THE NILE BASIN INITIATIVE – NBI
Eastern Nile Subsidiary Action Program -ENSAP
EASTERN NILE TECHNICAL REGIONAL OFFICE (ENTRO)

Flood Embankment Design, Operation and Maintenance Manual, Sudan

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1 Background

1.1 General

The Nile Basin Initiative (NBI) is based upon an agreed shared vision between the Nile Basin countries to achieve sustainable socioeconomic development through the equitable utilization of, and benefit from, the common Nile basin water resources. A strategic action program is introduced to translate this vision into investment activities and projects, currently under preparation in the Eastern Nile and the Nile Equatorial Lakes regions. The Eastern Nile countries (Egypt, Ethiopia and Sudan) have identified their first joint project, the Integrated Development of the Eastern Nile (IDEN). IDEN consists of a series of sub-projects to be implemented by ENTRO to address issues related to flood preparedness and early warning (FPEW); power development and interconnection; irrigation and drainage; watershed management; multi-purpose water resources development; and modelling in the Eastern Nile.

The objective of the FPEW project is to reduce human suffering caused by frequent flooding while preserving the environmental benefits of floods. The project is implemented by a Regional Flood Coordination Unit headed by a regional flood coordinator. In addition the FPEW project enhances regional collaboration and improves national capacity in the mitigation, forecasting, warning, emergency preparedness, and response to floods in the Eastern Nile basin. The project has four components which were identified during the project definition phase. The project is now in its implementation phase.

1.2 Objectives

A review of the World Bank mission to Sudan, observed that there is a lack of sound technical skills to design and build flood embankments in Sudan. In this effect, the FPEW project of ENTRO, employed the Hydraulic Research Station (HRS) of Sudan to:

a. Prepare a design, operation and maintenance manual for sound embankment design for flood protection or irrigation-cum-flood control.

b. Conduct training on the use of the manual with example projects to government and NGO’s staff working on flood protection.

1.3 Terms of Reference

ENTRO (the client) employed the Hydraulics Research Station (HRS) as a consultant to assist the Flood Coordination Unit in preparing the manual in accordance with the contract signed by both parties. The major works requested by the Coordination Unit is a preparation of a flood embankment design, operation and maintenance manual and to conduct training for Sudanese engineers:

Generally, the manual should give adequate guidance on the subjects listed below:

- Topographic surveying requirement

- Hydrological Analysis – surface, subsurface, and local drainage
- Hydraulic and geomorphologic consideration.
- Geotechnical and geomorphologic considerations
- Construction aspects
- Social and environmental considerations, and
- Operation and Maintenance

Secondly, the consultancy includes a training workshop to be conducted by the HRS addressing young engineers from Sudan. The teaching material to be based on the manual, covering all aspects of flood embankments: design, construction, maintenance, and impacts.

The workshop has been successfully completed in Khartoum, during the period 16-18 January 2010. The workshop participants include two groups: engineers from the regional states as well as from the Ministry of Irrigation and Water Resources MoIWR, and the second group include senior staff from the MoIWR. The first group raises many questions including the applicability of the manual in the field, while the second group provides constructive comments to improve the manual to serves the targeted objectives. All comments received during workshop have been incorporated in the final version of the manual.

1.4 Layout of the Manual

The Manual comprises nine chapters in addition to references and annexes. Chapter one is a general background, briefly describing the objectives of the manual and the terms of references specified by the Flood Unit. Chapter two is introduction as a basic knowledge regarding the hydrology of the Eastern Nile system and flood characteristics with historical flood records and damages associated to these flood incidents. The chapter also covers flood management and experiences in Sudan and the importance of having a flood management manual.

Chapter three is mainly allocated for the hydrologic issues including the rainfall-runoff processes, computation and analysis. It also covers modelling of flood routing and frequency analysis with emphasis on flood computation. Data needs, and flood forecasting have been discussed as well.

Chapter four covers the hydraulic and geomorphologic considerations with detailed description of flood embankments including the design of their different parameters and causes of failures. It also covers siltation and scouring problems related to river channels and means to design crossings and accessibility structures.

Chapter five is allocated for the geotechnical and soil issues with detailed soil compaction techniques, density methods and related stability analysis. The chapter also gives a brief description of the soils and geology of the Sudan. As well, this chapter presents methodology for foundation design of flood embankments.

Chapter six is dedicated for the construction aspects including the material, local knowledge and experience with highlights on recent methods of construction. The section also includes the economics of construction and issues of procurement and contracts as a prerequisite to initiate good construction works.

Chapter 7 is meant to cover the social and environmental issues related to flood embankments. It discusses issues of ownership and land tenure system in flood prone areas, and how this affect dyke building. It also covers community participation in flood management in rural areas. The chapter discusses the potential environmental impacts associated to flood embankments, and listed a check list for EIA as related to flood embankments.

Chapter 8 is allocated to cover operation and maintenance issues. It discusses the role of local communities in routine inspection and maintenance of flood dykes. The chapter discusses causes of embankment failures, and preventive maintenance methods. It also covers, inspection timing, and associated roles of stakeholders.

The manual is concluded in chapter nine and some recommendations deemed necessary are clearly highlighted. To avoid having a bulky report, some details of the covered issues, e.g., local data, design examples, and parameters values are annexed at the end of report. All references related to the manual topics are incorporated at the Reference section which is requested by the Unit for further details.

2 Introduction

The Sudan is occasionally hit by high floods caused by high river levels, or by heavy torrential rains over the villages and urban areas. Historically, high river levels caused huge flood damages in 1946, 1988, 1998, 2006, and 2007. While, heavy torrential rainfall caused widespread flooding in residential areas occurred in 1999, and recently in 2007 and 2009. Though rarely happens, the two sources of flooding may overlap. The high rainfall over Khartoum (over 120 mm in one day), occurred in mid August of 1988, was accompanied by high levels of the Blue Nile, that were consistently above flood level for 3 weeks. This exacerbates the suffering of the people and complicates flood mitigation efforts. Natural drainage of flood water to the Nile became very difficult because of high water level in the river.

The river flooding in Sudan occurs mainly along the banks of the Eastern Nile rivers, and specifically the Blue Nile, and subsequently downstream in the Main Nile, Figure 2.1. Less frequent river flooding occurs by other rivers, e.g., Bahr el Jabel in Southern Sudan, the Gash river in Eastern Sudan, and some of the Wadi's in western Sudan.

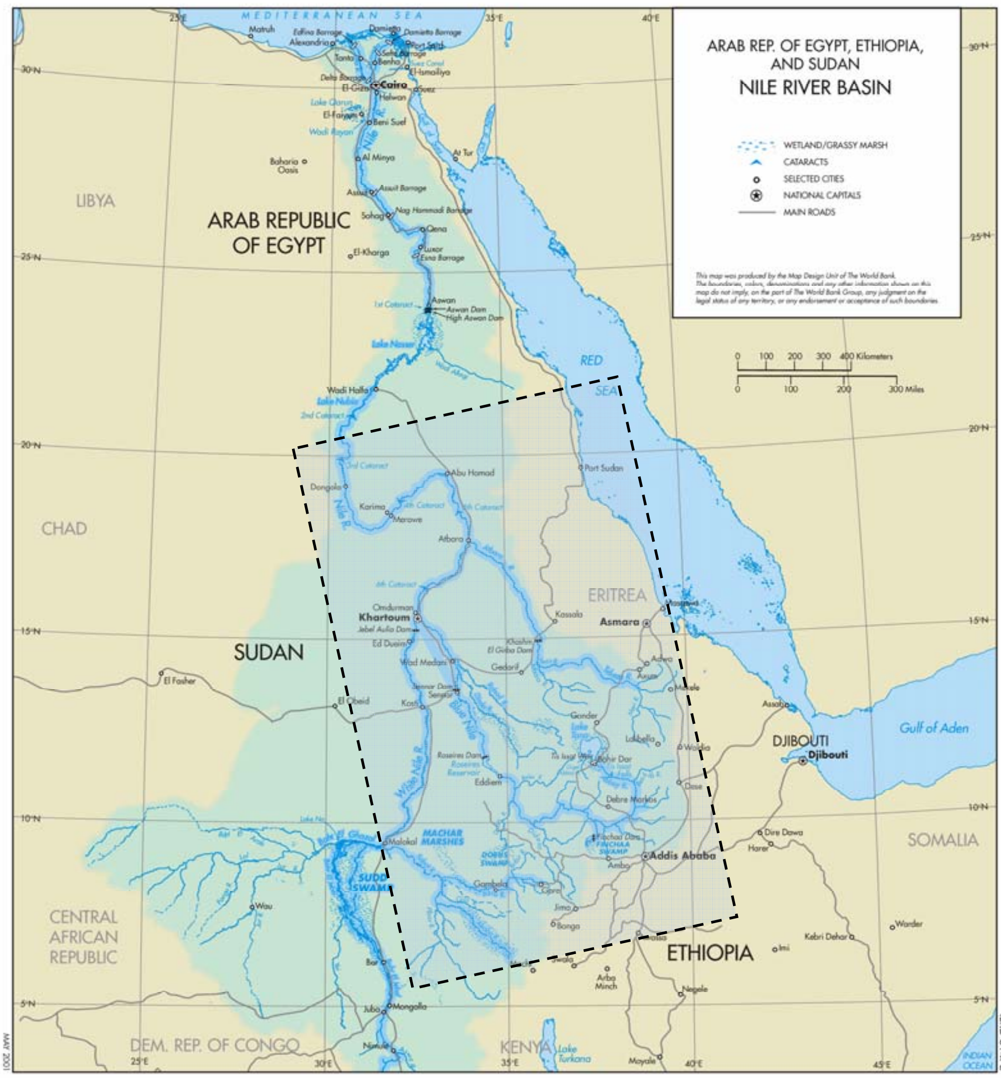


Figure 2.1: Location map of the Eastern Nile River System within the Sudan

The reported flood damages during high flood years, was mainly loss of property, damage to agricultural crops, drowning of livestock, and some people death in severe floods such as 1988 and 2007. The existing flood management activities are predominantly supported by the government in Urban areas, and communal efforts in the rural areas. Flood warning, although exist but improper and hardly used effectively to reduce anticipated flood damages. Therefore, a flood management manual will help government institutions and local communities to better cope with and mitigate high river flooding in the Sudan.

This is an introductory chapter of the manual, highlighting the general characteristics of the Eastern Nile floods within the Sudan, the historic flood damages, and the potential for flood management. The existing management experience (official and communal) is evaluated to highlight anticipated benefits from the preparation and implementation of a flood management manual in reducing future damages.

2.1 Eastern Nile River System in Sudan

The major part of the Eastern Nile catchment (Blue Nile, Atbara, Sobat) is located within the Sudan, Fig. 1. Out of the total catchment of the three rivers, more than two-third is located within the Sudan. The majority of the runoff is generated from the Ethiopian Plateau, and only small part from the Sudanese catchment. However, all the three rivers discharge their floods in the Sudan to eventually form the Nile River flowing northward to Egypt.

The catchment areas of the EN Rivers are shown in Figure 2.2. The three rivers originate from the high Ethiopian Plateau (> 1500 m amsl), and flow north, and north-west to enter the flat terrain of the Sudan plains at 500 m amsl. The river channel changes from gorge type of river courses in the Plateau to meandering rivers along the mild slopes of the Sudan plains. River cross sections are relatively narrow (200 to 400 m), and depths are relatively shallow, 5 to 10 m. Therefore, insufficient river capacity available to carry the high flood waters, which then spills on both sides of the river.

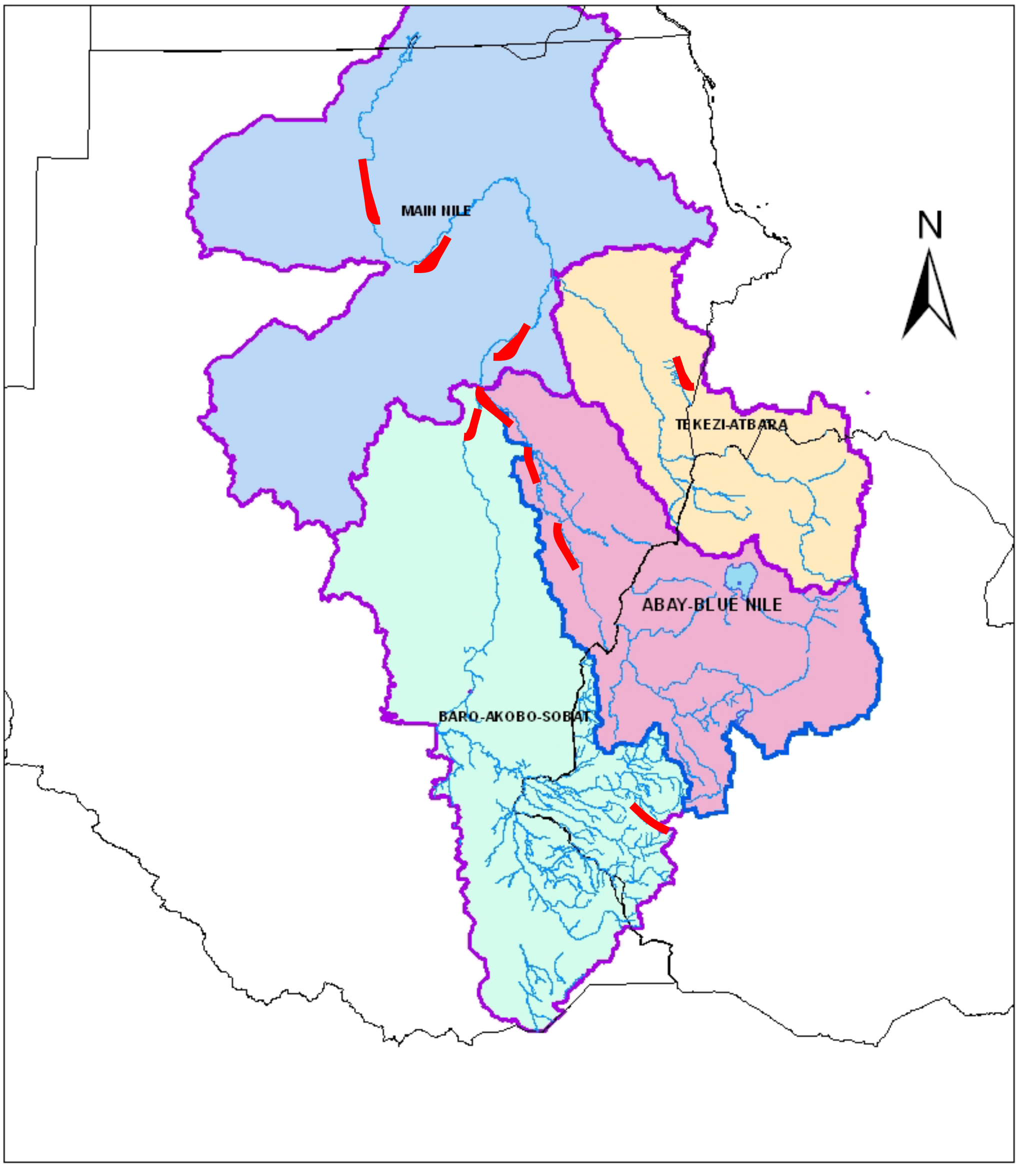


Figure 2.2: Location of frequent (riverine) flood areas in the Eastern Nile of the Sudan

2.2 Flood Characteristics

Flooding in the Sudan is caused by two sources: (i) River flooding, i.e., high river waters spill over the banks, (ii) Flash floods caused by torrential rainfall over villages and urban areas. This report emphasizes on river flooding, though methodology and procedure are well applicable to rainfall flooding.

