0.505	1.25
	Coarse	0.725	1.50
Bajri & Sand	Fine	0.888	1.75
	Medium	1.29	2.00
	Coarse	2.42	2.75
Gravel	Medium	7.28	4.75
	Heavy	26.1	9.00
Boulders	Small	50.1	12.0
	Medium	72.5	15.0
	Large	184	24.0

Table 4-1: Lacey's Sediment Factor

Based on data collected in 1962 to 63 it was proposed that the sediment factor (f) should be determined based on the measurements of channels exhibiting regime using the equations given in **Section 4.5** above. This is preferable provided that a similar channel in regime may be observed.

4.6 Hydraulic Slopes

From the regime slope equation, it may be observed that smaller channels require steeper slopes to transport the sediment bed load, and that the larger the size of the bed material, the steeper the required slope.

Whenever the regime slope is less than the natural ground slope, drops may be built. For small channels (i.e. minors and watercourses) exceptions may be made on grounds of economy, and the sediment factor increased a little, if this obviates the need for an expensive drop structure.

If there is insufficient available slope, which may be the case particularly for smaller channels, the following options are available:

- ✓ Adopt a “tighter” channel section (i.e. a smaller b/d ratio);
- ✓ Increase the sediment factor (f) a little;
- ✓ class the effected land as out of command.

The penalty of opting for a tighter section is that scour of the banks may occur with a tendency for meanders to develop and berms to be eroded, while adopting overly low sediment factor will result in sedimentation and the need for desilting.

4.7 Design Procedure

Once the sediment factor and the slope of the channel are fixed the next step is to calculate the bed width and depth of the channel. For alluvial channels, the side slope used in the hydraulic design may be quite steep at 1V : 0.5H, and is unlikely to be flatter than 1V:1.5H. This is because it is assumed that the channel would ultimately acquire this side slope due to deposition of silt, even if the initial (newly built) side slope has to be much flatter for stability during and immediately after construction (see Section 5.2).

The design procedure is as follows:

- ✓ Determine the design discharge (Q_d)
- ✓ Adopt suitable values for the sediment factor “f”, the width factor “e” and the channel side slope
- ✓ Determine the channel bed slope (S).
- ✓ Determine the water surface width (W_s), and then the channel bed width (b)
- ✓ By trial, determine a value for “d”, so that the channel can carry the required design discharge (Q_d)

5. HYDRAULIC DESIGN PROCEDURES FOR CHANNELS

5.1 Design of Artificial Channels

5.1.1 Approach

The determination of slope and section dimensions for artificial channels includes the following steps:

- ✓ Determine the design discharge, estimate Manning's "n", and select the bed slope, Sb. For a lined channel the bed slope can vary tremendously, and should be taken as the average natural land slope as a first estimate. Later the bed slope can be adjusted so that the average flow velocity is within recommended limits.
- ✓ Adopting values for bed width (b), channel side slope (m) and flow depth (d), calculate the area of flow (A), the wetted perimeter (P), the hydraulic radius (R), and the discharge (Q). Adjust the section parameters so that the calculated discharge equals the design discharge capacity required for the channel.

The adopted slope and section dimensions should result in the minimum possible cost, and also average flow velocities that are not so low that sedimentation and weed growth becomes a problem, or so high that the bed material is quickly damaged by scour.

For unlined channels the design approach may be the same as for lined channels, except that the b/d ratio is selected so as to provide a stable channel section, and the slope and section size must be sufficient to carry the design discharge while the average flow velocity remains non-silting non-scouring.

5.1.2 Width / Depth Ratio for Unlined Channels

The bed width/depth (b/d) ratio for a lined channel should be chosen to minimise the cost of lining. The following b/d and side slopes were recommended for unlined channels for preliminary design process. These are suited to channels built in colloidal soils (i.e. with some cohesion). Channels in alluvial soils would have to be flatter and wider (see Lacey Regime equations in Chapter 5 of this manual).

Discharge (m3/s)	Side Slope (1V : mH)	Bed Width / Water Depth (b/d)
0.15 to 1.5	1.0	1.0 to 1.8
1.5 to 10.0	1.5	1.8 to 3.9
10.0 to 40.0	2.0	3.9 to 9.0

Table 5-1: Recommended b/d Ratios for Unlined Channels

5.1.3 Average Flow Velocity in Unlined Channels

The average flow velocity should be close to the Kennedy velocity for channels built in fine alluvial material. For channels in other soils, including cohesive soils, higher flow velocities are possible, but should not exceed the values given in Section 4-5 (Maximum Permissible Velocities).

5.1.4 Design Procedures

For easy of presentation canal design procedures for water distribution canal system have been discussed in this section which later be discussed for natural channels.

Prior to starting design process, canal lay out should be drawn out on the maps, and decision made on the types and locations of canal structures. The procedures for the design have been summarised as follows:

I. Determine design discharge (See Hydrology Part)
II. Adopt suitable side slopes

Side slopes for trapezoidal cross section should be selected according to the type of soil through which the canal is cut. In fill sections, slightly flatter side slopes could be used.

Soil Type	Side Slope (V:H)
Rock	1: 0.25
Soft Rock	1: 0.5 to 1: 0.7
Heavy Clay	1: 0.5 to 1: 1
Loam	1: 1 to 1: 1.5
Sandy Loam	1: 1.5 to 1: 2
Loose Sand	1: 2

Table 5-2: Recommended Canal Side Slopes

III. Adopt bed width to depth ratio for canals
IV. Adopt suitable values for Manning's "n" (See Table 4-1)
V. Choose water surface slope

The most economical section is normally with water level close to natural ground level. The maximum slope that can be adopted with out causing scour can be checked using the Tractive Force Theory.

The unit tractive force, T, is given by:

$$T = CWRS$$

Where C = coefficient

W = water specific weight (10 000 N/m3)

R = hydraulic radius in meter

S = water surface slope

Permissible tractive force for various materials can be referred from the given reference materials.

VI. Carry out hydraulic design
VII. Appropriate values of freeboard need to be recommended

By entering the above data in the Manning equation, a complete hydraulic design of the channel can be obtained.

5.1.5 Output from the Design Procedures

a) General

The principal output from the design process for canals will be:

I. Longitudinal sections
II. Strip Plan
III. Typical cross sections
IV. Quantities
V. Locations, type and numbers of structures and design levels and flows for use in structure selection and design.

b) Longitudinal Sections

The longitudinal sections will present details of:

I. Natural features
   - ✓ Distance along the canal line
   - ✓ Ground levels along the canal line
   - ✓ Crossings (roads, rivers, streams, drains, etc)
   - ✓ Problem areas (land slip zones, faults, swamp areas, sandy soils, etc)

II. Canal Design Data
   - ✓ Discharge
   - ✓ Bed, bank top and water levels
   - ✓ Longitudinal slope
   - ✓ Section dimensions (bed width, flow depth, freeboard)
   - ✓ Cross section type (bank widths, side slopes, drainage, service roads, etc)
   - ✓ Structure positions, type and nominal sizes
   - ✓ Radii of bends
   - ✓ Details of embankments (widths, side slopes, roads, drains, cross falls, etc)
   - ✓ Reservation widths

It is more convenient if the canal design is carried out using standard canal design sheet as is shown in Figure 6-1.

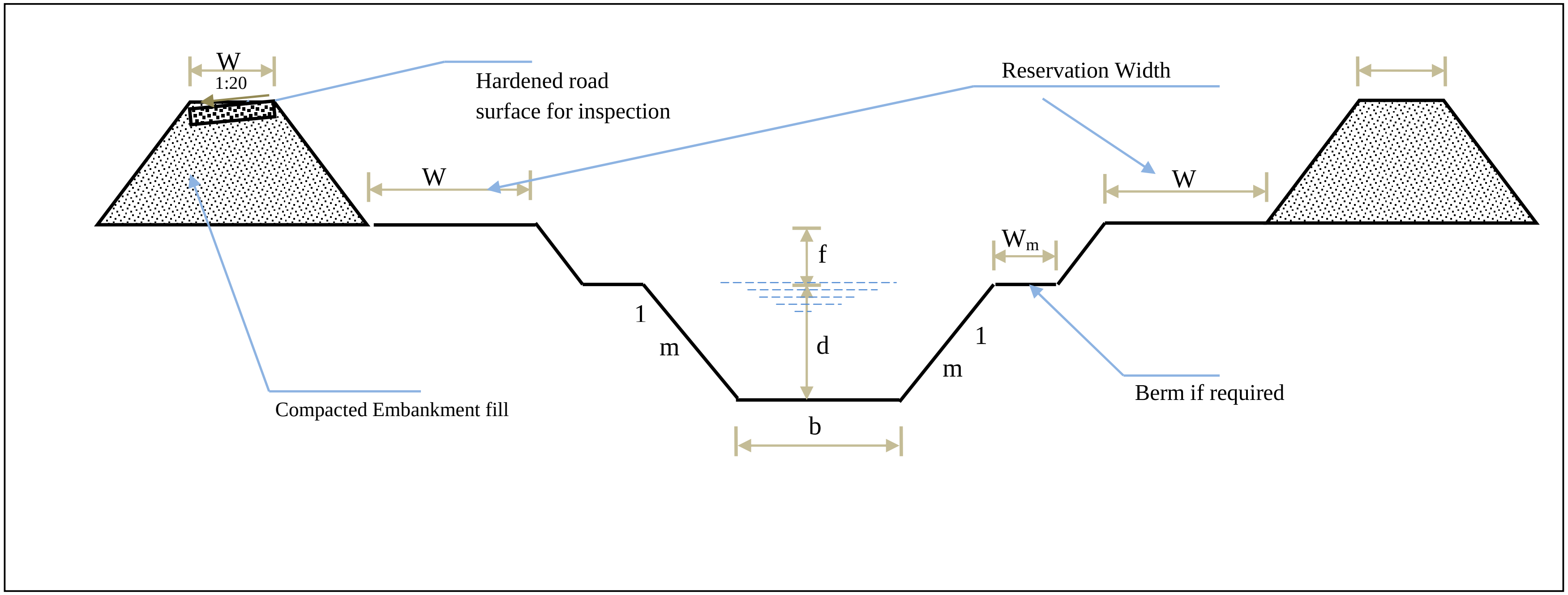


Figure 5-1: Typical Canal Cross Section

Canal Design Sheet

Project Name:.____________________ Project Code:.______________________

Canal Name:______________________ Sheet Number:________ of _________

Designed by:______________________ Checked by:_________________ Date:______________________________

Chainage	EGL	Bed Level	Bed Width b	Flow Area A	Free board F	Top of Embankment	Design Discharge Q	Side Slopes	Manning's n	Gradient	Water Depth d	Water Surface Level	b/d Ratio		Velocity V	Water Surface Width	Forude Number Fr		
(Km)	(m)	(m)	(m)	(m)	(m)	(m)	(m3/sec)				(m)	(m)	(m)	(m)	(m/sec)	(m)			

Figure 5-2: Canal Design Sheet

5.2 Design of Natural Channels

5.2.1 General

In the case of design of natural channels, the aim is to arive at reasonalble flood corridor and sizing of required flood embankment height. This requires ittirative procedure for sizing of the flood corridor and embankement height. When the floodway is large it is going to take larger land and small embankement height which implies less embankement cost irrispective of construction material.

The second option is less flood corridor and incresing embankement height which in turn implies larger embankement cost again irrispective of construction material. Therefore, the design process requires optimization of the land taken by the flood corridor and the construction cost of embankement.

5.2.2 Approach

For flood protection works the basic hydraulic principles may be the same as for the design of open channels. However, the approach for flood protection embankment starts from determination of the flood level before and after the protection works as shown in Figure 5-3.

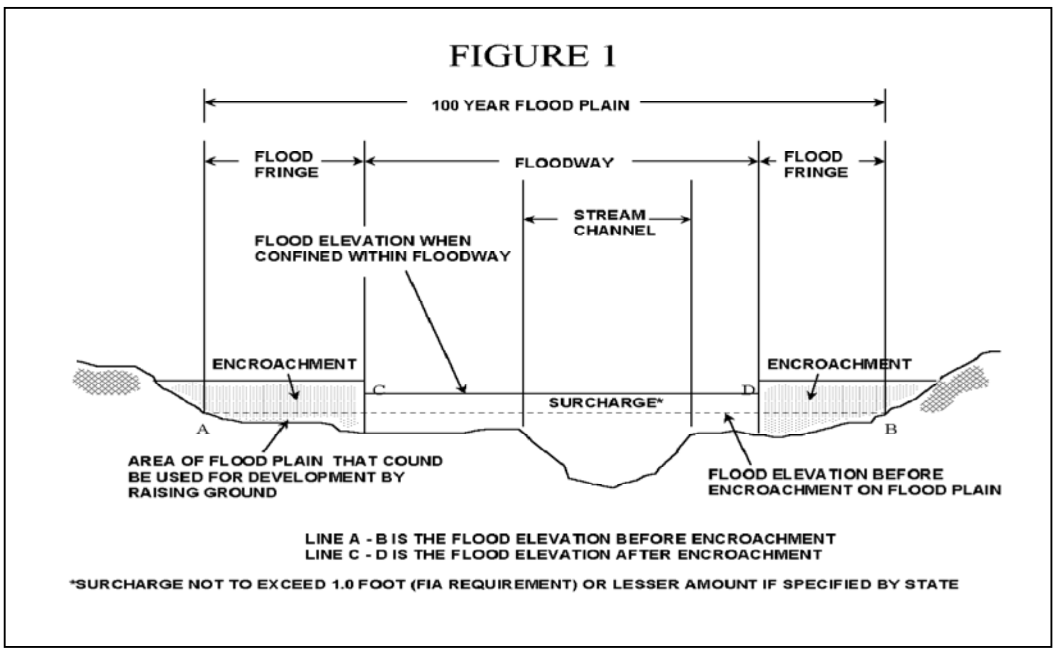


Figure 5-3: Typical Section of Flood Plain Channel

5.2.3 River Water Surface Flood Elevation

The basic principle for flood protection work is the determination of top embankment level, which depends on the design water surface flood level with in the flood corridor.

Therefore, first the design flood levels should be determined with out the flood protection embankment. In other words it is all to determine the size of flood volume that can be accommodated with in the river bank as it is shown in Figure 6-3.

The design water surface flood level may be determined by a uniform flow formula such as Manning, using average river bed slope as obtained from the topographic survey and suitable value of Manning's "n". Special care should be taken around bends as the river cross section (and bed slope) will vary.

The river surface flood level should be determined for at least three cross sections and the long section plotted showing flood level, (average) river bed level and design scour level.

It is usual for design flood levels to be determined both with and without the proposed flood protection works. This is particularly important if the waterway is being reduced to allow any adverse effects, for example to lands on the opposite bank of the river to that being protected, to be properly assessed.

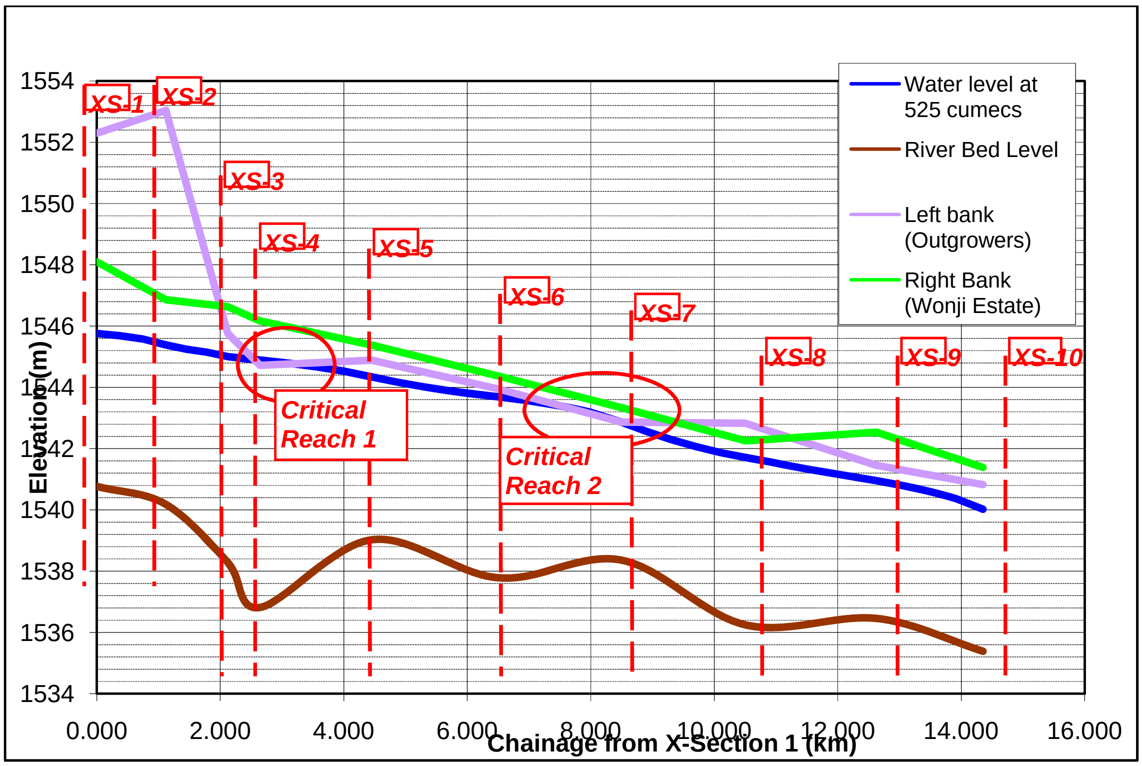


Figure 5-4: Long Section of Awash River at Wonji - Discharge at 525 m3/s

Source: Awash River Basin Flood Control and Watershed Management Project, Ethiopia

Figure 5-4 is self explanatory to note that from the profile of the Awash River around Wonji area, that there are two critical areas on the left bank. Plotting the long section and cross section of the river from the topographic survey and hydraulic calculation of the flood with in the river bank including the existing flood protection dyke has determined the critical flood prone areas.

5.2.4 Stage Discharge Curve of Natural Streams

The river channel properties can be well defined through developing stage discharge curve. Topographic map, river cross-sectional and longitudinal profiles of point of interest areas are the basis for developing stage discharge curve the stream. Manning's equation (See Section 3.2) can be used for computing and develop stage curve. Hence, one of the basic parameter to use Manning's equation is roughness coefficient and it needs careful consideration in selecting the appropriate value as it has been discussed in section 3.2 and Table 3.1. Developing stage discharge curve helps in knowing the tail water depth after construction of weir and enables to decide the arrangement of the weir and protective structures like bunds. It also helps in deciding flood embankment height in case of flood protection structures.

Once stage discharge curve is developed for a river channel of point of interest reaches, the flood embankment height is fixed considering the design flood level plus allowable freeboard.

Example

Meqala River located in Bale Zone Adaba Wereda of Oromia Region is planned for the construction of river diversion work for irrigation purpose. River channel properties have been studied 100m upstream and 100m downstream of the proposed diversion weir axis. From the topographic survey, the average river bed slope for 200m reach is 0.016. The river can reach is in mountainous stream with steep banks with bottom of gravels, cobles and few boulders. Upstream approach of the weir site has vegetation cover. Left side of the river bank is steep slope with rock outcrop. Right bank is wide and flat extending 70m having a plot of farm land with sugar cane.

Construct stage discharge curve of the given river for the analysis and design of diversion weir and estimate the height of flood for 100m3/sec.

Solution:

Column 1-4 are obtained from the topographic survey data and hydraulic radius R is A/P. Then the velocity can be calculated using manning formula. In order to use the formula it requires selecting Manning's roughness coefficient from Table 3.1.

Elevation	Depth (m)	Water Area (m2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Velocity (m/sec)	Discharge (m3/sec)
1	2	3	4	5	6	7
1842.130	0	-	-	-	-	-
1842.258	0.128	0.359	5.620	0.06	0.58	0.21
1842.445	0.315	1.760	9.448	0.19	1.18	2.07
1843.234	1.104	11.240	11.955	0.94	3.47	38.99
1843.670	1.540	18.876	25.267	0.75	2.98	56.17
1843.986	1.856	30.454	40.702	0.75	2.98	90.71
1844.840	2.710	78.570	70.520	1.11	3.88	305.17

Table 5-3 : Sample River Data for Stage Discharge Curve

The description of the river regime it coincides with a roughness coefficient in between 0.03 for minimum up to 0.05 for maximum condition. Therefore, considering an average of 0.035 is convenient for this case. The average river bed is given as 0.016. Hence work out the velocity and discharge for each stage and then draw the graph as shown in the following figure.

The flood height corresponding to 100 m3/sec is approximated to be 1844.05 from the graph given below.

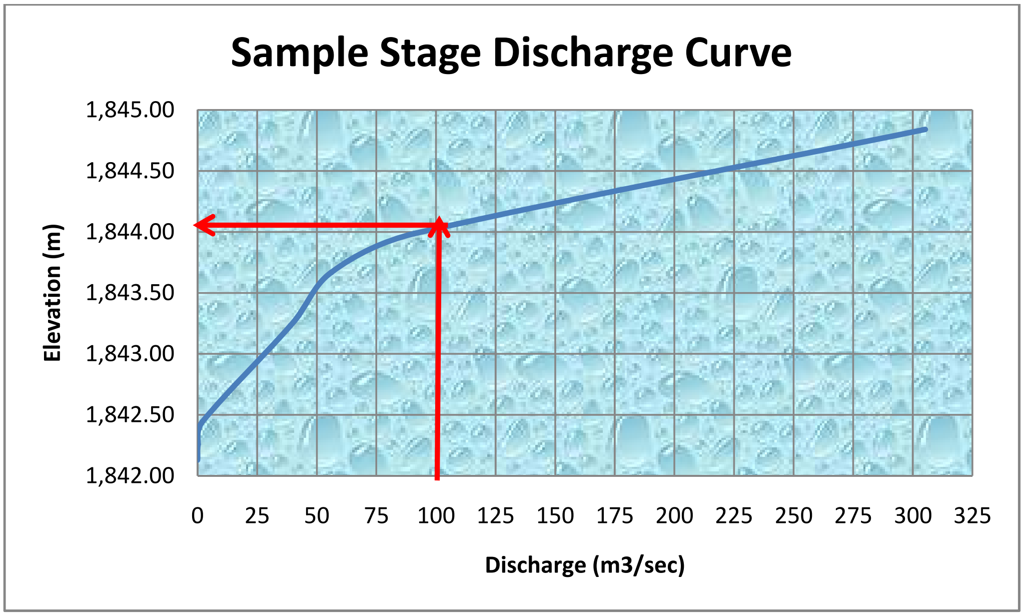


Figure 5-5: Sample Stage Discharge Curve

5.2.5 Embankment Freeboard

One of the basic criteria for fixing the top embankment level is to include freeboard over the high flood level which was calculated as in the previous section. There are a number of empirical procedures for the estimation of the freeboard basically done for large water storage reservoirs which can also be applied to smaller bodies such as embankments and NSRs.

Normally, the principle in determining the freeboard is wave height created by the flood water. The widely used empirical formula used to establish the design wave height according to the methodology of the Donelan/JONSWAP empirical formula [Carlyle, 1997]. The estimated effective fetch lengths along the diagonal, with the adopted design wind speed of 30 m/s for most of Ethiopia (if wind speed is not available at specific location of project areas), This method allows calculation of wave height for all shapes of reservoirs and embankments and wind directions not necessarily normal to the embankment.

The formula is given by:

$$H_S = \frac{U\sqrt{F}}{1\,760}$$

where: Hs = significant wave height in metres
U = design wind speed in m/s
F = fetch in metres.

The design wave height for any embankement crest level is obtained from:

$$H_D = f_1 \times f_2 \times H_s$$

where: f1 = factor to establish design wave height with allowance for permitted overtopping

f2 = factor to allow for wave run-up.

For the flood embankments the value of Hs varies depending on the width of the river bank of the reservoir. For 200 m river bank width the value of Hs is 0.24 m and for a 400 m river bank width the value is 0.34 m. The value of f_1 is set to 1.3. This is applicable to an earth fill embankment with random grass cover downstream face, and a grassed crest. The value of f_2 is set to 1.

To estimate the design wave height for the embankment top:

$$H_D = f_1 \times f_2 \times H_s$$

where: f_1 = 1.3 as above

f_2 = 2.2 for embankment with side slopes of 1:2 with smooth surface.

These give H_D for the embankment as between 0.32 and 0.45 m. A minimum of between 320 mm and 450 mm freeboard can be provided above the flood protection embankment to accommodate the design wave height.

5.2.6 Design Procedures

Prior to starting the design the canal/stream/river lay out should be drawn out on the maps, and decision made on the types and locations of flood mitigation structures. The procedures for the design have been summarised as follows:

I. Determine design discharge (See Hydrology part of this Manual)

II. Obtain River cross section and long section from topographic survey

Work out river bed slope and cross sectional area at each chainage

Work out the available flood corridor on both sides of the river bank form topographic map as explained in **Part 5 Topographic Survey** of this manual

III. Adopt suitable values of Manning's "n" for the river channel and flood plain

IV. Carry out hydraulic design

V. Appropriate values of freeboard need to be recommended

By entering the above data in the Manning equation, a complete hydraulic design of the channel including the flood corridor.

5.2.7 Output from the Design Procedures

a) General

The principal output from the design process for flood mitigation work will be:

I. Layout and alignment of the protection dyke including centre line, top embankment edge, toe of embankment;

II. Longitudinal sections including river bed, river bank top level, and flood level before and after protection, and top embankment as shown in Figure 5-4;

III. Longitudinal sections of the protection work including ground level, flood level, and top embankment level;

IV. Typical cross section of the embankment relative to the river;

V. Typical detail cross section of embankment showing excavation level, centre of embankment, top edge of embankment, toe of embankment and protections required;
VI. Type and location of structures along the embankment;
VII. Curves and curve details;
VIII. Quantities;
IX. Locations, type and numbers of structures and design levels and flows for use in structure selection and design.

b) Longitudinal Sections

I. The longitudinal sections along the river bed will present details of:
   - ✓ Distance along river bed,
   - ✓ River banks top level;
   - ✓ Flood level before and after protection,
   - ✓ Critical areas of the river; and
   - ✓ Proposed top embankment levels as shown in figure 6-4;
   - ✓ Problem areas (land slip zones, faults, swamp areas, sandy soils, etc)

II. Long sections along the centre line of the protection embankment
   - ✓ Distance along the centre of embankment line
   - ✓ Excavation level,
   - ✓ Bank top and water levels
   - ✓ Longitudinal slope

c) River Cross Sections

- ✓ Section dimensions (bed width, flow depth, freeboard)
- ✓ Cross section type (bank widths, side slopes, drainage, service roads, etc)
- ✓ Structure positions, type and nominal sizes
- ✓ Details of embankments (widths, side slopes, roads, drains, cross falls, etc) as shown in **Figure 3-4 of Part 8: Flood Protection Structures**
- ✓ Reservation widths.

This page is intentionally left blank

FLOOD PROTECTION DESIGN MANUAL
PART 8: FLOOD PROTECTION STRUCTURES

Table of Contents

CONTENTS OF OTHER PARTS

1. INTRODUCTION

1.1 General

This part of the Flood Protection Design Manual discuses the type of flood protection structures and presents standard design criteria and guideline for the design of flood protection works in Ethiopia.

Major irrigation development potential areas in Ethiopia are located along the major river banks and valley bottoms. Those along the river banks would be flooded during storm or high discharges in the rivers. In such a case, protection against flooding from the rivers is primarily provided by flood protection structure dykes.

Rainfall runoff from high grounds to the valley bottom and plain areas will cause flooding to infrastructures and settlement areas. In such a case, flood containing structures and flood diverting structures are required.

The area protected by a linked system of primary flood protection structures is called a dyke which may be associated with retaining walls, interceptor drains, guide bunds and crossing structures. Together these structures ensure the protection and safety of the project area both in flood plain areas and river banks.

This part of the manual discuses the type of flood protection structures, design of river containing, river training structures and some other associated structures like cross drainage works.

1.2 Concepts Background

Large scale flooding in Ethiopia is limited to the low land area where major rivers cross flood plain areas of the country. In such a case, the flooding occurs when the flood water overtops on either sides of the river banks and causing damage to development projects and infrastructures.

Similarly, several small streams originating in the mountain range and extending to relatively plain settlement areas like Addis Ababa and Dire Dawa. Torrential rains, common during the rainy season, cause flash flood of these streams, which bring about flood damages to settlements along their banks. One of the similar situations that affected the City of Dire Dawa on August 2006 is the recent flood damage example in urban area.

Both cases of flooding problem from overtopping of river banks and rainfall runoff from high grounds to the valley bottom and plain areas will cause large casualties and damage to property which will have economic effects on a national level

In such a case, flood containing structures and flood diverting structures are required for the protection of settlement areas, infrastructures and property.

1.3 Purpose of the Flood Protection Works

On most medium and large scale irrigation schemes and potential agricultural development areas in Ethiopia, flood protection works are required to protect the irrigation facilities, the command area, agricultural areas or housing areas. Where these potential development areas are being protected the opportunity may sometimes be taken to enclose a larger area and reclaim land washed away during past floods.

1.4 Level of Protection Works

To protect infrastructure, such as head works or village housing areas, flood protection works should be for a comparable design life depending on the purpose and type of infrastructure (i.e. 25-30 years). The design flood event should have a return period of 50 years (plus freeboard) or 100 years (without freeboard).

To protect land from inundation, the flood protection works would usually be designed to a much lower standard, for a five year design life, and for a design flood event with a return period of 20 to 25 years. However, if breaching of the flood protection works would not only result in land being inundated, but in valuable top soil being washed away or in densely populated settlement urban areas (like Dire Dawa), then a higher level of protection is justified

1.5 Classification of Flood Protection Works

Flood protection structures may be classified according to its purpose or function as either:

- ✓ River containing structures (for example stone protected earthen flood protection embankments where the top of the embankment is above design flood level); or
- ✓ River training structures (for example guide bunds and Groynes, which are sometimes designed to overtop during floods).

The types of structures are described in the following subsequent Chapters.

1.6 Hydrology and Meteorological Data

The Hydrological methods and procedures for estimating flood peaks for the purpose of flood protection embankments are described in **PART 4: HYDROLOGY** of this Design Manual.

In order to design flood protection works, the peak flood in the river for the design return period of 25, 50 or 100 years is needed depending on the level of protection required for specific areas.

The river flood level corresponding to the design flood event will also be required. River cross sections should be surveyed upstream and downstream of the critical section of the river reach requiring protection works. The approximate river flood level for the design flood event can then be estimated using Manning’s equation taking into account the surveyed cross sections, the slope of the river and an appropriate Manning’s coefficient “n”.

Where possible the river levels should be checked against flood marks left by floods, which should be noted when river cross sections are surveyed. The flood marks should normally be evaluated and correlated with the flood levels calculated for certain design period.

It is usual for design flood levels to be determined both with and without the proposed flood protection works. This is particularly important if the waterway is being reduced to allow any adverse effects, for example to lands on the opposite bank of the river to that being protected, to be properly assessed.

1.7 Alignment of Flood Protection Works

The alignment of flood protection embankment needs to be considered carefully, taking into consideration the following:

- ✓ Topography: for example high land, outcrops, etc
- ✓ The existing (and historical) river alignments;
- ✓ Farmers wishes, land ownership, location of other infrastructures such as roads, electric poles, houses, etc
- ✓ The regime river width (for alluvial river beds);

- ✓ The effect of the proposed works on “others” outside the scheme area, such as on the opposite bank of the river to that being protected;
- ✓ The stability of the river bed, location of any rock outcrops, etc.

For alluvial rivers, the width should not be reduced below the regime river which should be calculated for various flood flows, including flood flows with a return period of 10, 20, and 50 years.

For rivers with cohesive (colloidal) banks the natural width of river both upstream and downstream of the scheme should be measured, at locations with similar bed and bank material and slope. The adopted width should not be less than the (minimum) measured natural width.

Having adopted an alignment, the effect on “others” needs to be determined. A reduced waterway will increase flood levels and increase scour of the river bed.

2. RIVER MORPHOLOGY AND FLOOD PROTECTION

2.1 Introduction

This Chapter describes the nature of rivers and their formation and the consequences this has on flood protection works.

Intake structures and flood protection dykes are the most significant aspects on major irrigation development projects in Ethiopia. Success in best understanding of the river morphology and then appropriate consideration for design of intake as well as flood protection dykes allows irrigation schemes to function without disruption and long-term sustainability.

Failure to understand the river morphology during the design process will generate failure on the performance of structures during implementation.

Therefore, this Chapter explains the concept of river morphology and regime and then discusses river boundary conditions related to the effect of bed slope on river development and flood protection which is referred to as river training to be discussed in Chapter 4.

2.2 River Regime

Generally river regime is defined in relation to alluvial channels thus "The dimensions, width, depth and gradient of a channel to carry a certain amount of water loaded with a given silt charges". In other words, a river can adjust its width, depth, and gradient in order to achieve regime. As a result, the cross-sectional area, velocity, and sediment transport capacity result in neither deposition nor erosion of the bed or banks over a hydrological cycle.

2.3 River Bank Erosion

A river’s bank consists of upper and lower sections. The lower bank, the part below low water, acts as the foundation for supporting the upper bank and is generally more susceptible to erosion. Bank recession is caused by erosion of the lower bank, particularly at the toe. Recession can be fast, especially when there is a sandy substratum, as the sand is washed away by the current and the over-hanging bank collapses. The upper bank is the portion between the low water and the high water. Action on this portion of the bank is most severe when the current attacks normal to the bank. During high stages of the flood erosion also occurs due to a strong current along the bank.

There are various types of bank erosion as described below:

- ✓ River flow attack at the toe of the underwater slope, leading to bank failure and erosion. Usually the greatest likelihood of upper bank failure occurs during a falling river stage;
- ✓ Erosion of soil along the bank caused by current action;
- ✓ Sloughing of saturated banks with floods of long duration, due to rapid receding of the flood;
- ✓ Flow slides (liquefaction) in saturated silty and sandy soil;
- ✓ Erosion of soil by seepage out of the bank at relatively low channel flows/velocities;
- ✓ Erosion of the upper bank due to wave action caused by wind.

Figure 2-1: River Bank Erosion on small tributary river Gojam (Source: Consultant Field Visit 2009)

Figure 2-2: River Bank Erosion on Ribb River @ Upstream of the Main Bridge (Source: Consultant Field Visit 2009)

Understanding generally the river morphology and the type and causes of river bank erosion will help in recommendation of type of river training and bank protections.

2.4 Characterization of Rivers

Rivers can be characterized according to :

- ✓ Gradient;
- ✓ Valley shape;
- ✓ Bed and bank material;
- ✓ Plan form.

The gradient dominates the characteristics of the river and hence major three principal zones are discussed in this manual as below:

i. Mountainous reach: these reach are so steep that their beds are formed of rock outcrops and in most cases at boulder stage. They are highly turbulent so that gravel and finer material travels in suspension;
ii. Outwash reach: these are less steep and partly filed with smaller boulders, cobbles, gravels and sand brought from upstream and transport further downstream. However, the reach is known for high variability of sediment transport both phase of aggradation and degradation.
iii. Lowland reach: these are of lower gradient, generally the river travels through sand deposits of sand and silt. The regime here may be dynamic with sediment transport broadly in balance with the supply of finer material. Therefore in most cases, the reach is in aggradation nature as the sediment tends to deposit.

It is a continuous transition process between these categories.

2.5 Channel Stability

2.5.1 Stability of the Longitudinal Profile

Most rivers in Ethiopia originate from highlands of mountainous regions and are unstable in terms of sediment yield over time and over length. The mountains and steep reach of river areas are erosion river regime supply sediment to downstream to create an aggrading zone. The longitudinal profiles are therefore unstable, which means that it should never be assumed that the bed profile as observed at a particular time is stable. The aggrading river bed zone means the flood levels are always increasing, which increases the risk of overtopping the river bank or the existing flood protection dykes.

Experience on Amibara Irrigation Project is one of the lessons to be learnt the effect of river morphology on sustainability of intake structure and flood protection embankment.

Awash River bed is aggrading in the reach between Melka Sedi gauge station and Melka Werer, causing flooding and erosion along the toe of the existing flood protection embankment. The aggrading bed means the flood levels are getting higher, which increases the risk of the existing flood protection dykes overtopping. It is generally acknowledged that the continued rising of the dykes is not sustainable. Over time the aggradations of the river bed in the Amibara reach has completely buried the original Amibara diversion weir, intake, and scour sluice with sediment As shown in Figure 2-3.

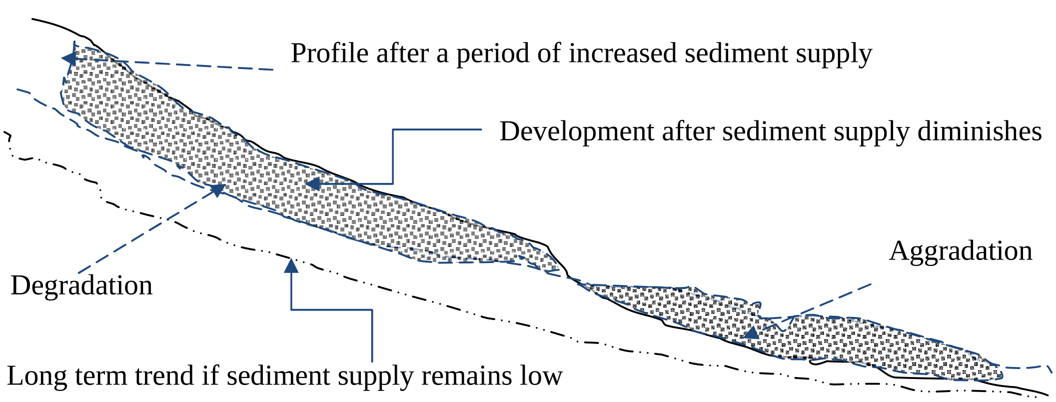


Figure 2-3: Instability of River Bed Profile

Figure 2-4: Awash River at Amibara Weir and Intake Site (Source: Consultant Library 2007)

Knowledge of the river regime therefore will help in design of appropriate river training and also the level of flood protection works.

The lessons on Amibara are clear:

- ✓ Understanding of river morphology is critical in the design of flood protection as well other river structures;
- ✓ Is critical in recommendation of best practice of river training type to accommodate such instability of river bed;
- ✓ Is critical in recommendation and design of sustainable diversion weir and intake structures.

The same aggradation nature of river bed is true for Ribb and Gumera Rivers at the Trans highway reaches.

2.5.2 Stability of Plan Form

There are several reasons and ways of channel instability of plan form of rivers. Gradual progression of instability occurs through bank erosion in meandered rivers causing the meander's progress downstream.

Rapid development of major new channels can also occur when the river breaks the flood protection to find lower route downstream. Bank erosion of major meandered reach permits escape in to flood plain area and creates new channel system or channel switching.

One of the functions of river training is to bring such insatiability of plan form under control. In general a relative stability is encountered at the point of curvature reversal, while on the contrary a definite instability is encountered at the downstream edge of bend due to an asymmetrical distribution of water velocity. This phenomenon is created by transverse water currents in bends where the water rises along the outer bank due to centrifugal force.

Figure 2-5: Meandering on Ribb River (Source: Consultant Field Visit 2009)

2.5.3 Natural and Artificial Controls

Natural outcrops of rock can control both vertical and plan stability. The artificial or natural controls create stability of the plan geometry of the river reach and are essential features for the development of river's resources. The natural controls can be used to advantage in selecting the location of intake structures. Artificial controls are used for stabilization of the river in plan or profile which is termed as river training to be discussed in the following sections of this manual.

3. RIVER CONTAINING STRUCTURES

3.1 Flood Protection Embankments

3.1.1 General

Flood protection dykes are earthen embankments constructed parallel or adjacent to the river channel for primary purpose of furnishing 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. The embankments are made of earth and are sometimes additionally protected with revetments.

Even though flood protection dykes are similar to small earth dams they differ from earth dams in the following important respects:

(a) a flood embankment may become saturated for only a short period of time beyond the limit of capillary saturation,

(b) flood embankment alignment is dictated primarily by flood protection requirements, which often results in construction on poor foundations, and

(c) borrow is generally obtained from shallow pits or from channels excavated adjacent to the levee, which produce fill material that is often heterogeneous and far from ideal. Selection of the levee section is often based on the properties of the poorest material that must be used.

3.1.2 General Design Procedures

Numerous factors must be considered in flood embankment design. These factors may vary from project to project, and no specific step-by step procedure covering details of a particular project can be established. However, it is possible to present general, logical steps based on past experience on flood protection projects that can be followed in the design. The procedures stated in this manual can be used as a base for developing more specific procedures for any particular project. Such a procedure is given in Table 3-1.

Steps	Procedures
1	Conduct preliminary assessment of the flood prone area based on available topographic maps, aerial photographs and satellite imagery and define the level of protection required and scope of investigation required including topographic survey requirement and geotechnical investigation.
2	Conduct topographical survey based on preliminary hydrological analysis including cross sections and long sections.
3	Conduct detail hydrological study based on available hydrological and meteorological data and topographic survey data. Define critical areas and alignment of the embankment based on the hydrological study. From this hydrological analysis define the flood water level and top level of embankment.
4	Conduct geological study based on a thorough review of available data including analysis of aerial photographs and satellite imagery. Initiate preliminary subsurface explorations
5	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
6	Initiate final exploration to provide: a. Additional information on soil profiles. b. Undisturbed strengths of foundation materials. c. More detailed information on borrow areas and other required excavations
7	Using the information obtained in Step 6: a. Determine both embankment and foundation soil parameters and refine preliminary sections of critical areas. b. Compute rough quantities of suitable material and refine borrow area locations.
8	Divide the entire dyke into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
9	Analyze each trial section as needed for: a. Under seepage and through seepage. b. Slope stability. c. Settlement. d. Accessibility of the embankment surface.
10	Design special treatment to prevent any problems as determined from Step 9. e. Determine surfacing requirements for the dyke based on its expected future use. f. Scour protection requirements based on critical areas determination; g. Structures requirements.
11	Based on the results of Step 10, establish final sections for each reach.
12	Design embankment slope protection
13	Compute final quantities needed; determine final borrow area locations.

Table 3-1: Major and Minimum for Design Procedures

3.1.3 Embankment Section

For dykes of significant height or when there is concern about the adequacy of available embankment materials or foundation conditions, embankment design requires detailed analysis. Low dykes and dykes to be built of good material resting on proven foundations may not require extensive stability analysis. For these cases, practical considerations such as type and ease of construction, maintenance, seepage and slope protection criteria control the selection of embankment crest width, slopes and height.

Standard embankment sections have been recommended as a guideline in this manual which is based on minimum requirements. The recommendation and adoption of standard embankment sections does not imply that stability analyses are not required. The standard embankment sections are applicable as a minimum requirement and to initial cost estimate, emergency and maintenance repairs

(a) Crest Width

The crest width of the flood embankment depends primarily on roadway requirements and future emergency needs. To provide access for normal inspection, maintenance operations and flood fighting operations, minimum widths of 3.00 to 3.50 m are commonly used with wider turnaround or bypass areas provided at specified intervals.

Figure 3-1: Flood Protection Embankment at Amibara Irrigation Project Awash River (Source: Consultant's Library 2007)

Figure 3-2: Flood Protection Embankment at Amibara Irrigation Project Awash River(Source: Consultant's Library 2007)

(b) Side Slope

The side slopes of the embankment must be stable under all conditions of construction, design flood discharge, rapid flood draw-down, and low flow level and earthquake forces. The stability depends on the strength of the fill soil foundation characteristics. The angle of slope should be selected to suit subsoil conditions, the angle of repose of the embankment material and the type of slope revetment to be provided.

Factors which influence the stability of an embankment include the following:

- ✓ Soil properties (angle of internal friction; cohesive strength; unit weight).
- ✓ Design flood level and low water level of the river.
- ✓ Phreatic line and pore water pressure.
- ✓ Surcharge on the embankments.
- ✓ Earthquake loading.

An embankment may fail for many reasons, including:

- ✓ Overtopping;
- ✓ Piping through the embankment and/or foundation during high floods;
- ✓ Cracking of the embankment due to settlement, either of placed fill material or the foundation;
- ✓ Front (upstream) side shear failure (sliding) due to weak foundations or over-steep front slope, most likely during rapid draw-down after a flood;
- ✓ Back (downstream) sides shear failure (sliding) due to weak foundations or over-steep back slope, most likely during the flood.

The recommended standard flood embankment sections are:

- ✓ Crest width: 3.0m
- ✓ River side slope: 2H:1V or 1.8H:1V
- ✓ Landside slope: 1.5H:1V

(c) Freeboard

Embankments are usually designed not to overtop. They are designed and built with crest level above maximum design flood level including a freeboard. Therefore, flood embankments and guide bunds need to be provided with freeboard as a safeguard against over-topping. The bund top height may be set as the river water level associated with:

- ✓ A design flood with a return period of 25-50 years to protect agricultural land against inundation; and
- ✓ A design flood with a return period of 100 years for guide bunds and/or marginal bunds at weirs, and to protect housing areas.

For bunds higher than 3.0m it may be necessary to flatten the back slope to ensure embankment stability, and/or place a counter berm along the back slope.

3.1.4 Cause of Embankment Failure

The principal causes of earth embankments failures are divided in to three major types as discussed below.

(a) Hydraulic Failure

- ✓ Overtopping. Water may overtop the embankment if the design flood is under estimated or the freeboard is insufficient.
- ✓ Erosion of river side batter. The waves developed near the top water surface due to wind try to notch out the soil from the river face and may cause the slip of the upstream slope. Stone pitching or riprap should be provided.
- ✓ Erosion of land-side by gully formation. Heavy rains falling directly on the land side may lead to formation of gullies, ultimately leading to embankment failure.

(b) Seepage Failure

- ✓ Piping through foundations. When highly permeable cavities or fissures or strata of coarse sand or gravel are present in the foundation, water may start seeping at huge rates through them. The concentrated flow at high gradient may erode the soil, thus increasing flow of water and soil, ultimately creating hollows below the foundation. The embankment may sink down causing its failure. Figure 3-3.

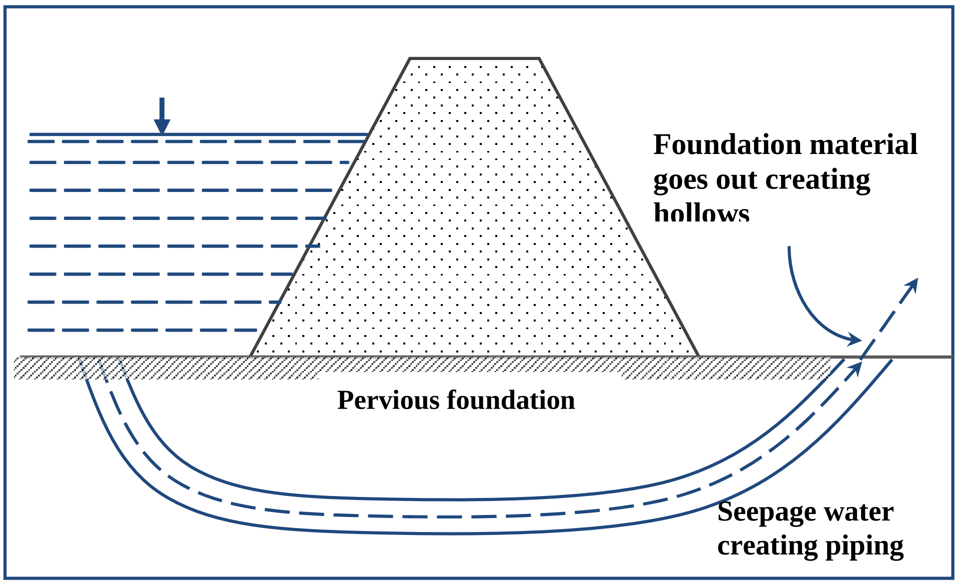


Figure 3-3: Piping through embankment foundation

- ✓ Piping through the embankment body. When the concentrated flow channel developed in the body of the embankment soil may be removed in the same manner as in foundation leading to embankment failure. Piping through the embankment body generally gets developed near culverts and pipes passing through the embankment body, Figure 3-4.

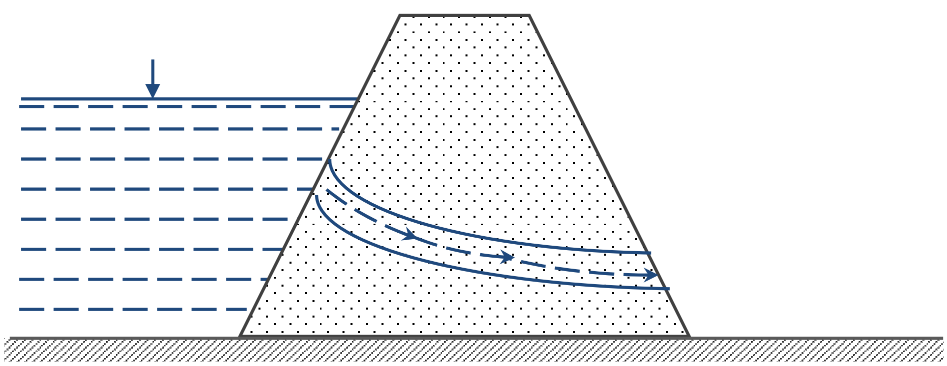


Figure 3-4: Piping through the embankment body

(c) Structural failure

- ✓ Foundation slide. When the foundation is made of soft soils such as fine silt, soft clay, etc. the entire embankment may slide over the foundation. The top of embankment gets cracked and subsides, the lower slope moves outwards forming large mud waves near the heel, Figure 3-5.
- ✓ Slide in embankment batters. When the embankment side slopes are too steep for the strength of the soil, they may slide causing the embankment failure. Sudden draw down of water level and lengthy flood wave are the most critical as discussed in section 3.1.5 and as shown in **Figure 3-6 and Figure 3-7.**

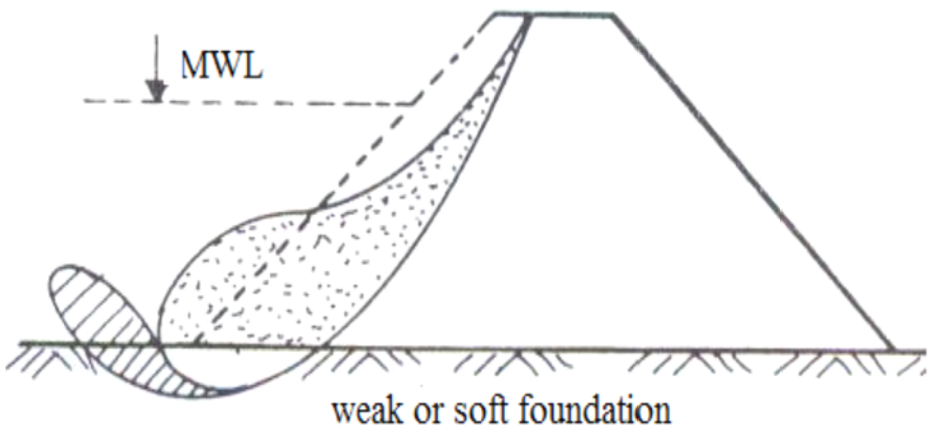


Figure 3-5: Sliding due to soft or weak foundation

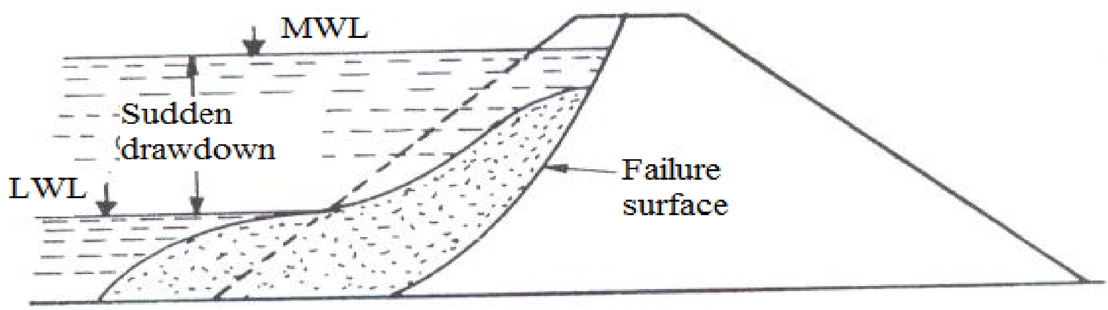


Figure 3-6: Upstream Slope slide due to sudden draw-down

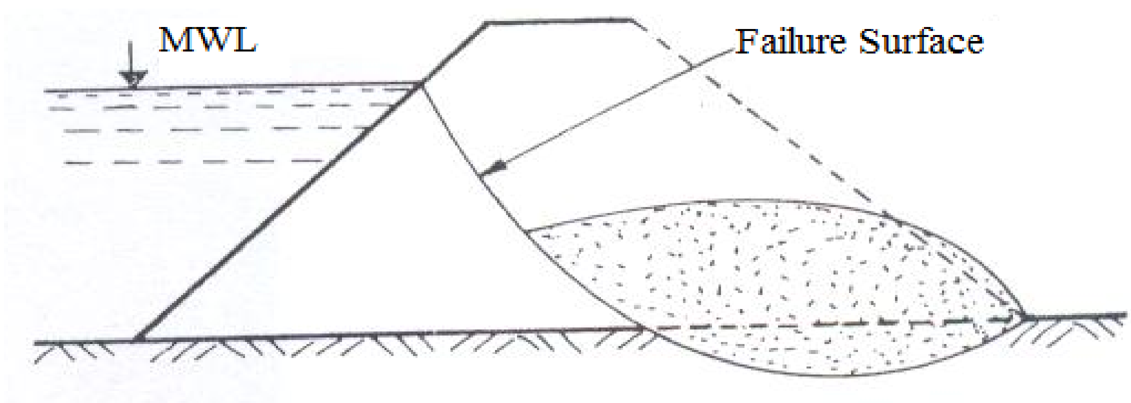


Figure 3-7: Downstream slope slide due to lengthy flood

3.1.5 Stability Analysis

If dykes are higher than 3.0 m then stability analysis has to be conducted for the recommended material with in the vicinity of the project. The various loading conditions to which a dykes and its foundation may be subjected has been discussed and which should be considered in analyses.

Steady State Seepage (SSS) Condition

This case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable.

Rapid Drawdown (RDD) Condition

Steady seepage from full flood stage (fully developed phreatic surface). This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability.

End of Construction (EOC) Condition

This case represents un-drained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-un-drained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.

Slope stability analysis may be carried out using Bishop's simplified method (of slices). The analysis should be carried out assuming that river bed material in front of the (pre-launched) placed apron is eroded away, for the case of rapid draw down after the design flood event, as well as for seepage through the bund during the design flood, with surcharge loading on top of the embankment, and (if considered applicable) for earthquake loading. The factor of safety to be adopted should be between 1.1 and 1.5, depending on the severity of the loading conditions adopted. The required factors of safety for different conditions of stability of the dykes are summarized in Table 3-2 below.

Alternatively, the design section may be designed based on the adoption of a safe hydraulic gradient (or seepage) line.

Analysis Condition	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)	1.3	Upstream and Downstream
Long-Term Steady State Seepage - with maximum storage pool - with maximum Surcharge pool	 1.4 1.5	 Downstream Downstream
Rapid Drawdown	1.1 – 1.3	Upstream

Table 3-2: Summary of Factor of Safety for Stability Analysis

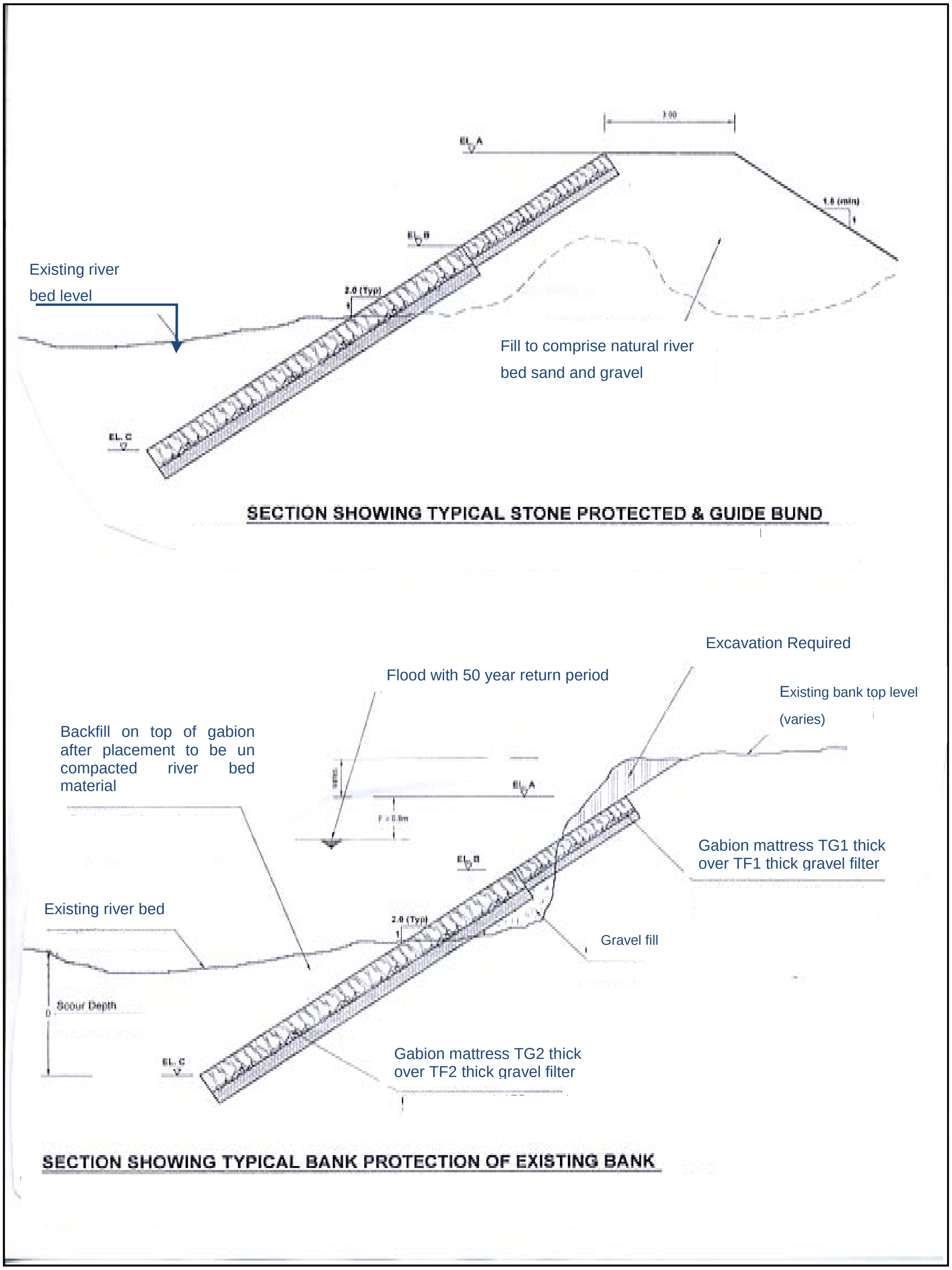
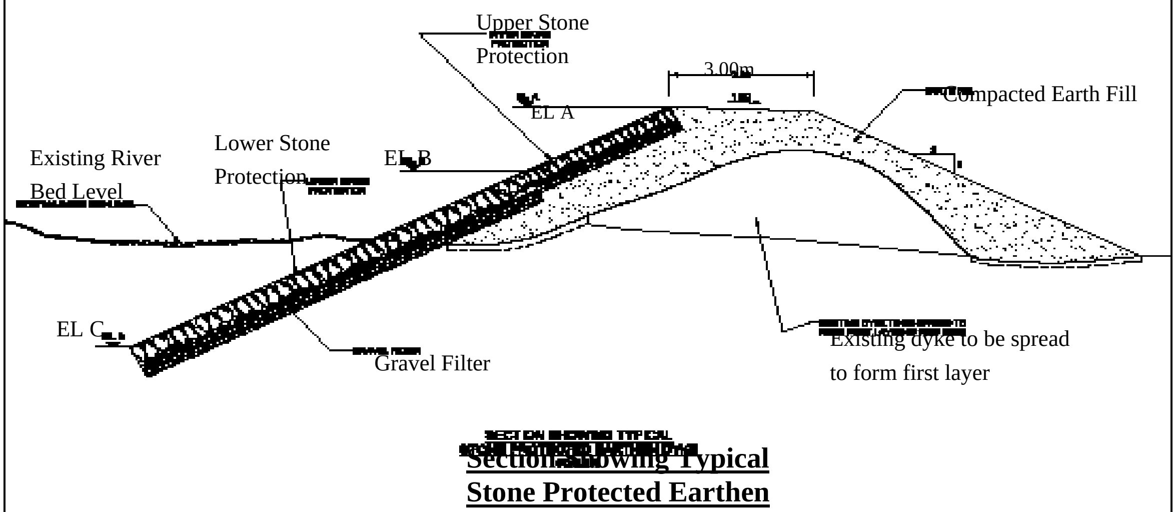


Figure 3-8: Gabion & Stone Pitched Embankments

3.2 Marginal Bunds

Marginal bunds are provided to contain river spill generated by the backup of water level upstream of a structure, such as a weir or crossing structures. They connect to high land upstream, and are usually anchored to guide bunds or to the main weir or siphon structure's abutment walls. They are not expected to be subject to high velocity flows and may have light stone protection, or even non-structural protection.



RECOMMENDED STONE PROTECTION THICKNESS

Average River Flow Velocity for 100 yr Flood, V, (m/s)	Lower Stone Protection: Gabion TG2 (m)	Lower Stone Protection: Filter TF2 (m)	Upper Stone Protection: Gabion/Stone (m)	Upper Stone Protection: Filter TF1 (m)
V<3.2	0.45	0.15	0.30 (Stone Pitching)	0.15
3.2< V < 4.0	0.60	0.20	0.45 (Gabion)	0.15
4.0< V < 4.5	0.75	0.20	0.45 or 0.60	0.15 or 0.20
4.5< V< 5.0	0.90	0.20	0.60 or 0.75	0.20

ESTERN NILE TECHINICAL REGINAL OFFICE (ENTRO) FLOOD PROTECTION & EARLY WARNING PROJECT (FPEWP) FLOOD EMBANKMENT DESIGN MANUAL IN ETHIOPIA	DESINGED BY EPHREM T. CAD BY : DAWIT S. CHECKED BY : APPROVED BY : REVISION NO:	DATE Nov / 2008 FIGURE 3-4

Figure 3-9: Typical Cross Section of Earthen Dyke and Recommended Stone Protection

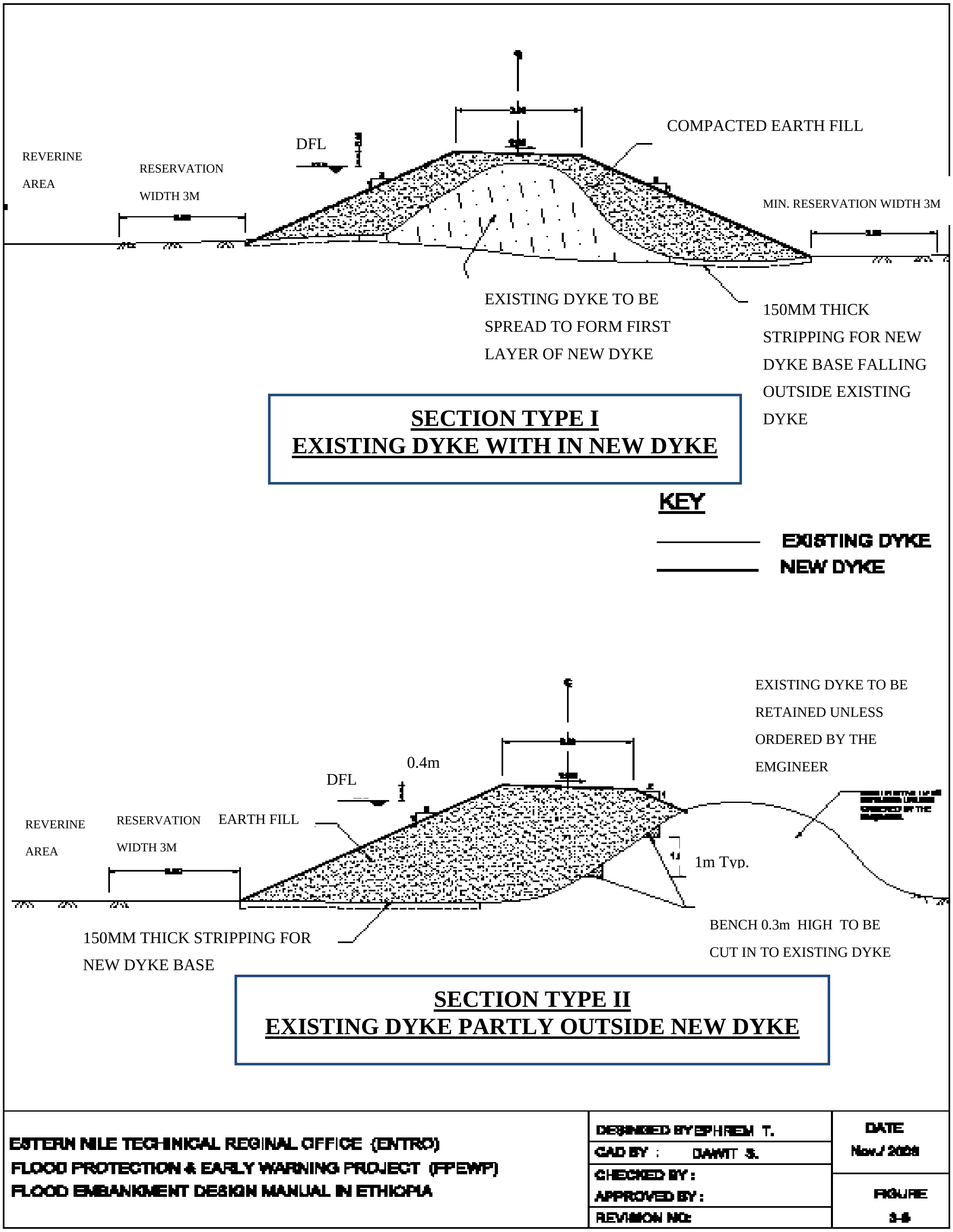


Figure 3-10: Upgrading of Existing Earthen Dyke

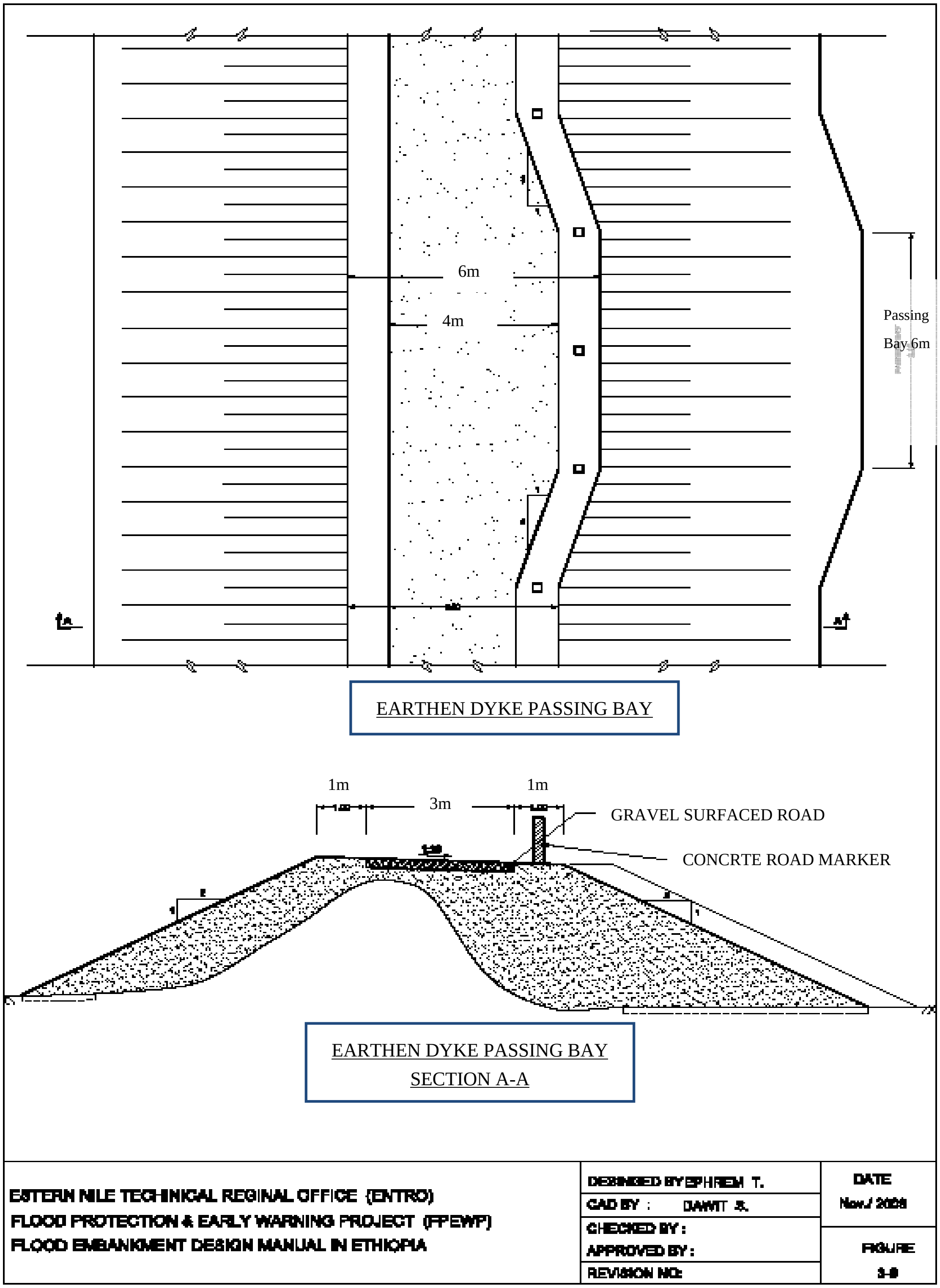


Figure 3-11: Earthen Dyke Passing Bay

4. RIVER TRAINING AND BANK PROTECTION

4.1 General

River training and bank protection works covers structures constructed along a river to guide and confine the flow to the river channel. River training works are also, to control river bed configuration for safe movement of floods and river sediment.