Historic river flooding occurs along the EN (Main Nile, Blue Nile, Atbara) are shown in Fig. 2.2, highlighted in red. Though less frequent, but severe flash flooding by Wadi's (Khors) may occur in some areas in central Sudan. The so-called Khor Dunya, a small tributary feed into the Blue Nile just downstream of Roseires dam, frequently wash out the highway and railway lines during high flood events, and sometimes water carries drowned livestock.

River flooding is seasonal, following the high rain over the Ethiopian Plateau between July to September. The flood wave propagates downstream along the Blue Nile and Atbara, then into the Main Nile facing towards Egypt. The celerity (speed) of flood wave reduces from about 200 km/day near Sudan-Ethiopia border to less than 100 km/day further north in Dongola. Therefore, typical travel time of the flood-wave is given in Table 1, which can be longer or shorter depending on magnitude of the flood, and condition of the river.

Table 2.1: Estimate of typical travel time of the flood wave in the Sudan (July to September).

River Reach	Travel time in days
Eldeim - Crosiers	0.5 to 1.0
Rosieres - Sennar	1.0 to 2.0
Sennar - Khartoum	1.5 to 3.0
Khartoum - Merowe	3.0 to 4.0
Merowe - Dongola	3.0 to 4.0
Girba - Atbara	1.0 to 2.0

Historically high river flooding occurred in different years, 1946, 1988, 1998, 2006. As can be noticed, the frequency of high floods increases in the last few decades. The Blue Nile hydrographs at Khartoum for those years is given in Figure 2.3.

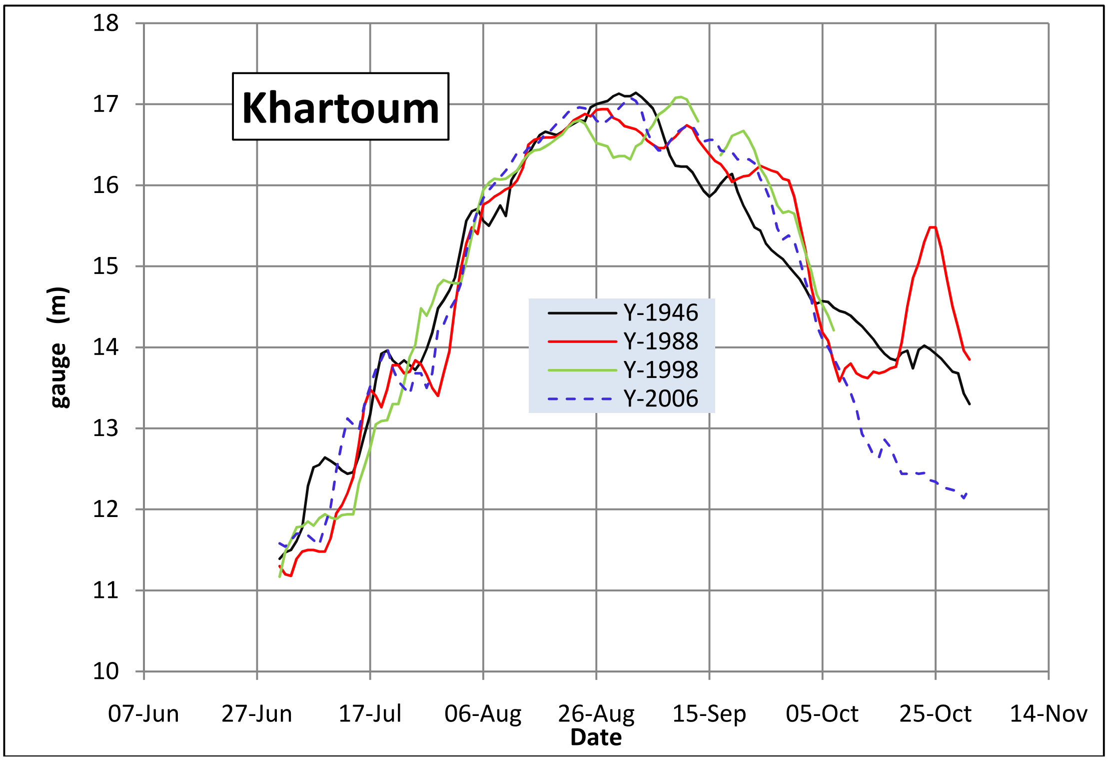


Figure 2.3: Flood hydrographs at Khartoum for different high years

2.3 Historic Flood Damages

Although it is hard to assess accurately the flood damages, in particular in the far past, huge flood damages were reported during high flood years. Table 2 shows high flood peak flows at key places in the Sudan; Eldiem, Khartoum, and Dongola. Flood damages amount to 100's of million dollars were reported for those years. Damages constitute: loss of agricultural lands (crops and fruit trees along the river banks), loss of livestock, damages to buildings (houses, schools, and roads). Following a high flood, usually accompanied by wide spread of diseases, such as malaria, diarrhea, bilharzias, and other water borne diseases.

Table 2.2: Highest flood peak at key locations for selected high floods (mcm/day).

Flood	Eldiem	Khartoum	Dongola
1946	NA	NA	NA
1975	842	955	1095
1988	801	891	978
1994	721	664	930
1996	833	679	776
1998	794	714	1086
2001		811	1038
2003	750	727	775
2006	955	912	924
2007	699	879	963

In recent past, huge flood damages have been reported by different national and international agencies. e.g., The river flooding of 1998, had affected 1 million people, and displaced around 100,000. The devastation reported to be more than in 1988 and 1946 flood. The associated recovery plan prepared by the UN agencies, Sudan Government, World Bank, and IMF had requested US$ 230 million to recover the associated flood damages (Source: University of Pennsylvania - African Study centre). Around mid August of 2006, the high river levels forced more than 1,100 families to flee their homes (Source: Sudan Tribune Newspaper, 15/08/2006).

In some years, rainfall flooding (flash floods) caused more damages than river flooding, e.g., while river floods is below critical level in 2007, the torrential rainfall over the country brought what the Sudanese Government called the worst floods in living memory (photo-1). The United Nations reported large damages in different regional states: 64 people died, and about 30,000 houses destroyed, about half million people affected, (source: Sudan Floods - Bulletin # 2, by United Nations)

Photo 2.1: Flooding in Khartoum, in 2007 (Source: Canadian Red Cross Press Release 20/07/2007).

2.4 Potential for Flood Management

Given the immense losses because of floods, there is a great potential to reduce damages through appropriate flood preparedness and management activities. The location of Sudan, at the central part of the basin, allows relatively long lead times for flood prediction. Accurate flood early warnings will provide essential information for downstream people to evacuate valuable properties, and get prepared for the incoming flood. Therefore, early warning can be the first tool in flood management for the Sudan. However, flood bunds should be built permanently on low lying areas along the river banks. As discussed in following chapters, proper design and construction, as well as routine maintenance of the bunds is important to guarantee correct functioning against high floods.

It should be emphasize that, due to limited storage capacity of existing reservoirs, opportunity to store flood water in the Sudan is very limited. e.g., the capacity of Rosieres reservoir is only 6% of the Blue Nile flow. Furthermore, the high silt content during flood times, makes water storage a risky business, as to the danger of loosing live storage by the deposited silt.

2.5 Government experience and communal practice for flood management

The Sudanese people have a long history dealing with floods, in particular as associated with flood basin irrigation experienced along the Main Nile River. Obviously, this is an extension of the old Egyptian (Pharaohs) floodplain agriculture, migrated further south by the Nubian Kingdoms in the Sudan. The government policy towards flood management, it is the responsibility of the Ministry of Interiors (Civil Defence Department), who cooperates -with varying degree, with the relevant ministries, e.g., the Federal Ministry of Health, Ministry of Irrigation and Water Resources, Ministry of Defence, and regional governments. The Ministry of Irrigation and Water Resources, provides information on river stages and discharges. The Sudan Meteorological Department provide weather information and the rainfall forecasts in particular. Other ministries are involved directly or indirectly in the flood management within the Sudan. However, during flood events, it is not uncommon to see a sectarian division on activities, and that appropriate coordination is sometimes missing.

The Ministry of Irrigation and Water Resources (MoIWR) forms a committee from high officials within the ministry, called the high flood committee, responsible for the day to day management of the flood, including flood early warning, proper operation of reservoirs, and in some places involves in construction/enhancement of flood bunds, e.g., using they the MoIWR machineries to build flood dykes along the Blue Nile near Wad Medani. The flood event of the Gash River in 2006, had triggered huge support from the MoIWR. The governments of the regional states affected by floods play an important role in flood mitigation, e.g., Sennar State, Gezira State, Khartoum State, as well as other states affected by the high floods.

The local practice of flood management at village level is through the so called "nefeer". Responding to calls from those close to river banks, the villagers voluntarily participate in defending the flood water, through building of flood dykes, or evacuating people (especially women and children) to higher grounds, and providing food and shelter to the displaced people. Locally this is an effective method, in particular at remote areas where government support is normally delayed or not existing. National and International Organizations (NGO's) also play a essential role in mitigating flood damages, those include: The Sudanese Red Crescent Society (SRCS), The Red Cross, Arab Assistance Funds, among many others.

2.6 Benefits of a Flood management Manual

A manual which includes proper procedure on how to manage floods in the Sudan is indispensible by both government institutions, and local communities. A clear procedure how to act during flood event will substantially reduce damages and panic of the poor living along the rivers. The flood management manual discussed below, includes different parts: (i) How to be informed about the incoming flood wave, which requires accurate and early flood warnings, as well as effective media to communicate the information, (ii) How to act at national and regional level (central and regional governments), as well as at the local level (villages and towns). This includes all activities before, during and after the flood event. The manual will include information on how to design, build and maintain a flood bund. How to manage a flood bund at different responsibilities levels (central, regional, and local). Important issue is how to raise awareness about flood management, and conduct relevant training for different stakeholders.

The expected benefits from effective flood management using the flood manual are immense. Loss of life will decreased because of early waning. Loss of property (buildings, crops, and livestock) will be minimized because of precaution measures. After all, the resilience of people and institutions will be enhanced tremendously to cope with flooding.

3 Hydrologic Considerations

Flood embankments must be high enough to avoid their overtopping by peak floods. Therefore, determination of the probably maximum flood level is an essential step in designing flood embankments. Various hydrological methods have been developed and used for estimating peak floods. Some of them are based on the characteristics of the drainage basin, and others are based on the theory of probabilities applied to the previous known flood data. Others are based on a study of the rainfall and runoff data. Several of these methods are often employed together, and the right value of the design flood is chosen, to suit a certain problem. A good example is to select one or more of the methods: Empirical Formulae; Unit Hydrograph; Envelope Curves; and Statistical or Probability methods. This chapter gives a brief introduction on hydrological processes, and then discusses the runoff methods, and their application to estimate flood peaks, in particular in un-gauged catchments.

3.1 Hydrological processes

During a given rainfall, hydrologic losses such as infiltration, depression storage, and detention storage must first be satisfied before surface runoff begins. As the depth of surface detention increases, overland flow may occur in portions of the watershed. Water eventually moves into small rivulets, small channels, and finally the main stream of the watershed. Some of the water that infiltrates the soil may move laterally through upper soil zones until it enters a stream channel. This portion of runoff is called interflow or subsurface storm flow. Figure 3.1 shows the hydrological cycle, and runoff process. However, in our region (Eastern Nile) there is no snowmelt runoff component.

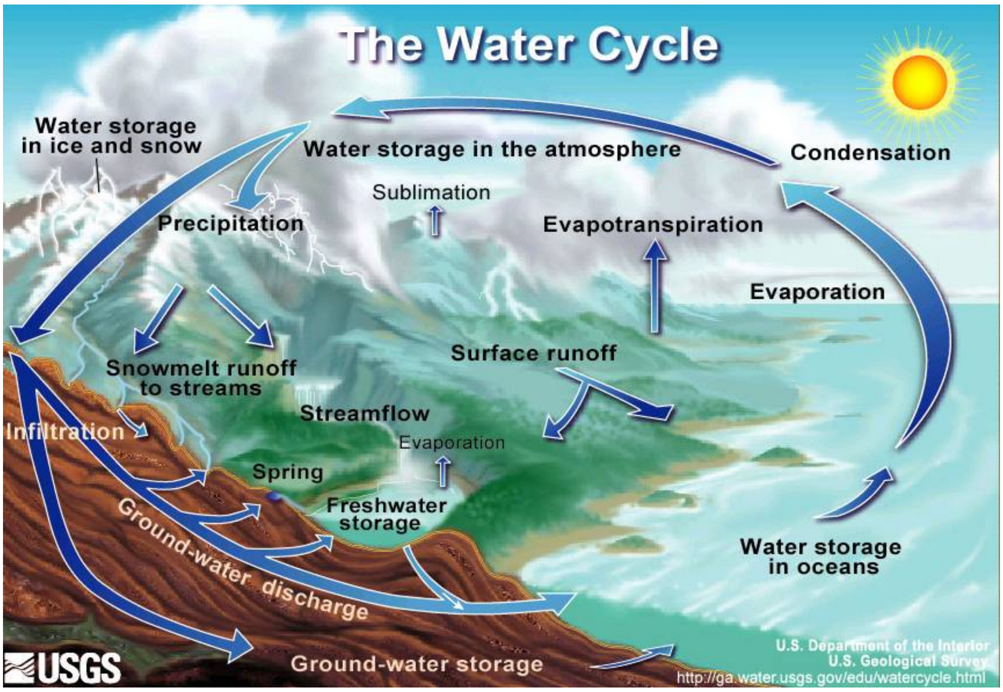


Figure 3.1: Hydrological cycle, and runoff Processes (Source: USGS).

3.2 Runoff methods

A detailed knowledge of the magnitude and time distribution of both rainfall and runoff is required for most flood control or flood management studies, especially in urban watersheds (Bedient & Huber, 1988). Countless hydrologic methods are available for estimating peak discharges and runoff hydrographs. The omission of other methods from this manual does not necessarily preclude their use. The methods include:

1. **Rational method** for watershed areas less than 0.81 km2
2. **Hydrograph method** for watershed areas up to 2500 km2
3. **Runoff Curve Number Methods**. Recommended for catchments area greater than 0.5 km^2.
4. **Empirical methods** are developed for specific catchments in which catchments characteristics are correlated with river flow data and the relations can be used for un-gauged catchments in the same region.
5. **Frequency analysis** methods are used when historical River flows data record are available (25 years).