River training and bank protection works are often required at the intake sites to stabilize the river channel upstream and downstream of the structures and at the toe of flood protection embankment to protect scouring of banks.

4.2 Guide Banks

A river in alluvial material generally flows in a wide flood plain, defined by the historical movements of the river between two high banks, and it is necessary to narrow down and restrict its course to flow centrally over the weir (or other structure) placed across it. In most case guide bunds are placed symmetrically in plan.

If there is high ground close by, a stone armoured guide bund would simply extend to this high ground. If not, it may be cheaper to build a Bell type guide bund and a marginal bund, where the marginal bund is un-armoured but will prevent outflanking of the weir by floodwater. The Bell bund extends upstream and downstream from the weir a sufficient distance to prevent a meander scour from reaching the marginal bund. The Bell bund is of course stone armoured.

Guide Banks are used to:

- ✓ confine the flow to a single channel;
- ✓ improve the distribution of discharge across the weir or intake;
- ✓ control the angle of attack;
- ✓ control meander patterns; and
- ✓ prevent erosion of embankments.

The length of the guide bunds depends on the following:

- ✓ The distance necessary to secure a straight and normal approach flow to the structure so as to minimize the obliquity of current;
- ✓ To safely protect the approach banks, or marginal bunds, on both sides of the structure from river meander;
- ✓ To ensure that the turbulence, likely to be created by the spreading out of flow downstream of the guide banks, do not endanger the structure;
- ✓ The length necessary to prevent the edge of the river bend or meander reaching below the center line of the structure, behind the guide bund.

The recommended standard guide bund section is the same as for flood protection bunds, i.e.:

- ✓ Crest width: 3.0m
- ✓ River side slope: 2H:1V or 1.8H:1.0V
- ✓ Landside slope: 1.5H:1V

The types and overall layouts of guide bunds are shown on Figure 4-1. Figure 4-2 also shows a typical embankment section for a guide bank. The angle of slope should be selected to suit subsoil conditions, the angle of repose of the embankment material, and the type of slope revetment provided. The end of guide banks should be rounded as shown in Figure 4-2b.

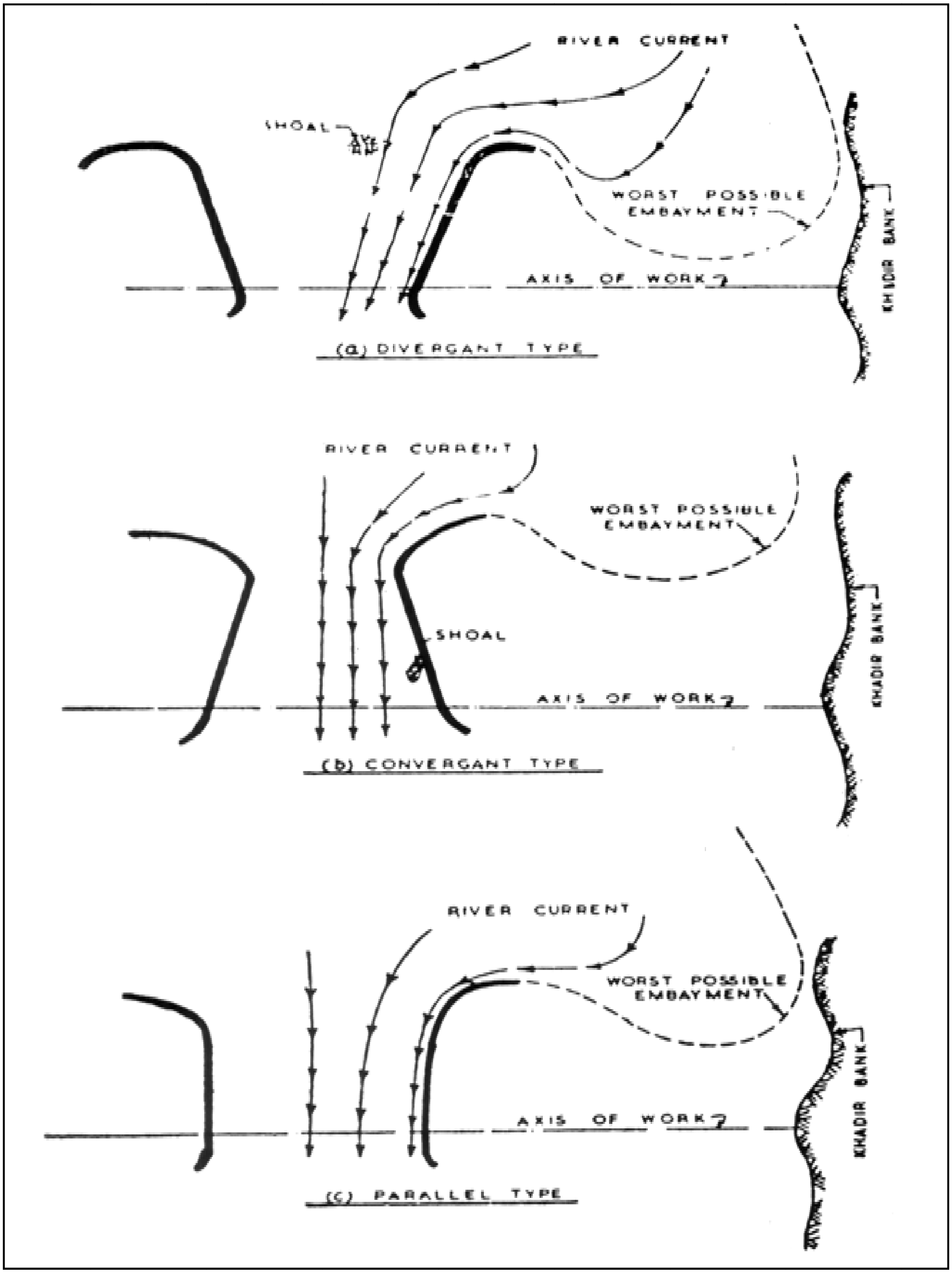


Figure 4-1: Types of Guide Bunds (Source: Halcrow Group Ltd, Flood Protection Structures Design Manual

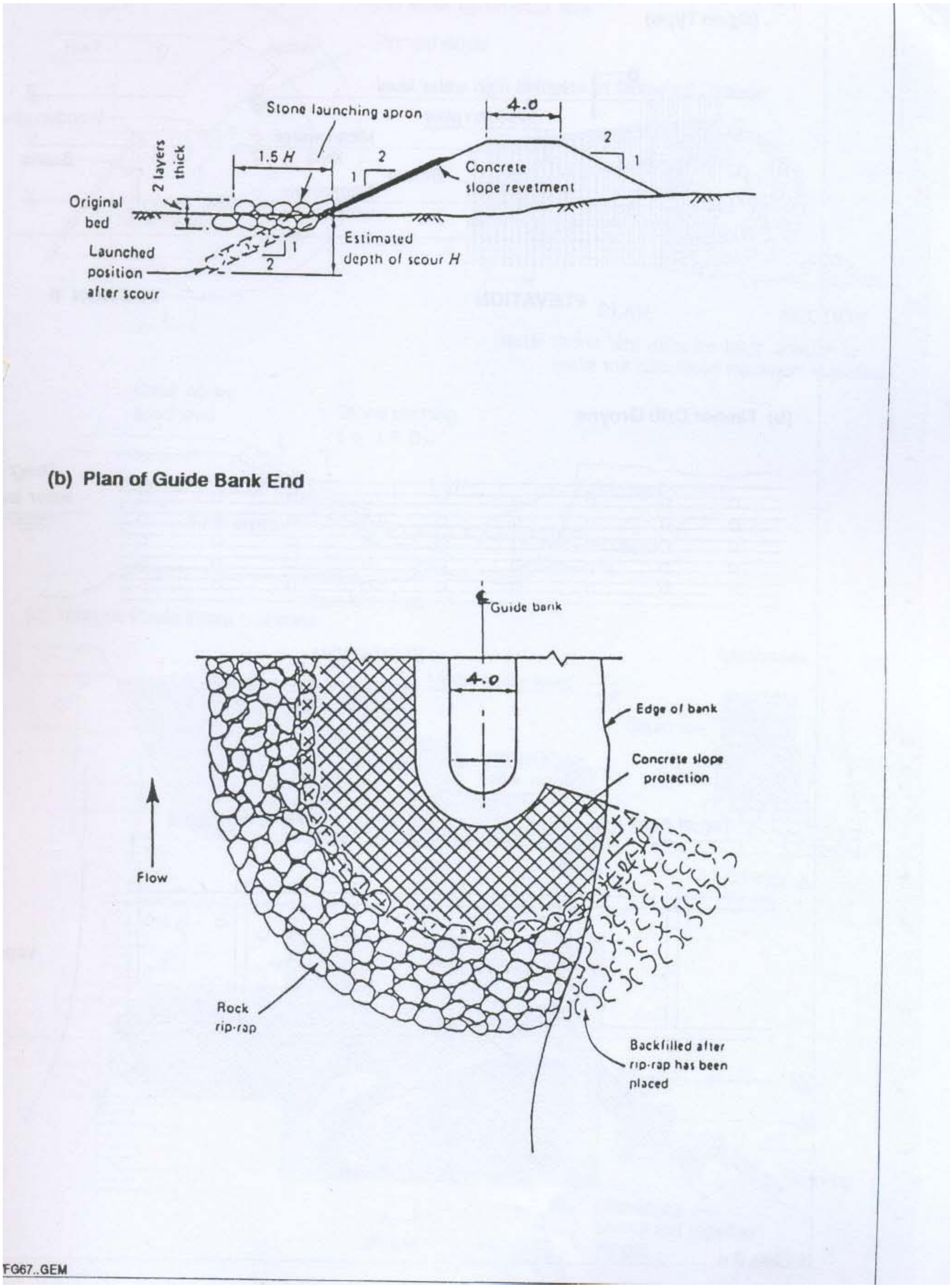


Figure 4-2: Typical Embankment Section for Guide Bank (Source: Reference Number 8)

4.3 Groynes

Groynes are structures placed transverse to the river flow and extend from the bank into the river to protect the erosion of embankments, dykes, or natural river banks, or as supplement to guide banks to direct the flow in to the direction of intakes. These are widely used for the purpose of river training and serve one or more of the following functions:

- ✓ Training the river along a desired course by attracting, deflecting (or repelling) and holding the flow in a channel. An attracting groynes creates deep scour near the bank; a deflecting groynes shifts deep scour away from the bank, and a holding groynes maintains deep scour at the head of the groynes;
- ✓ Creating a zone of slack flow with the object of silting up the area in the vicinity of the groynes;
- ✓ Protecting the river bank by keeping the flow away from it.

The requirements of groynes are:

- ✓ Optimum alignment and angle consistent with the objective.
- ✓ Availability of a high river bank to anchor (or tie) the groynes back, by extending it into the bank a sufficient distance to avoid it being outflanked.
- ✓ Sufficient freeboard provision (in case of non-submerged groynes).
- ✓ Fairly stable flow entry condition upstream.
- ✓ Adequate protection to nose/head against anticipated scour.
- ✓ Shank protection with stone pitching and stone apron for the length which is liable to parallel the flow attack upstream.

Depending upon the purpose, groynes can be used singly or in series. They can also be used in combination with other training measures. Their use in series is introduced if the river reach to be protected is long, or if a single groynes is not efficient/strong enough to deflect the current and also not quite effective for sediment deposition upstream and downstream of itself. The structure located the farthest upstream in a series of groynes is much more susceptible to flow attack both on the riverward and landward ends. Thus it should be given special treatment to ensure its structural stability.

The position, length and shape of groynes depend on site conditions, and require significant judgement on behalf of the designer. No single type of groynes is suitable for all locations. The illustration of types of groynes is shown Figure 4-5 and 4-6.

4.3.1 Alignment of Groynes

Groynes may be aligned either perpendicular to the bank line or at an angle pointing upstream or downstream.

A groynes angled upstream repels the river flow away from it and is called a repelling groynes. These are preferred where major channel changes are required. A groynes originally angled upstream may eventually end up nearly perpendicular to the streamlines after development of upstream side silt pocket and scour hole at the head. Repelling Groynes need a strong head to resist the direct attack of swirling current. A silt pocket is formed on the upstream side of the groynes, but only when the Groynes are sufficiently long. Repelling Groynes are usually constructed in a group to throw the current away from the bank. Single Groynes are neither strong enough to deflect the current nor as effective in causing silt deposition upstream and downstream.

A groynes angled downstream attracts the river flow towards it and is called an attracting groynes as shown in Figure 4-3. The angle of deflection downstream ranges between 30 to 60 degrees. The attracting groynes bear the full fury of the frontal attack of the river on its upstream face, where it has to be armoured adequately. Heavy protection is not necessary on the downstream slope. It merges into the general stream alignment more easily. The scour hole develops off the riverward end of the structure.

When the upstream angled groynes is of short length and changes only the direction of flow without repelling it, it is called deflecting groynes. It gives local protection only.

The angle which the groynes make with the current may affect the results. A groynes built normal to the stream usually is the shortest possible and thus most economic. An upstream angle is better to protect the riverward end of the groynes against scour. A downstream angle might be better for protecting a concave bank, especially if spacing and the lengths of the Groynes are such to provide a continuous protection by deflecting the main currents away from the entire length of bank.

4.3.2 Spacing of Multiple Groynes

The spacing between Groynes depends on the length of the groynes from the bank, its projected length. General recommendations are:

In a straight reach the groynes spacing should be about five (5) times the projected groynes length.

Groynes may be spaced further apart, with respect to their projected lengths, in a wide river than in a narrow river, having similar discharge.

The location of Groynes affects their spacing. The recommended spacing for convex bends is 2 to 2.5 times the projected groynes length; and for concave bends, equal to the projected groynes length.

4.3.3 Length of Groynes

No general rules can be formulated for fixing the length of Groynes. It depends entirely on the corresponding conditions and requirement of the specific site. The length should not be shorter than that required to keep the scour hole formed at the nose away from the bank. Too short a length may cause bank erosion upstream and downstream of the groynes due to eddies formed at the nose. A long groynes may encroach into the main river channel and would not withstand flood attack from discharge concentration at the nose and a high head across the groynes. Normally Groynes longer than one fifth (1/5) the river width are not provided.

4.3.4 Types of Groynes

The different types of Groynes commonly used, named according to the shape of their head, and are listed below:

- ✓ Bar groynes
- ✓ Mole-head groynes
- ✓ Hockey groynes
- ✓ Inverted hockey groynes
- ✓ T-head groynes
- ✓ Sloping groynes
- ✓ T cum hockey-sloping groynes
- ✓ J – head groynes
- ✓ Guide-head groynes

4.3.5 Choice of Type of Groynes

Various factors which influence the choice and design of Groynes are as below:

- ✓ Gradient and velocity of river.
- ✓ Available construction materials.
- ✓ Type of bed material carried by the river (i.e. shingle, sand or silt).
- ✓ Quantity of silt load in river flow.
- ✓ River width or waterway available at high, medium or low discharge.
- ✓ Depth of waterway and flood hydrograph.

Permeable Groynes are best suited to erodible bed rivers normally carrying heavy-silt-laden flow. These are not suitable for small rivers, having steep gradients or deep rivers carrying light-sediment load.

Impermeable Groynes are most suitable for confining a river to a defined channel.

In a straight reach of the river, a series of Groynes are required to provide bank protection. For a curved reach of the river it could be trained by a limited number of Groynes.

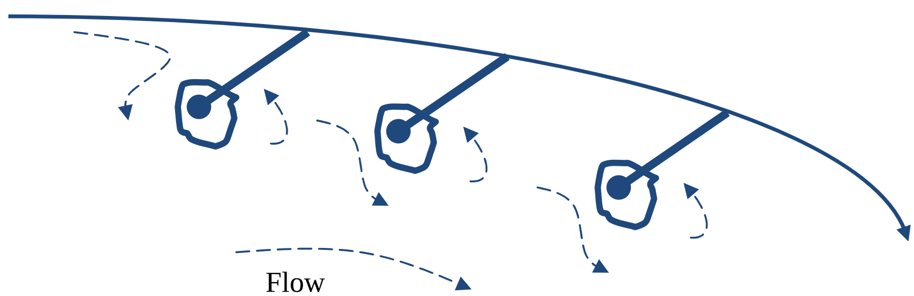


Figure 4-3: Groynes Pointed Upstream

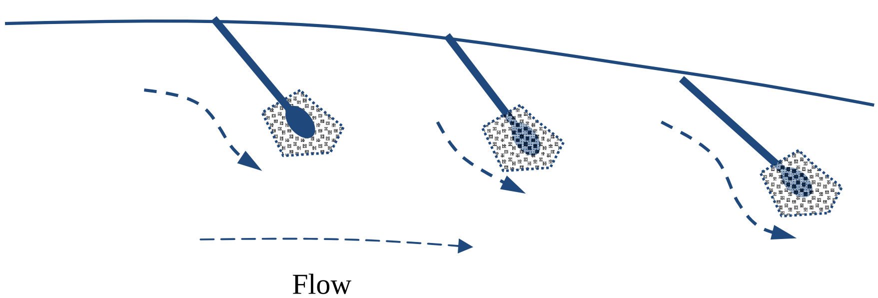


Figure 4-4: Groynes Pointed Downstream

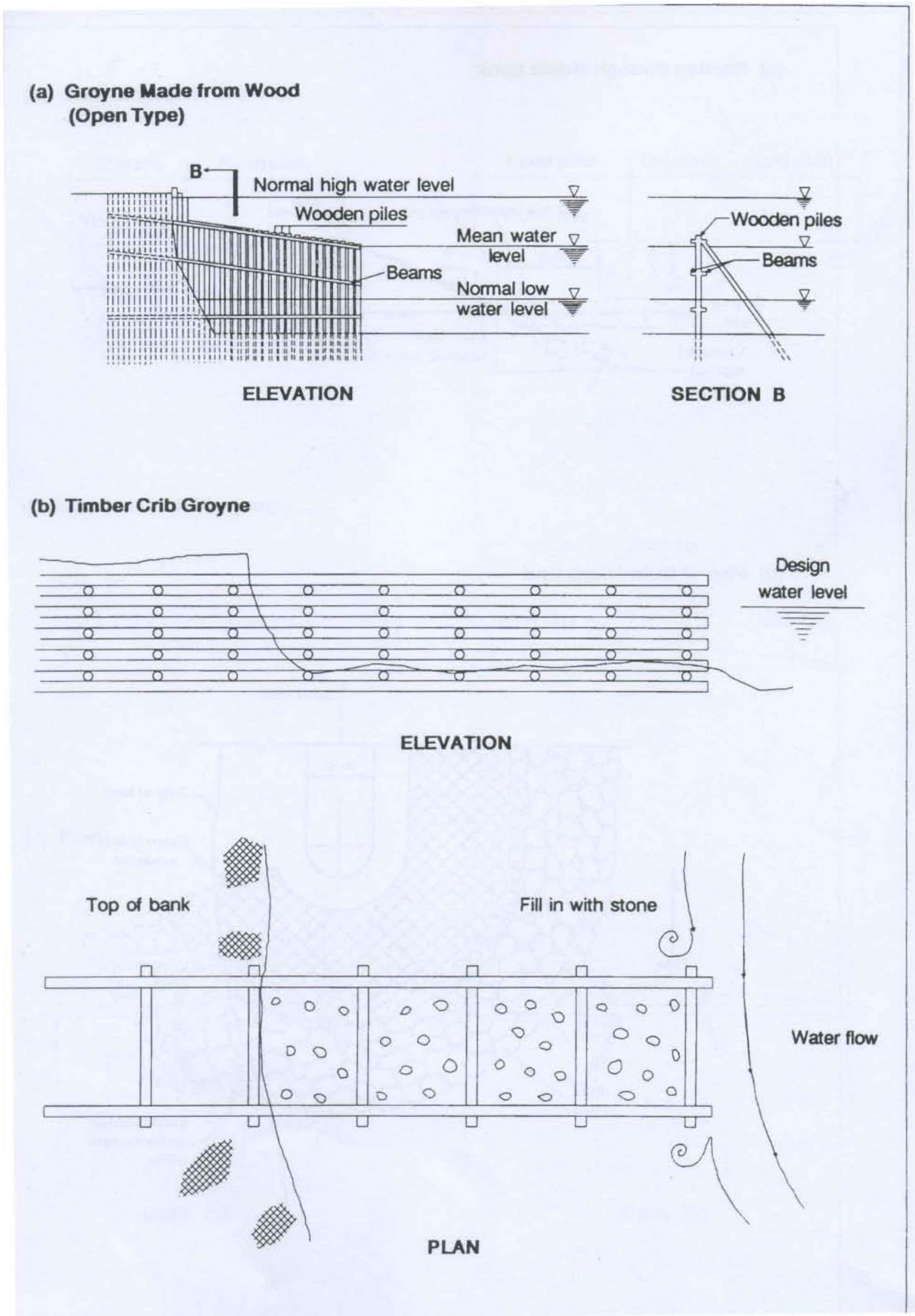


Figure 4-5: Types of Groynes Made from Wood and Timber
(Source: Reference Number 8)

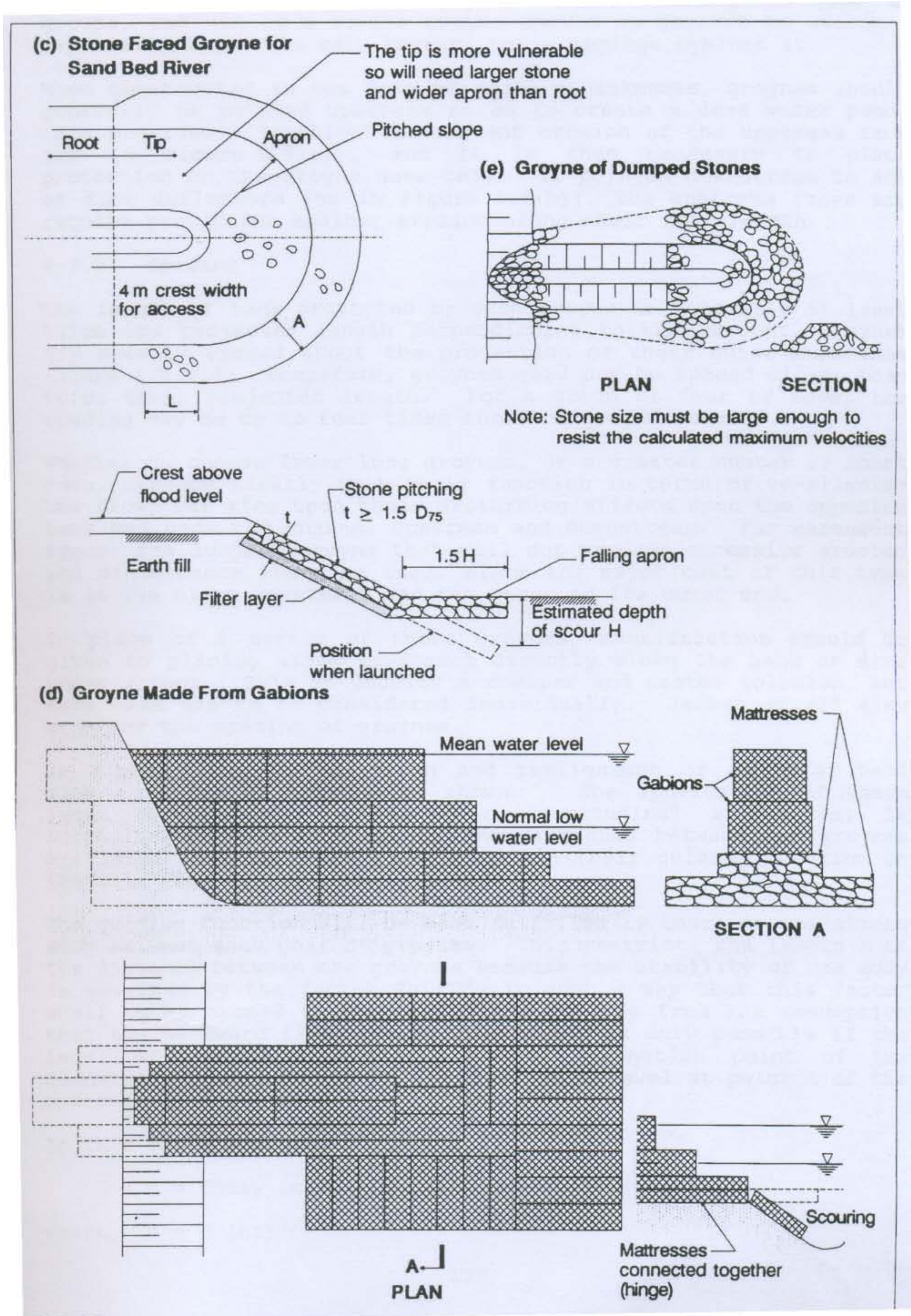


Figure 4-6: Types of Groynes Made Stone and Gabion

(Source: Reference Number 8)

4.4 Retaining Walls

On some schemes, on aggressive rivers having steep gradients and high velocities, concrete, masonry or gabion retaining walls may be provided, either to contain or to train river flow, particularly where protection is subject to frontal river attack. Gabion walls are often used because of their flexibility and cheapness. Masonry or gabion retaining walls are designed as gravity walls.

The various types of retaining walls for river bank protection are discussed in the following section.

4.4.1 Masonry Retaining Walls

Masonry retaining walls are widely used for new construction. Weep holes and an adequate filter behind the wall should be provided, unless the wall is designed to withstand hydrostatic pressures.

4.4.2 Masonry with Dry Stone Panels

Walls with dry stone panels can be provided on the uphill side of the canal, especially where seepage or slumping is a problem.

4.4.3 Concrete Retaining Walls

Mass or reinforced concrete walls are likely to be more expensive than masonry walls and have few advantages.

4.4.4 Dry Stone Walls

Dry stone masonry walls are often used on farmer's canals. They are relatively cheap but canal water is not retained and seepage can occur. The walls often have foundation problems. They can be used for uphill bank support.

4.4.5 Gabion Walls

Gabion walls are more expensive than dry stone walls, but have greater strength and flexibility. Gabion walls should be built as dry stone walls with stones being carefully chosen and packed. The crates should be braced with partitions and tied at frequent intervals to adjacent crates using binding wire. Some times a retaining walls with a gabion for the upper sections and with dry stone foundation may be appropriate.

Longitudinal bank protection structures in gabions can be classified on the basis of their structural and functional characteristics into the following types:

- ✓ Massive structures;
- ✓ Linings;
- ✓ Combined structures;
- ✓ Emergency works; and low level structures.

Massive structures and combined structures have been discussed as they are widly used in most of protection works. However, the user of this manual is advised to refer more on reference number 6 given at the end of this part of the manual for more appropriate solution. The most appropriate solutions may be determined by analysis the nature and characteristics of the river morphology (watercourse) and its surrounding as it is discussed in Section 2 of this manual.

Figure 4-7: Gabion Retaining Wall on Ribb River Upstream of the Main Bridge Site

Source: Consultant Field Visit 2009

Figure 4-7 shows massive gabion retaining wall structure for river bank protection on Ribb River upstream of the main Bridge. The river bank erosion may be attributed to the following major three types direct river flow attack, erosion by the current action and erosion of the upper bank due to wave action as discussed in the Section 2.3 of this part of the manual. Therefore, basic understanding of the erosion characteristics will be the basis for the type of recommendation and detail design of gabion protection structures.

4.4.5.1 Massive Retaining Wall Structures

Is required within the more unstable reaches of a watercourse (Figure 4-7) or where a protective structure is required and has to be provide also earth or river bank retention, it is desirable to use structures of wide cross section.

It is also required in the case where a river follows the foot of an unstable slope or cliff. If the river be cannot be rerouted and erosion by the stream is the cause of instability it is necessary to build a sturdy structure to control the erosion and support the unstable material.

Figure 4-8: Unstable Reaches of Ribb River Meandering Reach

a) Structure with Deep Foundation

In Such a case the wall must be founded at a level that is not affected by water scouring problem as shown in Figure 4-9. Generally a direct foundation of this type is only advisable where the river bed material is virtually in-erodible or where it consists of solid rock.

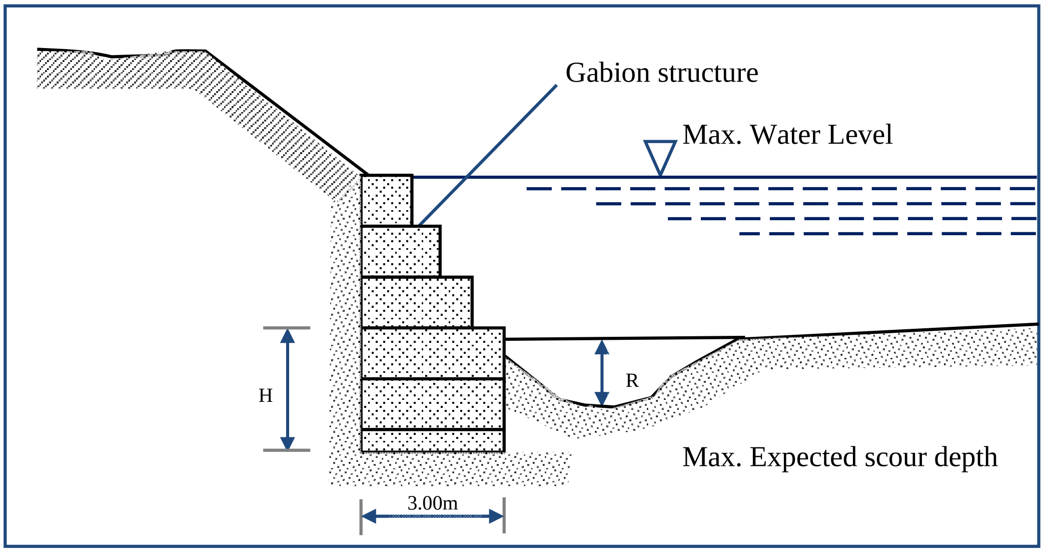


Figure 4-9: Gabion Retaining Wall with Deep Foundation

It is also appropriate in mountainous regions and especially in the more wild torrents where the heavy bed load could damage elements of toe structures such as aprons protruding into the stream bed.

This type of foundation is also appropriate where the rock and gravel deposits can provide an adequate foundation.

b) Structure built on anti-scour aprons

A gabion apron extending horizontally into the bed so that it can adjust to possible scouring provides and adequate foundation for protective structures built upon it as shown in Figure 4-10.

Some times in fact the apron provides an additional margin of safety as compared with direct foundations since the scouring action is kept at a distance from the toe of the structure therefore affording a wide range of response.

A properly sized apron should extend horizontally beyond the vertical front face of the protection in to the river be 1.5 times to 2 times the maximum depth of expected scour.

For projects exceeding 3-4 m in height an apron gives better performance both as regards safety and economy than a direct foundation. The spread of the apron is less costly than excavating for the foundation and increasing the height of stem portion of the structure.

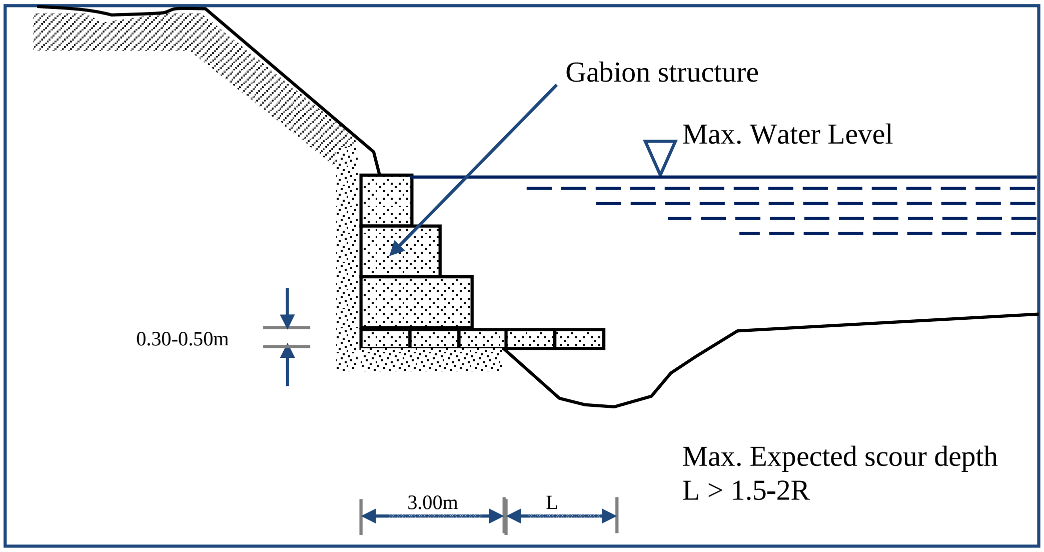


Figure 4-10: Gabion retaining wall built on an apron

c) Structure built on loose rock platform

It is not always possible to build the foundations of river bank improvements in the dry. In such cases a platform can be built of loose rocks and the vertical portion of the protective structure can be conveniently built on top as it is shown in Figure 4-11.

The use of very large boulders is recommended in mountainous reaches where the impact from large bed material in motion could damage the gabion wire mesh.

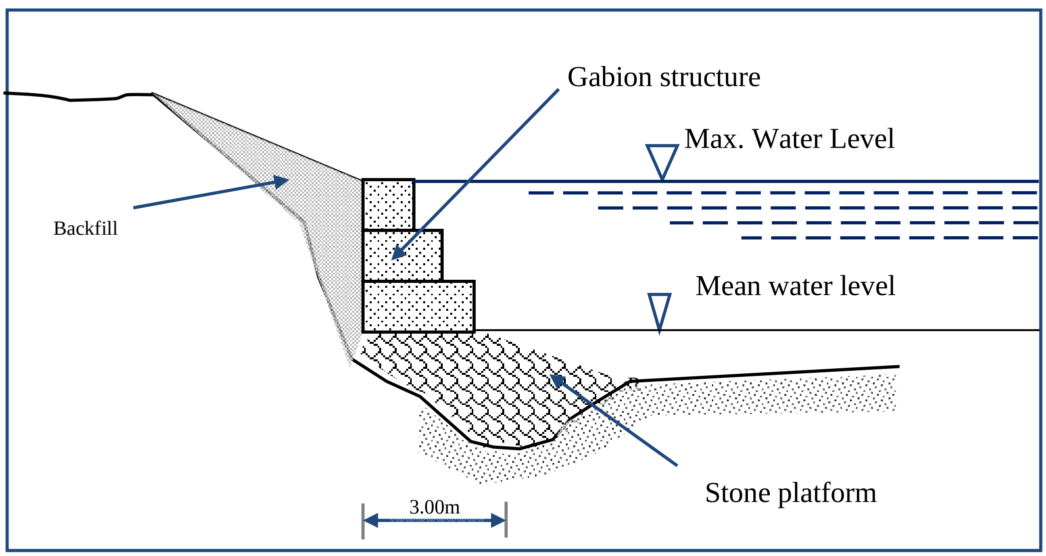


Figure 4-11: Gabion retaining wall built on loose stones

In such cases it is important to choose a rock size to ensure the stability of the supper structure. Rock size requirements are that their size be larger and well assorted so as to prevent movement and washing away of fines.

4.4.5.2 Combined Structure

As discussed in section 4.4.5 of this part of the manual the possible combinations of structures are gabion retaining wall massive structures and Reno mattresses for lining. The linings are simply laid on the slopes of an embankment to be protected. The thickness of the lining can be dictated by the hydraulic conditions of the water course and it may vary from 0.15 m to 0.30 m.

The possible combinations of gabion wall and Reno mattresses are numerous. For instance projects are often built with gabions up to the level of ordinary high water and above that a Reno mattress lining can be installed up to a level slightly above that of maximum high water as it is shown in the following figures.

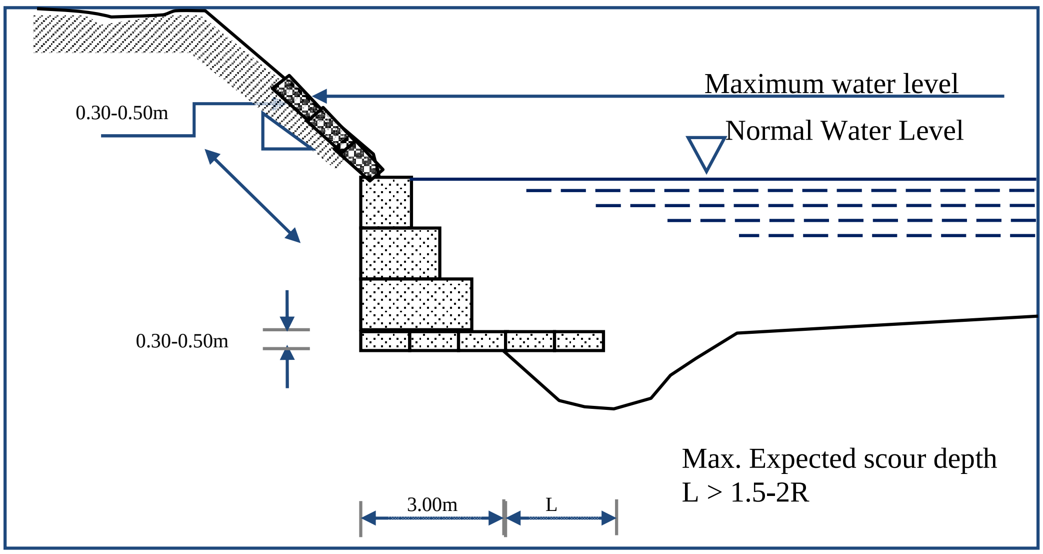


Figure 4-12: Combining Reno mattress and gabion on a foundation platform

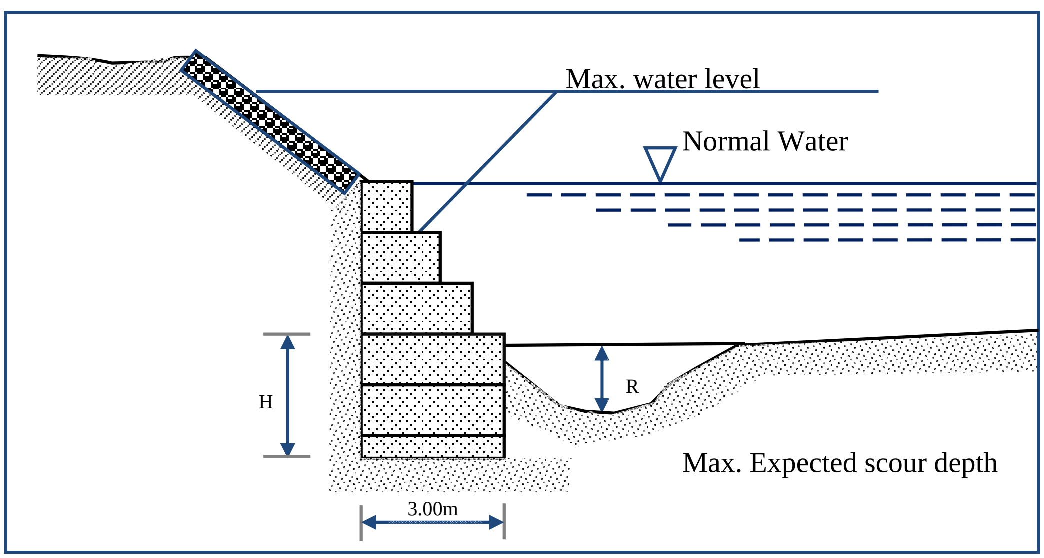


Figure 4-13: Reno mattress and gabion wall structure with deep foundation

4.4.6 Fundamentals for the Selection of Type of Structures

The main function of a bank protection structures is to provide a substantial interface between the water flow and the containing river bed or ground.

To achieve this improvement in the river protection work must satisfy the following basic requirements

Stability: - The work must be capable of supporting the imposed loads, to stabilize the underlining soil and to prevent erosion.

Flexibility: - The ability to absorb settlement deformation without the harm of its other functions

Durability: - The structure should remain effective for the duration of the required design life period.

Maintenance: - The design should allow for maintenance including the repair of local damage and the replacement of deteriorated materials. The elements which require periodic maintenance should be easily accessible for inspection and replacement.

Safety: - During design consideration must be given to eliminating potential risks to the labour force and the public. All factors relating to safety should be incorporated, including consideration of all possible activities that may be taking place on and around the site.

Local Availability of Construction Material: - It is always advisable to select locally available construction material with in the vicinity of the project area.

Environmentally acceptable: - The work will be part of the environment and of the ecological system. To satisfy this requirement the project should not be designed with purely technical considerations in mind.

Cost: - The project will need to fulfil all the functional requirements while staying within budget for both constructions and maintenance.

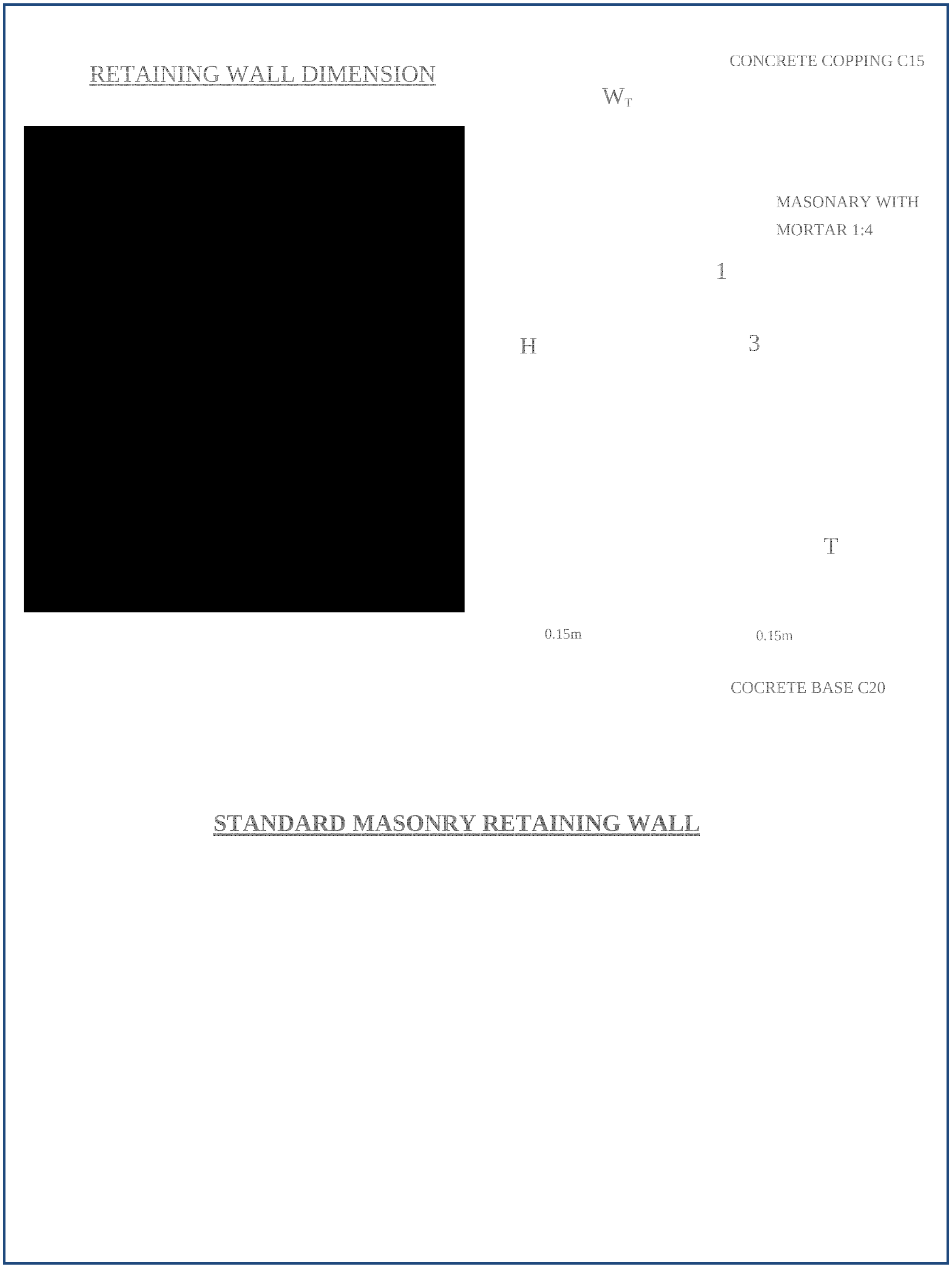


Figure 4-14: Standard Masonry Retaining Wall

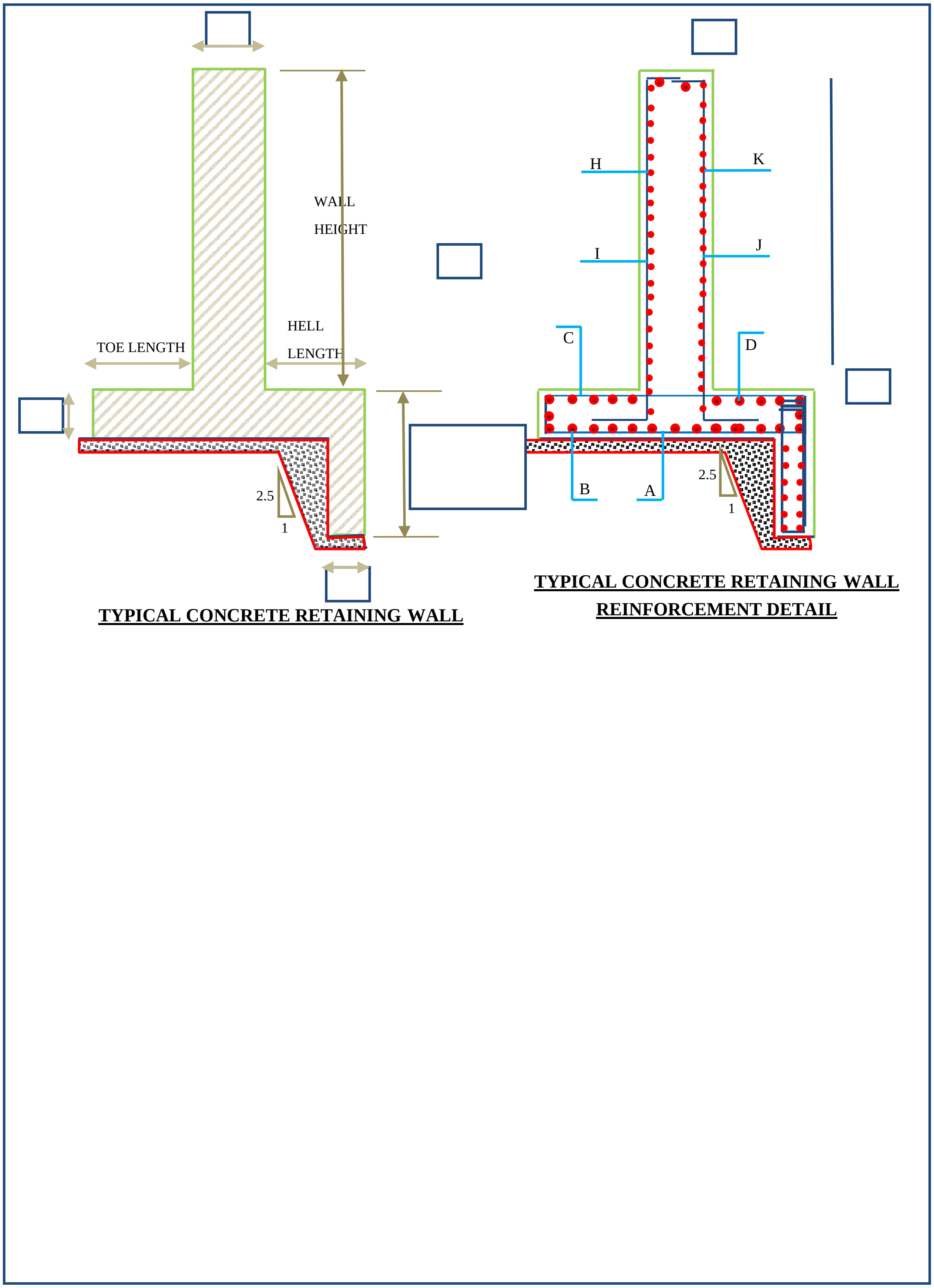


Figure 4-15: Standard Concrete Retaining Wall

4.5 Erosion Protection Works

4.5.1 River Bank Erosion

As has been discussed in the previous section of this manual, the regime of river may not be in equilibrium due to natural and or man made conditions. This creates conditions in which rapid longitudinal or transverse currents erode banks or toe of banks rendering both slopes and embankments unstable.

It is therefore, necessary to understand the river morphology and investigate the conditions that triggered river bank erosion and instability and then recommend the most appropriate erosion protection remedies.

In general stability of river regime or river bank can be achieved through control of meanders (plan) and long sections which are termed as protection works and river stabilization. Additionally, maintaining normal flow condition through creation of uniform channel cross section will lead to river stabilization.

Hence, various types of bank erosion and remedial options have been discussed in the following sections.

Figure 4-16: River Bank Erosion and Sediment Deposition on Small Tributary River Gojam (Source: Consultant Field Visit, 2009)

The various types of bank erosion are classified as follows:

- ✓ River flow attack at the toe of the underwater slope, leading to bank failure and erosion. Usually the greatest likelihood of upper bank failure occurs during a falling river stage.
- ✓ Erosion of soil along the bank caused by current action.
- ✓ Sloughing of saturated banks with floods of long duration, due to rapid receding of the flood.
- ✓ Flow slides (liquefaction) in saturated silty and sandy soil.
- ✓ Erosion of soil by seepage out of the bank at relatively low channel flows/velocities.
- ✓ Erosion of the upper bank due to wave action caused by wind.

River Bank Failure Surface

Figure 4-17: Sliding Failure of River Bank at Ribb River at the Bridge Site, (Source: Consultant Field Visit, 2009)

Probable failure surface

Dyke

Original river bank profile

Max. WL

Normal WL

Figure 4-18: Sketch of Sliding Failure of River Bank (Ribb River)

The river bank failure in figure 4-11 may be attributed for one or more reasons:

- River flow attack at the toe;
- current action;
- floods of long duration; and
- the load by the Improper dyke construction on the edge of the river bank

River bank protection may be classified as direct and indirect:

- ✓ Direct protection: includes works done on the bank itself such as providing armoured protection, or non-structural protection, depending upon the severity of the current;

- ✓ Where the river is not aggressive, it may be practicable to protect the banks with non-structural protections such as bushes and trees.

For aggressive rivers, the banks should be protected by sufficient armoured protection (on a sloping earthen embankment) to resist the tractive shear stress exerted by the flowing water. To prevent outflanking, the revetment should be provided with curved heads cut well into the bank.

Direct protection materials may be gabion and/or stone pitching or stone masonry. The consultants recommend that gabion and/or stone masonry be used as this can accommodate some settlement of the embankment. Masonry is liable to cracking when placed on a sloping earthen embankment and is usually more expensive.

Indirect protection: includes the works installed in front of the banks to reduce the erosive force of the current, either by deflecting the current away from the bank or by inducing silt deposition in front of the bank. This is usually done with Groynes.

4.5.2 Scour

Lack of protection against scour is a frequent cause of failure of river training works. Scour along side banks and training works is liable to occur in the following situations:

- ✓ Alongside channel banks and the main convex part of guide banks where flow is roughly parallel to the face of the bank;
- ✓ Around the noses of guide banks, with flow largely parallel to the bank but with spiral component around the nose itself;
- ✓ Around the end of groynes, with the main flow impinging more or less directly at the groyne axis;
- ✓ Under a bank on the outside of a gentle river bed;
- ✓ Under a bank subject to direct impingement of flow, so that a major change of direction occurs;
- ✓ at a box like structures such as intake, at the edge of a river without guide banks.

Scour depends largely on the following factors:

- ✓ The particles size of the channel bed material;
- ✓ The intensity of the flow per unit width;
- ✓ The velocity of flow and tractive force or bed shear;
- ✓ The degree of turbulence.

4.5.3 Scour Depth

There are various methods of calculating scour for different situations (Scour at bridge piers, irrigation structures etc). For general scour, the Lacey scour formula can be used. The Lacey empirical equation was developed from research carried out in India and may be used to compute the depth of scour. The design scour depth below water flood surface level (Rs) is given by:

Design scour depth (Rs) = XR [metric units]

Where:

X = scour factor dependent on type of reach (see Table 4.1 below)

R = depth of scour measured from the water surface = 1.35 (q^2/f) ^1/3

q = the maximum discharge per unit width [m2/s]

f = Lacey's silt factor

Type of Reach	Mean Value of Scour Factor "X"
Straight	1.25
Moderate bend (Most transitions)	1.50
Severe bend (also Shank protection at Groynes)	1.75
Right angled bend (and pier noses and groynes	2.00
Nose of Guide Banks	2.25

Table 4-1: Scour Factors

The Lacey silt factor (f) can be calculated from

$$f = 2.46\ V^2 / Dm$$

Where, V = Velocity

Dm = mean depth = A/Ws

A = flow area

Ws = Water surface width

Or it can be read from the following table for the corresponding river bed material.

Soil Type	Lacey's Silt Factor "f"
Large boulders and shingle	20.0
Boulders and shingle	15.0
Boulders and gravel	12.5
Medium boulders, shingle and sand	10.0
Gravel and cobles	9.0
Gravel	4.75
Coarse cobles and sand	2.75
Heavy sand	2.0
Fine cobles and sand	1.75
Coarse sand	1.5
Medium sand	1.25
Standard silt	1.0
Medium silt	0.85
Fine silt	0.6
Very fine silt	0.4
Clay	5.0

Table 4-2: Lacey’s Silt Factor

4.5.4 Scour Protection Works

Stone protection to bunds or banks may comprise a combination of stone pitching and gabion mattresses. This stone protection is placed on the front (upstream) 2H: 1V slope of the embankment, extending down to the design scour depth as calculated in Section 4.5.3. The protection is placed in its launched position by excavating and backfilling the river bed. If river diversion costs prevent placing in a launched position, then a launching apron may be placed on the river bed.

It is suggested that gabions should extend from the design scour level to at least the normal flood level, with pitching continuing to the design flood level, (i.e. the top of the bund) as shown in Figure 4-9.

4.5.5 Local Scour and its Protection Methods

The basic principles of protection works have been discussed in the following sections. At this point, the methods of scour protection works will be discussed for more information. There are basically three methods of protection works against scour around river training works and groynes, and at the foot of embankments and other hydraulic structures.

i. Excavate and continue the slope revetment down to a resistant material or to below the calculated scour depth as shown in Figure 4-8. This method is the most permanent, but it may be impractical or uneconomical if deep scour is expected.
ii. Drive a cut-off wall of sheet pilling from the toe of the revetment down to a resistant material or to below the expected scour level as shown in Figure 4-9. Such walls are subject to risk of failure from earth pressure on the bank side after scour occurs on the channel side, and tend to cause deeper scour than paved slopes. The risk of failure resulting from unforeseen scour can be reduced by tying back the piling to deadmen or similar anchors. Sheet piling is not recommended in coarse bed channels.
iii. Lay a flexible launching apron horizontally on the bed at the foot of the revetment, so that when scour occurs the materials will settle and cover the side of the scour hole on a natural slope. This method is recommended for cohesionless channel beds where scour is expected, as being generally the most economical. This launching, or falling apron, may be formed of a gabion mattress in cases where scour is expected to be no more than, say 2 m.

4.5.6 Stone Size

The size of stones for protection works designed to resist the average channel velocities. USBR recommends the following formula for determining the size of stone that will not be dislodged under turbulent flow conditions:

$D50 = (V_{avg} / 4.915)2$ (turbulent flow conditions) [metric units]

Where:

V_{avg} = average velocity of flow for maximum discharge [m/s]

D50 = average stone size [m]

The specific gravity of the stones is assumed to be 2.65 (i.e. density of 2,650kg/m3).

If less dense stone is used, then the stone size should be increased correspondingly.

For low-turbulent flow conditions, such as exist along the shank of a flood protection bund, the required stone size will be less than that given above. A reduction in the D50 stone size of 40% is probably acceptable.

4.5.7 Grading of Stone

The grading of the stone pitching should be as follows:

- ✓ Maximum stone size = 1.5D50
- ✓ Minimum stone size = 0.5D50
- ✓ Not more than 40% of the stone should be smaller in size than D50

4.5.8 Thickness of Stone Pitching / Gabions

The thickness of the pitching should be 1.5 times the stone D50 size. It is usual to place the pitching on a filter layer (0.15m to 0.20m thick), to prevent fines being washed out from the embankment during the flood recession.

An alternative empirical equation for stone thickness is:

Thickness = 0.06 QT 1/3 (m)

Where:

QT = flood flow with a return period of T years [m3/s]

The thickness of stone pitching and gabion protection specified for a range of average flood flow velocities are recommended as shown in Table 4-3.

4.5.9 Filter Layer

Stone protection placed on embankments should be laid on a filter layer to prevent piping. When one filter layer is sufficient it is called a “graded filter”. When more than one filter layer is used, the coarser filter is placed on top of a finer filter (i.e. the permeability increases outwards), and the filter is called an "inverted filter".

The gradation of a graded filter should conform to the following guidelines established originally by Terzaghi:

- ✓ d15 filter / d85 soil < 5;
- ✓ d15 filter / d15 soil > 5; and
- ✓ d50 filter / d50 soil < 25

Where d85 is the sieve size which will pass 85% of the material, and similar for other percentages (d15 and d50).

The above criteria relate respectively to:

- ✓ stability (i.e. preventing the movement of soil particles into the filter);
- ✓ permeability; and
- ✓ Uniformity.

If this cannot be achieved with a singe filter layer, then two layers shall be used, where the upper layer of the filter is designed using the above criteria, where the soil parameters are replaced by the parameters relating to the filter below.

Launching Aprons

Width and Thickness

The amount and size of the (tipped) stone that makes up the launching apron must be sufficient so that after launching into the scour hole it remains to protect the face of the scour hole.

The following width and thickness for the apron may be adopted.

Width = 1.5 Db [metric units]

Thickness = 1.9T [m]

Where:

Db = Lacey scour depth below bed level (=Ds – Y) [m]

Y = average depth of flow [m]

T = thickness of pitching/gabions as calculated in Section 3.4.4.

The specific gravity of the stones was assumed to be 2.65 (i.e. density of 2,650kg/m3). If less dense stone is used, then the stone size should be increased correspondingly.

For low-turbulent flow conditions, such as exist along the shank of a flood protection bund, the required stone size will be less than that given above. A reduction in the D50 stone size of 40% is probably acceptable.