3.2.1 Rational Method

The Rational Method was first introduced in 1889. Although it is often considered simplistic, it still is appropriate for estimating peak discharges for small drainage areas of up to about 1.3 km^2 in which no significant flood storage appears.(Marek, 2009). The Rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as:

$$Q = \frac{CIA}{3.6} \quad \text{……………………………………………………………………………………} \quad (3.1)$$

Where;

Q is the maximum rate of runoff (m^3/s)
C is the runoff coefficient as outlined in Runoff Coefficient below
A is the drainage area (km^2)
I is the average rainfall intensity (in./hr. or mm/hr.).

The ***assumptions*** of the rational methods include:

1. The rate of runoff resulting from any constant rainfall intensity is maximum when the duration of rainfall equals the time of concentration.
2. The rainfall intensity is constant and uniformly distributed over the entire drainage area.
3. The fraction (C) of rainfall that becomes runoff is independent of rainfall intensity or volume.

Time of Concentration

Time of concentration T_C is defined as the time required for storm runoff to travel from the hydraulically most remote point of the drainage basin to the point of interest. T_C is typically the cumulative sum of three travel times, including:

1. Sheet flow travel time,
2. Shallow concentrated flow travel time,
3. Channel flow travel time,

Sheet flow

Sheet flow is flow of uniform depth over plane surfaces and usually occurs for some distance after rain falls on the ground. The maximum flow depth is usually less than 20 - 30 mm. For unpaved areas, sheet flow normally exists for a distance less than 25 – 30 m. An upper limit of 91 m is recommended for paved areas. A common method to estimate the travel time of sheet flow is

$$T_t = \frac{6.92 L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}} \quad \text{...} (3.2)$$

Or

$$T_t = \frac{5.476 L^{4/5} n^{4/5}}{P_2 S^{2/5}}$$

Where

T_t = Travel time (minutes)

L = Length of flow path (m)

i = design storm rain fall intensity (mm/hr)

S = Slope of flow m/m

n = Roughness coefficient for sheet flow from Table(A1-11)

P_2 = is the 2-year, 24-hour rainfall depth in mm.

Shallow Concentrated Flow Travel Time

The Upland Method is commonly used when calculating flow velocity for shallow concentrated flow travel time in which:

$$V = K\sqrt{S} \quad \text{......................................} (3.3)$$

Where

S is the slope in percent and K (m/s) is an intercept coefficient depending on land cover

$$T_t = \frac{L}{60V} \quad \text{......................................} (3.4)$$

Where

T_t = Travel time (minutes), L = Length (m), V= the velocity (m/s), K= intercept coefficients for shallow concentrated flow Table (A1-12)

Channel flow Travel Time

When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning equation, assuming bank full conditions.

Channel length L=longitudinal slope $\times$ elevation change

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{0.5} \quad \text{(3.5)}$$

$$T_t = \frac{L}{60V}$$

Another alternative is to use Kirpich equation (1940) which is an empirical equation to calculate time of concentration of small drainage areas, given by:

$$T_t = 0.01947 L^{0.77} S^{-0.385} \quad \text{(3.6)}$$

S=overall catchment slope in m/m=(H2-H1)/L
L= the length of the catchment along the longest river channel

Rainfall Distribution

Regional rainfall time distributions should be developed for a 24-hour period. This period is chosen because of the general availability of daily rainfall data that are used to estimate 24-hour rainfall amounts. Normally a rainfall duration equal to or greater than T_c is used. Therefore, the rainfall distributions are designed to contain the intensity of any duration of rainfall for the frequency of the event chosen. That is, if the 10-year frequency, 24-hour rainfall is used, the most intense hour will approximate the 10-year, 1-hour rainfall volume. The Rainfall intensity is typically found from Intensity/Duration/Frequency curves for rainfall events in the geographical region of interest. A 10-yr, 25-yr, 50-yr, or even 100-yr storm frequency may be specified as shown in Figure 3.2. Example of rainfall analysis for El Managil Town is shown in Appendix 2

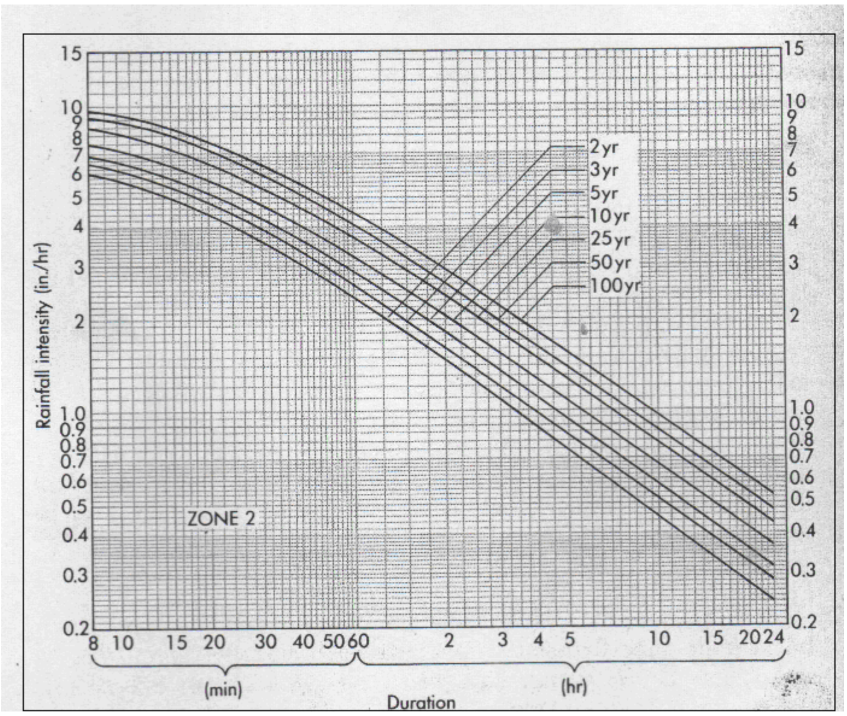


Figure 3.2: Typical Intensity-duration frequency curves for Florida region (Source:Weldon,1985).

Runoff Coefficient

The runoff coefficient defined as runoff divided by rainfall, varies with topography, land use, vegetal cover, soil type, and moisture content of the soil. In selecting the runoff coefficient, consider the future characteristics of the watershed. If land use varies within a watershed, one must consider watershed segments individually, and the can calculate a weighted runoff coefficient value (equation 3.7).

$$C = C_r + C_i + C_v + C_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.7)}$$

Where:

C_r a factor for land slope, C_i a factor for soil infiltration, C_v a factor for vegetal cover, and C_s a factor for soil depression

An example of runoff coefficients is listed in Table A1.1 (Appendix 1), showing values of runoff coefficients for Urban Watersheds.

Table A1.2 (Appendix 1) shows an alternate, systematic approach for developing the runoff coefficient. It applies to rural watersheds only, addressing the watershed as a series of aspects. For each of four aspects; C_r, C_i, C_v and C_s, make a systematic assignment of a runoff coefficient "component." Using Equation (3.7), add the four assigned components to form an overall runoff coefficient for the specific watershed segment.

Tables A1.1 and A1.2 apply to storms of two-year, five-year, and 10-year frequencies. Higher frequency storms require modifying the runoff coefficient because infiltration and other abstractions have a proportionally smaller effect on runoff. Adjust the runoff coefficient by the factor C_f as indicated in Table A1.3 (Appendix 1). The product of C and C_f should not exceed 1.0. The Rational formula now becomes:

$$Q = \frac{CC_f IA}{3.6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.8)}$$

Procedures for Rational Method

The following procedure outlines the rational method for estimating peak discharge:

1. Determine the watershed area (km^2).
2. Determine the time of concentration, with consideration for future characteristics of the watershed. e.g. Future changes in land use , vegetation cover or major development affects the time of concentration.
3. Assure consistency with the assumptions and limitations for application of the Rational Method.
4. Determine the rainfall intensity from intensity duration frequency curve or any developed formula for the geographical location to calculate the rainfall intensity (mm/hr), e.g., Figure (3.2).

5. Select or develop appropriate runoff coefficients for the watershed. Where the watershed comprises more than one characteristic, you must estimate C values for each area segment individually. You may then estimate a weighted C value using equation 3.9 below. Note that the runoff coefficient is a dimensionless quantity.

$$C = \frac{\sum_{n=1}^{m} C_n A_n}{\sum_{n=1}^{m} A_n} \qquad \text{(3.9)}$$

Where;

C = weighted runoff coefficient
n = n^{th} sub-area
m = number of sub-areas
C_n = runoff coefficient for nth sub-area; and
A_n = n^{th} sub-area size (km^2).

6. Calculate the peak discharge for the watershed for the desired frequency using equation 3.8.

An illustrative example of the rational method is given in Appendix (2)

3.2.2 Hydrograph Method

A discharge hydrograph is a continuous plot of instantaneous discharge vs. time. It represents the integrated effects of climate, hydrologic losses, surface runoff, interflow and groundwater flow. Base flow is separated and subtracted from the total storm hydrograph to derive the direct runoff hydrograph. Discharge from rainfall excess, after losses have been subtracted, makes up the direct runoff hydrograph. The direct runoff hydrograph represents the response of the watershed to rainfall excess, with the shape and timing of the direct runoff hydrograph related to duration and intensity of rainfall as well as storage factors (Figure 3.3).

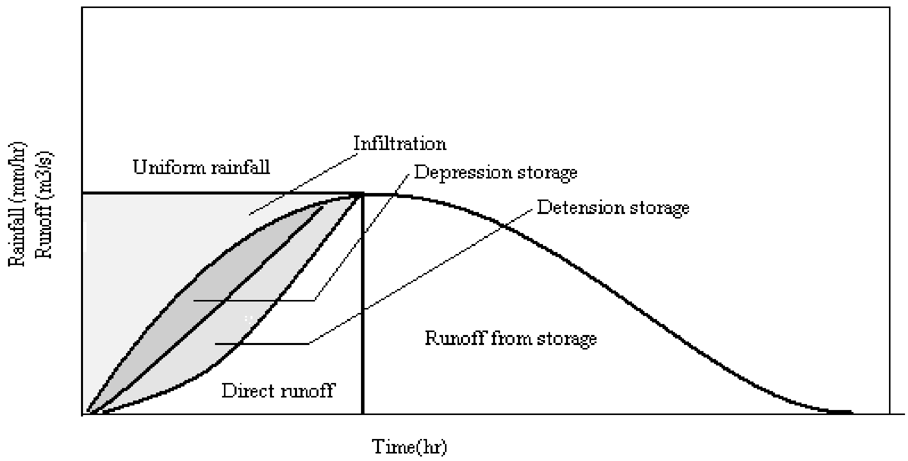


Figure 3.3: Runoff Hydrograph (Source: Hydrology and Flood Plain Analysis, 1988).

The unit hydrograph is defined as "a direct runoff hydrograph resulting from 1 inch (cm in SI units) of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration. The derivation of unit hydrograph is a well known approach in rainfall-runoff analysis.

The following ***assumptions*** are made when unit hydrograph concepts are applied:
- Constant rainfall intensity in time;
- Uniform distribution in space of the excess rainfall;
- The time base of the direct runoff hydrograph is independent of the rainfall intensity and depends only on the effective rainfall duration.
- Simultaneous observations of both precipitation and stream flow must be available.
- Watershed sizes should fall between 4-2500 km2
- Direct runoff should range from 12mm to 50mm.

Hydrograph Components:

Direct runoff
- Surface Runoff
- Interflow

Baseflow
- Delayed interflow
- Groundwater runoff

Direct runoff is usually considered to be the sum of surface runoff and interflow. Unit hydrograph analysis refers only to direct runoff.

Baseflow, or groundwater flow, is the flow component contributed to channel by groundwater. This process is extremely slow as compared to surface runoff.

Hydrograph time characteristics

1. Time to peak t_p : time from beginning of rising limb to peak discharge
2. Time of concentration t_c (or T_C): time required for the water to travel from the hydraulically remotest point in the basin to the basin outlet.
3. Lag time t_L: time between center of mass of effective rainfall and center of mass of direct runoff hydrographs
4. Time base t_b : duration of direct runoff hydrographs.

The time to peak is largely determined by drainage basin characteristics such as drainage density, slope, channel roughness, and soil infiltration characteristics. Rainfall distribution in space also affects the time to peak. (Bedient et.al,1988)

Baseflow separation

Baseflow separation is the process of separating the surface runoff from the baseflow. There are several subjective methods. The simplest one consists in arbitrarily selecting the beginning of the rising limb as the value of the baseflow and connecting this point with a horizontal line to a point in the recession limb of the hydrograph. Another method consists in arbitrarily selecting the beginning of the groundwater recession on the hydrograph and connecting this point with a straight line to the beginning of the rising limb. One last example of subjective methods consists in extending a line from the beginning of the recession to a point directly beneath the peak discharge and then connecting this point to the beginning of the rising limb.(Bedient et.al,1988)

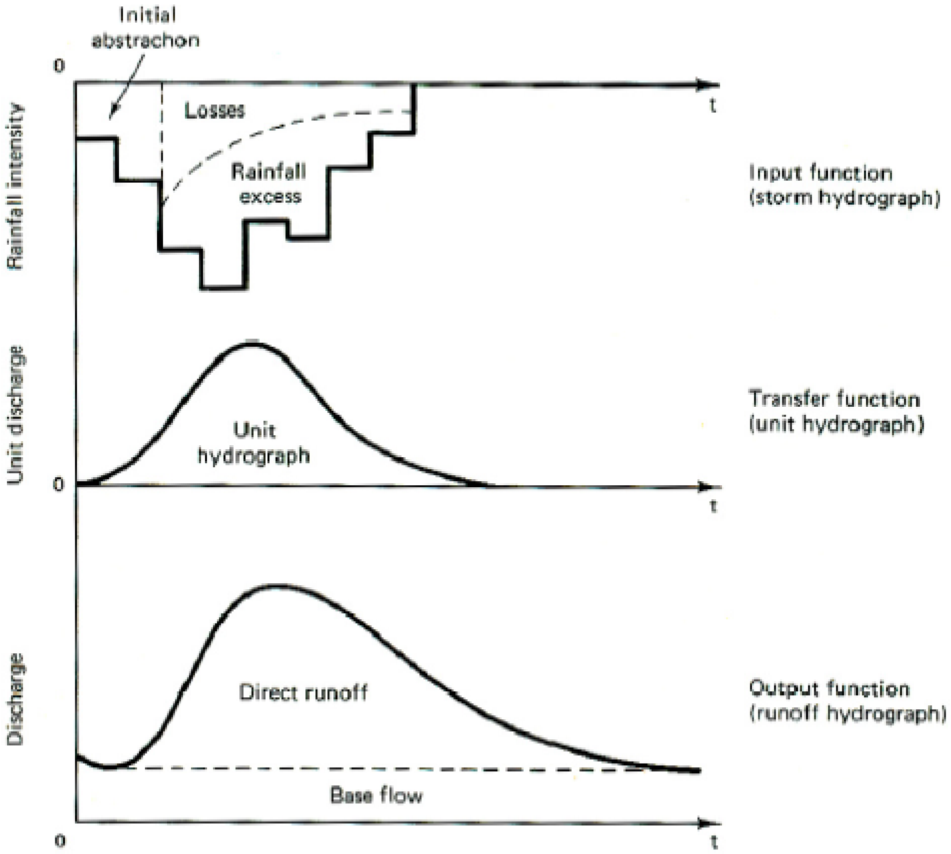


Figure 3.4 Relationships of storm, Unit and direct runoff hydrographs

Empirical unit hydrograph derivation

The following are essential steps in deriving a unit hydrograph from a single storm:

1) Separate the baseflow and obtain the direct runoff hydrograph.
2) Compute the total volume of direct runoff. Convert this volume into equivalent depth (in inches or in centimeters) over the entire basin.
3) Normalize the direct runoff hydrograph by dividing each ordinate by the equivalent volume (in or cm) of direct runoff (or effective rainfall).
4) Compute effective rainfall (e.g., using the Φ-index) and associated duration of the effective rainfall hyetograph. This duration is the duration associated with the unit hydrograph.

Unit hydrographs are intimately linked with the duration of the effective rainfall event producing them. They can only be used to predict direct runoff from storms of the same duration as that associated with the UH.

Unit hydrograph for different effective duration

A unit hydrograph for a particular watershed is developed for a specific duration of effective rainfall. When dealing with a rainfall of different duration a new unit hydrograph must be derived for the new duration. The linear property implicit in the UH analysis can be used to generate UH’s of larger or smaller duration. This procedure, which is explained using linear systems theory, is sometimes referred to as the S-curve Hydrograph method. The S-curve method allows construction of a unit hydrograph of any duration. Assume that a UH of duration D is known and that a UH for the same basin and of duration D' is desired. The first step is to determine the S-curve hydrograph by adding a series of UH's of duration D, each lagged by D. The resulting superposition represents the runoff resulting from a continuous rainfall excess of intensity 1/D.

By shifting the S-curve in time by an amount D' and subtracting ordinates between the two lagged S-curves, the resulting hydrograph must correspond to a rainfall event of intensity 1/D and of duration D'. Thus, to convert this hydrograph into a unit hydrograph of duration D', its ordinates must be normalized by multiplying them times D/D'. The resulting ordinates represent a unit hydrograph associated with an effective rainfall of duration D'.

Once a unit hydrograph has been derived for a catchment, it may be used to predict the surface runoff for any storm event by the process of convolution.

Use of Unit Hydrographs

The discrete convolution equation allows the computation of direct runoff on given excess rainfall P_m and the unit hydrograph U_{n-m+1}.

$$Q_n = \sum_{m=1}^{n \le M} P_m U_{n-m+1} \qquad \text{(3.10)}$$

The reverse process, called deconvolution, is needed to derive a unit hydrograph given data on P_m and Q_n. Suppose that there are M pulses of excess rainfall and N pulse of direct runoff in the storm considered; then N equation can be written for Q_n, n = 1,2,….,N, in terms of N – M + 1 unknown values of the unit hydrograph. If Q_n and P_m are given and U_{n-m+1} is required, the set equations is over determined, because there are more equations (N) than unknowns (N – M + 1).

$$\begin{aligned}
Q_1 &= P_1 U_1 \\
Q_2 &= P_2 U_1 + P_1 U_2 \\
Q_3 &= P_3 U_1 + P_2 U_2 + P_1 U_3 \\
Q_M &= P_M U_1 + P_{M-1} U_2 + \dots + P_1 U_M \\
Q_{M+1} &= P_M U_2 + \dots + P_1 U_{M+1}
\end{aligned}$$

An example of Computation of runoff using Hydrograph method is shown in Appendix(2)

3.2.3 Runoff Curve Number Methods (NRCS)

NRCS methods produce the direct runoff for a storm, by subtracting infiltration and other losses from the total rainfall. It is one of the techniques adopted by U.S. Department of Agriculture and Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS).

The primary input variables for the NRCS method are as follows:

- Drainage area size (A) in square miles (square kilometres)
- Time of concentration (T_c) in hours
- Weighted runoff curve number (RCN)

- Rainfall distribution
- Total design rainfall (P) in (millimetres).

The NRCS Rainfall Runoff equation represents a relationship between accumulated rainfall and accumulated runoff and is represented by:

$$R = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \ldots\ldots (3.11)$$

Where;
R = accumulated direct runoff (mm)
P = accumulated rainfall (potential maximum runoff) in mm for a 24-hour duration storm, for the relevant frequency (2, 5, 10, 25, 50, and 100-year frequencies).
I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff (mm)
S = potential maximum retention (mm) computed by equation 3.12 below:

$$S = Z\left(\frac{100}{RCN} - 1\right) \qquad \ldots\ldots (3.12)$$

Where;

Z	=	10 for English measurement units, or 254 for metric
RCN	=	runoff curve number described below.

Equation 3.12 is valid if S < (P-R). Therefore, this method is appropriate for estimating direct runoff from 24-hour or one-day storm rainfall.

Generally, I_a may be estimated as follows:

$$I_a = 0.2S \qquad \ldots\ldots (3.13)$$

Substituting this in Equation 3.11 gives:

$$R = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \ldots\ldots (3.14)$$

Runoff Curve Number (RCN)
Rainfall infiltration losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors known as runoff curve numbers**.** The higher the RCN, the higher the runoff potential. Tables A1.4 through A1.7 (Appendix 1) provide an extensive list of suggested runoff curve numbers. The RCN values assume medium antecedent moisture conditions (RCN II). If necessary, adjust the RCN for wet or dry antecedent moisture conditions. Use a five-day period as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period. For help determining which moisture condition applies, Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons shown in Table A1.8 (Appendix 1).

For expected dry soil condition use:

$$RCN\left(I\right)=\frac{4.2RCN\left(II\right)}{10-0.058RCN\left(II\right)} \qquad \text{(3.15)}$$

For wet soil condition use:

$$RCN\left(III\right)=\frac{23RCN\left(II\right)}{10+0.13RCN\left(II\right)} \qquad \text{(3.16)}$$

Soil Group

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. NRCS divides soils into four hydrologic soil groups based on infiltration rates (Groups A-D). Remember to consider effects of urbanization on soil groups as well.

Group A. Group A soils have a low runoff potential due to high infiltration rates even when saturated (0.30 in/hr to 0.45 in/hr or 7.6 mm/hr to 11.4 mm/hr). These soils primarily consist of deep sands, deep loess, and aggregated silts.

Group B. Group B soils have a moderately low runoff potential due to moderate infiltration rates when saturated (0.15 in/hr to 0.30 in/hr or 3.8 mm/hr to 7.6 mm/hr). These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures (shallow loess, sandy loam).

Group C. Group C soils have a moderately high runoff potential due to slow infiltration rates (0.05 in/hr to 0.5 in/hr or 1.3 mm/hr to 3.8 mm/hr if saturated). These soils primarily consist of soils in which a layer near the surface impedes the downward movement of water or soils with moderately fine to fine texture such as clay loams, shallow sandy loams, soils low in organic content, and soils usually high in clay.

Group D. Group D soils have a high runoff potential due to very slow infiltration rates (less than 0.05 in./hr or 1.3 mm/hr if saturated). These soils primarily consist of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, shallow soils over nearly impervious parent material such as soils that swell significantly when wet or heavy plastic clays or certain saline soils.

3.2.4 Graphical Peak discharge procedure

The procedure is used for relatively homogeneous watersheds with a maximum time of concentration of 10 hours (600 minutes) and runoff amounts more than 38 mm and runoff curve number more than 60. Additionally, the range of curve numbers should be small (say 20%) to reasonably conform to the assumption of homogeneity. Use the following procedure to determine the graphic peak discharge:

1. Determine the drainage area (A) (square kilometres).
2. Determine the soil classification based on runoff potential (Type A, B, C, or D).
3. Determine the antecedent soil moisture conditions (AMC).

4. Classify the hydrologic condition of the soil cover as good, fair, or poor. For Urban Areas, Cultivated Agricultural Land, Other Agricultural Lands, and Arid and Semi Arid Rangelands (Table A1.4 through A1.7 of Appendix 1).
5. Determine the RCN for the AMC II soil classification. If appropriate, adjust for AMC I or AMC III using Equation 3.15 and 3.16, respectively. If necessary, determine a weighted value by dividing the sum of the products of the subarea sizes and RCNs by the total area. This process is similar to the weighting of runoff coefficients in the Rational Method.
6. Estimate the watershed time of concentration in hours (T_c).
7. Determine the potential maximum storage (S). Use equation 3.12 to calculate the potential maximum storage.
8. Determine the initial abstraction (I_a). These are the losses that occur before runoff begins and include depression storage, interception, and infiltration. Use equation 3.13 to calculate I_a.
9. Determine the rainfall distribution type based on the location of the watershed.
10. Determine the total rainfall (P) for watershed location.
11. Determine the accumulated direct runoff. Use Equation 3.14 to compute R. This value, when multiplied by the watershed area, will indicate the total volume of the rainfall that appears as runoff.
12. Determine the unit peak discharge using equation (3.17) and Table (A1.9) with the relevant distribution type from step 10 to determine the unit peak discharge (q_u) using time of concentration (T_c) and the ratio I_a/P.

$$q_u = \left(10^{C_0 - 3.36609}\right)\left(T_c^{\,C_1 + C_2 \log T_c}\right) \qquad \text{.................................(3.17)}$$

Where:

q_u = unit peak discharge (cfs/sq.mi./in. or $m^3/s/km^2/mm$)
T_c = time of concentration (hours)

13. Determine the pond adjustment factor (F). Use Table (A1.10) to determine F (Appendix 1). This adjustment is to account for pond or swamp areas within the watershed that do not interfere with the time of concentration flow path.
14. Compute the peak discharge (Q) using:

$$Q = q_u ARF \qquad \text{..(3.18)}$$

Where;

Q is the peak discharge (cfs or m^3/s)
q_u is the unit peak discharge (cfs/sq.mi./in. or $m^3/s/km^2/mm$) from step 12
A is the drainage area (sq.mi. or km^2) from step 1
R is the runoff volume (in. or mm) from step 11
F is the ponding factor from step 13

3.2.5 SCS dimensionless hydrograph

The SCS dimensionless hydrograph is a synthetic UH in which the discharge is expressed by the ratio of discharge, Q, to peak discharge, Q_p and the time by the ratio of time, t, to time to peak of the UH, t_p. Given the peak discharge and the lag time for the duration of the excess rainfall, the UH can be estimated from the synthetic dimensionless hydrograph for the given basin. The dimensionless UH has been obtained from the UH's for a great number of watersheds Figure 3.5

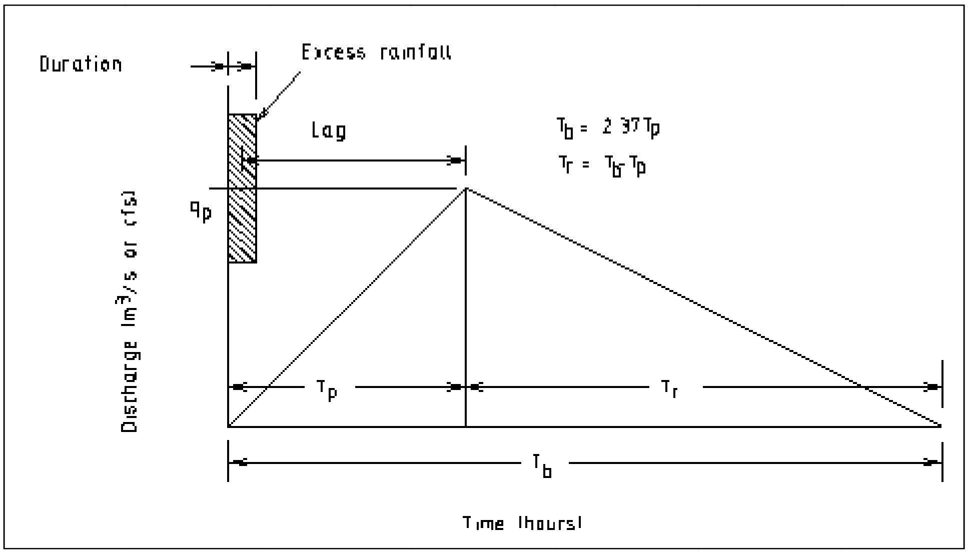


Figure 3.5 Dimensionless triangular unit hydrograph

Peak flow: $Q_p = \dfrac{CA}{t_p}$ ... ……………………………………(3.19)

Time to peak: $t_p = \dfrac{D}{2} + t_l$ ……………………………… ……(3.20)

Lag time: $t_l = \dfrac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.3}}$ ……………………………………(3.21)

Time base: $t_b = 2.67t_p$ ……………………………………(3.22)

The values of Q_p and t_p. can be estimated using a simplified model of a triangular unit hydrograph whose height is equal to Q_p and whose base (time base, t_b) is equal to 2.67 t_p. The time is usually expressed in hours (SCS), and the discharge in m^3/s/cm (or cfs/in). The recession duration of 1.67 t_p is suggested by the SCS after analysis of a great number of UH's.

C = 2.08 (483.4 in the English system) and A is the drainage area in square kilometers (square miles). D is the duration of rainfall.

In the above expression for basin lag in hours, *l* is the length to the divide in ft, Y is the average watershed slope in percent, and S is the potential maximum retention in in. S = (1000/CN)-10 and CN is the Curve Number for various soil/land use (see Table (A1-4) through (A1-7) Appendix 1)

3.3 Time series analysis

A set or a series of any data (sequenced) describing the pattern of a certain phenomena, e.g. river water levels and flows, daily temperature at a certain locality, etc., within a time span is known as the Time Series (TS). Analyses of such time series is essential for identifying the nature of the phenomenon (trends) and hence predicting future values of the time series variables. In case of flood embankment design, operation and maintenance, such predicted values of yield and hence river water levels are very important and useful to look at.

Time series analysis requires consistent and homogenous set of observations (data). According to Min (2006), detection of inconsistency or non homogeneity in the observation series commonly employs statistical tests, either parametric or non-parametric. The choice between the two families of tests is based on the expected distribution of the data involved. If data is normally distributed, parametric tests are preferred. Hydrological data is often not normally distributed (Helsel and Hirsch, 2002), the non parametric test will be used. Transformations are sometimes used to make data more normally distributed, prior to performing a parametric test. The advantages and disadvantages of both parametric and non parametric tests can be found in the literature.

Serial correlation test is one of the important test to identify whether the TS is dependent or independent one. In case of dependent TS, the well-known autoregressive family of models is preferable in describing the process (e.g ARIMA), while the statistical distribution models such as normal, Gumbel, Pearson, etc are suitable ones to select for independent time series.