Average River Flow Velocity for 100 Year Floods, V(m/s)	**Lower Stone Protection**		**Upper Stone Protection**	
	Gabion TG2 (m)	Filter TF2 (m)	Gabion Stone Pitching TG1 (m)	Filter TF1 (m)
V<3.2	0.45	0.15	0.30 (stone pitching)	0.15
3.2< V 4.0	0.60	0.20	0.45 (Gabion)	0.15
4.0< V 4.5	0.75	0.20	0.45 or 0.60	0.15 or 0.20
4.5< V 5.0	0.90	0.20	0.60 or 0.75	0.20

Table 4-3: Recommended Stone Protection for Revetment

Class 1		
Nominal 300 mm diameter of 35 kg mass		Allowable Velocity up to 3 m/s
Grading	100% < 450 mm (135 kg)	
	20% >350 mm (70 kg)	
	50% >300 mm (35 kg)	
	80% >200 mm (10 kg)	
Class 2		
Nominal 500 mm diameter or 100 kg mass		Allowable Velocity up to 4 m/s
Grading	100% < 750 mm (700 kg)	
	20% >600 mm (600 kg)	
	50% >500 mm (180 kg)	
	80% >300 mm (35 kg)	
Class 3		
Nominal 750 mm diameter or 180 kg mass		Allowable Velocity up to 4.5 m/s
Grading	100% < 1200 mm (2300 kg)	
	20% >900 mm (1150 kg)	
	50% >750 mm (680 kg)	
	80% > 500mm (180 kg)	

Table 4-4: Stone Riprap Grading for Stream Bank Revetment and Bed Protection

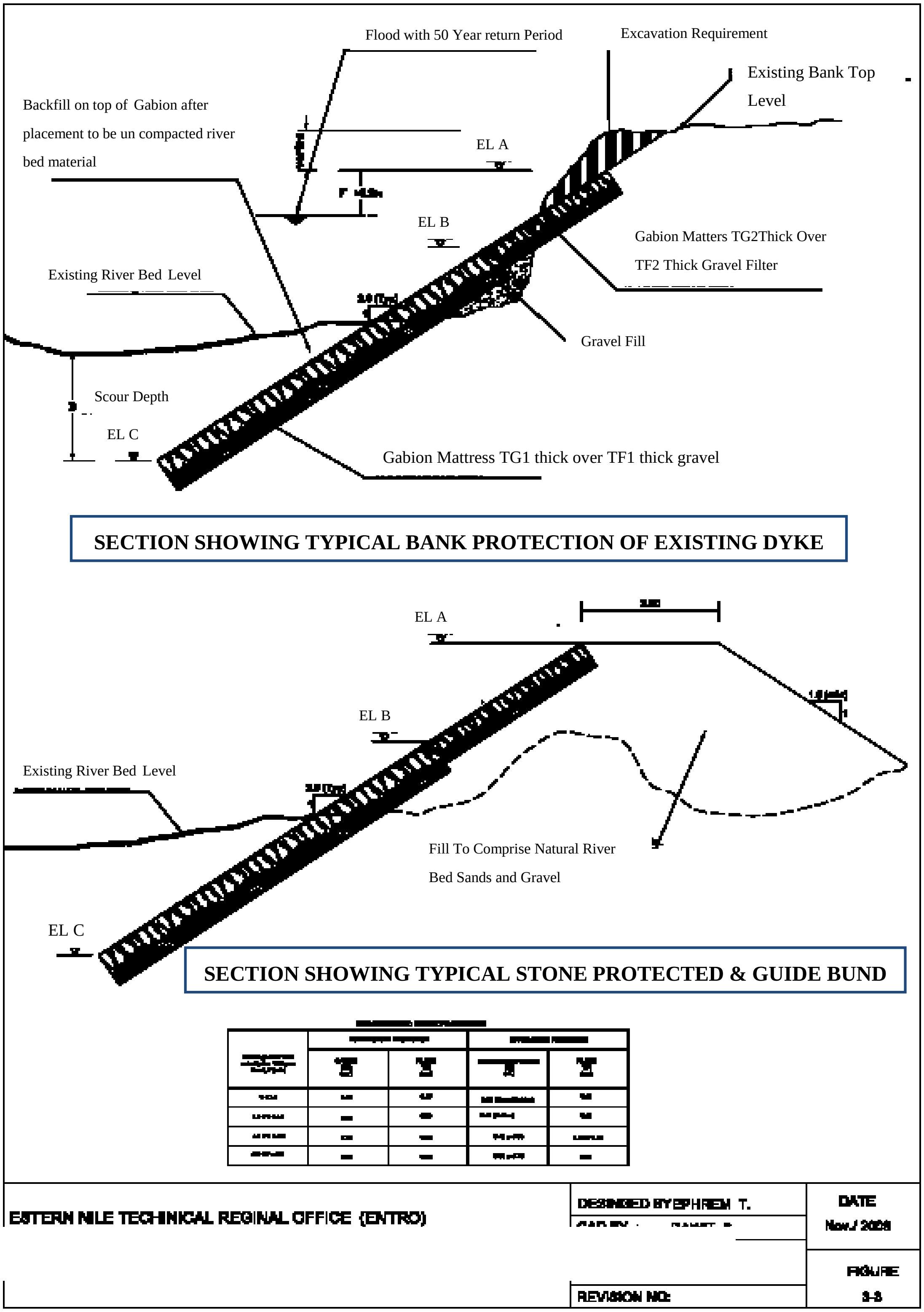


Figure 4-19: Scour Protection at toe of Embankment

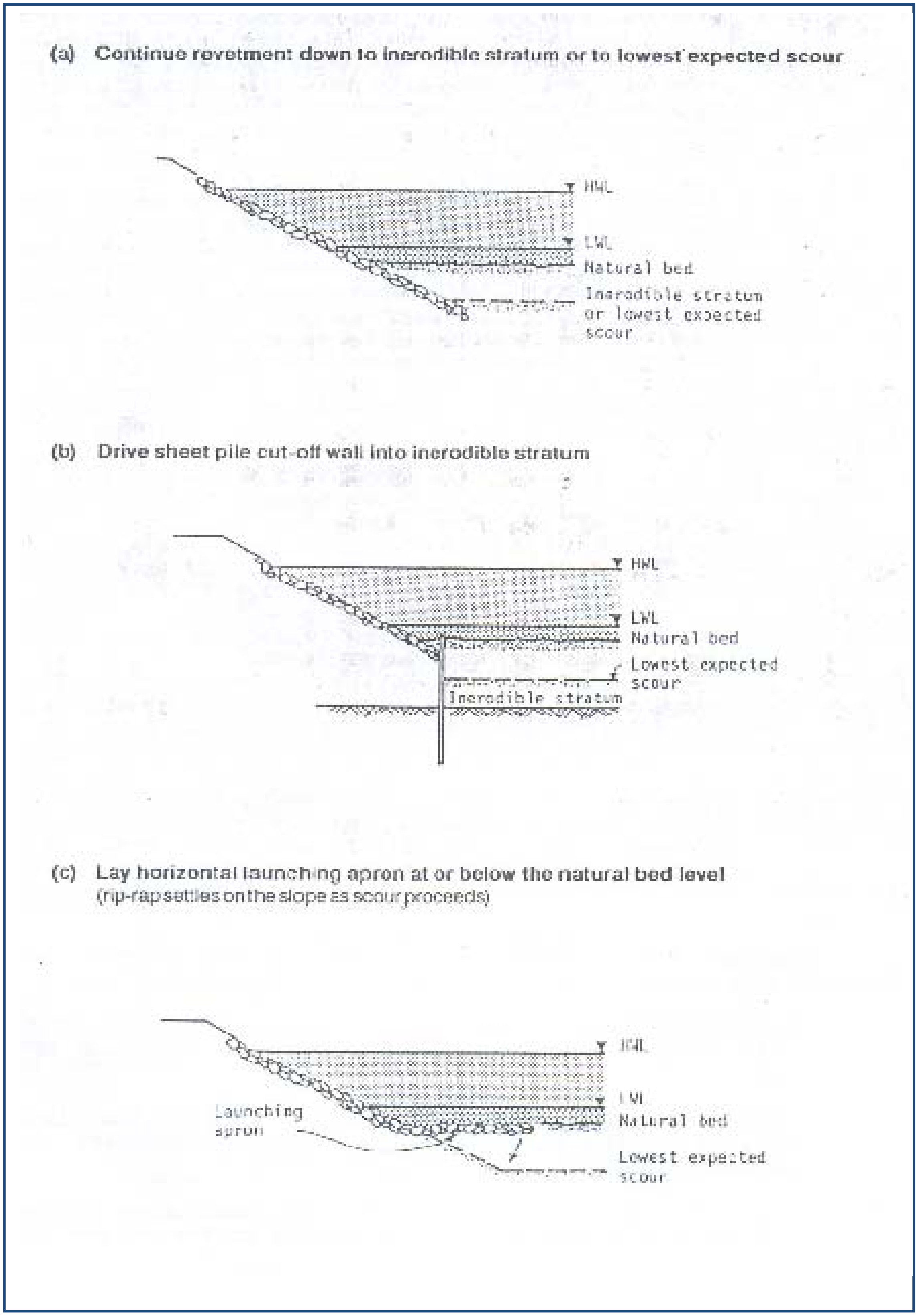


Figure 4-20: Methods of Protecting Bank Revetment against Scour

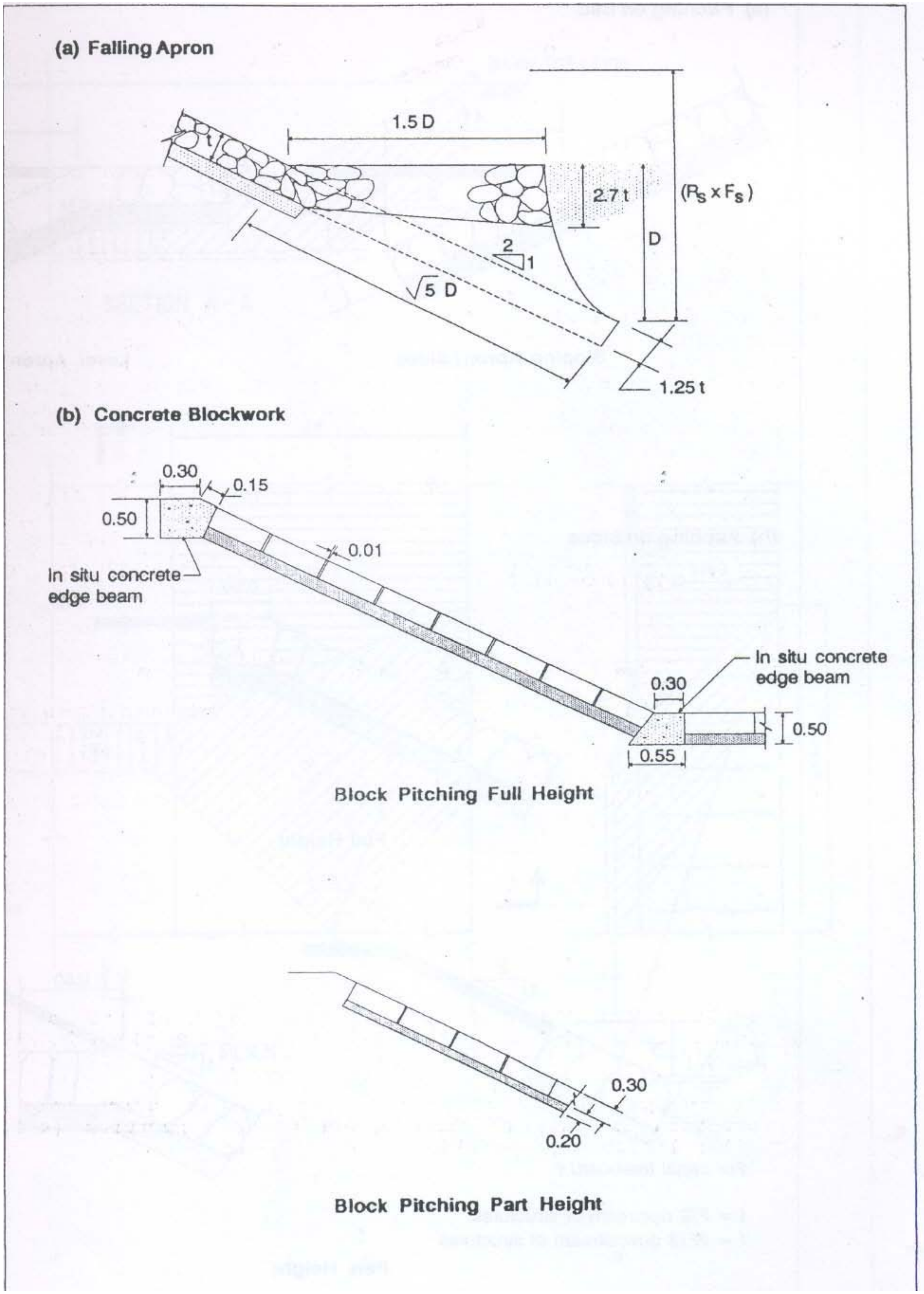
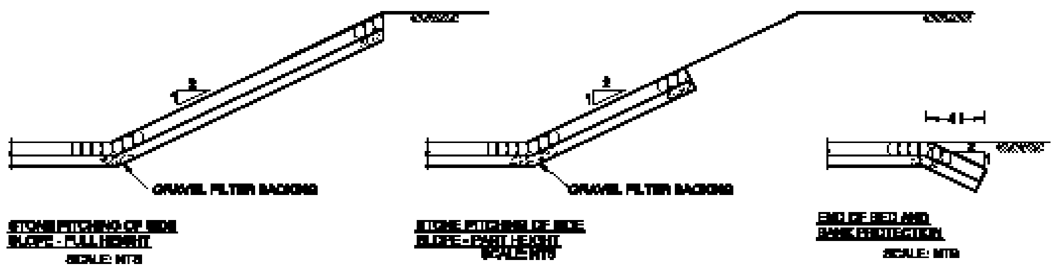


Figure 4-21: Flexible Protection Details for Structures



DESIGN DISCHARGE (M^3/S)	DOWNSTREAM PROTECTION			UPSTREAM PROTECTION	THICKNESS OF PITCHING (M)
	TURNDOWN DEPTH (M) D_T	BED L_2 (M)	SIDES L_3 (M)	BED & SIDES, L_1 (M)	
0.10	0.0	1.0	1.0	0.6	0.2M ON 0.15M GRAVEL BACKING
0.15	0.40	1.0	1.0	1.0	
0.25					
0.40					
0.60					
0.90	0.50	2.0	4.0		
1.50					0.3M ON 0.15M GRAVEL BACKING
2.90	0.60	3.0	6.0	2.0	
4.20					
7.00					
10.00					
10.60		4.0	8.0	3.0	
20.00					
33.00		6.0	12.0	4.0	
50.00					
80.00					

ESTERN NILE TECHINICAL REGINAL OFFICE (ENTRO) FLOOD PROTECTION & EARLY WARNING PROJECT (FPEWP) FLOOD EMBANKMENT DESIGN MANUAL IN ETHIOPIA	DESINGED BY EPHREM T. CAD BY : DAWIT S. CHECKED BY : APPROVED BY : REVISION NO:	DATE Nov./ 2008 FIGURE 4-7

Figure 4-22: Stone Pitching Details for Structures

5. CONSTRUCTION OF EMBANKMENTS / DYKES

5.1 Introduction

The construction management aspect of flood protection works in general have been discussed in **PART 9 CONSTRUCTION ASPECT** of this manual. This Chapter presents the technical aspects of construction methods and procedures for flood embankment.

5.2 Foundation Preparation

Minimum foundation preparation for dykes consists of clearing and grubbing, and most dykes will also require some degree of stripping.

Clearing consists of complete removal of all objectionable and/or obstruct ional matter above the ground surface. This includes all trees, fallen timber, brush, vegetation loose stone, abandoned structures, fencing, and similar debris. The entire foundation area under the levee and berms should be cleared well ahead of any following construction operations

Grubbing consists of the removal, within the dyke foundation area, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectionable matter. Roots or other intrusions over 38 mm in diameter within the dyke foundation area should be removed to a depth of 1 m below natural ground surface. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill, consisting or material similar to adjoining soils, should be placed in layers up to the final foundation grade and compacted to a density equal to the adjoining undisturbed material.

After foundation clearing and grubbing operations are complete, stripping is commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil. The depth of stripping is determined by local conditions and normally varies from 152 to 305 mm. All stripped material suitable for use as topsoil should be stockpiled for later use on the slopes of the embankment and berms.

5.3 Borrow Area

Dykes will be constructed with materials excavated from adjacent canals and borrow areas to balance excavation quantities and minimize construction costs. Where possible, riverside borrow areas are generally preferred being wide and shallow. While riverside borrow is generally preferable, required landside borrow from pond areas, ditches, and other excavations should be used wherever possible.

Selection of borrow area should be based on material type, available quantities, haul distance and much more attention should be given for environmental aspects, increasing land value, overall dyke stability, and erosion.

In computing required fill quantities, a shrinkage factor of at least 25 percent should be applied (i.e., borrow area volumes should be at least 125 percent of the dyke cross-section volume). This will allow for material shrinkage, and hauling and other losses. Right-of-way requirements should be established about 4 to 6 m beyond the top of the planned outer slope of the borrow pit. This extra right-of-way will allow for flattening or caving of the borrow slopes, and can provide maintenance borrow if needed later.

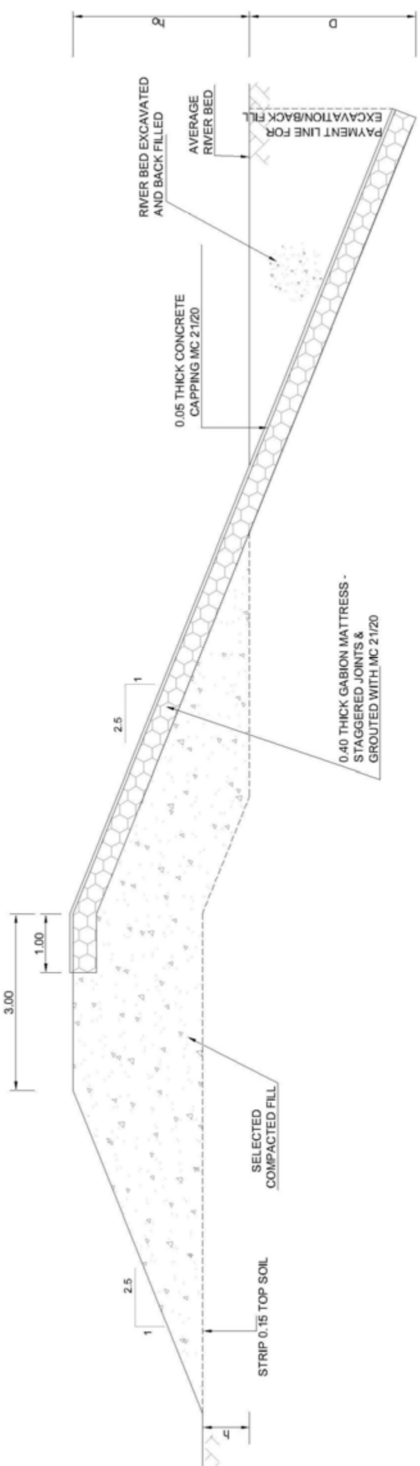
Excavation slopes of borrow areas should be designed to assure stability. The slopes of the upstream and downstream ends of riverside borrow area should be flat enough to avoid erosion when subjected to flow at high water stages

5.4 Practical Considerations

The ends of flood protection or guide bunds should never be left “hanging”. Either they should be tied into (adjacent) high ground, or a bulbous nose should be provided, with stone protection extending all around the nose.

Some practical drawings from "Awash Flood Control and Watershed Management Study Project" have been shown for in the following pages for visualization of flood protection design concepts.

These figures are flood protection type bunds, and flood walls designed for urban flood protection Dire Dawa.



NEW GABION BUND

Goro River Left Bank

From	To	Chainage from	Chainage To	Reach Length	Approx Bank Height (h)	Height above River Bed (ho)	Scour depth (D)
		(m)	(m)	(m)	(m)	(m)	(m)
	Section 12	2395	2608	213	2.1	3.0	2.4
Section 12	Section 13	2608	2846	238	1.7	1.9	5.4
Section 13		2846	2963	117	1.8	2.1	1.4

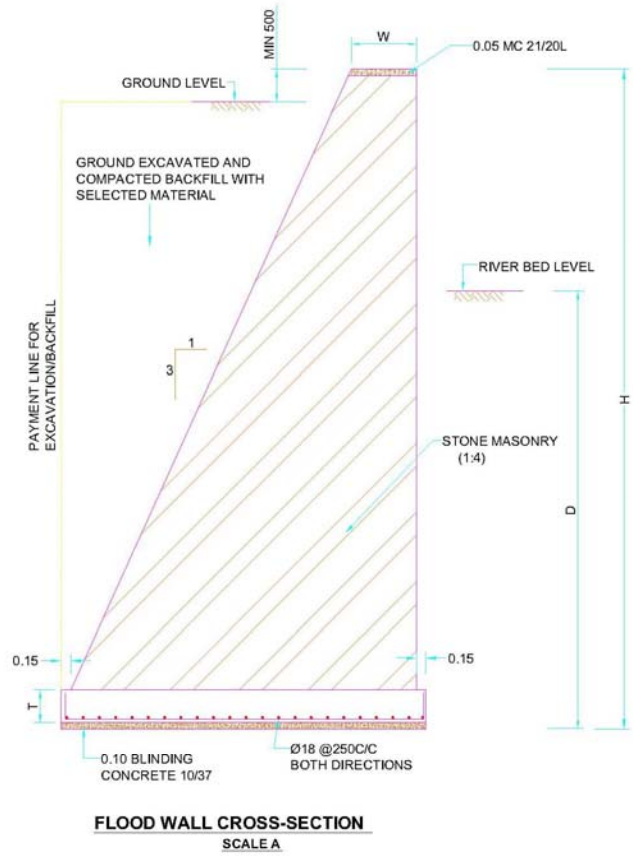
NEW GABION BUND

Goro River Right Bank

From	To	Chainage from	Chainage To	Reach Length	Approx Bank Height (h)	Height above River Bed (ho)	Scour depth (D)
		(m)	(m)	(m)	(m)	(m)	(m)
	Section 13	2670	2846	176	1.8	2.4	6.8
Section 13	Section 14	2846	3090	244	1.7	2.1	1.4

Figure 5-1: Goro River Gabion Bund

(Source: Awash Flood Control & Watershed Project Dire Dawa Flood Protection)



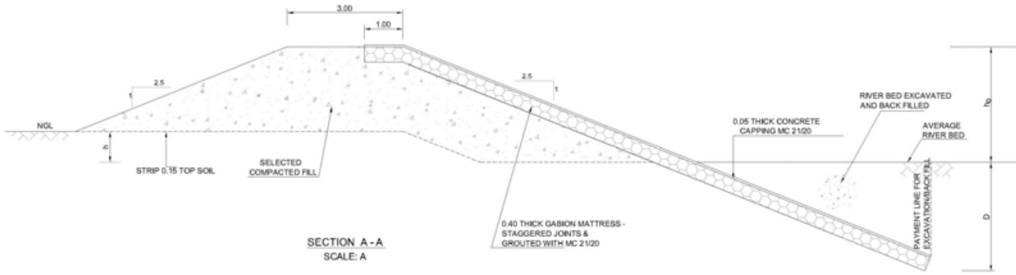
DECHATU RIVER - NEW FLOOD WALL

Chainage From (m)	Chainage To (m)	Reach length (m)	Wall below river bed D (m)	Total wall height H (m)	Base slab thickness T (m)	Crest width W (m)
LEFT BANK						
3,284	3,294	10	4.1	8.0	0.7	1.4
3,294	3,400	106	4.1	8.0	0.7	1.4
3,400	3,514	114	4.1	8.0	0.7	1.4
3,514	3,764	250	4.8	8.0	0.7	1.4
3,764	3,894	130	4.3	7.2	0.6	1.2
3,894	3,904	10	4.4	7.3	0.6	1.2
3,904	4,014	110	4.6	8.0	0.7	1.4
4,014	4,248	234	4.6	8.0	0.7	1.4
4,258	4,278	20	5.7	7.9	0.6	1.2
4,278	4,528	250	5.2	9.5	0.8	1.5
4,528	4,670	142	4.7	8.2	0.7	1.4
4,670	4,778	108	4.5	8.1	0.7	1.4
5,028	5,262	234	5.4	8.9	0.7	1.4
5,262	5,272	10	6.1	9.0	0.8	1.5
6,248	6,308	60	7.4	11.3	1.0	1.7
6,308	6,318	10	3.8	8.0	0.7	1.4
6,318	6,568	250	3.7	7.9	0.6	1.2
6,568	6,600	32	4.5	7.5	0.6	1.2
6,600	6,818	218	4.2	7.3	0.6	1.2
6,818	7,068	250	4.2	7.6	0.6	1.2
9,094	9,107	13	4.8	6.5	0.6	1.2
9,107	9,117	10	3.6	7.4	0.6	1.2

Chainage From (m)	Chainage To (m)	Reach length (m)	Wall below river bed D (m)	Total wall height H (m)	Base slab thickness T (m)	Crest width W (m)
RIGHT BANK						
2,650	2,750	100	2.9	5.5	0.5	1.0
2,750	2,780	30	3.9	5.8	0.5	1.0
2,780	3,000	220	4.8	8.0	0.7	1.4
3,000	3,284	284	3.7	6.7	0.6	1.2
3,284	3,294	10	2.9	6.3	0.6	1.1
3,294	3,400	106	2.9	6.3	0.6	1.1
3,400	3,514	114	2.9	6.3	0.6	1.1
6,200	6,248	48	5.7	9.1	0.8	1.5
6,248	6,308	60	5.7	9.1	0.8	1.5
6,308	6,318	10	3.8	8.0	0.7	1.4
6,318	6,568	250	3.7	7.9	0.6	1.2
6,568	6,600	32	4.5	7.5	0.6	1.2
6,600	6,818	218	4.2	7.3	0.6	1.2
9,094	9,107	13	4.8	6.5	0.6	1.2
9,107	9,157	10	3.6	7.4	0.6	1.2

Figure 5-2: Dechatu River Flood Wall

(Source: Awash River Basin Flood Control & Watershed Project Dire Dawa Flood Protection)

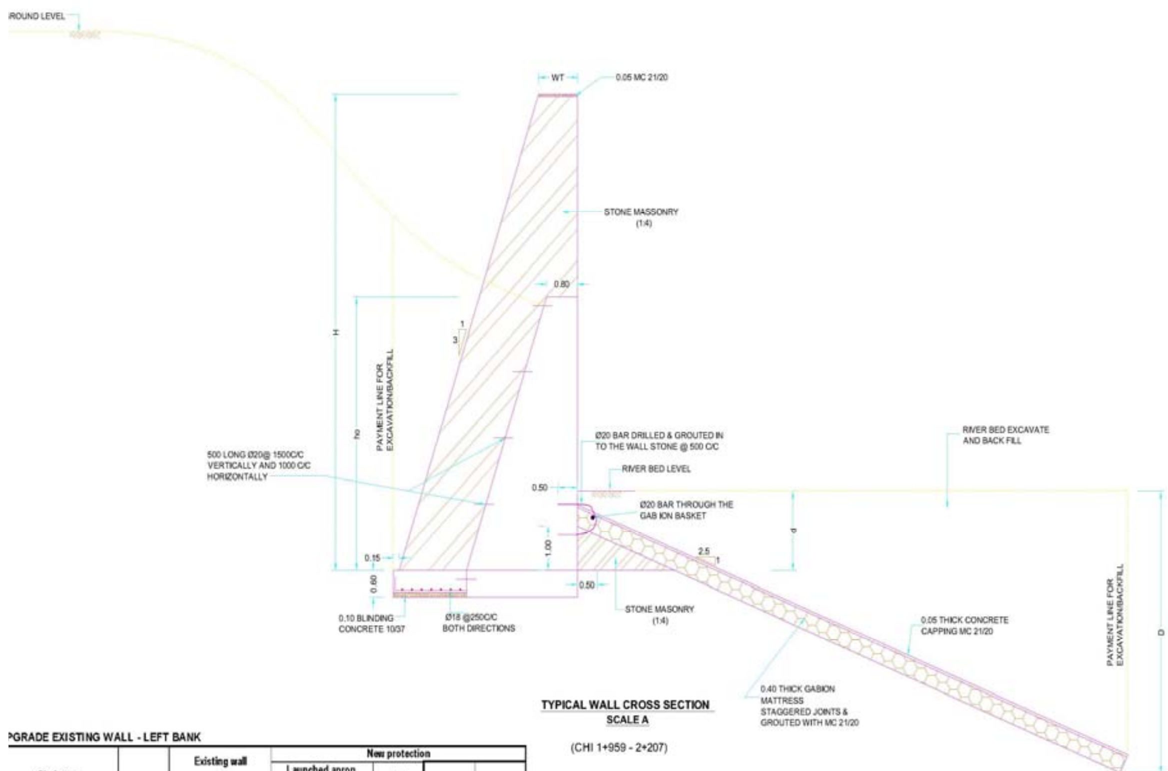


DECHATU RIVER - NEW FLOOD DYKE

Chainage		Reach length	Gabion below river bed	Dyke height above river bed	River bed depth below NGL	Chainage		Reach length	Gabion below river bed	Dyke height above river bed	River bed depth below NGL
From	To		D	ho	h	From	To		D	ho	h
(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)
LEFT BANK						*RIGHT BANK*					
7,068	7,318	250	3.2	5.8	3.0	6,818	7,068	250	4.2	7.6	2.9
7,318	7,568	250	3.1	5.8	2.8	7,068	7,318	250	3.2	5.8	2.8
7,568	7,828	260	3.2	6.1	2.9						
7,828	7,950	122	2.4	4.4	2.7						
7,950	8,078	128	1.8	3.7	2.5						

Figure 5-3: Dechatu River Flood Dyke

(Source: Awash River Basin Flood Control & Watershed Project, Dire Dawa Flood Protection)



PGRADE EXISTING WALL - LEFT BANK

Chainage		Reach length	Existing wall		New protection				
					Launched apron		Wall raising (h_1)	Top Width (WT)	Base slab thickness (T)
From	To		Below river bed (d)	Total wall height (h_0)	Depth below river bed (D)	Width			
(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)
50	373	323	2.0	4.0	0.0	0.0	4.3	1.5	0.6
1657	1722	65	2.0	4.0	4.3	8.8	2.4	1.9	1.0
2267	2370	103	2.0	4.5	4.0	8.1	1.1	1.5	0.6
2370	2395	25	2.0	4.5	2.4	3.9	0.6	1.5	0.6

PGRADE EXISTING WALL - RIGHT BANK

Chainage		Reach length	Existing wall		New protection		
					Launched apron		Wall raising (h_1)
From	To		Below river bed (d)	Total wall height (h_0)	Depth below river bed (D)	Width	
(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)
1980	2207	227	2.0	4.4	5.2	11.2	0.0
2207	2267	60	2.0	5.1	3.0	5.4	0.0
2267	2370	103	2.0	5.1	3.0	5.4	0.0
2370	2395	25	2.0	5.1	2.4	3.9	0.0
2395	2608	213	2.0	5.1	2.4	3.9	0.0
2608	2670	62	2.0	5.1	6.8	15.6	0.0

Figure 5-4: Dechatu River Flood Wall

(Source: Awash River Basin Flood Control & Watershed Project Dire Dawa Flood Protection)

This page is intentionally left blank

FLOOD PROTECTION DESIGN MANUAL

PART 9: CONSTRUCTION ASPECTS

Contents

CONTENTS OF OTHER PARTS

CHAPTER-1 INTRODUCTION

1-1 General

A project has three development phases in the entire life cycle: the planning phase (see Part-1), the implementation or construction phase (this module) and the operation and maintenance phase (see Part-10). The construction stage of a project starts immediately after the award of the contract. This construction juncture is the conversion of ideas, designs and dreams into physical reality where the owner invests money to get feedbacks as services or incomes.

As in the planning stage; construction phase also has design modifications, partial design changes, omissions, additions, etc to acquire the planned objectives. There is no single construction work which has a complete design or which is built as designed.

There are some types of constructions, for instance underground works, tunnels, grouting, etc which are very difficult to fully anticipate the activities at the planning stage even to half of its magnitude. Sometimes in some projects, the construction stages may be more difficult than the planning stages. If the planning stage works, i.e., design works & the agreement writings are poorly done the construction stage will be full of mess and litigations. There will be also unnecessary costs, delays and disputes to the owner.

1-2 Concepts of Construction Management

Management in general is an activity process, composed of some basic functions, for getting the objectives of any entity or enterprise accomplished through and with the efforts of its personnel. Or in other terms management is a science of arranging various activities and group of people in an organization to achieve a common goal. Wherever and whenever objectives are to be achieved through organized and cooperative endeavor, management becomes essential for directing and unifying efforts towards a common purpose.

Like in enterprises, constructions also need proper management. Despite the methods of applications of the management functions, the same concepts hold true for constructions as there are people, resources, time, etc in the realization of a project.

As discussed in Part-1 of this guideline, classical construction management concerns mostly were concentrating on important three points (angles); these are: managing time, managing costs and managing quality. Actually these are the three strategic body and principal indicators of project management.

However, nowadays project management knowledge areas are increased from three to nine angles (Tsegaye D. 1999). These are

- goal management;
- time management;
- cost management;
- quality management;

- risk management;
- human resource management;
- communication management;
- procurement management and
- integration management.

The above management areas are subjects of vast knowledge and discipline courses. Although they are very important areas of management applicable to flood protection works, the elaborate discussions and inclusions here perhaps make the guideline bulky and lengthy, and it is therefore left for further reading.

1-3 Functions of Construction Management

The aims of construction management are to facilitate the execution of work in a planned and efficient manner as per designs and specifications within the prescribed time limit and within the greatest possible economy at the specified quality standard. To achieve these aims the management must have the following functions:

(i) Planning, (ii) Organizing, (iii) Staffing, (iv) Leading and (v) Controlling.

Some management authors and literatures put management functions as six roles (POSCOM): Planning, Organizing, Staffing, Communicating, Regulation and Motivating.

These functions are closely interlinked and interwoven in character. All executive or managers, regardless of their areas and positions, are to discharge these functions. These functions are the identifying marks by which a manager can be differentiated from a non-manager. Of the five functions, however, the upper level managers (see Part-1, Sec. 2-4) or executive are mostly preoccupied with the first two functions-planning and organizing while the lower ranking executive are largely busy with the remaining three functions. However, all levels of mangers exercise the functions despite the degree of application.

Planning is the rational and orderly thinking about ways and means for the realization of certain goals (see Part-1 of this guideline). It involves thoughts and decisions pertaining to a future course of action. It anticipates and precedes action rather than making a reflective thinking about the past events. Absence of planning before doing implies rashness, in-prudence or short-sightedness in the performance of work. Before undertaking any work, it is worthwhile to ascertain *what* the work is, *how, when* and *where* the work is to be done, and *who* is to do the work. In considering these points, the managers have to clarify objectives and goals and to evolve policies and procedures for guiding those who do the work. They have to chart the proposed lines of actions with proper time schedules for the execution of the work. For providing a factual basis for future action, the mangers have to map out the program indicating the best course of action to be followed, fixing the targets and standards for work performance therein, and evolving the strategies and remedies for possible hindrance to the smooth flow of work. In other words programs provide a complete road map for the guidance of managers to get things done through operators.

Organizing provides the mechanism or apparatus for purposive, integrated and cooperative action by two or more persons with a view to implement any plan. With a few persons, organization calls for the allocation of tasks to individuals and integration of their efforts towards the common purpose. When the job expands and requires the efforts of many people, several departments or implementing crews come into existence under the charge of different managers who are tied together neatly by authority relationships for integrated action. That is, organization involves the division and subdivisions of construction activities into smaller sections and jobs as well as the integration of the activities and positions into a coordinated whole.

The concept of organization has two aspects: *technical aspect* pertaining to activities and *social or humanistic aspects* pertaining to people. For the personal contentment and social satisfaction of people, organization calls for matching of jobs with individuals and vice versa.

Staffing is the function of management which deals with appointment of the required number and quality of personnel to fulfill the responsibilities laid down by the organization structure. It lays down the process by which the persons are selected, trained, promoted and retired, keeping in mind the volume of work and the available funds. A right person on the right job should be the motto for a project not to be impaired due to lack of knowledge on that particular job.

Manpower planning involves an estimate of present and future requirements of staffs in the project. The recruitment of staffs or making ready of personnel for a particular construction site needs the knowledge of the work and the timeframe of each activity. Over staffing clearly affects the cost of the project, and similarly under staffing not only affect costs but also the schedule of the project completion time.

Leading or leadership may be defined as the process of influencing the activities of a group of individuals for achieving goals in a give situation. Effective direction is not possible by managers without the aid of leadership. In contrast to authority and power, managerial influence is exercised through persuasions, suggestions and advice for affecting the behavior of subordinates. In the case of influence, however, subordinates have the option to reject or to accept the propositions.

Some of the common nature and characteristics of leadership are:

- co-existence with fellowship;
- understanding feelings and problems of followers;
- assumptions of responsibility;
- objectivity in relations through fair play and absolute justice in all affairs;
- self-awareness on preference and weakness of leaders, and
- specific situations of the work.

Control is a fundamental managerial function that usually follows other functions. But like planning and other managerial functions, control is never ending activity on the part of managers. By forcing events to conform to plans, control becomes intimately connected with planning and has the same feature of unity, continuity, flexibility and pervasiveness as are to be found in the case of planning. Plans are not capable of self-actuating and do not lead to automatic accomplishment. With a view of to achieving organizational objectives and completing project plans, the managers have to regulate work assignments, to review work progress and to check operations falling under their jurisdiction.

Control is concerned with securing good individual performance and organizational performance. Good performance calls for identification of some desirable characteristics of such performance and the establishment of a level of par for each characteristic. This is effected through setting control standards which indicate expected results pertaining to all important aspects of any project activity.