The literature shows many software's for time series analysis. One of the useful tool for testing the homogeneity, consistency and independence of hydrological time series data is Spell-Stat software, developed by Jorge GUZMAN and CHU, Universidad Industrial de Santander, Bucaramanga, Colombia. The Spell-Stat can perform the following:

- determination of the basic statistics of the TS (mean, variance, skewness and kurtosis);
- plotting of frequency distribution (histograms);
- statistical distribution fitting (the normal distribution, the general extreme value distribution and POT (exponential distribution) or partial duration series);
- statistical tests:
    - Non-parametric test (Standard Normal Homogeneity Test (SNHT)); and
    - Parametric test (Spearman’s rank correlation test, Pettitt test, F-test and t-test).
- Development of correlogram and periodogram.

The Tables A3.1 through A3.4 in Appendix 3 give the basic long-term statistical records at some of the gauging sites on the Blue Nile, the White Nile, the Atbara River and the Main Nile, respectively, for the period 1965 to 2000. On the other hand, the minimum and maximum recorded stages throughout the last four decades are also presented in Table A3.5 as given in MoIWR records.

3.3.1 Flood Frequency Analysis

Estimation of peak floods is essential for the design of the flood embankment. This necessitates the availability of considerable flow discharge records from well-representing sites. The ministry of Irrigation and water Resources is the responsible body for monitoring, development and assessment of the flow data, while the Meteorological Authority is responsible for rainfall data within the country.

For gauged catchments with long records (e.g. greater than 25 years) the techniques of flood frequency analysis may be applied directly to determine the magnitude of any flood event (Q) with a specified return period (T). Therefore, the main task of the Flood Frequency Analysis (FFA) is to estimate a flood magnitude (Q) corresponding to any return period (T) of occurrence, i.e. to establish the Q-T relationship.

The FFA methods are statistically based. In these methods, the prediction for the future floods is made on the basis of available records of the past floods. These methods can be safely used to determine the maximum flood that is expected on a river with a given frequency of occurrence. There are many different methods among such category. Some of these are discussed briefly in the following sub-sections.

Probability plotting on Empirical relations likes California Formula:

$$E = m/N \quad \text{……………………………………………… 3. 23}$$

$$T = (1/p) = N/m \quad \text{………………………………………… 3. 24}$$

Where;

E	is the exceedance probability;
T	is the recurrence interval;
m	is the ranking number; and
N	is the number of floods (data point).

After the values of recurrence interval (T) for different floods are calculated, a graph can be extended if desired, as to extrapolate the value of flood magnitude corresponding to any high value of frequency. Beside that flood discharge for any other frequency can also be determined from the plotted graph.

Theoretical Probability Distribution Methods

This method used for larger extrapolations. Thus, for determining extreme flood events, specific extreme value distributions are assumed, and the required statistical parameters are determined from the available data, from which the flood magnitude (Q_T) for any specific return period (T) can be determined. Chow (1951) has shown that most frequency–distribution functions applicable in hydrologic studies (Normal, Gumbell, Pearson, Lognormal, etc.) can be expressed by the general equation of hydrologic frequency analysis Eq. (3.15):

$$Q_T \quad = \quad Q^{-} + K_T\sigma \quad \text{……………………………………………… 3. 25}$$

Where;

Q_T	is the peak discharge for return period (T);
Q^{-}	is the mean discharge ($\sum Q/N$);
K_T	is the frequency factor; and
σ	is the standard deviation of discharge

In case of Gumbel's distribution:

$\sigma \quad = (y_T - \bar{y}_n)/S_n.$
$y_T \quad = -\{\ln \ln(T/(T-1))\};$
$\bar{y}_n$ is a tabulated reduced mean with a maximum value of 0.57, depending on N; and
S_n is a tabulated standard deviation with a maximum value of 1.2825, depending on N.

To avoid large scale calculations, the (K_T) values are usually tabulated or they can be found from plotted curves. Such tables and curves are available in most of stochastic hydrology books and published journals.

Steps for FFA

- Identify the maximum flood series (one value every year) from the available time series (daily) records;
- Carry statistical analysis to determine the mean, Q- (= $\sum Q/N$) and the standard deviation, σ, of discharge;
- Select alternative frequency distribution functions;
- Test the fitting of each distribution to the available data and select the best fitted one;
- Use the K_T-T relationship table/curve and equation 3.25 to establish the Q-T relationship curve;
- Use the Q-T curve as a design tool.

Application Example

In this example the daily discharge (Time series) at the different gauge sites on the river Blue Nile, for the period 1965-2000, was used. The annual maximum flood peak series was identified for each gauging site.

Table 3.1 below gives the basic statistical parameters of the flows computed at the gauging sites. The Pearson Type III frequency distribution function was selected and the discharges for different return periods are given in tables 3.2.a & 3.2.b (Values of K_T for Pearson Type III are taken from the hydrology references).

Table 3.1: Statistical parameters related to flood frequency analysis (1965-2000).

Gauging Site	**Mean Discharge (Mm^3/day)**	**Standard Deviation (Mm^3/day)**	**Skewness Coefficient**
The Blue Nile System			
Eddeim	668	138	0.09
Roseires D/S	611	105	0.34
Sennar D/S	584	112	0.38
Khartoum	637	151	0.47
Gwasi @ Dinder River	048	014	0.83
El Hawata @ Rahad River	014	002	0.27

Table 3.2.a: Magnitude of discharges corresponding to various return periods (1965-2000).

Return Period (Years)	*Eddeim*		*Roseires D/S*		*Sennar D/S*		*Khartoum*	
	K_T	Flow (Mm^3/ day)	K_T	Flow (Mm^3/ day)	K_T	Flow (Mm^3/ day)	K_T	Flow (Mm^3/ day)
2	-0.02	666	-0.06	605	-0.06	577	-0.08	625
5	0.84	784	0.82	697	0.82	675	0.81	759
10	1.29	847	1.31	748	1.32	731	1.32	836
25	1.78	914	1.84	804	1.83	788	1.89	921
50	2.10	959	2.23	844	2.25	835	2.30	983
100	2.39	999	2.57	880	2.60	874	2.66	1038
200	2.66	1036	2.89	914	2.93	911	3.01	1091
1000	3.22	1113	3.58	986	3.64	989	3.77	1205

Table 3.2.b: Magnitude of discharges corresponding to various return periods (1965-2000).

Return Period (Years)	*Gwasi*		*El Hawata*					
	K_T	Flow (Mm^3/ day)	K_T	Flow (Mm^3/ day)				
2	-0.14	46	-0.04	14				
5	0.78	59	0.83	16				
10	1.34	67	1.31	17				
25	2.00	76	1.84	18				
50	2.47	82	2.20	19				
100	2.91	89	2.52	19				
200	3.34	95	2.83	20				
1000	4.29	108	3.48	21				

3.4 Hydrological Modelling

Broadly speaking, the hydrological models serve the following purpose (Dingman, 2002; Agor, 2003):

- Management tool: To assess the impact of human interventions on hydrological systems;
- Predictive tool: To predict the (near-) future state of hydrological systems; and
- Research tool: To gain and encapsulate knowledge on the hydrological processes.

Models can be either deterministic or stochastic. In deterministic models, each parameter has one specific value, which means for two equal sets of input the model will generate the same output, provided that initial conditions are the same. In stochastic models on the other hand, some parameter values are randomly generated, so in general two equal sets of input lead to different output. Furthermore, models are labelled either as distributed or lumped. Lumped models use parameters which represent the "average value" of entire catchments, whereas distributed models take spatial variability into account by dividing the catchments into a number of elements.

Models can also be classified according to what extent the model takes into account the physics that occur in the catchments:

- Black box: Model relating input and output series without consideration of the physics involved in the hydrological system of interest.
- Conceptual models: Models using process simulation techniques and parameters which are physically meaningful, but mostly not directly measurable and often they are lumped on the catchment scale.

Physically based distributed models, are models based on physical laws, such as the conservation laws of mass, momentum and energy. The distributed characters of these models enable the parameters to be directly measurable and the equations to be solved by numerical solution technique.

3.5 Un-Gauged Catchments

Due to financial and some technical limitations, missed data and un-gauged sites may exist within the flow records of a certain system. Regional flood frequency analysis is one of the reasonable solutions for flood frequency estimation in such un-gauged catchments.

It is worth to mention that the regional flood frequency curves for some identified homogeneous Nile Basin regions, which developed by Wiellems P. et al (2006) within Phase I of the FRIEND/NILE project, can be applied for un-gauged regions within the basin. Regression analysis was applied to develop a relationship between the mean annual flood and catchment characteristics. The developed regional frequency curves Fig(3.6) and regression models can be applied for the estimation of floods with various return periods in un-gauged catchments homogeneous with the regions under consideration. Therefore the best relation considered suitable to the eastern Nile Rivers could be given by:

$$MAF = 2\mathrm{E} - 0.7(A \times MAR)^{1.4529} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.26$$

Where:

MAF= Mean annual flood (m^3/s)
A=Catchment area (km^2)
MAR=Mean annual rainfall (mm)

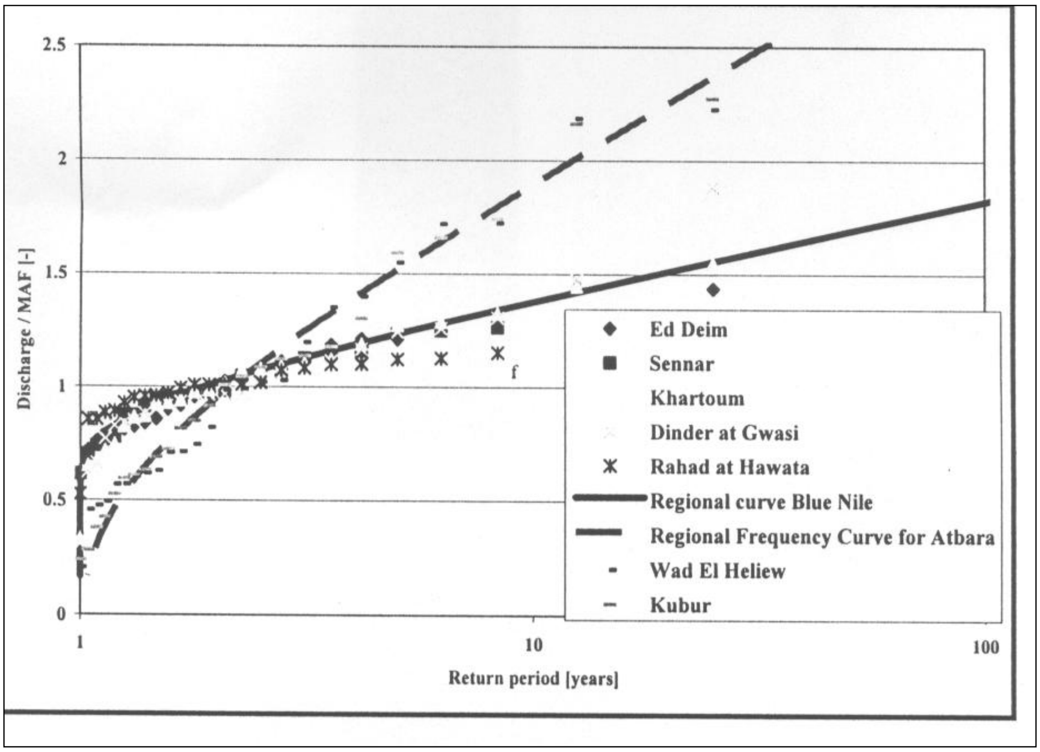


Figure (3.6): Pooled regional data and regional frequency curves. Source(Flood Frequency Analysis of the eastern Nile Rivers/ FRIEND NILE Project,2006)

In case of wadi hydrology and the absence of long record, the maximum peak discharge could be estimated using the Manning formulae as followed:

Step 1: Identify the high water mark (Hw) from the surrounding environment;
Step 2: Carry land survey to establish the average cross section up to the high water mark.
Step 3: Determine the wetted area (A) and the hydraulic radius (R).
Step 4: Estimate the longitudinal slope of the channel by selecting reasonable straight reach;
Step 5: Estimate the manning roughness coefficient (n);
Step 6: Apply Manning formulae to estimate the peak discharge ($Q = \{(1/n)AR^{2/3}S^{1/2}\}$

3.6 Flood routing

Flow routing is a procedure to determine the time and magnitude of flow (i.e., the flow hydrograph) at a point on a watercourse from known or assumed hydrographs at one or more points upstream. If the flow is a flood, the procedure is specifically known as flood routing. In a broad sense, flow routing may be considered as an analysis to trace the flow through a hydrologic system, for the given input.

The hydrologic analysis of problems such as flood forecasting, flood protection, reservoir design and spillway design invariably include flood routing. In these applications two broad categories of routing can be recognized. These are; Reservoir routing and Channel routing.

A variety of routing methods are available and they can be broadly classified into two categories. These are; hydrologic routing and hydraulic routing. The hydrologic routing methods employ essentially the equation of continuity. On the other hand, the hydraulic methods employ the continuity equation together with the equation of motion of unsteady flow.

3.6.1 The Reservoir routing

For a hydrologic system (a reservoir), the mathematical relationship between the amount of inflow (input) I(t), the outflow (output) Q(t) and storage S(t) at any time (t) is given by the following equation (if all possible losses are negligible):

$$dS/dt = I(t) - Q(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.27)$$

If the inflow hydrograph, I(t), is known, Eq. (3.27) can not be solved directly to obtain the outflow hydrograph, Q(t), because both Q and S are unknown. A second relationship, or storage function, is needed to relate S, I, and Q; coupling the storage function with the continuity equation provides a solvable combination of two equations and two unknowns.

The specific form of the storage function to be employed depends on the nature of the system being analyzed. The general form, in case of a reservoir is as follows:

$$S = f(Q) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.28)$$

Alternatively, in a small time interval Δt the difference between the total inflow volume and total outflow volume in a reach is equal to the change in storage in that reach.

$$\bar{I}\Delta t - \overline{Q}\Delta t = \Delta S \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.29)$$

Where;

$\bar{I}$ = average inflow in time interval Δt;
$\overline{Q}$ = average outflow in time interval Δt; and
Δ S = change in storage.

By taking $\bar{I} = (I_1 + I_2)/2$, $\overline{Q} = (Q_1 + Q_2)/2$ and $\Delta S = S_2 - S_1$ with suffixes 1 and 2 to denote the beginning and end of time interval Δt Eq. (3.27) is rewritten as follows:

$$[(I_1 + I_2)/2]\Delta t - [(Q_1 + Q_2)/2]\Delta t = S_2 - S_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.30)$$

The time interval Δt should be sufficiently short so that the inflow and outflow hydrographs can be assumed to be straight lines in that time interval. Further Δt must be shorter than the time of transit of the flood wave through the reach.

3.6.2 Hydrologic channel routing

In channel routing the storage is a function of both outflow and inflow discharges. The total volume in storage within a channel reach can be considered as a prism storage plus wedge storage (Fig. 3.7). The prism volume that would exist if uniform flow occurred at the downstream depth, i.e. the volume formed by an imaginary plane parallel to the channel bottom drawn at the outflow section water surface, while the wedge storage is the wedge-like volume formed between the actual water surface profile and the top surface of the prism storage.