The characteristics of control may be briefly stated as:

- last function of other managerial roles;
- exercised by all managers (unity);
- continuously ongoing;
- flexible;
- pervasiveness;
- forward looking, and
- worker focused.

CHAPTER-2 CONSTRUCTION PARTIES IN FLOOD PROTECTION WORKS

2-1 General

An individual cannot complete a construction work alone. The construction work requires a team of workers with their specific duties and responsibilities. The roles of each of the constituents of the construction team depend up on the size and type of the work and also method of its execution. Based on the duties, responsibilities and obligations of the team in a construction activity, the team is broadly classified into three categories called construction parties as: Employer (equivalently called Owner or Client), Consultant (commonly named as Engineer or sometimes as Architect) and Contractor.

Their relationships with respect to implementing the activity and among the parties are indicated in Fig 2-1 below. The detail relationship and assignments are as discussed in Secs. 2-2, 2-3 & 2-4.

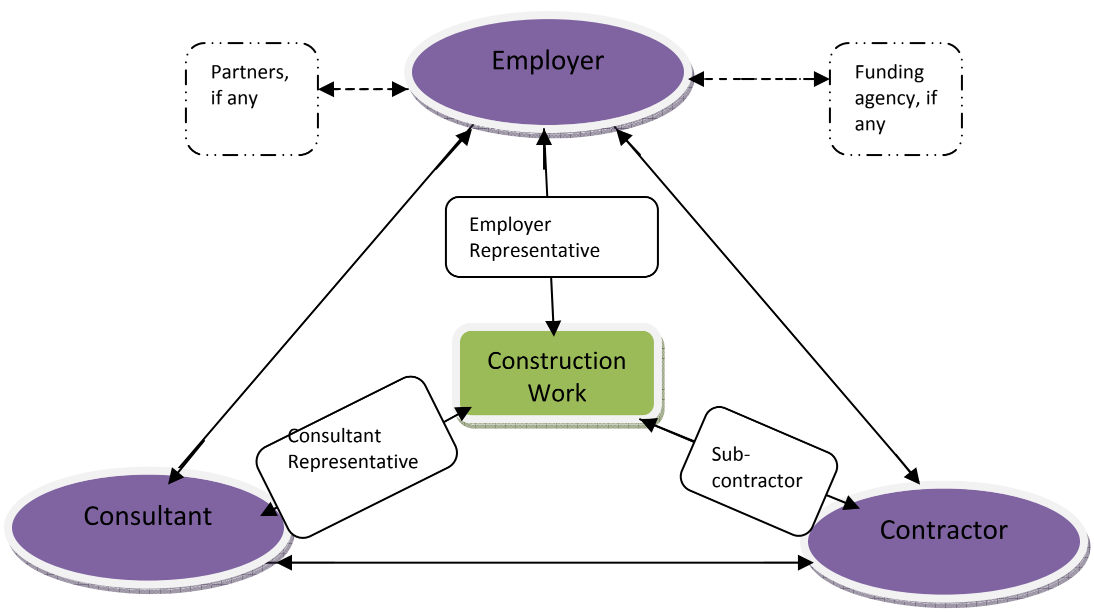


Fig. 2-1 Interactions between the Parties and Construction Activities

2-2 Employer

Employer may be an individual, group, public, government or private body which promotes the work or project and provides finance and facilities for the execution of the work. In most standard conditions of contracts the Employer is defined as "*the party named in the tender proposal who has called for tenders to build or construct or erect or deliver the works and who will employ the Contractor and the legal successors in title to the Employer but not (except with the consent of the Contractor) any assignee of the Employer*." The Employer has usually little or no technical knowledge or has little time in hand to handle and specify the work or has manpower shortage which is to the subject of the contract. The Employer therefore employs a Consultant for advice and to prepare the necessary designs, drawings, specifications, quantities, etc which are to form the basis of the contract. Although the Consultant is not legally a party to the contract, this party obviously occupies a very important position in between the Employer and the Contractor, and usually given a complete power to ensure that the Consultant's designs and contracts are faithfully followed.

The duties and liabilities of the Employer are mainly as follows:

- to appoint a Consultant and give power to work on the Employer's behalf. The Employer may hire also other staff in consultation of the Consultant to carry out the project work;
- to enter into contract with the Contractor by signing the contract document;
- to give possession of the site to the Contractor;
- to render necessary help for the Contractor for obtaining permission of electric, water, telephone, etc connections from other organizations;
- to safeguard the progress of the work from outside interference that are beyond the control of the Contractor;
- to make payment to the Contractor on submission of certified bills from the Consultant;
- not to interfere the progress of the work directly and make any addition or alteration without the consent of the Consultant;
- to take over possession of the completed work timely from the Contractor;
- in case of conflict with the Contractor, the Employer will appoint lawyer for defending the Employer's side, etc

In large flood protection works the Employer could be local administration, city governments, society, federal government or other donors to relief the grievances caused by flood disasters.

2-3 Consultant (or Engineer)

The Consultant or Engineer is a professional legal body and responsible for the safe and economic design of the work and implementation of the work under the supervision of the Consultant. The contract shall recognize that the Consultant is a juridical assigned body by the Employer to act on the behalf of the Employer.

The field of activities of the Consultant or Engineer includes designing, drawing preparation, bill of quantity preparations and prior cost estimations, approval of construction plans by the local authority, tender and contract document preparation, assist the Employer on contract evaluation and awards, supervise the supply of materials according to the specifications and drawings, control the quality of the work, measurement of the work done and approve payments, etc.

In addition to the above activities the main roles of the Consultant or Engineer are:

- professional adviser of the Employer;
- acts as an agent of the Employer;
- judge or arbitrator between the Employer and Contractor;
- to plan and supervise the work from the inception to the commissioning period.

It is important that the Consultant carrying out the wishes of the Employer does not in any way make oneself personally liable. Although the Consultant is paid by the Employer for the proper work and full value of the Employer money but on the other hand the Consultant shall judge fairly between the Employer and Contractor. It is most important that the Consultant shall give decisions without bias or prejudice and holds the balance fairly between the two parties. Under such condition it is unlikely that any decision of the Consultant will be upset by an arbitrator or by a Court of Law.

2-4 Contractor

A Contractor or builder means the person or persons firm or company whose tender has been accepted by the Employer and includes the Contractor’s personal representative, successors and permitted assignees.

The Contractor shall, with due care and diligence, design (to the extent provided for by the contract), execute and complete the works and remedy any defects therein in accordance the provisions of the contract. Depending upon the type of contract, the Contractor shall provide all superintendence, labor, materials, plant, Contractor’s equipment and all other things, whether of a temporary or permanent nature, required in and for such design, execution, completion and remedying of any defects, so far as the necessity for providing the same is specified in or is reasonably to be inferred from the contract.

The Contractor shall give prompt notice to the Consultant, with a copy to the Employer, of any error, omission, fault or other defects in the designs of or specifications for the works which the Contractor discovers when reviewing the contract or executing the works.

In addition to the aforementioned duties and obligations of the Contractor, some specific supplementary responsibilities and liabilities of the Contractor are:

- to inspect the site and study soil conditions before tendering. The Contractor should also investigate the accessibility, availability of electric power, water supply condition, local construction materials, and the local law and order conditions;
- shall collect local rates of materials, labor, etc;

- to go through the different clauses of the conditions of contract, specifications, drawings, plants, etc before submission of the tender;
- after acceptance and signing of the contract to mobilize and immediately start the work;
- to designate representative as Project Manager who shall be responsible pertaining the contract;
- to follow the labor, environmental, etc acts truly;
- to execute the work according to the contract and hand-over the completed works, etc.

In general the Consultant is the interpreter of the plans and specifications and as arbitrator of disputes between the Employer and the Contractor. The Consultant on the behalf of the Employer supervises the materials, workmanship, progress of work, etc. Therefore the Contractor shall satisfy the Consultant the work under contract. In case the Contractor has any grievances against the Consultant, the Contractor can approach the Employer as a high authority, but the disputes have to be settled according to the contract. When any alterations, additions and modifications during construction period of a project are required by the Employer, the Employer shall first consult the Consultant and the Consultant shall direct the Contractor. In case, the Consultant deems that certain modifications are required in the design the Consultant shall first take approval of the Employer and communicate with the Contractor for such changes. The Contractor is not supposed to approach directly to the Employer in connection with the work except when there is dispute with the Consultant.

In all cases and for smooth accomplishment of the works the Employer, Consultant and Contractor shall maintain a good relationship from the time of entering the contract till its completion. The Employer shall give timely payment to the Contractor through the approval of the Consultant. The Consultant shall always maintain fair and unbiased judgment for anything arises during the construction period and shall give also timely instructions to the Contractor during the execution of the work. The Contractor shall also accomplish with a professional discipline and complete the work timely to the full satisfaction of the Employer and Consultant.

2-5 Other Adjoin Associates for the Parties

In a construction projects besides the three principal parties there are adjoin contract bodies with considerable influence for the full and timely execution of the works. The adjoining parties may be attached or members or partners of the main parties as outlined below.

Funding agency is the body which gives the project budget as donation or loan. It is usually attached with the Employer (see Fig. 2-1 above). For instance, it can be World Bank, African Development Bank, International Monetary Fund, Some Foundations, Individuals, etc. The funding agency has influence and control on the cash flow through the Employer following the accomplished activities. The relationships with the Employer can be as partner or funding agency may or may not appear in the contract.

Consultant's (Engineer's) Representative is a person(s) appointed from time to time by the Consultant (Engineer) to assist in carrying out the duties and authorities of the Consultant. The Consultant shall notify the Employer and Contractor the names, duties and scope of authorities of such persons. The Engineer's representatives have different ranks and authority such as

Resident Engineer, Office Engineer, Supervisor, Quantity Surveyor, Surveyor, AutoCAD Expert, etc. The Engineer's personnel are actually staff of the Consultant and cannot be considered as adjoin parties.

Sub-contractor means any person or firm named in the contract as a Sub-contractor for a part of the works or any person or firm to whom a part of the works has been subcontracted with the consent of the Consultant and the legal successors in title to such person, but not any assignees.

In most contracts the Contractor shall not subcontract the whole of the works. Except where otherwise provided by the contract, the contractor shall not subcontract any part of part of the works without the prior consent of the Consultant and Employer. Any such consent shall not relief the Contractor from any liability or obligation under the contract and the Contractor shall be responsible for the acts, defaults and neglects of any subcontractor, subcontractor agents, servants, workmen.

Insurance is a legal contract that protects people from the financial costs that result from loss of life, loss of health, lawsuits, or property damage. Insurance provides a means for individuals and societies to cope with some of the risks faced in everyday life.

Therefore in construction the Contractor shall, without limiting the contractor or Employer obligations and responsibilities, insure, in the joint names of the Contractor and the Employer, against liabilities for death of or injury to any person or loss of or damage to any property arising out of the performance of the contract other than the exceptions. In the course of tendering, agreement or payments contractors always provide security in the form of bid bond, advance guarantee, performance bond, third party insurance, etc.

CHAPTER-3 CONSTRUCTION COMMENCEMENT

3-1 Construction Site Handover

The common time of commencement in most construction projects is 21-calender days after site hand-over, and sometimes when the project is urgent, and negotiating with the Contractor, the commencement time can be less as 14 or 7-calendar days. This commencement time is given to the Contractor for earlier preparations for site organization and mobilizations of equipment, materials and labors.

Site handover is therefore the transfer of the project site to the Contractor to commence the work according to the contract and schedules. During transferring the site, the Employer (assisted by the Consultant) has to make the site free of obstructions and any right-of-ways problems. Any invisible existing underground infrastructures shall also be indicated in the drawings for the Contractor’s care or shall be removed before the commencement of the works; otherwise it will impede the Contractor’s activities.

The Contractor shall commence the works on the site within the period named in the appendix to the tender after the receipt by the Contractor of a written order to this effect from the Consultant and shall proceed with the same with due expedition and without delay, except as may be expressed sanctioned or ordered by the Consultant, or wholly beyond the Contractor’s control.

In some projects, like flood protection works, the construction site could be large, say river training through city or farmland, the extent of portions of the site of which the Contractor to be given possession will be on accomplishment and submitted program bases. Otherwise if the whole of site is handed over to the Contractor once, there will be mess to the project because of interference of the work with residences and different infrastructures. Therefore the Contractor shall write notice to the Consultant from time to time as the work proceeds as per the agreed program. The Employer and the Consultant shall prepare the sites ahead and always be ready to hand over according to the notice and schedule of works. If the Contractor suffers delay or incurs costs from failure on the part of the Employer to give possession in accordance with the terms of the schedule and notices of the Contractor, the Consultant shall grant an extension time for the completion of the works and certified sum as, in the Consultant opinion, shall be fair to cover the cost incurred, which sum shall be paid by the Employer.

Construction site handover usually done with formal written document (see Fig. 3-1) and shall be kept as evidence for delays or early accomplishment. The standard format, almost can be used for any type of project, is presented below.

HANDOVER OF CONSTRUCTION SITE

This________ {*Insert Day*} date of ________{*Insert Month*} 20_____{*Insert Year*} the work for the construction of ____ {*Insert the project name and lot, if site handover is done progressively*} has been officially handed over to the ____{*Insert Name of the Construction Company*} where it is represented by:

Mr./Mrs. ____

WHEREAS the transfer of the site is done by the Employer's representative:

Mr./Mrs. ____ .

The Contractor therefore, acknowledges the taking over of the site and start immediately the work with all its explanation clearly defined in the specifications and drawings.

The Contractor, hereafter, shall be responsible for the damages that occur as a result of his/her fault, carelessness or negligence in connection with untimely commencement of the work.

The Contractor, in addition, acknowledges the completion of the said works shall count as from the date mentioned above and comply with the terms and conditions of the contract to the satisfaction of the Employer.

INWITNESS HERE OF THIS DOCUMENT HAD BEEN SIGNED BY ALL PRESENCE IN SIX COPIES OF WHICH THE EMPLOYER HAS ONE, THE SUPERVISOR ONE. THE CONTRACTOR ONE AND THE CONSULTANT THREE.

____________________________ ______________________________

(Employer Repr. name & signature) (Contractor Repr. name & signature)

Witnesses

1)______________________________

2)______________________________

3)______________________________

Fig. 3-1 Construction Site Handover Format

3-2 Site Organization

The construction site has to be organized by the Contractor in consultation and approval of the Consultant. Site organization includes:

- delineation of the net working areas and the required working spaces;
- allocation of the entrance and exit to the site;
- allocation of different positions of construction equipment and plants;
- allocation of different construction materials sites and stores with due regards to the construction equipment and plants positions;
- allocation of labor camps, labor hats, clinics, offices, guard houses, fuel tank, toilets, temporary works, etc.

A well planned and organized site reduces shifting and reshuffling of the different site allocations mentioned above thereby reducing maneuvering distances to the Contractor, minimum interferences during the work progress, optimal use of the land, efficiency in time utilization, etc. In general it is the advantage of the Contractor in reducing costs that could incur due to poor site organization.

In preparation of the job layout for the resources of construction, the planner has to have the following points in his/her mind:

- materials should be stored near the places of their utilization;
- machines and equipment should be placed at the most advantages and most suited position with regard to materials to be stored, accessibility by crane or other transport, permanency throughout the project time, etc;
- the site circulation be in proper order;
- accommodation for adequate storage areas, etc.

Therefore, after the site handover, the Contractor has to immediately submit proposed job layout plan showing the positions of the construction facilities for the approval of the Consultant and the Contractor precedes mobilization activities according to the general agreed plans.

3-3 Mobilization

Construction mobilization is the transfer of construction resources (materials, equipment and labors) to the specific project site.

Mobilization takes time and costs to the Contractor. In most cases mobilization has cost items to be filled by the Contractor.

Mobilization is effected according to the agreed site organization job layout mentioned above and also according to the different schedules mentioned in Sec.3.6 below. In mobilization of construction resources for flood protection works the following two core points have to be remembered:

- time of mobilization and
- progressive mobilization.

The season or time of mobilization is a very determinant factor for the commencement works. For flood protection works or even for any other construction types the dry season is the best construction time due to the following reasons:

- there is no or minimum influence from flooding;
- groundwater disturbances (dewatering) is a reduced amount;
- compaction can be done effectively;
- accessibility and use of dry weather road is possible;
- most lands are free from crops, etc.

The concept of progressive mobilization is the delivery of construction resources (materials, equipment & labor) at a progressive stage depending upon the need of the resource. Dumped all resources once at the beginning not only require capacity shortage to the executor but also it is inadvisable due to idle times for equipment and labor, shortage of space for construction materials, expiry and deterioration problems for construction materials (e.g., cement, etc), etc.

3-4 Construction Accidents and Safety Measures

In any construction site accidents are unavoidable occurrence. Starting from nail punch to loss of life or big injuries is a day to day phenomenon unless there are prudent safety facilities and regulations.

An accident may be defined as an event that happens all of a sudden unexpectedly. For example, a laborer falling from a ladder, cutting of arms while a worker working on a machine, falling of masonry blocks on workers head or foot, etc can be cited as an accident. The accident causes injury, deaths, damages to the equipment, loss of man hour/machine hours/production, etc and affects the life of individual and his family.

There are some special and unexpected accidents that could occur on a worker. There are various reported cases like dozer operators working in a forest bitted by snakes which cause to death. Wild bees may swarm these dozer operators through the ventilation and any openings of the machine and on involuntary escape trial by the operators the machine may chop the worker.

Accidents therefore are un-controllable if safety measures are not adopted. But the number and degree of the accidents can be reduced considerably if proper safety measures and regulations are adopted. From past statistics it can be inferred that accidents are caused due to unsafe acts and negligence and carelessness on the parts of the works, and absence of follow ups or provisions of safety facilities and regulations by the companies.

Safety shall be understood that precautions or cares taken by the workers before starting the work. To be safe from accidents one has to understand the causes of accidents. In general the following are the main causes of accidents:

- machinery accidents due to absence of safety devices, defective machines, insufficient working space, etc;
- poor working conditions due to lack of illumination, abnormal temperature, poor ventilation, unsafe work speed, long working hours, etc;
- use of improper tools such as blunt and worn out, undersized or oversized for the job, brittle, loose/without handle, etc;
- carelessness during using of materials such as being too hot, inflammable, poisonous, emitting foul gases, etc;
- poor working environment such as slippery ground, loose and exposed electrical cables, lack of discipline among workers, noisy and other disturbances, unhygienic conditions, etc;
- related to poor uniforms such as very loose dress, sleeves and shirts without button, slippery shoes, etc;
- some physiological causes such as poor eye-sight, over work, poor health, lack of experiences, old age, intoxication (drugs and alcohols), being handicapped, absenteeism (lacking safety information), etc;
- some psychological causes such as being worried, mental tension, emotional attitude, impulsiveness, nervousness, fear, over confidence, carelessness, frustration, etc.

The first and prior control of accident is the creation of safety rules and regulations at the construction site. The Contractor has to be abiding by the national labor law of workers on safety protection of jobs by provision of safety defense facilities and imposing safety regulations and follow ups throughout the project life; whereas the Consultant also has to enforce the Contractor on the safety of the workers and inspect whether the safety regulations are followed or not.

With regards to flood protection works the common safety measures and precaution areas are:

- care for the emergence of flush floods;
- care during excavation works and working on earthwork machineries (while shoring and timbering, collapse and slide of land mass, follow up of machines control units, oils, lubricants, etc);
- care during drilling and blasting (use of explosives, storage, transportation, loading, etc);
- care during use of ladders and scaffoldings;
- care during R.C and masonry work construction,

- care during site clearing and demolition activities, etc.

3-5 Environmental Protections

The Consultant and the Contractor have to be aware the environmental concerns and the necessary protections on the project site just starting from the beginning of the work. Therefore during the commencement of the work the Consultant has to give a verbal and written notice on the basic environmental issues and protection precautions to the Contractor.

The principal impacts and the corresponding mitigation measures during the construction stage are:

i. Waste discharges resulting from Contractor's camp:
The impact of waste discharges and refuse disposal can be limited by the imposing appropriate restrictions on the Contractor and having suitable clauses included in the Contract documents and a strict supervision.

ii. Noise and dust resulting from the construction activity:
Noise and dust are a short-term and localized problem. The impact of noise pollution can best be limited by the imposing maximum noise levels on the Contractor and by the restriction of working hours, particularly with respect to locally recognized days of rest.

iii. Spillages of lubricants, fuel, paint, other chemicals and scrap metal:
A waste management plan should be produced showing how waste oils, lubricants, other chemicals and scrap metal would be removed to a waste management facility where they could be recycled, incinerated, decontaminated and/or decomposed safely. If no such facility is available, the Contractor should propose and demonstrate an environmentally safe system of hazardous waste disposal.

The Contractor should account for the quantities of hazardous (potentially polluting) materials (explosives, acid, paint, and solvents) brought to the site. Either they are used (and seen to be used) or removed.

iv. Soil erosion due to digging of trenches and borrow materials:
- Appropriate erosion prevention structures have to be provided wherever necessary;
- Non-hazardous waste should be compacted and entrenched at an approved landfill sited over impermeable sub-soil, well away from a drainage course;
- Disturbed top soil should be preserved and restored and adequate precautions should be taken to prevent soil erosion.

v. Interruption to the existing water supply, electric, sewer, telephone, etc systems:
If the flood protection work is done I a city or dwelling places, some interruptions to the existing infrastructure systems are inevitable especially during excavation resulting unforeseen destruction or interruption. However, this could be minimized through proper planning of works before construction is started and creating awareness on the envisaged affected community beforehand, like announcement to inhabitants, temporary connections, etc.

vi. Loss of land and destruction of vegetables and trees for the construction of temporary houses, embankments, access roads, borrow sites, etc

There should be proper control from the Consultant side and due concern and awareness from the Contractor so that the environment shall not be disturbed unnecessarily. In the detail design and implementation phases, selection of routes of the access roads, and sites for constructing the camp facilities is assumed to take into account minimum environmental disturbance requirements. However, it may not be possible to completely avoid the impacts because of technical and other factors. It is, therefore, important to set up monitoring techniques at the time of construction to minimize disturbance of the environment.

The environmental concerns during the planning phase, long term impacts of flood protection works, quantifications of EIA, etc are discussed in detail in Part-9 of this guideline and the readers are advised for further reference.

3-6 Preparation of Construction Schedule

Construction scheduling is a planning process of setting the sequential order of various operations in a construction project by fixing the dates of starting and completion of each operation of the works in such a manner that the whole work should be done in an orderly and systematic way. The schedule should also indicates quantity of work involved, materials to be supplied, labor to be mobilized, equipment to be delivered to the site, money required and allotted time to each operation of the work. The actual progress can be recorded from time to time and can indicate if the work is lagging behind the schedule or going accordingly or ahead of the plan.

i. Preparation of construction schedules:

The entire project is divided into different operations or activities, and the sequence of the different activities to be performed is decided after knowing their-interrelationship according to the methods of construction adopted. The quantity of the work is computed from the drawings or bill of quantities of the project, rate of performance of each operation is estimated and duration of each operation is worked out. A suitable time of starting and finishing of each operation is decided from the worked out estimates and fair margin should be given for unexpected delays due to bad weather, lack of proper labor, unforeseen events, etc.

ii. Advantage of scheduling:

Scheduling is indispensible for construction work of any type. The following are some of the main advantages of a scheduled planning:

- all possible methods of construction can be examined at the planning stage so that most economical methods can be adopted;
- since the time of starting each activity is known, therefore, prior and adequate arrangement for provision of resources can be made;
- scheduling also indicates the quantities of the works duration of various operations for which plant or equipment can be arranged in due time to avoid unnecessary expenditure for keeping the plant idle time for dates on which it will not be required;

- it helps in arranging all types of labors, materials & money;
- It indicates the critical activities, total completion time and any float times;
- it can indicate whether the project is going according to the planned time or not;
- it is easy to modify or revise from existing schedule; etc.

iii. Resources and activities to be scheduled:

In construction activities resources or things to be scheduled are classified as:

- the work (construction) schedule;
- material schedule;
- labor schedule;
- equipment schedule' and
- financial flow schedule.

Work (construction) schedule is the master plan or roaster prepared showing the execution of the different activities of the project in the order of their accomplishment. In this schedule the work is sub-divided into many sub-heads or operations. The quantity work of each operation is calculated and time is estimated based on the performance of the method of execution used. Generally the amount of work, rate of completing the work making allowance for unforeseen circumstances, number of labors required, types of equipment needed, materials to be delivered and interrelationship of various operations are calculated to arrive to prepare the work schedule. Therefore work schedule governs the preparation of other schedules.

Material schedule is preparing plan for delivering and storing of materials in any form is known as materials schedule. It is a schedule which shows the dates of the deliverance of each type of materials. Material schedule is guided by the master or work schedule and strictly precedes the start of each activity operations.

Labor schedule is the plan and presentation of the labors required on a certain days or weeks or months of the project time. The types of labors required (skilled or unskilled) during the scheduled time depends also on the activities indicated on the work schedule. The labor schedule shows the number, type and durations of workers allotted for the activity.

Equipment schedule is similarly indicates the types and quantity of equipment required on particular dates in the entire project time. If equipment schedule is properly used it generally avoids machine idle times and it is helping tool for equipment rental arrangement and cost saving.

Financial schedule shows the flow money or cash into a project. Generally the Contractor always expects payment of the completed works at the end of each month.

ILLUSTRATION-1:

As a simple demonstration of the above types of schedules, a flood protection retaining wall is required to divert repeated flush floods coming to a city. The activities, according to the order of precedence, are provision of simple access road (about 12km); construction of simple labor hats, store and temporary office at the diversion site (three houses, each 40m^2); excavation on common soil for gully diverting and training masonry works (about 100m long stone masonry retaining wall & 900m long stone paved ditches); and construction of the masonry protection walls and lined ditches until it reaches the natural water course. Table 3-1 shows a sample demonstration on the breakdown of the activities, performance rate and anticipated time of completion.

Table 3-1 Illustrating Example for Schedules Preparation

S. no	Item of activity	Qty	Performance		Estimated completion time (days)			Remarks
			Unit	Rate	Qty/Rate	Contingency	Total	
1.	Temporary access road provision by dozer for 12km including ditches and crossing culverts	12km	km/dozer/day	1	12	2	14	
2.	Construction of stores, labor hat, office and toilets from simple corrugated iron sheets, by 2-crews	3 houses	house/per crew/week (a crew= 2-carpentors, 2-masons and 10-laborers)	1	10.5	3.5	14	
3.	Excavation for diversion masonry walls and then lined ditches using an excavator (1km).	3350m^3	m^3/day	250	13.4	0.6	14	
4.	Construction of stone masonry retaining wall for 100m length, by 10-crews.	300m^3	m^3/crew/day (a crew =2-masons, 3-helpers & 3-laborers)	2	15	6	21	
5.	Construction of stone lining to ditch 900m length, by 10-crews.	2700m^2	m^2/crew/day (a crew =2-masons, 3-helpers & 3-laborers)	20	13.5	7.5	21	
6.	Site clearing and demobilization	LS	-------	-------	-------	-------	14	

3-7 Methods of Planning and Scheduling

The most common methods of planning and scheduling of construction projects are

i. Using Charts

(a) Bar or Gantt Charts

(b) Milestone Charts

ii. Network Techniques

(c) Critical Path Method (CPM)

(d) Program Evaluation Review Technique (PERT)

(e) Line of Balance system (LBS)

Bar chart has been the conventional method of construction planning and scheduling. Henry Gantt developed this method around 1919 (J.L. Sharma, 1998). This method is sometimes called Gantt chart after the name of the inventor. In this method the various activities involved in a construction project are listed and the duration of execution of each of the activity is shown by a scaled horizontal bar graphically extending over the period of the activity. Fig. 3-2 shows a bar chart representation of work, equipment, labor and material schedules.

The main benefits of bar chart are

- it is very easy to draw and conceive;
- each item of work (activity) is shown separately;
- it is very easy to establish inter relationship between various activities; it is very easy to compare the progress and the original schedule, and
- it is very easy to modify the chart if required.

Some of the limitations of bar chart are

- interdependence of activities cannot be clearly shown while using the bar chart, i.e., parallel drawing of the activities doesn't indicate that they are related or completely independent;
- a bar chart doesn't show the uncertainty or tolerances in the estimated duration;
- the bar chart doesn't show the actual progress of work because the graph only presents the time elapsed in a particular activity;
- the critical activities cannot be shown by bar charts;

Fig. 3-2 Sample Schedule Planning Using Bar Chart

A) Work Schedule

Item No.	Project Activities	Quantity of work	Activity Duration (weeks)	Project Months															
				Month-1				Month-2				Month-3				Month-4			
				w-1	w-2	w-3	w-4	w-1	w-2	w-3	w-4	w-1	w-2	w-3	w-4	w-1	w-2	w-3	w-4
1	Mobilization	LS	1																
2	Temporary access road clearing and provision by dozer for 12km including ditches and crossing culverts	12km	2																
3	Construction of stores, labor hat, office and toilets from simple corrugated iron sheets, by 2-crews	3-houses	2																
4	Excavation for diversion masonry walls and then lined ditches using an excavator (1km).	3350m^3	2																
5	Construction of stone masonry retaining wall for 100m length, by 10-crews.	300m^3	3																
6	Construction of stone lining to ditch 900m length, by 10-crews.	2700m^2	2																
7	Site clearing and demobilization	LS	2																
	B) Material Schedule																		
1	Cement	700qt					110		200	200	190								
2	Sand	250m^3					40		70	70	70								
3	Gravel	40m^3					40												
4	Water	150m^3					10	10	30	30	30	30	7	3					
5	Stone masonry for houses, stone pitching and retaining wall	1100m^3					50		100	400	400	150							
6	Rafters and posts (10mm diameter)	810m					810												
7	Purlins (4x7cm)	900m					900												
8	Corrugated iron sheets	290pcs					290												
9	Timber posts for struts and falseworks (8-10mm diameter)	450m							200	200	50	50							
	C) Equipment Schedule																		
1	Dozers				1	1								1					
2	Excavators								1	1									
3	Mixers						2	2	2	2	2								
4	Plate compactors						2	2	2	2	2								
5	Small vehicles				1	1	2	2	2	2	2	1	1	1					
6	Water trucks						1	1	1	1	1	1	1	1					
	D) Manpower Schedule																		
1	Project Manager			1	1	1	1	1	1	1	1	1	1	1					
2	Construction Engineer			1	1	1	1	1	1	1	1	1	1	1					
3	Administrator					1	1	1	1	1	1	1	1	1					
4	Acountant						1	1	1	1	1	1	1	1					
5	Surveyor				1	1	1	1	1	1	1	1							
6	Quantity surveyor					1	1	1	1	1	1	1	1						
7	General foreman			1	1	1	1	1	1	1	1	1	1	1					
8	Operators				1	1	4	4	5	5	4			1					
9	Drivers				1	1	2	2	2	2	2	1	1	1					
10	Earthwork foremen				1	1			1	1				1					
11	Carpentery foremen						1	1	1	1	1	1	1	1					
12	Carpenters						4	4	1	1	1	1	1	1					
13	Masonry foremen						1	1	1	1	1	1							
14	Masons						4	4	20	40	40	20							
15	Store men								1	1	1	1	1						
16	Helpers						4	4	60	60	60	60							
17	Laborers						6	6	60	60	60	60							
18	Time keepers						1	1	1	1	1	1	1	1					
19	Guards						5	5	5	5	5	5	2	2					
20	Others						1	1	3	3	5	2	1						
	Total			3	7	9	40	40	168	188	188	160	14	13					

Milestone chart: There are some shortcomings of the bar carts. To avoid these inadequacies the bar charts have been modified by adding some new elements such as events with numbering and explanatory notes are provided as legend. These key events are known as milestones.

Critical Path Method (CPM): Large flood protection works and any huge civil works involving many activities and professions within the main project, the earlier bar charts are not sufficient to handle the planning and scheduling. Therefore CPM planning and scheduling is the application of network technique by avoiding the delay matters that could be created by the bar chart methods. Network technique means a diagram which shows the inter-relationship and interdependence of the activities and events of the whole construction project.

The *critical path* is the line of sequence of activities which must be completed on schedule in a network analysis of a construction project. Any delay on the critical path activity will definitely delays in other activity. The critical path always indicated with bold or unique color representing a series of activities on which each activity has zero float time. This path determines the minimum project completion time by summing the duration of all activities falling on the critical path. The other lines are non-critical lines and may have free float times.

The method of estimating the duration of each activities are similar with the bar chart methods, that is; estimation of quantity of work, setting of the performance of machineries and/or labor and then finally anticipate the time required to do the activity. This method is generally use deterministic approach where less emphasis is given in fixing the uncertainties.

There are numerous terminologies and concepts to be understood in using the CPM approach. For example, activity and event, dummy activity, duration, earliest start (ES) time, earliest finish (EF) time, latest start (LS) time, latest finish (LF) time, float times (free float, negative float, total float, independent float), slack time, etc nomenclatures have to be clearly understood to apply the method.