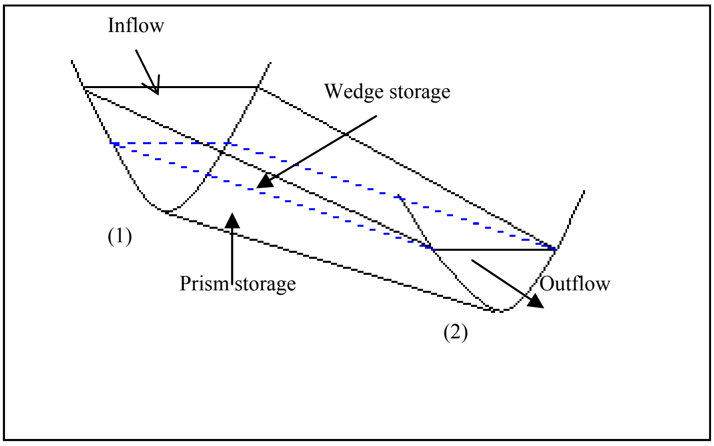


Figure 3.7: Storage in a channel reach.

The most widely used method of hydrological stream routing is the Muskingum method originated by McCarthy in 1938. The routing equation of this method is described as follows:

$$Q_{i+1} = C_o I_{i+1} + C_1 I_i + C_2 Q_i \qquad (3.31)$$

Where;

$$C_o = (-Kx + 0.5\Delta t)/(K - Kx + 0.5\Delta t)$$
$$C_1 = (Kx + 0.5\Delta t)/(K - Kx + 0.5\Delta t)$$
$$C_2 = (K - Kx + 0.5\Delta t)/(K - Kx + 0.5\Delta t)$$

K = proportionality coefficient
X = weighting factor
Δt = routing interval

Note that $C_o + C_1 + C_2 = 1$

The above equation is known as Muskingum routing equation and provides a simple linear equation for channel routing. It has been found that for best results the routing interval Δt should be so chosen that K > Δt > 2Kx. If Δt < 2Kx, the coefficient C_0 will be negative. Generally, negative values of coefficients are avoided by choosing appropriate values of Δt.

3.7 The Easter Nile Sub-System

The Blue Nile sub-system consists of the Blue Nile river and its two major tributaries; namely; Rahad and Dinder Rivers. This sub-system is represented by the following discharge gauging sites:

1. Eddeim;
2. Roseires Dam;
3. Sennar Dam;
4. Wad Medani;
5. Khartoum;
6. Gwasi on the Dinder River; and
7. El Hawata on the Rahad River.

The White Nile Sub-System d/s Malakal. Note that, the Baro-Akobo-Sobat is part of the Eastern Nile.

1. Hillet Dolieb
2. Malakal; and
3. Jebel Aulia Dam.

The Atbara River Sub-System:

1. Wad el Hiliew;
2. Kubur;
3. Khashm el Girba Dam; and
4. Atbara at K.3 from mouth.

The Main Nile Sub-System u/s Dongola:

1. Tamaniat;
2. Shendi;
3. Hassanab;
4. Atbara;
5. Dongola.

Figure 3.8: Main gauging Stations at the Nile River within Sudan.

During flooding season (Kharief), flow levels are monitored every two hours at all gauging sites (MoIWR). Rainfall data is also monitored (MA). Some important Hydrological parameters- river Nile and its tributaries are shown in Appendix 3.

It is evident that the Blue Nile river is the main contributor to flooding in Sudan. It originates from Lake Tana in Ethiopia, its basin lying on the Northern side of the Ethiopian plateau. The average contribution of Lake Tana to the Blue Nile is about 4 Billion cubic meter annually which is approximately 7% of the total flow of the Blue Nile. On its way to join the White Nile, the Blue Nile meets the Dinder and Rahad rivers inside Sudan. Its average annual contribution to the main Nile is 59% while during flood time the percentage changes appreciably to 68%. The Blue Nile average seasonal variation at Roseires for the period 1911-1962 ranges from 125 m3/sec to 6300 m3/sec.

Some other hydrological characteristics of the river are:

Average annual flow	= 50 billion m^3
Average flood flow	= 39 billion m^3
Average flood discharge(10 days average)	= 6300 m^3/sec
Yearly average discharge	= 1590 m^3/sec
Monthly average peak discharge	= 5840 m^3/sec
Monthly average peak sediment concentration	= 3.9 g/l
Monthly average peak sediment load	= 23 tons

The flow of the Blue Nile river is measured at El Diem about 100 km upstream Roseires reservoir. The features of the recent floods are:

- Most floods are caused if the reading at El Diem gauge station is > 12.25m for a period of more than 2 days.
- Floods can be mitigated by Rosieres dam even if El Diem gauge station readings > 13.0 m, but for a period that does not exceed one day.
- The operation of Jebel Aulia reservoir during the flood period has a great effects on the water level fluctuations of the area downstream the dam to Mogran.
- For the rejoins downstream Atbara city, floods will occur if the release downstream Khashm El Girba reservoir exceeds 300 Mm^3/day together with a discharge > 700 Mm^3/day of the Blue Nile.

Figure 3.8 below shows the gauging sites on the River Nile and its tributaries within Sudan, which are summarized as follows:

3.7.1 Travel time of flood wave

The flow time of travel from one point on a stream to another can be deduced from the flow distance and velocity (celerity of flood wave). The following table presents the approximate times of travels from a gauging site to the next along the Blue Nile as published in "The Nile Basin, vol. VII, 1946".

Table (3.3): Approximate Time of Travel along the Blue Nile River.

Month		days		days		Days	
January		1½		4		5	
February		1¾		4½		5½	
March		2		5		6	
April		2		5½		6½	
May		2		5		6	
June	Eddeim	1¾	Roseires Dam	4½	Sennar Dam	5½	Khartoum
July		1½		3½		4½	
August		1		2		3	
September		½		1		2	
October		¾		1½		2½	
November		1		2½		3½	
December		¾		3		4	

These figures can be taken only as indications of the average times of travel of fluctuations in flows. Actual times of travel in individual cases may vary appreciably from these, according to conditions at the time, particularly those of the magnitudes of the flows and whether they are rising or falling.

3.8 Flood Early Warning Systems

In the Sudan traditionally flood protection work was not considered as an important activity both for governments and local inhabitants until the devastating flood of 1988. An Emergency Flood Reconstruction Program was developed soon after the high flood of 1988. The Ministry of Irrigation and Water Resources (MoIWR) involved in this program by developing a very sophisticated flood early warning system (FEWS) as well as on many flood protection works.

The World Bank has funded flood forecasting work on the Blue Nile undertaken by Delft Hydraulics, in association with the Sudan Water Resources Department (Grimes et al, 1993; El Amin El Nur et al, 1993). An 11 region distributed model of the Blue Nile was developed, based on the Sacramento Watershed model (Grijsen et al, 1992). The name of this forecasting system is FEWS. Sudan Flood Early Warning System (FEWS) is in operation since August 1992, providing an advanced operational tool that gives three days ahead the water levels and flows at El Deim, using remote sensing and hydrologic modelling techniques. However there is a new version of FEWS software mor advanced that is not yet available in the Sudan.

The old FEWS which is used in Sudan encounters some problems nowadays like:

- The bathymetric survey has been done in 1991 to support the development of the FEWS model. Two bathymetric surveys (on reservoirs) have been done after that and none of them included in FEWS model.
- The capacity of Roseris Dam and Sinnar Dam were changed because of the great amount of sediment that comes from the upper catchment during the flood.
- Changes of dams’ operation rules experienced since 1991, not updated to the model.
- New dams were not included to the model, e.g., Merowe.
- The most important input after the development of the model is the satellite data of cold cloud duration (Meteosat). However, the access to the CCD images is interrupted frequently.

The engineers in the General Directorate of the Water Resources in the Ministry of Irrigation and Water Resources are doing a great effort to make FEWS running. As a result there is need for a new model to predict the flood, or upgrade the current model.

4 Hydraulic and Geomorphologic Considerations

The hydraulic chapter covers, definition of key hydraulic parameters that are required for the design of flood embankments, including: layout, foundation, crest and batter treatment, as well as related morphological changes of scouring and sedimentation. It is essential to provide the correct information of the hydraulic parameters for design of flood embankments in order to:

- Serve the purpose they are constructed for;
- Avoid unnecessary construction cost due to over design; and
- Avoid embankment failures due to water flow.

4.1 Flood Embankment

Flood embankments are characterised as:

- Flood embankments are the oldest and most frequent type of flood protection works.
- They can be constructed on one or both sides of the river.
- Their primary purpose is to furnish flood protection from seasonal high flood water
- They are subject to high water for only a short period of time
- Fill material is usually (poorly graded) obtained from adjacent borrow pits.

There are different types of levees and embankments, and are broadly classified according to location, the area they protect, or the use. e.g., urban or agricultural levees because of different requirements for each. Levee types according to use are given in Table 4.1.

Table (4.1): Classification of Levees According to Use. (Source: US Army Corps of Engineers, 2000)

Type	Definition
Mainline and tributary levees	Levees that lie along a mainstream and its tributaries, respectively.
Ring levees	Levees that completely encircle or “ring” an area subject to inundation from all directions.
Setback levees	Levees that are built landward of existing levees.
Sub-levees	Levees built for the purpose of under-seepage control. Sub-levees encircle areas behind the main levee which are subject, during high-water stages, to high uplift pressures and possibly the development of sand boils.
Spur levees	Levees that project from the main levee and serve to protect the main levee from the erosive action of stream currents. Spur levees are not true levees but training dikes.

Embankments types according to fill material*:*

1. Homogeneous embankments
Composed of one kind of sufficiently impervious material, Figure (4.1)

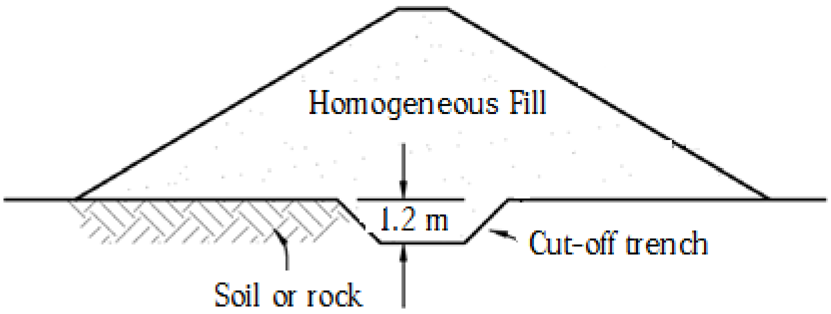


Figure 4.1: Typical homogenous embankment

Zoned embankments contain a central impervious core, flanked by zones of more pervious material, called shells. See Figure (4.2)

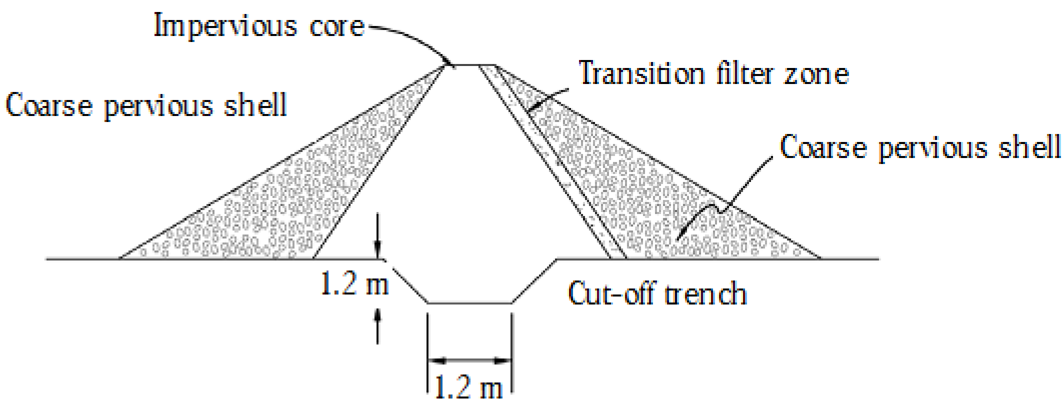


Figure 4.2: Zoned embankment

4.2 The Design of flood embankments

Many factors must be considered in the design of flood embankments. The general steps for designing a levee embankment are given in Table A4.1 of Appendix 4.

4.2.1 Hydraulic parameters

The hydraulic and geometric parameters required for the design of flood embankments are:

A. Flow characteristics
   - bed roughness (bed forms)
   - Bed lope
   - turbulence
   - flow velocity
   - hydraulic radius

B. Sediment characteristics
    fall velocity of the bed and bank material
    grain size of bed-load and suspended sediment
    transport of sediment

C. seepage forces in the bed and the bank
D. the longitudinal profile of the river channel
E. meander properties if any
F. the shape of the cross-section (width, depth, side slopes)

4.2.2 Hydraulic design

Embankment hydraulic design is the dimensioning of the embankment based on the river's hydraulic parameters.

Step 1: Determine the design flood water height H:

Using the design discharge Q_T (e.g. Q_{100}) computed in chapter 3 and the hydraulic and geometric characteristic of the river at the site, compute the flood height H. For wide streams or rivers:

$$Q_T = \frac{1}{n} \times B \times H^{5/3} \times \sqrt{S} \quad \text{.........(4.1)}$$

Where;
B= stream width
n= Manning's coefficient of roughness (can be estimated from Table 4.4 below).
S= stream bed slope

Step 2: Determine the freeboard F:

The freeboard is defined as the vertical distance between the crest of an embankment and the maximum flood level. It protects embankments and dams from overflow caused by wind-induced tides and waves. Depending on the importance of the structure, the amount of freeboard will vary in order to maintain structural integrity and the estimated cost of repairing damages resulting from overtopping.

Freeboard is generally estimated based on maximum probable wind conditions. Three basic considerations are generally used in establishing freeboard allowance viz. wave characteristics, wind setup, and wave run-up. The minimum height of the freeboard for wave action is generally taken as $1.5h_w$, where h_w is given by:

$H_w=0.032\sqrt{(V.F)} + 0.763 - 0.271\,(F)^{3/4}$ for F<32 km and,
$H_w=0.032\sqrt{(V.F)}$ for F>32 km

Where:
hw=height of water from top of crest to the bottom of trough in meters
V= wind velocity in km/hr

F= Fetch or straight length of water expansion in km

A freeboard of 1.0 to 1.5 m is usually used.

The height of the embankment H_T= H + F.

Step 3: Determine the embankment side slopes V:H.

For levees that do not exceed height of 4 m, and have no particular foundation problem, the recommended river-side slope is 1:3 – 1:3.5, and land-side 1:2 – 1:2.5. The recommended values of side slopes for earth dams given by Terzaghi and shown in Table (4.2) below can be used for flood embankments.