The procedures to be followed in using CPM method are:

- the project should be divided into various activities or operations;
- the duration of time required for completion of each activity is estimated;
- the sequence of performance of different activities is established;
- the network is drawn by interconnecting the various activities according to their order of execution;
- in drawing the diagrams arrows should be cared not to cross each other;
- assignment of numbers to events should be done, making sure that the number at the head of the arrow should be greater than the number at the tail;
- make forward pass and backward pass through network to establish the different ES, LS, EF, LF, etc times of activities;
- establish the critical path and fix the project completion time and

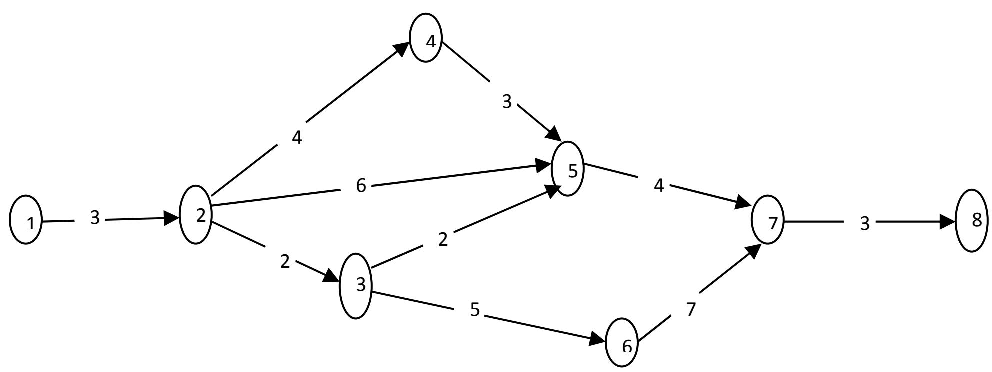
- carefully follow the activities during construction and immediately correct any lags.

Note also the following:

- an event cannot be completed unless or until all the activities leading to an event are completed;
- times flow from left to right;
- the length of arrow representing an activity does not show the time required for its completion. No information is provided by the length and shape of the arrows;
- tail events have lower number than head events;
- checking of activities locations by asking question like what? (i) has happened, (ii) is happening & (iii) will happen and check again by asking what? (i) had to be done, (ii) can be done now;
- check also the network errors of dangling, looping and redundancy.

ILLUSTRATION-2

A flood protection project has a network as shown in Fig.3-3(a) below. Determine the critical path and project completion time.

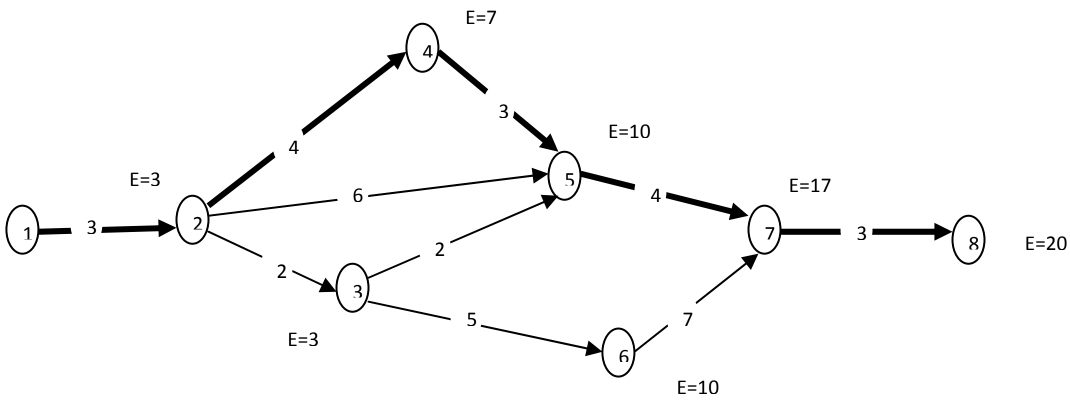


(a)

Solution: The event durations, E, are calculated and indicated at each node.

Critical path is the path 1-2-4-5-7-8.

Project completion time is 20unit time (see Fig.3-2(b) below).



(b)

Program Evaluation Review Technique (PERT): The PERT method is developed to overcome the shortcomings associated with planning projects for which duration of specific activities could not be estimated properly and reliably. PERT is like the common CPM and assumes the specific activities and their precedence relationships. PERT uses powerful mathematical tools, i.e., stochastic models of statistics and probability. PERT uses a fairly complex probability density functions known as (-) beta function or distribution which handles the variable data which are bounded or bracketed and exists only in certain ranges.

The bounded or bracketed variables are three times estimates for an activity. These are

i. best or optimistic time (O);

ii. the worst or pessimistic time (P)’ and

iii. the most likely or most probable time (M).

To find a single expected time the three time estimates can be compounded together as follows:

$$t_e = \frac{O + 4M + P}{6}$$

Where t_e = expected (average) time or duration;

O = optimistic time;

M = most probable or likely time' and

P = pessimistic time.

Except the method of determination of the duration time the remaining network construction is similar with CPM. However, when the project has long construction period, lots of uncertainties & possible bad circumstances (bad weather, material supply constraints, etc) the PERT method is preferable and most powerful for planning the schedules.

CHAPTER-4 CONSTRUCTION SUPERVISION

4-1 General

Supervision is one of the construction management activities conducted at a project site. The managers at this level are called supervisors and it represents the lowest level in the managerial hierarchy such as top level, middle level and lower level or supervisory management (refer Part-1: Project Planning). The supervision activities depending upon the complexity of the project undertakes all the managerial tasks such as quality management, time management, cost management, goal management, risk management, human resource management, communication management, procurement management and integration management.

Quality, time and cost managements are the three main corner angles of the supervisory management. Supervision activity is usually conducted by the Consultant and sometimes the Employer and Contractor also may assign site supervisors separately for better accomplishment of the works. To discharge the aforementioned construction supervisory management responsibilities by a supervisor, important sections are allotted for major work items encountered in flood protection works such as demolition, site clearance, earthworks, concrete works, masonry works, gabions, etc as discussed below.

4-2 Demolition and Site Clearance Works

Demolition: is a destruction and scrap removal activity of pre-existing or old or nonfunctional manmade structures from the new construction site or accesses to the project. Demolition can be done manually or by machine depending on the size of the structure to be destroyed.

With regard to flood protection works demolition is not a major activity if the works are located outside of cities. However, if river training works are done inside a city demolition can be encountered. For instance,

- Demolition of failed previous flood protection works impairing the new construction activities;
- Demolition of houses due to area requirements of the new works;
- Demolition and renovation of undersized or old culverts, bridges, retaining walls, etc;

The amount and type of demolition activity to be performed has to be specified in the bill of quantities and drawings, and the Contractor also has to visit the site prior to bid proposal submission. Unless it is very small amount, which can be considered in earthworks, demolition is a separate cost and payable to the Contractor as lump sum bases.

Some demolition activities need care and the scraps have to be transported to the specified place for reuse. Therefore the scraps are the property of the Employer and the Contractor has responsibilities to stockpile and protect from theft action or damage by the construction activities. Voids formed by the demolition shall be treated as indicated in the drawing and bill of quantities.

Site clearance: is the removal of the weak and organic top soils, bushes, hedges and trees and stumps. Site clearance can be done manually or machine depending up on the size of the work. During the site clearance activity maximum care must be exercised both by the Contractor and Consultant to protect natural vegetations, trees, wildlife habitats, etc.

In site clearing activities has to be specified in the bill of quantities (BoQ) and categorized based on the method of measurement and type of tasks as follows:

- organic and weak top soil removal in area stating the depth of clearing (20-30cm);
- bush clearing including grubbing in area;
- hedge removal in area, and
- cutting of trees including cutting, uprooting and raking, in number, stating the diameters of the trees. Trees' diameter less than 80mm are usually considered in site clearance by area.

Trees, bushes and hedges as much as possible must be protected, and if it is unavoidable the bushes, hedges, trees and plant roots are the properties of the Employer and must be stockpiled or burnt according to the construction agreement. Holes left by stumps and roots shall be filled with suitable material and compacted to the approval of the Consultant.

Termite hills shall be removed and disposed to appropriate tip. The cavity formed by the removal of termite hills shall be treated and filled with suitable material and compacted to approval of the Consultant. Termite hill works shall be measured by the volume of earth removed and disposed off.

4-3 Earthworks

Earthwork is the activities done on soils and/or rocks in digging and get out, stockpiling, spreading, watering, fill, compaction and disposal for surplus items.

Excavation means the digging and get out process. Excavation can be mentioned and classified by type of work done as described below.

Removal of top soil: top soil shall mean the top 20-30cm depth layer of soil containing more than 5% organic matter by weight (BaTCoDA, 1991). Removal of top soil is measured in area occupied the work to be placed on the cleared area. Therefore, removal of top soil shall be understood as including the clearing pushing aside of the working area, stockpiling in near distances and wheel spreading as directed by the Consultant.

Trench excavation: is an excavation length wise with limited width for masonry, pipes, strip footing, etc construction. It is measured in volume as the net void created by the excavation with deduction made for existing voids. For trench excavation structures, such as masonry work, having bedding shall be measured after allowing 25cm working space in both directions whereas if the trench is buried without in ground without bedding shall be measured without allowing working spaces. Quantification and preparation of BoQ shall be measured in successive stages of 150cm from the starting ground level.

Pit excavation: is a rectangular, circular or other shape vertical excavations made for foundation works. It is measured in volume by allowing 25cm in both directions at 150cm successive layers.

Bulk excavation: is large volume excavation for foundation or land leveling. The limits of bulk excavation for the measurement shall be shown in the drawing. Bulk excavation is measured in volume allowing working space specified in the drawing if such provisions are not provided a maximum of 50cm is allowed in both directions. There is no depth specification in bulk excavation.

Excavation classification: excavation can be termed as common soil, soft rock or hard rock. "*Common soil"* or "*ordinary soil" is* the ground material yielding to ordinary excavation machinery or pick axes. These are any type of soils, sand, gravel, highly weathered and decomposed rocks, etc.

"Soft rock" shall mean from highly to moderately weathered rocks which can be excavated using excavators, tractor drawn tines, bulldozers, etc for breaking up the materials.

"Hard rock" shall mean from slightly to intact mass rocks requiring the use of wedges, prismatic tools, blasting to enable the excavation work to be carried out.

"*Boulder*": shall mean isolated volume of hard rock in ordinary soil and soft rocks or above ground *greater* than 0.5m3 in volume

Embankment Fill: fill to excavations or to make up level as embankment shall be made from suitable materials and capable of being compacted. Fill shall be placed in successive stages in 20-30cm lift thickness, watered and compacted to the standard maximum dry density stated in the BoQ or specifications. Usually this standard is from 90% -95% of the laboratory Proctor compaction test results depending up on the quality of fill required. The moisture content of the fill material shall be adjusted as necessary to achieve the required compaction. Any material which after repeated compaction, does not fulfill the requirements, shall be removed and replaced.

The final fill levels shall be adjusted, graded and prepared to receive the subsequent fill or foundation. Fill shall be measured as equal to the net volume of voids to be filled and shall be understood as including the stockpiling and haulage of materials from location of fill.

Fill materials is normally classified as selected excavated fill and selected borrow fill. The selected excavated fills are the materials which are not expansive and excessively compressible arising from the excavation done before, Whereas selected borrowed materials are same suitable materials but imported from outside.

Quality assurance of fill: the quality assurances to be seen during the supervision of fills are

- fill material types;
- foreign matters in the fill (as lumps, roots, rocks, etc);
- moisture content of the fill materials;
- relative density of the compaction;

- finish levels and volumes, etc

Disposal: all unsuitable and surplus suitable materials arising from excavation are disposable item and shall be disposed off the site according to the BoQ. It is measured in net volume arising from the void created by the excavation less backfill and spread within the site. Disposal includes stockpiling, loading, transporting, dumping and wheel spreading at tip.

4-4 Concrete Works

Concrete: concrete is a construction material obtained by mixing cement, aggregates and water in a suitable proportions and then curing this plastic mixture to a hard mass. Concrete can be placed on site or precast type.

Concrete structures for flood protection works may be required for RC retaining walls, culverts, ditch lining, etc. Concrete works are done as mass concrete or reinforced concrete.

The main supervision activities in concrete works are as follows:

- type of cement to be used (Portland, Portland Pozolana, etc) as specified in the drawing, specifications and BoQ;
- storage conditions (with good ventilation and far from dampness);
- gravel quality (strong, durable, non-flakey and free from deleterious materials);
- sand quality (strong, sound, durable and free from deleterious matters);
- water quality (must be free from oils, acids, alkalis, organic and inorganic impurities, mud, vegetables, sulphate, sewage, soaps, etc);
- admixtures, if any;
- formwork placements and removal (formwork designs, straightness, tightness, dimensions, smoothness, temporary opening, spacers, etc);
- reinforcement bending and placement (size of bars, numbers, bending details, surface nature/rusting, etc);
- mixing, transporting and placement of concrete (mix proportion, mixing equipment, transportation facilities, dumping & segregation conditions, vibration, curing, etc);
- provision of concrete ancillary materials such as expansion joints, construction joints, water stops and any other embedded materials;
- formwork removal periods (for soffits, vertical members, props, etc);
- standard of concreting (class, finish type, joints, etc);
- quality of workmen (for concreting, formwork, at batching plant & bar benders); etc

Concrete and materials quality assurance tests: the supervision of concrete constructions works need quality conformation of the finished concrete products and the constituent materials.

For example,

Gravel: (i) gradation checking, (ii) abrasion tests, (iii) flakiness and elongation indices, etc

Sand: (i) gradation checking, (ii) soundness tests, (iii) bulking tests, etc

Water: (i) sulphate test, (ii) visual observations, etc

Reinforcement: (i) tensile strength, (ii) elongation index, etc

Concrete: (i) cube or cylindrical strength tests for 7-days, 21-days, and 28-days (ii) water absorption tests, (iii) non-destructive field tests

4-5 Stone Masonry Works

A stone masonry work is placing of coursed or rubble stones and mortar together to form hardened and monolithic construction structure. The stones to be used for masonry are obtained from quarries and the stones shall be hard, sound, free from vents, cracks, fissures, discoloration or other defects that will adversely the strength or appearance.

Some of the common types of rocks that are used for masonry works are ignimbrite, trachyte, granite, basalt, sandstone, slate, limestone, etc. In addition to the strength requirement the rocks shall have the quality to be quarried, shaped and dressed.

Stones for various masonry works shall be selected, shaped and placed as follows:

- stones for facing work shall generally be selected for consistency in grain color, and texture throughout the work;
- stones for below the ground work concealed from view shall be chiseled natural stone average size of 450mm;
- stones for rough dressed exposed faces shall be fair chiseled in average size of 450mm and individual not less than 380mm;
- stones to receive other finish shall be chiseled natural stone in average 450mm and individual not less than 380mm;
- stones for massive masonry works may not require strict sizes for the central parts, however, overlapping and interlocking with large size stones are required;

The mortar to be used for stone masonry shall be 1:3 or 1:4 and sometimes for low strength masonry 1:5 mix cement to sand proportion. The ingredient of mortar shall be measured in accurate gauge boxes for volume. Mortar shall be mixed in an approved mechanical batch mixer. Dry ingredients of mortar shall in the first instance, be mixed until there is a uniform consistency in color. Water shall be added and the mixing continued until there is a uniform distribution of the materials and the mass is uniform in color and consistency. In no case shall mixing be carried out for less than 2-minutes after adding water. Sufficient water shall be added to the mix to produce a pure consistency.

In instances where hand mixing is unavoidable, the cement content shall be increased by some amount. The dry and wet mixes shall be turned over sufficient number of time to produce the respective consistency as required by batch mixers. Mortar shall not be allowed to stand more than 1-hour without mixing. Cement mortar shall be used within half hour of adding cement to the mix. The mortar of stone masonry works shall be less than 20mm in thickness and shall be distributed to fill the cavities formed by joining stone.

In stone masonry works some ancillary works required are weep holes, drainage gravels, concrete foundations, anchors, etc.

Masonry works are measured in volume for the finished work including the mortar.

Quality assurance tests for stone masonry works: some of the common quality control works to be followed in stone masonry constructions are

- unconfined, triaxial or point load compressive strength tests for the rocks;
- compressive strength tests for mortar;
- quality tests for sand and cement, etc

4-6 Gabions

Gabions are wire meshed boxes filled with stones and tied together to form basic structures for flood protection works. Gabions are used as a substitute for concrete or masonry works. Gabion structure should be built with the same principles of good foundation, stability and quality control. The advantages of gabions are their simplicity of construction (requiring low levels of skills), ability to let moisture pass through avoiding the built up of water pressure and flexibility (should minor settlement occur) and low costs.

Their principal uses are

- retaining walls;
- drifts;
- erosion protection, etc.

Gabion boxes may be made from:

- purpose made gabion cages;
- welded steel mesh sheets;
- galvanized chain link fencing.

Some of the construction supervisions required in gabion construction are

- the preparation of the foundations,

- width height relationship provided in the drawings;
- diameter, strength and opening of the wire mesh;
- size of the stones; etc.

CHAPTER-5 PAYMENTS AND SITE COMMUNICATIONS

5-1 Payment Types

As explained in Part-1 of this guideline, one of the *Essential Elements* of contract is that both parties of the contract should get benefits to be a lawful agreement. Otherwise it cannot be enforceable at law and may be breached at certain period of the execution of the work. Therefore, the actual replacement of construction costs and profit payments are the gains of the agreement to the Contractor.

To effect payments the Contractor shall submit to the Consultant mostly after the end of each month (FIDIC, 1987) or any agreed arrangements, statement signed by the Contractor showing:

- take off sheet and priced BoQ;
- the values of temporary works;
- any separate material supplies;
- any agreed day work executed, and
- an amount reflecting any changes.

The types of payments that commonly endorsed to contractors are:

i. advance payment;

ii. monthly payment;

iii. payment on the certificate of completion, and

iv. payment after expiration of period of maintenance.

The advance payment is mainly meant to boost the capacity of the Contractor, and released just after signing of the contract and upon presentation of advance guarantee and disbursement schedules. The amount of advance payment depends whether the contract follows the set magnitude by the country or the specific agreement entered in the data sheet. This payment usually deducted from the monthly payment according to the withdrawn percentage of the project cost and must be finished mostly just at payment on the certificate of completion.

The second payment type is the monthly based imbursement to the Contractor. It is handled within less than 30-days of presentation to the Consultant which is the net interim release from the 100% values of executed works and materials on site up to the end of the previous month after deducting percentage of advance and agreed retention percent.

The third type of payment is released when the Consultant has granted a certificate of completion for the whole of the works. Certificate of completion is given when the whole of the works are substantially completed, functional and have satisfactorily passed any final tests prescribed by the agreement. The Consultant finally approve payment after balancing the values of the whole executed works with all previous payments and agreed percentage of final retention money. All previous payment errors are corrected in this stage of payment.

The last payment to be endorsed to a Contractor is up on the expiration of the period of maintenance, or, if different periods of maintenance shall become applicable to different sections or parts of the works, the expiration of the latest such period. The payment at this stage usually is the final retention money retained during the third payment.

5-2 Payment Processing

The procedure of payment processing is generally as follows. The Contractor submits the detail calculated payment certificates (for the case of advance and final account the calculation based on the agreed percentage) to the Consultant, then immediately the Consultant approves and forwards to the Employer after making any errors passed into the certificate and finally the Employer endorses the said sum of money after making communications and clarifications with the Consultant.

In payment processing it is unavoidable to encounter items having

- variations or;
- omissions, or
- additions.

The processing of these matters into the payment are usually trigger altercations among the construction parties. If the tender documents have loop holes and the agreement has ambiguity, some Contractors intentionally plan claims and make the construction project a battlefield. Most General Conditions of Contract (GCC) have brief description in handling these matters and also the different powers of the Consultant in resolving the problems.

5-3 Construction Reports

In construction projects report mostly is a written document submitted to other organization or within the same organization indicating the stage of certain activity or activities. The body to be reported for is informed for subsequent actions.

Construction reports to a project can flow in to two major directions. One direction of flow of reports is from the Contractor to Contractor's head office and the other route of flow is from the Consultant to the Employer. But sometimes in big construction parties and projects reports manifest to flow from Engineer's Representative to head office then from head office to the Employer and/or donors. Furthermore there are also report correspondences from the Contractor to the Consultant or vise versa.

Construction site reports (for both routes) mainly categorized into three types: progress reports, completion report and circumstantial reports. The details of the types are as discussed below.

Progress Report: it is a time based tales and information transfers to a higher body about the stages of the work and about important events encountered in the project. The progress may be submitted weekly bases, monthly bases, quarterly, etc; however, the longer timeframe reports encompass the smaller timeframe earlier reports. The main ideas and accomplishments to be reported, together or separately, include:

- introduction about the project general salient features and the agreement for general readers;
- percent of the work executed in terms of physical and/or monetary expressions;
- payment endorsed against executed work;
- time elapsed and evaluation with the schedule;
- quality controls conducted and their evaluations (including for sand, gravel, water, cement, concrete, masonry works, reinforcements, embankment & foundation compactions, etc);
- existing manpower, equipment, materials, cash, etc status;
- weather conditions and any other interferences;
- instructions given (variations, suspensions, design changes, addition and omission, material changes, rectifications and modifications, etc);
- any claims and resolutions;
- meeting conducted and conclusions reached;
- and any concerns of the project.

Completion report: it is a reported delivered immediately after the provisional or temporary acceptance of the work. A construction project completion report declares and encompasses:

- introduction about the project general salient features and the agreement for general readers;
- important project date benchmarks (commencement & completion);
- parties attending the completion handover process;
- minutes of meeting of the attendants;
- account status (previous payments, penalty, rebates and net payments);
- as built drawings,
- comments made by the attending parties, and many more information.

Circumstantial report: This report is occurs occasionally to solve incidental problems happened on a project site or investigations to be conducted on the site by a team for strengthening the construction process.

For all cases of reports types the writer has to bear in mind the following points:

- to establish good report format (for the progress & completion report types) or adopt standard reporting formats from renown projects;
- to include report no., date, table of contents, conclusions and strong recommendations, reporter name & signature, etc on the document;
- to beautify the cover of the report by photos from the project or related designs;
- the reporting, as much as possible, must be short and precise. Flow of ideas must be continuous; tables, charts and figures must be mentioned in the body of the text and captioned on the spot, etc.

5-4 Site Correspondences

Site correspondences are interchange of information, instructions and work orders between the Contractor Project Manager or delegated personnel and the Resident Engineer or delegated Engineer Representative Supervisor.

Site correspondences are multi-various in nature and some of the communications are

- documenting the daily diaries;
- methodology submittals by the Contractor;
- notices and letters correspondence between the Contractor and Engineer Representative;
- written and oral work orders;
- discussions and meetings;
- quality control field and laboratory tests and evaluations;
- certifications of work items;
- modification designs and drawings; etc

It is always a good habit to have a written, dated and signed site communications because of the possibility of to forget events or being prudent for the case of disagreements. For both parties the correspondence documents must be filed in a separate folder sequentially. The Consultant or the Contractor site office should have the following documentation facilities for good communications and keeping documentations for future references:

- shelves where the different folders can be put (for instance folders for meetings, correspondence letters, work orders, field and laboratory tests, payments, progress reports, site inspection certificates, methodology, etc);
- site photo albums;
- computers;
- fax, telephone & internet connections;
- boards for schedules and site photo presentations, etc.

CHAPTER-6 PROJECT COMPLETION AND COMMISSIONING

6-1 General

When the whole of the works have been substantially completed and have satisfactorily passed any final test that may be prescribed by the contract, therefore the Contractor may give a notice to that effect to the Consultant or Consultant's Representative accompanied by an undertaking to finish any outstanding work during the period of maintenance.

At this circumstance the Consultant prudently check, although the Consultant's Representative is always at site except a period supervision type, all the conditions and provisions of the contract and the full functionality of the completed works. The Contractor personnels and the Consultant Representatives jointly verify the completed works item by item and component by component for the whole work and if the said completed works have corrections the Contractor immediately rectify, otherwise, the Consultant immediately write a letter to the Employer for joint acceptance process.

6-2 Site Clearing Ups

It is understood that during the progress of the works, the Contractor shall always keep the site reasonably free from all unnecessary obstructions and shall store or dispose of any construction plant and surplus materials and clear away and remove from the site any wreckage, rubbish or temporary works no longer required. Moreover, inspection shall be carried out daily to ensure that sufficient workmen, tools and facilities are provided to maintain the standard of hygiene.

However, before arrangement of provisional acceptance process involving the Employer, the Contractor (inspected by the Consultant) shall clear all unwanted leftovers and create normal operation conditions by removing

- plants, equipment, tools, etc;
- temporary works;
- rubbishes, surplus earth, stones, sand, gravel, containers, etc;
- stores, Contractor/Consultant offices (if unwanted by the Employer or not in agreement);
- remove or fill temporary toilets, fill and level any depressions created by the Contractor, etc

After cleaning and restoration of the construction site (if it in the agreement) by landscaping and planting trees, grasses, platforms, etc the completed project will be ready for services.

6-3 Completion Acceptances

Project completion acceptances have two stages. The first one is mostly called provisional acceptance or temporary acceptance and the second one is called final acceptance. The former acceptance type is the one which is done immediately upon satisfactory completion of the project and after the site clearing aforementioned above, whereas the final acceptance is the one which is made after maintenance and defect liability period.

There are standard project completion handover formats which depend up on the type of the acceptance. Matters to be checked and written on each acceptance type are different because the concerns are at a dissimilar timeframe. Any format can be followed but the check lists to be written under the two acceptance forms are presented below.

Matters to be checked and issues to be written on provisional acceptance form are

- project name and acceptance type as heading;
- general project salient features next to heading (construction parties, costs as main contract/supplementary/variations, etc);
- members attending the provisional acceptance representing the construction parties;
- things examined by the construction parties (say finished works, contract, drawings, specifications, etc);
- timeframes (commencement, site possession, completion, extension times based on justified delays, delay and penalty);
- financial accounts (executed work values, previous payments, rebates, penalty, retention and net payment);
- other issues like performance bonds, manufacturers guarantees, etc
- comments by the provisional acceptance members, and
- conclusions.

The contract is not considered as complete until a final acceptance form (maintenance certificate) has been signed by the construction parties. This shall be started before the expiry of the maintenance and defect liability period.

The final acceptance process can be initiated by any of the construction parties before the expiration of maintenance and defect liability period. If the serving project is in good condition, the Contractor may initiate the final acceptance process to collect the retention money whereas if there are new problems impairing the service including comments mentioned during the provisional acceptance process the Employer through the Consultant may initiate after examining the maintenance to be accomplished listing defects to be rectified.

Matters to be checked and issues to be written on final acceptance form are

- project name and acceptance type as heading;
- general project salient features next to heading (construction parties, costs as main contract/supplementary/variations, etc);
- members attending the provisional acceptance representing the construction parties;

- things examined by the construction parties (say remarks from provisional acceptance, rectified works, contract, drawings, specifications, etc);
- time frames (date of final acceptance and proposed date of final acceptance);
- final examination of the works and general remarks (any remaining defects, maintenance done, new orders, etc)
- final financial accounts;
- act of renouncement by the contractor for full and final cost balance, and
- conclusions.

6-4 Commissioning

After completion of the flood protection work and provisional acceptance process the project is handed over to the promoter for its operation. The initial start and demonstration process shown to the Employer, for the proper functioning of the completed, is termed commissioning.

In flood protection works there is no as such commissioning procedures because mostly the construction work is done in dry season, and whereas flooding is a natural process that occurs stochastically at unexpected certain day. However, some possible testing action that could be done for flood protection works is the observation of the action of the rejoined river, if any, that was diverted during the construction period. The observations are inspecting the smooth flow, possibility of scouring, forming ponds, etc for the reinstating river.

This page is intentionally left blank

FLOOD PROTECTION DESIGN MANUAL
PART 10: INSPECTION AND MAINTENACE

Table of Contents

CONTENTS OF OTHER PARTS

1. INTRODUCTION

1.1 General

The objective of the manual is to bring a measure of consistency to the design of flood protection structures in Ethiopia to ensure that minimum standards of the design are achieved.

This part of the Design Manual will enable engineers and operators to better understand what is required to maintain a flood protection project and the commitment necessary to provide flood protection to the project in concern. It provides the user with the tools to meet standard requirements for inspection and maintenance of flood protection projects.

Standard dyke management includes a range of activities aimed at ensuring flood protection works are repaired and well maintained, and that advance preparations are made to protect safety of infrastructures during flood events.

1.2 Purpose of the Flood Protection Project Maintenance

A regular maintenance program of flood protection work is the key to ensuring the performance and service life of flood protection project. Keeping the flood protection structures in as-built condition will provide continued flood protection benefits to the community, development projects and to the investments. The operation and maintenance program must include inspection and maintenance plan.

The purpose of this manual is therefore, to provide general instructions, methods, techniques and data pertinent to the inspection, operation and maintenance of the flood protection works on specific project area.

1.3 Goal and Objective

The inspection and maintenance goal of flood protection works is to develop, implement and manage an effective maintenance program for the project that will continue to provide flood protection to the project in concern. Maintenance guideline presented in this manual will ensure proper performance throughout the successful life of the project.

The task for operation and maintenance team of flood protection project is to keep the flood protection project in as built condition with an acceptable condition rating through a program of regular maintenance and minor repairs. The project should be ready to perform during the next high water event.

Therefore, the operation and maintenance guidelines presented in this manual will enable you any responsible authorities to better understand what is required to maintain a flood protection project and the commitment necessary to provide flood protection to the project in concern.

2. INSPECTION OF FLOOD PROTECTION WORKS

2.1 General

Periodic inspection of dykes is extremely important. Responsible authorities should be advised to make a thorough visual inspection of the dykes at least twice a year, before and after the flood season. A closer inspection of the embankment surface can usually be made during the winter months when the vegetative cover is dormant and during the summer immediately after mowing. Items to look for during inspection have been emphasized in this Chapter.

Included in this section are suggested forms to record inspection observations. Operation, maintenance, rainfall and flood level records are covered. The inspection forms have been set up with common problems identified for each area inspected.

Four forms are provided on which to record inspection observations: one for embankments, dykes, or levees: one for spillways and drains or outlet works; one for miscellaneous items such as the watershed, lake shoreline, downstream area, monitoring devices, etc.; and a separate form for concrete dams. General instructions are provided on the back of each sheet. The fifth form is for recording operation, maintenance, rainfall, and pool level events.

2.2 Annual Inspection

At least once a year the entire dyke system should be inspected in detail by the responsible authority for need of routine maintenance. This inspection should be scheduled prior to the high flow season, and early enough to allow adequate time for any required work to be completed prior to possible flood events.

The inspection should note and record any conditions that might affect the performance or operation of the flood protection system including: access, obstructions to the dyke and along the dyke crest including the extent of vegetation growth and any trees that affect visibility of the slopes or that may weaken the dyke or bank protection; low spots along the dyke crest; animal burrows; sloughing, cracking or damage to the slopes of dyke and bank protection; signs of erosion of the riverbank or damage to the existing bank protection; particular attention should be given to:

- ✓ loss or disturbance of rock from the existing protective layer;
- ✓ weathering or abrasion of rock particles;
- ✓ slumping of the slope and scour at the toe of the slope;
- ✓ erosion or scour of the riverbank upstream, and downstream;
- ✓ erosion or significant changes to overbank areas.
- ✓ structural and functional condition of pumps, pump stations and seepage control works including debris, sedimentation or other problems at intakes, outlets and trash racks;
- ✓ the condition of flood box flap gates and their ability to open and close freely providing a watertight seal;
- ✓ condition of scour or erosion around bridges and similar structures;
- ✓ damage to, reference level, and visibility of water level gauges and monitoring devices; and
- ✓ Unauthorized excavation or construction in, on or adjacent to the dyke.

A written report on the results of the annual inspection should be prepared and documented with in the archive of the responsible authority. Photos of damage or problems may be included. Arrangements must be made by the authority for additional investigation, design and approvals as required ensuring necessary work is completed prior to the next high water period

2.3 High Water Patrol Inspection

Patrol inspections should be carried out during floods and high water events to monitor the performance of the flood control works and take corrective action as required. Dyke Patrol Log has been shown at the end of this chapter which is indicative what to be done during patrol inspection.

2.4 Construction Controls

The responsible authority for flood protection dyke management and maintenance should control development or construction on, through or in the vicinity of the flood protection works, so that such activity does not reduce the existing standard of flood protection. Activity outside the dyke right-of-way should be in accordance with accepted floodplain management practice as established by the

2.4.1 Excavation Controls

Excavation adjacent to dykes, bank protection or other flood protection structures should be discouraged, but where such excavation is necessary, advice from the responsible authority should be obtained to ensure that the excavation does not destabilize the flood protection works.

2.4.2 Pipes Controls

Where pipes, cables or other works must pass through or along the dykes, the correct use of seepage collars and compacted backfill materials is mandatory. Rupture resistant pipe, with mechanical or equivalent joints which will not separate under settlement, shall be used where pipe is laid within the design dyke section. Material excavated to install the works should be replaced with backfill material of equivalent flood resistant quality and in a manner that will not reduce the standard of protection.

2.4.3 Encroachment Controls

Trees or tall shrubs should not be allowed to encroach on the dyke. Buildings or other obstructions should not be allowed within the right-of-way or situated in a position that would impede dyke maintenance work or the functioning of designated floodway corridors.

2.5 Post Flood Inspections and Evaluation

An inspection of the protective works should be undertaken after flood events, and particularly when damages occur to works, after water levels have recede. There may be a need for professional evaluation, design and correction of major damages. Efforts should be made to locate and record a complete high water profile along the dyke after significant flow events. This information should be utilized to assess the dyke crest level and freeboard.

2.6 Special Inspections

Special inspections may be needed at other times of the year to monitor and react to particular situations such as storms, earthquakes, stream channel sedimentation, and debris accumulations.