Table (4.2): Recommended side slopes for earth dams (Source: Garg, 2005)

Type of Material	U/S (V:H)	D/S (V:H)
Homogeneous well graded	1:2.5	1:2
Homogenous course silt	1:3	1:2.5
Homogenous sillty clay		
Height less than 15m	1:2.5	1:2
Height more than 15m	1:3	1:2.5
Sand or sand and gravel with central clay core	1:3	1:2.5
Sand or sand and gravel with R.C. diaphragm	1:2.5	1:2

Step 4: Determine the seepage line V:H

The seepage line depends on the soil type and its compaction. It starts 0.3 m above the design water level with the following slopes:

Clayey Soil 1V:4H
Clayey sand 1V:5H
Sandy Soil 1V:6H

The Sudan MoIWR experience on seepage line is:

Central Sudan (Gezira Canals) 1V:7H
Eastern Sudan (Gash training embankments) 1V:6H
The embankment section should be designed in such a way to keep the seepage line inside the body of the embankment well below the top surface of the embankment.

In northern Sudan states a seepage line of 1:5 is used with no failure for many years. This is attributed to the soil type and wetness conditions (intermittently wet).

Step 5: Determine the embankment Crest width B

Embankment crest width (B) = road width on embankment crest
Embankment crest width (B) = width necessary to keep the seepage line well within the embankment at the design water level.
Whichever is large (usually 2.5-5 m)

Table (4.3): Recommended levee crest width

Embankment Height (m)	Crest width (m)

4.2 or less	2.4
4.5 -5.7	3.0
6 – 7.2	3.6
7.5 or more	4.5

The crest width should be increased till the seepage line lies well inside the body of the embankment and passes below the top surface of the embankment.

In designing flood embankments wetness conditions are very important. Two types should be distinguished viz. always wet or intermittently wet. The differences in the design of the two types are summarised in the following paragraphs.

a. The embankment is always wet:
Should be design as an earth dam embankment;

1. Design the embankment using seepage line slope as recommended by MoIWR (1V:7H);
2. Design an upstream filter for relieving the pore pressure;
3. Downstream filter is optional;
4. Diaphragm embankments with clay core can be used.

b. Embankments subject to wetness intermittently

1. Seepage failure is rare. Therefore they can be designed using textbooks seepage line slope (clay1:4, clayey sand 1:5 and sandy soils 1:6). In the northern states of Sudan experience showed seepage line 1:5 is used without failure for many years.
2. Use homogenous soils;
3. Avoid the use of clay cores as the clay will crack when dry leading to embankment failure
4. Upstream filters are not necessary.

In designing flood embankments in urban areas where space limitations exists, steeper slopes can be used provided that adequate slope protection is used.

Seepage cut-off walls can also be used. Stability berms can be used to avoid very steep side slopes for more economic design.

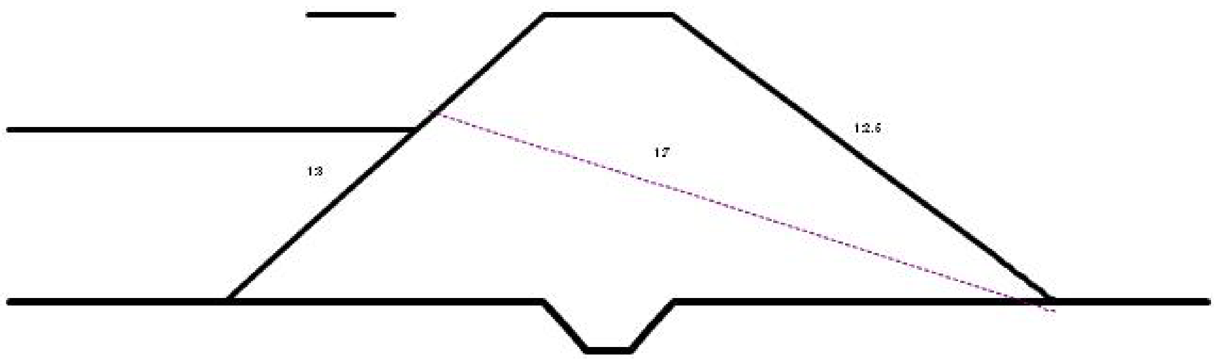


Figure (4.3): A typical Flood Protection Embankment for Abbassia (Kordufan, Sudan)

{Crest width B=3.0 m, Freeboard F=1.2 m, Design water Height H=1.8 m, Side Slope (S_{in} 1:2, S_{out} 1:2.6) Seepage line 1:7}

4.2.3 Flood water way

A floodplain is the flat land adjacent to a stream that is occasionally or periodically flooded. It includes the floodway and the flood fringe. The floodway consists of the stream channel and adjacent areas that carry flood flows. The importance of defining these terms is that, the floodway parameters are used in calculating the maximum water level resulting from the design discharge. It is also important for setting rules e.g. living in the floodway is prohibited and those living in the flood fringe are protected by flood embankments.

In the case of over-bank flow the discharge consists of two parts (Figure 4.4):

1. Main channel discharge:

$$Q_{mc} = \frac{(b_c - b_{mc})}{2}\frac{1}{n} i b^{1/2} h^{5/3} \quad \text{.......(4.2)}$$

2. Floodplain discharge

$$Q_{fb} = \frac{(b_c - B_{fp})}{2}\frac{1}{n} i b^{1/2} (H - h)^{5/3} \quad \text{.......(4.3)}$$

Assuming that the floodplains have the same slope and the same bottom roughness as the main channel, the total discharge becomes:

$$Q = \frac{(b_c + b_{mc})}{2}\frac{1}{n} i b^{1/2} \left\{ h^{5/3} + \left[\frac{(b_f + B_{fp})}{(b_c + b_{mc})} - 1.0\right](H - h)^{5/3} \right\} \quad \text{.......(4.4)}$$

Solve equation (4.4) for H.

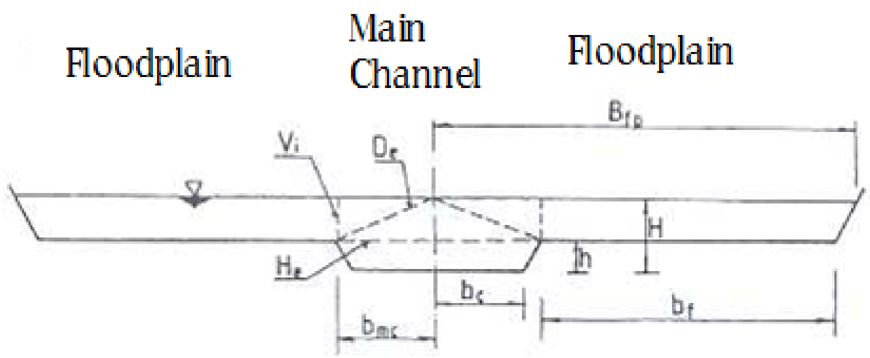


Figure (4.4): Compound channel cross-section (Przedwojski et.al,2005)

Table (4.5): Values of Roughness Coefficient "n"

Nature of Surface	n

	Min	Max
Canals		
Dredged in earth, smooth	0.025	0.033
In rock cuts, smooth	0.025	0.035
Rough beds and weeds on sides	0.025	0.040
Rock cuts, jagged and irregular	0.035	0.045
Natural streams		
Smoothest	0.025	0.033
Roughest	0.045	0.060
Very weedy	0.075	0.150

(Source: Ven Te Chow, Open Channel Hydraulics, 1959)

4.2.4 Dyke layout and location

When levees are used as the only measure for flood protection of a meandering river or braded river, and not as part of comprehensive scheme including channel regulation, realignment, etc., they must **not** follow the river alignment but rather form a levee belt. The levee should be placed at a fair distance from the main channel in order to avoid erosion by migrant channels. The inter-levee distance should remain more or less constant along the reach (Figure 4.5). On the land between the levee and the river banks, seasonal crops may be grown, so that it should not be seen as a total loss from the agricultural point of view.

Levees or dykes designed as a part of flood protection works which also include channel regulation and alignment viz. river training the axes of the levee can follow the river alignment e.g. Gash River (Figure 4.6 & Figure 4.7)

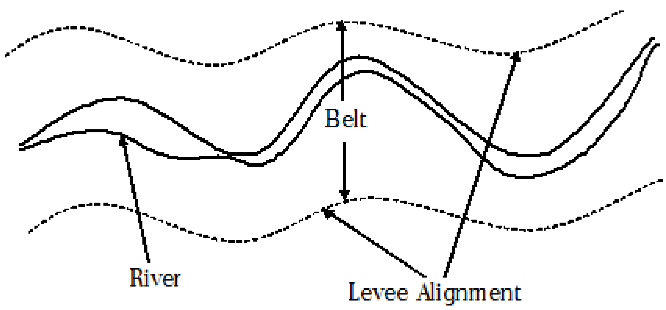


Figure 4.5: Levee alignment

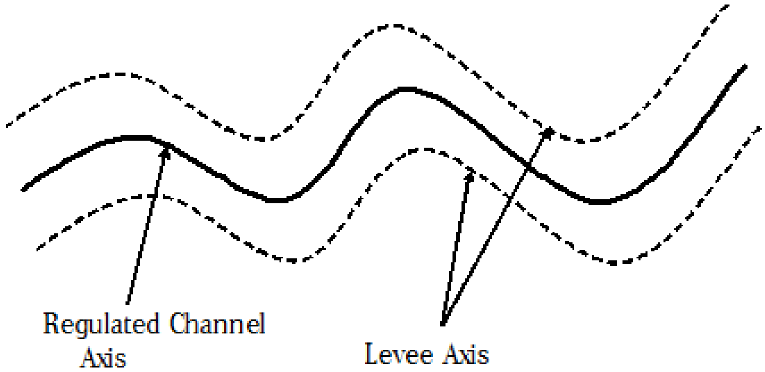


Figure 4.6: Levee along regulated smaller river

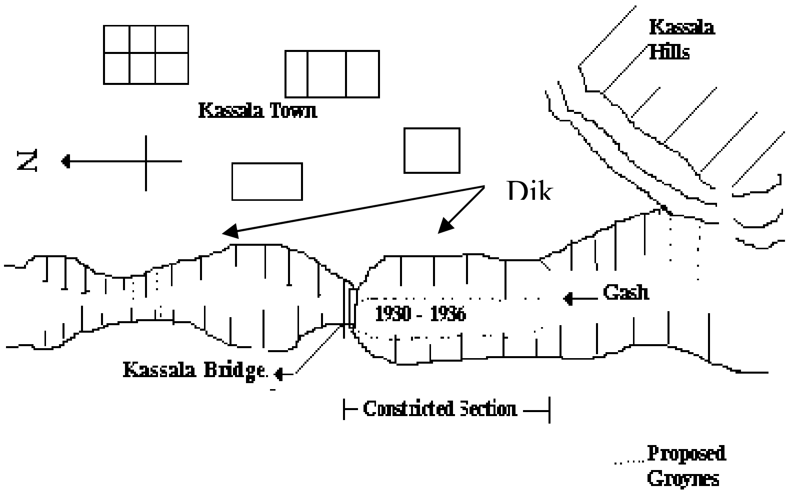


Figure 4.7: The system of spurs and dikes of the Gash River training (Kassala, Sudan)

4.3 Embankment Crest and batter treatments

The inclined face of an embankment is called batter, while the flat top of an embankment is called the crest.

Surface drainage of the crest should be provided by a crown of at least 7.5 cm, or by sloping the crest to drain toward the riverside slope. A camber may be provided along the crest to ensure that the freeboard will not be diminished by foundation settlement or embankment consolidation. It is based on the amount of foundation settlement and embankment consolidation expected. Surfacing is the placing of a layer of selected fine rock or gravely material at least 10 cm thick on top of the crest for protection against damage by wave splash and spray, rainfall, wind, and traffic when the crest is used as a roadway. If the crest constitutes a section of highway, the width of roadway and type of surfacing should conform to those of the highway.

4.4 Slope Protection

1. **Riprap**
Riprap Figure (4.8) is loose armour made of randomly placed quarried rock made up of durable stones 10 – 50 cm diameter and 10 – 500 kg. Its stability depends on the size and mass of the stones, their shape and gradation.

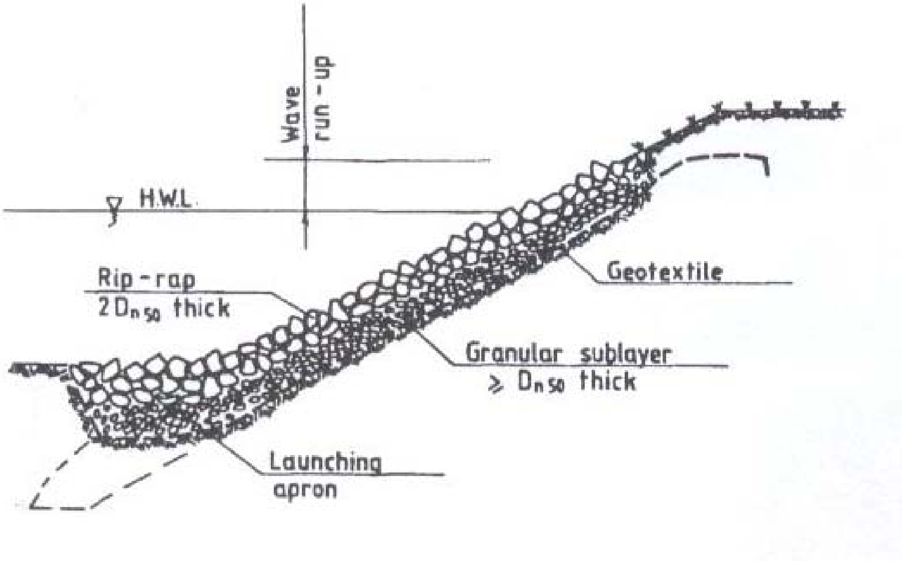


Figure 4.8: Use of riprap in slope protection (Source: Przedwojski, 1995)

2. **Pitched stone**

Stones are placed in rows at first along the lowest row then the upper rows bounding with the lower ones. The size of stones varies from 20 to 50 cm. The maximum unevenness should not exceed 5cm. The pitching is placed in dry by hand, more labor consuming and often more expensive than riprap; Figure (4.9). The Sudanese experience in the Gash is to fill the spaces in the riprap and pitched stones with gravel. This will accelerate the sediment deposition and the sealing of the pitched stones.