Flood Protection Works Management Checklist

No:	Rev:
By:	Date:
Check By:	Date:

Project: ____________________

Authority: ____________________

O & M Supervisor: ____________________

Tel: ____________ Fax: ____________

Dyke Reach (Name): ____________________

O & M Manual ☐ Yes ☐ No

1. Records:

   Design Reports: ____________________

   Plans: ____________________

   Drawings: ____________________

   Files No.: ____________________

   Flood Plain Maps: ____________________

2. Rights-of-ways: ☐ Yes ☐ No

3. Annual Budget: ☐ Yes ☐ No

4. Follow-up Last Inspection

   By: ____________________

5. Annual Inspection Report: ☐ Yes ☐ No

6. Approvals/Changes: ☐ Yes ☐ No

7. Maintenance:	Required	Ok
Access:	☐	☐
Gates:	☐	☐
Dyke Crest:	☐	☐
Damges/Repairs:	☐	☐
Dyke	☐	☐
Bank Protection	☐	☐
Drain Structures	☐	☐
Pumps	☐	☐
Drainage	☐	☐
Vegitation Control	☐	☐
Animal activity	☐	☐
Debris	☐	☐
7. Flood Response Plan:		
Flow Forcasting:	☐	☐
Contact list	☐	☐
Materials	☐	☐
Equipment	☐	☐
Comunications	☐	☐
Flood Patrol	☐	☐
Warning	☐	☐
Evacuation Plan	☐	☐

8. Follow-up Action Complete: ☐ Yes ☐ No

Date: ____________________

Signed: ____________________

Rev 1

Flood Protection Inspection Report

Project: ______	No:	Rev:
Authority ______	By:	Date:
	Check By:	Date:

Dyke Name: ______

Dyke Length: ______

Reach: ______

Date Inspected: ______ Signed: ______

The condition of the flood protection works is as reported below:

1. Dykes: (Access,gates, vegitatioon growth, gravel surface, height, slopes, erossion, animal burrows seepage, trash, berms)

______

______

2. Bank Protection: (Loss of rock, gabions, settlement, slumping)

______

Page 1

______

3. FloodBoxes/ Pump Stations: (Inlet and outlet channels, gate operation, trash racks, debris, erosion, corrosion, structures, discharge structures)

______

______

4. Work Required:

______

______

5. Additional Information (Attachements, sketch, photos, etc.)

______

______

6. Work Completed: Date : ______ Signed: ______

to be Submited to the Project Manager/Basin Authority

Rev 1

Flood Protection Works Management Checklist

No: Rev:
By: Date:
Check By: Date:

Project: ____________________
Authority: ____________________

O & M Supervisor: ____________________

Tel: ____________ Fax: ____________

Dyke Reach (Name): ____________________

O & M Manual ☐ Yes ☐ No

1. Records:

Design Reports: ____________________

Plans: ____________________

Drawings: ____________________

Files No.: ____________________

Flood Plain Maps: ____________________

2. Rights-of-ways: ☐ Yes ☐ No

3. Annual Budget: ☐ Yes ☐ No

4. Follow-up Last Inspection

By: ____________________

5. Annual Inspection Report: ☐ Yes ☐ No

6. Approvals/Changes: ☐ Yes ☐ No

Rev 1

7. Maintenance:	Required	Ok
Access:	☐	☐
Gates:	☐	☐
Dyke Crest:	☐	☐
Damges/Repairs:	☐	☐
Dyke	☐	☐
Bank Protection	☐	☐
Drain Structures	☐	☐
Pumps	☐	☐
Drainage	☐	☐
Vegitation Control	☐	☐
Animal activity	☐	☐
Debris	☐	☐
7. Flood Response Plan:		
Flow Forcasting:	☐	☐
Contact list	☐	☐
Materials	☐	☐
Equipment	☐	☐
Comunications	☐	☐
Flood Patrol	☐	☐
Warning	☐	☐
Evacuation Plan	☐	☐

8. Follow-up Action Complete: ☐ Yes ☐ No

Date: ____________________

Signed: ____________________

Dyke Patrol Log

No:		Rev:	
DES By:		Date:	
Check By:		Date:	

Project: ______________________________

Authority ______________________________

Date: ______________________ Inspector: ______________________

Time Comenced: ______________________ Time Completed: ______________________

1. Gauge Height	**Design WL**	**Time**	**Water Level**
Gauge________	________	________	________
Gauge________	________	________	________

2. Land Side Seepage			**Comments/Location**
Boils	☐ Yes	☐ No	Clear: Dirty: Piping
Ponding	☐ Yes	☐ No	________
3. Land Side Slope			
Cracking	☐ Yes	☐ No	________
Sloughing	☐ Yes	☐ No	________
Seepage	☐ Yes	☐ No	________
4. Dyke Crest			
Accessable	☐ Yes	☐ No	________
Cracking	☐ Yes	☐ No	________
Settlement	☐ Yes	☐ No	________
Freeboard	☐ Yes	☐ No	________
5. River Side Slope			
Erosion	☐ Yes	☐ No	Dyke Fill: ________ Riprap: ________
Instability	☐ Yes	☐ No	________
6. Floodboxes/Structures			
Gates	☐ Open	☐ Closed	Leakage: ________ Flow Estimate: ________
Pumps			
Inlet/Outlet			Open: ________ Obstructed: ________
Operating	☐ Yes	☐ No	Flow Estimate: ________
7. Others:			
Debris	☐ Yes	☐ No	________
Sediment Deposition	☐ Yes	☐ No	________

8. Required Action: ______________________________

Notification : ________________ To Whom: ________________ Time: ________________

Rev 1

3. MAINTENANCE OF FLOOD PROTECTION DYKES

3.1 Dykes/Flood Embankments

Dykes are a common feature of many flood protection projects and are designed to protect against flood flows. They are constructed of well-compacted, relatively impervious soil to minimize seepage. The structural integrity of dykes must be maintained at all times. Proper maintenance of dykes insures their overall condition and provides the level of flood protection for which the project was designed. Not only should the eroded areas be repaired, but also the cause of the erosion should be addressed to prevent a continuing maintenance problem.

Erosion might be aggravated by improper drainage, settlement, pedestrian traffic, animal burrows, or other forces. The cause of the erosion will have a direct bearing on the type of repair needed. This section therefore discusses causes of failures and repair guidance for the type of failures. During the inspection of the flood protection project, special attention should be given to the following maintenance items:

3.1.1 Paths from Pedestrian Traffic

Paths from pedestrian traffic are problems common to many embankments. If a path has become established, vegetation in this area will not provide adequate protection and more durable cover will be required, unless the traffic is eliminated. Small stones, asphalt, or concrete have been used effectively to cover footpaths. Treated wood beams into an embankment slope to form steps is one of the most successful and inexpensive methods used to provide a protected pathway

3.1.2 Depressions

- ✓ Dykes should be maintained to the shape and grade as constructed.
- ✓ All low areas found on the dyke slopes or on top of the dyke that ponds water, should be filled and brought to grade with impervious soil.
- ✓ Topsoil and sod should be removed and the existing dyke surface should be roughened before impervious type soil is added in approximately six inch layers, then compacted by hand or mechanical method and brought back to its original shape.
- ✓ The area should then have the topsoil replaced, seeded and mulched.

3.1.3 Erosion

Erosion prevention measures shall be used during and after construction preparation and then provide adequate water courses.

- ✓ All erosion gullies should be brought back to the original grades. The ground should be roughened, then backfilled with impervious soil.
- ✓ Backfill material should be thoroughly compacted by hand or mechanical methods, and then placed in 15 cm layers to the original alignment and elevation.
- ✓ The area should then be covered with the appropriate size rock.
- ✓ Soil added to restore an embankment should be properly "keyed" into the base material. This can best be accomplished by removing

Vegetative cover and all unsuitable material until a good, firm base in undisturbed soil is uncovered. Unsuitable materials include wet, soft, porous, organic, and improperly compacted soils. The surface should then be roughened with a disk or similar device to obtain a good bond between "old" and "new" materials. The new soil should be successively placed in thin layers (15 to 20 cm thick) and each layer compacted before adding more material.

3.1.4 Slope Stability and Cracking

There are two types of cracks: transverse and longitudinal.

a) Transverse cracks appear across the embankment and indicate differential settlement within the embankment. Such cracks provide avenues for seepage water and piping could develop quickly.
b) Longitudinal cracks run parallel to the embankment and may signal the early stages of a slide or slump on either face of the embankment. In recently built structures, these cracks may indicate inadequate compaction of the embankment during construction.
   - ✓ Small cracks, as they appear, should be documented, examined by an engineer, and then sealed. The seal will prevent surface water from entering the cracks, causing saturation of embankment material problem. Sealing can be accomplished by compacting clay in the cracks. Unless the cracks are large (wider than 25mm), this can usually be done in few minutes using a shovel and a compacting tool. After the cracks have been sealed, the areas should be monitored frequently to determine if movement is still occurring. Slides or crack locations can be documented by staking and/or with photographs. Continued movement is an indication of a more serious problem such as a slide.

3.1.5 Slope Stability and Sliding

- ✓ Slides and slumps are serious threats to the safety of an embankment. Slides can be detected easily unless obscured by tall vegetation. Arc-shaped cracks are indications that a slide or slump is beginning. These cracks soon develop into a large scarp in the slope at the top of the slide.
- ✓ Any deep-seated sliding, a movement of a portion of the dyke down the slope towards the toe, requires removal and replacement of that section of the dyke or stabilization of the dyke toe with a soil or rock berm.
- ✓ Shallow surface slides can be repaired by removing the slide material and rebuilding the slope to original grade with well compacted previously excavated material. The cause for any slide should be fully determined before implementing permanent repairs to the slope

3.1.6 Seepage or Saturated Areas

The location of the seepage or wet area on the embankment or abutment is often a primary concern. Excessive seepage pressure or soil saturation can threaten the stability of the downstream slope of an embankment.

- ✓ When soil on the land side toe of the dyke becomes wet and spongy during high water, seepage or saturated areas will develop. This condition must be checked often to determine if seepage flow is increasing or muddy.
- ✓ If the seepage is carrying soil or if longitudinal cracks appear, a movement of a portion of the dyke down the slope towards the toe may occur. Such a condition may cause a dyke breach. Also, seepage through the dyke or its foundation may cause pond of water to occur.
- ✓ When the water is up on the water side of the dyke, and a boil or water begins seeping from the tow on the land side, a ring embankment should be constructed around the boil with sandbags. The embankment should be raised until the water stops rising in the ring dyke. A supply of sandbags for this effort should be maintained.

3.1.7 Animal Burrows

- ✓ Measures should be taken to remove burrowing animals from the dyke.
- ✓ Upon removal of animals, the burrows should be excavated and backfilled with a cementitious backfill material.

3.1.8 Unwanted Growth

- ✓ Trees, brush cover and woody vegetation cannot be allowed to grow on the dyke.
- ✓ Care must be used to not denude the entire dyke of the protective grass cover.

Figure 3-1: Unwanted Prosopies Growth on the Dyke Prevented Access at Amibara Awash River (Source: Consultant Library 2007)

3.2 Slope Protection Works (Riprap)

Slope protection is provided on channel side slopes, dyke side slopes or other areas where stream velocities are expected to cause loss of embankment (scour) and erosion of soil and grass cover. Slope protection can be provided by several methods including concrete slope paving, concrete injected mattresses or stone riprap protection (stone placed adjacent to each other to form a protective cover).

Regardless of the method used, its purpose is to protect against scour and erosion that could lead to slope failure.

3.2.1 Stone Riprap Protection

- ✓ Riprap should be kept free of woody growth, voids, displacement and deterioration. The most effective way to control brush and woody growth is through the periodic application of a chemical herbicide applied by a State Certified Pesticide Applicator.
- ✓ Any areas of stone riprap that have settled, moved or been damaged by erosion should be filled in with hard, durable rock of suitable size or with a 15 cm filter blanket layer or a layer of geo-textile fabric under the stone riprap between the soil and the rock. These sub-layers allow the water to pass from the soil, while preventing the fine soil from washing out from under the stone riprap.

3.2.2 Placement

- ✓ Place stone for riprap on the filter blanket or on the sheet of geo-textile material, in the dry, and conforming to the lines and grades of the construction drawings.
- ✓ Place the stones in such manner as to produce a reasonably well-graded and uniform surface.
- ✓ Place stone to its full course thickness in one operation and without displacing the underlying filter material.
- ✓ Do not place stones in layers.
- ✓ The finished work will be free from objectionable pockets of small stones and clusters of larger stones.
- ✓ Smaller stones will be well distributed in order to fill the voids between larger stones.
- ✓ Do not place riprap by dumping stones into chutes or by similar methods likely to cause segregation of various sizes.
- ✓ Do not use a tractor equipped with a bulldozer blade, stone rake or any similar equipment.
- ✓ The desired distribution of the various sizes of stones throughout the mass should be obtained by selective loading of the material at the quarry, by controlled dumping of successive loads during final placing or by other methods.

4. CONCRETE STRUCTURES

Concrete structures include: concrete channels, walls and slabs, box culverts, inlet and outlet drainage structures, wing walls and floodwalls. Concrete structures will require repairs for erosion at joints and for cracks at fence posts caused by freeze-thaw action over time. Deterioration and erosion of channel slabs are often caused by acid mine drainage and by fine sands and gravel acting on the concrete surface. Repairs to concrete structures are made with concrete or concrete repair products.

4.1 Alignment

- ✓ Concrete structures shall be on line and grade as shown on the construction drawings, with no abrupt changes at adjacent sections.
- ✓ Where visible, both sides and the top of the walls are to be inspected regularly for structural failure or misalignment. If any is observed, it shall be reported to the responsible authority as soon as possible.

4.2 Erosion and Undermining

Adequate erosion protection is required along the contact between the upstream and downstream face of structures. A well graded mixture of rock with stones 25 to 30 cm in diameter or larger placed on a sand filter generally provides the best erosion protection both upstream and downstream of structures. Otherwise, it will create erosion and undermining (piping) of the foundation of structures.

Severe undermining of the structures can displace sections of pipe and create differential settlement on the structure which eventually leads to complete failure of the structures.

Repairs must be promptly initiated. Eroded and undermined areas around structures can sometimes be repaired by filling these areas with large stone (as to the original design). Stone that are large enough to be effective may size 45 to 60 cm in diameter depending on the calculated size normally that relates to velocity.

In case stone of this size may not available or is expensive to buy and haul, concrete slurry can be used to bind smaller stones together to increase their effective size and weight. Gabions have been used successfully in areas where the velocity is low, but should not be used where high velocity and turbulence are expected. Gabions require careful foundation preparation and experienced personnel for installation.

4.3 Cracks in Concrete

- ✓ Any chipped or eroded areas are to be repaired immediately with the proper repair material.
- ✓ Minor cracks should be sealed with appropriate material to prevent water from entering the cracks and causing damage to the concrete for further damages.

4.4 Settlement

- ✓ Backfill should have no settlement or depressions greater than 30 cm.
- ✓ Repair backfill areas with impervious soil material so the backfill around structures and culverts can form an integral part of the dyke or dam embankment.
- ✓ Place backfill in the dry ground, within a distance of 125 cm above concrete pipes; 60 cm above corrugated metal pipes; as necessary over culverts; adjacent to concrete structures; and within the horizontal limits of the trench.

- ✓ Place backfill in loose layers not exceeding 10 cm in depth, and compact each layer with vibratory compactor, mechanical tamper or other acceptable method.
- ✓ Do not use a roller or heavy construction equipment in these areas.
- ✓ In other areas, and where working clearance permits, backfill shall be placed in loose layers not exceeding 20 cm in depth. Do not drop backfill materials, but scatter and bring up equally on opposite sides of culverts, wall, anti–seepage collars and similar structures.
- ✓ Add water or dry the backfill materials as necessary to attain optimum moisture content during compaction.
- ✓ Moisture content of the pervious materials shall be such that no free water drains off and adversely affects the underlying or adjacent materials.
- ✓ Spread the layers evenly before compaction and thoroughly compact each layer.

4.5 Joints

- ✓ There should be no deterioration or displacement of expansion joint material, and no water stop or reinforcement bar should be exposed.
- ✓ Joints may be resealed with several brands of non-sag elastomeric sealants that allow the joint to expand and contract.

4.6 Concrete Surface Repairs

- ✓ Perform concrete repairs with a polymer-modified Portland cement mortar that will provide a minimum of 2500-psi compressive strength in two hours.
- ✓ For repairs greater than 4 cm deep, add 1 cm clean washed pea gravel to the mix at an amount determined by the manufacturer so that the aggregates do not result in variations of the physical properties of the mortar. Limestone gravel is not acceptable.
- ✓ All surfaces to be patched shall be structurally sound, clean and free of loose debris, oils, vegetation, paints, sealers and all other contaminants.
- ✓ Remove all deteriorated concrete to a minimum of 0.5 cm in depth.
- ✓ Cut edges should be square with the concrete surface.
- ✓ Surfaces should be sufficiently rough to ensure a good bond.
- ✓ Any existing reinforcing bars that are exposed should be thoroughly cleaned.
- ✓ If required, existing concrete should be removed to fully expose the reinforcing bar. Sandblasting will be required if there are no other means of cleaning reinforcing bars.
- ✓ All surfaces should be thoroughly saturated, and freestanding excess water should be removed before applying the repair material.
- ✓ The material should be placed in the prepared area starting from one side of the repair and working to the other side.
- ✓ Work the material firmly into the bottom and sides of the repair.
- ✓ Level the material to the desired elevation and close up edges of the repair with a trowel. Finish the material to match the existing concrete finish.
- ✓ For vertical areas, trowel the material in an upward motion over the repaired area. Successive applications must be troweled against the previously placed material just prior to hardening. Build up the material to the thickness desired.
- ✓ Finish the material to match the existing concrete finish.
- ✓ Remove any material applied or spilled beyond desired areas.
- ✓ All exposed surfaces should be thoroughly saturated with water immediately after finishing.

4.7 Unwanted Growth

- ✓ There should be no unwanted growth, grass or weeds in joints or weep holes.

- ✓ Trees and brush cover that exist adjacent to a concrete structure should be removed within 3 m of the structure for inspection and so any repair can be made.

4.8 Obstructions

- ✓ All debris and trash should be removed from the waterway area so it does not collect and create an obstruction downstream, reducing the waterway area.
- ✓ Large trees should be removed as soon as possible before they lodge under a bridge or in a culvert.

5. DRAINAGE STRUCTURES

Drainage structures are used to transport accumulated runoff from the land side of the dyke or dyke to the stream or river side. They function mainly during normal stream flow and after the stream recedes following a storm event.

Drainage structures allow gravity discharge of internal drainage water from behind the dyke into the main watercourse during times when the external water level is lower than the level behind the dyke, while preventing backflows when stream flows are high.

Drainage structures generally consist of a pipe or culvert through the dyke with a flap gate at the waterside outlet and may have trash racks fitted at the inlet and/or the outlet.

Figure 5-1: Outlet of Drainage Outfall with Flap Gate at Amibara Irrigation Project (Source: Consultant's Private Library 2007)

Figure 5-2: Sediment Deposition at the Inlet of Drainage Outfall Structure (Source: Consultant's Private Library 2007)

5.1 Drainage Structures at Dykes

- ✓ These drainage structures consist of an inlet on the land side with a concrete headwall that a gate may be mounted on, a concrete apron in front of the headwall (to prevent erosion) and possibly a sluice gate or slide gate.
- ✓ The pipe or box, usually a concrete pipe, transports local drainage through the embankment to the river side of the dyke.
- ✓ The outlet structure on the river side of the dyke has a concrete headwall to mount a flap gate and a concrete apron in front of the headwall to prevent erosion.

5.2 Drainage Structures at Concrete Walls

- ✓ Drainage structures consist of an inlet box with a wall mounted sluice gate on the land side. The box will have a roadway grate on top to catch local drainage and convey it to the water side.
- ✓ On the water side, a flap gate is mounted to the concrete wall. It may be installed in a separate chamber to shield the gate from the water flow.

5.3 Inlet/Outlet Channel and Structures

- ✓ Maintenance of flood boxes includes cleaning the inlet and outlet of accumulated debris and sediment to ensure water flows freely. The flap gate should be annually cleaned and lubricated to ensure that it swings easily and closes properly with a good seal, and be periodically painted with rust resistant paint.
- ✓ The dyke slopes adjacent to flood boxes should be kept clear of trees and brush to allow unimpeded inspection of the inlet and outlet of the flood box.
- ✓ Repairs shall be made to erosion and/or undermining of the inlets and outlet structure and sloughing of slopes which might block flows.
- ✓ All gates, trash racks and miscellaneous metal parts or works should be examined periodically
- ✓ The outlet structure is usually concrete with a flap gate attached to the headwall. All gates shall be seated correctly, greased and painted annually.

6. CONTINGENCY EMERGENCY PLANNING

6.1 General

Provision should be provided in the plan for the possibility of uncontrolled failure of flood protection works, including backup measures, and procedures for possible emergency evacuation of the affected population.

Sources of sandbags and fill material must be identified in advance together with local contractors, materials, and heavy equipment, pump and generator suppliers. Required delivery notice, costs and setup times is also valuable information.

The responsible authority for O & M must be prepared to carry out any work essential to dyke or bank protection integrity as emergency measures. Emergency measures include high water flood patrols, inspections, and implementation of contingency emergency planning and emergency response measures

6.2 High Water Patrol Inspections

Patrol inspections should be carried out during the pick flood seasons and high water events to monitor the performance of the flood control works and identify needed corrective actions

During high water events, local water level gauges should be monitored regularly and the readings recorded for future reference. Dyke patrol frequency should increase as flow and/or water levels approach critical conditions, and should be continuous while the level is within about 1.0 m of the dyke crest. The patrol crews are to observe and report to the authority any conditions or occurrences that could signal a weakening of the works such as:

- ✓ Settlement of the dyke crest and slopes causing ground depressions or sinkholes;
- ✓ Areas of low freeboard due to variable river profile or loss of dyke fill;
- ✓ Cracking of the dyke crest or slopes;
- ✓ Gulling: Observations should be made for sloughing and/or erosion of the dyke slopes;
- ✓ Seepage through the dyke and/or at the landside toe of the dyke. Close attention should be paid to seepage, as the safety of the dyke can be threatened by an increase or concentration of seepage flows;
- ✓ Erosion of the riverbank adjacent to the dyke;
- ✓ Sloughing, erosion and/or loss of rock from bank protection works. Critical areas should be closely inspected during and after high water events;
- ✓ Debris accumulation at flood boxes, flap gates and trash racks;
- ✓ Stream blockages or redirection of flows due to logs, debris, gravel and/or sediment jams, especially near bridges or other constrictions.

6.3 Emergency measures

As the river rises to critical levels, crews should be prepared to undertake emergency repairs such as treatment of active boiling, excessive slope seepage, riverside erosion, saturation, local overtopping, and pumping of internal drainage

6.3.1 Local Overtopping

As the prediction of flood profiles is uncertain, and because dykes often have varying freeboard, patrols should be advised to pay close attention to lower than average freeboard.

Once water flows over a dyke crest, fill is usually rapidly washed away creating a breach. This generally increases rapidly in size and will not generally be practical to close until water levels equalize.

Sandbags are usually considered for raising low sections of dyke, however, progress is slow and considerable hand labour is required. Sandbags should normally only be used for raising short sections. Possible alternatives to sandbags include reinforced plastic sheeting to contain loose granular or other fill, and water-filled dams.

Heavy equipment and trucks can be used to raise a dyke provided the work is done well in advance of high river levels, however, due to possible vibration and saturation effects, heavy equipment is not generally advisable on a dyke when the water level is near the crest.

6.3.2 Excessive Slope Seepage and Saturation of Fills

Where seepage on the dyke landside slope leads to soggy unstable conditions, free draining fill berms may be needed.

If high water levels are sustained for extended periods and the dyke fills becomes saturated, it may be necessary to restrict traffic on the crest road.

6.3.3 Riverside Erosion

Where river currents cause erosion of the face of the dyke or nearby over bank, rock riprap may be placed with an excavator or end-dumped provided the site is accessible to heavy equipment and safe for operation.

6.3.4 Internal Drainage

Local runoff and drainage will not escape once flood boxes close in protected areas which lack adequate permanent pumping facilities.

Temporary pumping of local drainage, or interception and diversion of inflow from higher elevations, may be necessary to alleviate this condition.

6.3.5 Debris and Sedimentation

Dykes are generally designed for open water conditions. Accumulations of sediment and/or debris either between floods or during events must be monitored to ensure detrimental deflection of currents and/or blockages do not occur. Action may be necessary to break up and/or remove accumulations especially at bridges, constrictions and bends.

6.3.6 Emergency Warning and Evacuation

Should the possibility of uncontrollable dyke failure arise, the local authority and Police must be alerted immediately in accordance with the local authority contingency emergency plans. Unless directed otherwise by the Police, the maintenance crew and responsible authority should generally confine its efforts to preventing flooding while ensuring the safety of its workers. The Police would generally be responsible for advice to the public in coordination with the local authority.

If conditions threaten to exceed local resources, a request for federal assistance should immediately be directed to the Federal DPPC.

This page intentionally left blank

APPENDIXES

List of Appendixes

APPENDIX A.

The following two tables bring together some useful lists of data and conversion factors for easy reference as they are always applicable in hydraulic as well as all engineering works.

APPENDIX A -1: USEFUL DATA

Density of Water	= 1,000 kg/m^3	= 62.4 lb/ft^3
Nominal weight of reinforced concrete	= 23.6 kN/m^3 (2,400 kg/m^3)	= 150 lb/ft^3
Nominal elastic modulus of concrete	= 14 kN/mm^2 (140 x 10^3 kg/cm^2)	= 2 x 10^6 lb/in^2
Co-efficient of Linear expansion of concrete	= 10 x 10^6 per ^{o}C	= 5.5x10^6 per ^{o}F
Acceleration of gravity, g	= 9.806 m/s^2	= 32.3 ft/s^2

APPENDIX A -2: CONVERSION FACTORS

Length		
1 inch	=	25.4 mm
1 foot (12 inches)	=	0.3048 m
1 mile (5,280 ft)	=	1,609 m
Area		
1 ft^2	=	0.093 m^2
1 acre (43,560 ft^2)	=	0.4047 hectares (4047 m^2)
1 sq. mile (640 acres)	=	259 hectares
Volume		
1 ft^3	=	0.028 m^3
35.315 ft^3	=	1 m^3 (=1,000l)
Imp. gallon (=0.16 ft^3)	=	4.546 l
1.0 US gallon	=	3.785 l
Discharge		
1 cusec (ft^3/s)	=	0.028 cumecs (m^3/s)
1 Imp. gallon/minute	=	0.076 l/s
Weights		
1 lb	=	0.454 kg
2.2 lb	=	1.0 kg
1 ton (US)	=	907.2 kg (0.907 tonnes)
Force		
0.2248 lbf	=	1 N (0.1020 kgf)
0.06852 lbf/ft	=	1 N/m (0.1020 kgf/m)
145.0 lbf/in^2	=	1 N/mm^2 (10.20 kgf/cm^2)
Moment		
0.7376 lbf ft	=	1 Nm (0.1020 kgf m)

APPENDIX B.

The following tables bring together some useful lists of reference materials for further reading on each part of the manual as they are useful for further reading.

APPENDIX B - 1: RECOMMENDED REFERENCES FOR PART 2

1	M. Chakraborti, 1996:	Estimating, Costing, Inspection And Valuation In Civil Engineering: Principles and practices, Bhaktivedanta Book Trust, 13th Edition, India
2	Tsegaye D, (1999):	Project Management Guideline for the Ethiopian Millennium (Amharic Version), Berlin, Germany

APPENDIX B - 2: RECOMMENDED REFERENCES FOR PART 3

1	Kefyalew Achamyeleh, 2003	Integrated flood management case study, Ethiopia, December, 2003.
2	Mott MacDonald 1990	*Design Manual For Irrigation Project in Nepal (United nations Development Programme (NEP/85/013/World Bank), Social and Environmental Consideration. Mott MacDonald, February 1990.*
3	EPA, 2002	*Environmental Impact Assessment Proclamation, Proclamation No. 299/2002 , December 2002.*
4	EPA, 1999	*Environmental Policy of Ethiopia (EPE), April 1999, Addis Ababa*
5	MoWR, 2000	*Ethiopian Water resource Management proclamation, Proclamation No. 197/2000.*
6	EPA, 1995	*Federal Environmental Protection Authority (EPA) establishment proclamation, Proclamation No. 9/1995.*
7	EPA, 2002	*Environmental Protection Organs Establishment Proclamation, Proclamation No. 295/2002.*
8	MoA, 1994	*Proclamation on Conservation, Development and Utilization of Forests, Proclamation No. 94/1994), 1994.*
9	MoA, 2005	*Rural Land Administration and Land use Proclamation, Proclamation No. 456/2005.*
10	UNICEF-News Letter August 29, 2006	*UNICEF – News letter, By Jane O'Brien , 29 August 2006*
11	MoWR, 1999	*Water Resource Policy of Ethiopia , 1999*

APPENDIX B - 3: RECOMMENDED REFERENCES FOR PART 4

1	Alemayehu Tefera, (1992):	Foundation Engineering, Addis Ababa University Press, Ethiopia
2	Blight, G.E, (1997)	Mechanics Of Residual Soils, A Guide to Formation, Classification and Geotechnical Properties of Residual Soils, with advice for geotechnical design, A.A. Bulkman, South Africa.
3	Bowles, J.E, (1988):	Foundation Analysis And Design: McGraw-Hill Book Company, 4th Edition, New York.
4	EBCS-8, (1995):	Ethiopian Building Code Of Standards: Design of Structures for Earthquake Resistance, Ministry of Works and Urban Development, Addis Ababa, Ethiopia.
5	ESCP, (1983):	Ethiopian Standard Code Of Practice For Foundations, Issued by Ministry of Construction, Addis Ababa, Ethiopia.
6	R. Gopal & A.S.R Rao, (1991):	Basic And Applied Soil Mechanics, Wiley Eastern Limited, New Delhi, India
7	S. Kumar Garg, (1989):	Irigation Engineering And Hydraulic Structures, Khanna Publishers, Delhi, eighth edition, India.
8	USBR, (1998):	Earth Manual: A Guide to the Use of Soil as Foundation and Construction Materials for Hydraulic Structures, US Department of Interior Water and Power Resources Service, 3rd Edition, CBS Publishers and Distributors, New Delhi, Indian Edition.

APPENDIX B - 4: RECOMMENDED REFERENCES FOR PART 5

1	Concert Engineering: 2001	Design of Topographic Survey, Guideline for the Design of Irrigation, Concert Engineering Plc, 2001

APPENDIX B - 5: RECOMMENDED REFERENCES FOR PART 6, 7, 8 & 10

1	CIRIA, 1976	*Design of Riprap Slope Protection Against Waves*. Hydraulics Research Station, Wallingford, UK, publish by CIRIA, December 1976.
2	Chow, V.T	Applied Hydrology, McGraw-Hill International, New York, 1954.
3	DEP, 2008	*Hand Book for Maintaining Flood Protection Projects, Pennsylvania Department of Environmental Protection, Bureau of Waterways Engineering*
4	ERA, 2002	*Drainage Design Manual, Ethiopian Road Authority ERA, Addis Ababa, 2002*
5	Gebyhu Admasu, 1989	*Regional Flood Frequency Analysis, Hydraulics Laboratory, Royal Institute of Technology, Stockholm Sweden, 1989.*
6	Halcrow, 2004	*Ache Design Manuals, Design Manual, Part 3: Water Resources, (ADB LOAN NO. 1579-INO***).** Halcrow Ltd,2004
7	HEC No. 19, 1984	*US Department of Transportation, Federal Highway Administration. Hydraulic Engineering Circular No. 19, Hydrology, 1984*
8	ICE, 1978	Floods and Reservoir Safety: an Engineering Guide, ICE, London
9	Mott MacDonald 1990	*Design Manual For Irrigation Project in Nepal (United nations Development Programme (NEP/85/013/World Bank)). Mott MacDonald, February 1990.*
10	MoE, 2008	*Guidelines for Management of Flood Protection Works in British Columbia). Province of British Columbia, Ministry of Environment, Lands and Parks.* March 1999.
11	TRL Overseas Road Note 14	*Hydrological Design Manual in the Tropics, 1997*
12	USBR, 1987	*Design of Small Dams, Third Edition.* USBR, Denver Colorado,1987
13	USBR, 1987	*Operation Maintenance and inspection Manual for Dams, Dykes, and Dykes.* Ohio Department of Natural Resources, Division of Water, Dam Inspection Section, 1987.
14	USBR, 1978	*Design of Small Canal Structures, Third Edition.* USBR, Denver Colorado,1978

APPENDIX B - 6: RECOMMENDED REFERENCES FOR PART 9

1	BaTCoDA, (1991):	Technical Specifications And Methods Of Measurements For Construction Of Buildings, Building & Transport Construction Design Authority (BaTCoDA), Addis Ababa, Ethiopia.
2	FIDIC, (1987):	Conditions Of Contract For Works Of Civil Engineering Construction: With Forms & Agreement, 4th Edition.
3	GCC, (1994):	Standar5d Conditions Of Contracts For Construction Of Civil Work Projects, Ministry of Works & Urban Development, Addis Ababa, Ethiopia.
4	Sharma J.L. (1998):	Construction Managements And Accounts, Satya Prakashan, New Delhi, India.
5	Tsegaye D, (1999):	Project Management Guideline for the Ethiopian Millennium (Amharic Version), Berlin, Germany