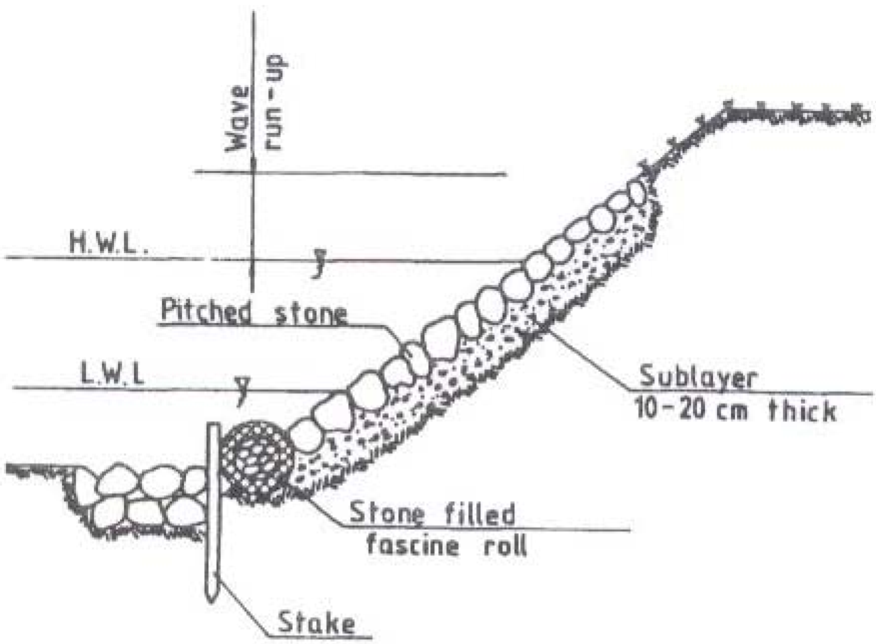


Figure 4.9: Slope protection with pitched stone (Source: Przedwojski, 1995)

3. **Grouted stone and masonry**

Riprap and pitched stones can be bound with cement grout (mortar) or bitumen to improve the revetment stability.

Surface grouting fills approx. 30% of surface voids over the whole area
Pattern grouting fills from 50% - 80% of cover layer-voids.
Full grouting fills 100% of cover-layer voids; Figure (4.10).

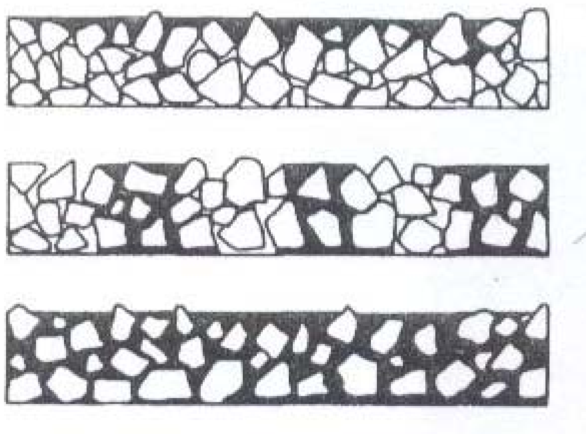


Figure 4.10: Grouted stone: a) Surface grouting b) Pattern grouting c) Full grouting

4. **Bags**
Bags made of jute or woven synthetic fabrics fill with cement or sand forming bag blanket revetment are placed directly on slope in one or two layers.

5. **Gabions**
Gabions are made of stone baskets of wire or plastic netting. The most commonly used steel wire may be woven or welded. Nominal mesh size varies from 440x60 mm to 100x120 mm. The filter stone must have its nominal diameter as 1.5 times mean mesh size. Individual units are assembled before placing them in position; Figure (4.11).

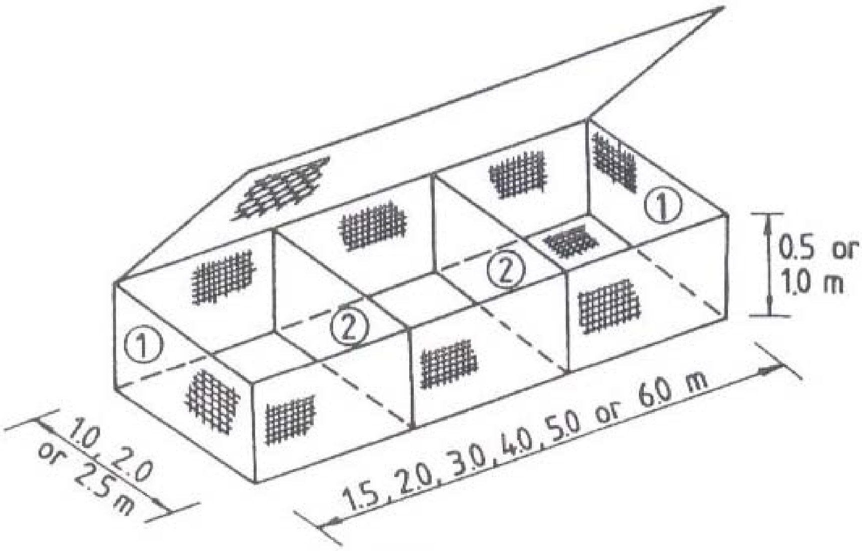


Figure 4.11: Framework and typical dimensions of classical gabions (Source: Przedwojski, 1995)

6. **Geotextiles**
Geotextiles are made from artificial fibers. They are used for producing various elements of river training and slope protection in the form of sheets, cloth, netting, webbing, bags baskets,

strands and other structural elements. They may be incorporated with different composites, geotextile-bitumen, geotextile-concrete etc.

7. **Brush wood revetments**:
Traditional brushwood revetments, known as Libbash, were practiced in the Gash River since long time ago. They are effective in bank stabilization and reducing bank erosion as they reduce the energy of the flowing water. Libbash are brushwood bundles made from local bushes and shrubs tied together and fixed to the banks using palm leaves or synthetic ropes.

4.5 Siltation and Scour of channels and embankments

Sedimentation of river channels results in raising the bottom level of the river and as a consequence the design water level increases. To keep the same freeboard, the embankment should be heightened by an amount equals to the sediment thickness. On the other hand, the scour depth determines the embankment's foundation depth and slope protection works (e.g. geotextile).

To study the morphological changes of a river, where flood embankment is proposed to be a construction site, the avail of the following information will help a lot in prediction the future changes that might take place at the proposed locality

Historical studies (social, aetiology, ecological, hydrological, etc;
Local information and data;
Thematic and aerial maps (for different periods)
Satellite imageries (for different periods);
Future river and its catchment development strategies;
Climate change prediction.

4.5.1 Sedimentation of channels

Sediment deposition in the floodplain and the main channel reduces the discharge capacity and storage capacity of the river channel. In the long run this sedimentation may necessitate the heightening of the levee embankment. Deposition of sediment occurs in regions where the transport capacity of the river is too small to convey the quantity of supply. As a result the bottom level of the river will start to rise increasing its longitudinal slope and flow velocity until transport capacity equals the supply of sediment. Thus protected floodplain areas by means of levees, the river and floodplain areas between dikes may rise above the level of the protected areas of the valley. For example, the level the riverbed of Gash has been raised due to sedimentation more than 3m above the ground level of Kassala town. The system of groins and dykes used for Gash training and Kassala protection was heightened several times to keep it functioning.

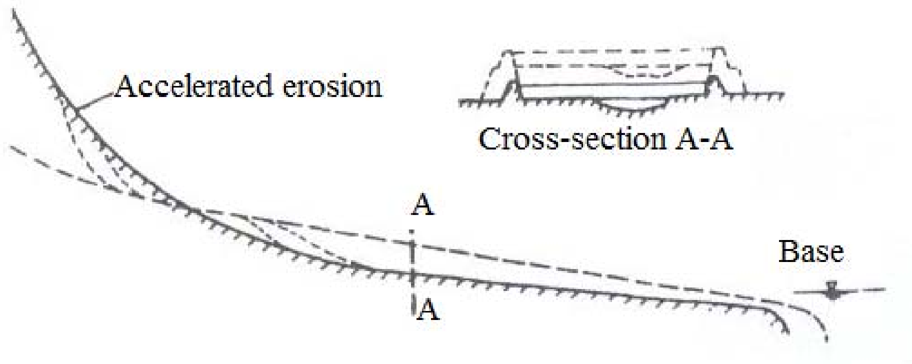


Figure 4.12: Deposition due to accelerated erosion in upstream area

The ultimate rise can be estimated by using a simplified transport formula. For the accelerating erosion of the head waters, Chezy formula for steady and uniform flow can be used:

$$Q = CBh^{3/2} i_b^{1/2} \quad \text{.....(4.5)}$$

Or

$$U = Ch^{1/2} i_b^{1/2} \quad \text{.....(4.6)}$$

The sediment transport is expressed by:

$$S_b = aBU^b = aBC^b (hi_b)^{b/2} \quad \text{.....(4.7)}$$

The two equations after elimination of h, give:

$$\frac{S_b}{Q^{b/2}} = aC^{2b/3} B^{(3-b)/3} i_b^{b/3} \quad \text{.....(4.8)}$$

Where: Q= Discharge, U= Velocity, h= Flow depth, i_b= Bed slope, C= Chezy roughness coefficient, S_b = sediment transport, b = exponent, B = River width. If assuming Q, a, b, C, and B, constants, thus

$$i_b = S_b^{3/b} \quad \text{.....(4.9)}$$

For $b = 5$ the equilibrium slope increases by 50%.

4.5.2 Scour of river channels

In general scour of a river channels is due to:

- Removal of sediment from flow by reservoirs or gravel and sand mining in rivers and flood plains;
- Long contractions causing an increase in specific discharge and velocities;
- Increase in discharge, depths and slopes during flood flows;
- Increase of slopes owing to decrease in roughness by channel regulation, elimination or destruction of drop, weir, etc.

The scouring results in lowering bed and water levels causing exposure of foundation and pipelines, loss of adjacent land, abandonment of water intakes, etc.

The Lacey's formula, for estimating normal scour depth in alluvial streams during floods, developed mainly on the basis of canal data, can be adopted for the design of flood protection embankment is:

$$R = 0.473\left(\frac{Q}{f}\right)^{1/3} \quad \text{.......(4.10)}$$

Where R is the normal scour depth below H.F.L., Q is the design flood discharge and f is the silt factor given by Eq. 4.11, (Weighted mean diameter in mm of the bed material)

$$f = 1.76\sqrt{d} \quad \text{.......(4.11)}$$

The above scour equation is applicable only when the river width equals the regime width given by Eq. 4.12, (P in meter, Q in m^3/s).
:

$$W = P = 4.75\sqrt{Q} \quad \text{.......(4.12)}$$

For other values of active river width, the normal scour depth is given by Eq. 4.13, (q is discharge per meter width)

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad \text{.......(4.13)}$$

Illustrative example 1:
What is the scour depth in an alluvial river having a flood discharge of 65 m^3/s, a width of 38.3 m, and average grain size of the bed material of 0.34 mm? What will be the new scour depth if the river width is 42m?

Solution:

Given Q= 65 m^3/s, W= 38.3 m, d= 0.34 mm
Required R=?
The regime width of the river is:
W= 4.75√Q.=4.75√65 = 38.3 m , and
f.= 1.76√d=1.76√0.34=1.03
The width equals the regime width, therefore apply equation (4.10)
R= $0.473(Q/f)^{1/3}$= $0.473(65/1.03)^{1/3}$=1.89 m
If W=42 > regime width, use equation (4.13)
R= $1.35(q^2/f)^{1/3}$= $1.35((65/42)^2/1.03)^{1/3}$=$1.35((1.55)^2/1.03)^{1/3}$ = 1.79 m

Local scour below a dam:

A simplified procedure of calculation of river bed degradation below dams is developed by Levi. At first the lowering of water level (Δy) is calculated assuming that below the dam protection measure the maximum scour depth is established. The maximum water depth is:

$$h_1 = \frac{Qf}{BU_C} \qquad (4.14)$$

Where; Qf is the design water discharge, B is the width of the scoured cross-section, Uc is the critical (threshold) velocity given by Levi as:

$$U_C = 1.7\sqrt{gD}\left(\frac{h_0}{D}\right)^{0.2} \qquad (4.15)$$

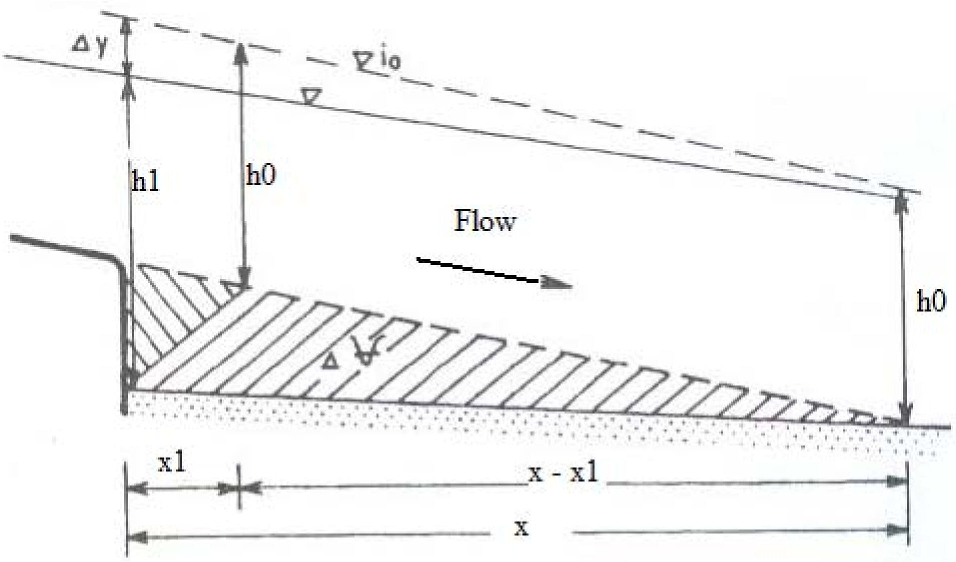


Figure 4.13: Sketch of degradation below a dam

Scoured volume of bed material can be estimated as:

$$\Delta V = \frac{B}{2}\left[(h_1 - h_0)(X - X_1) + X\Delta y\right] \qquad (4.16)$$

ΔV is transported downstream of the eroded reach at the rate of sediment discharge (St). For the scoured bed material roughness $\frac{D}{h_0} > 0.001$ Levi recommended his own relationship in the form:

$$St = \frac{0.002B}{(gD)^{1.5}}\left(\frac{D}{h_0}\right)^{1.25}\left(\frac{Qf}{Bh_0}\right)^{4}\left(\frac{h_1 - h_0}{h_1}\right) \qquad (4.17)$$

The time needed for scouring ΔV is equal to

$$\Delta t = \frac{\Delta V}{St} \text{ ....}(4.18)$$

Illustrative example2:
Estimate the lowering of water surface elevation and the maximum depth of general scour below a dam. Input data: a channel with rectangular cross-section of width B= 50 m, initial bed and water surface slope i_0= 0.0004, initial water depth h_0= 2.1 m at the bank-full discharge Qf= 100 m^3/s, mean duration of one bank-full peak flow tpf= 11 days and its recurrence T=1 year. Length of scour hole x1= 200m, and mean diameter of uniform bed material D= 0.5 mm.

Solution
1. The maximum scoured depth (h1) is computed as:
The mean critical velocity Uc by Levi:

$$U_C' = 1.7\sqrt{9.81 \times 0.0005} \times \frac{2.1}{0.0005} = \underline{\underline{0.64}}\, m/s$$

The first approximation of the water depth h1:

$$h'1 = \frac{100}{50.0.64} = 3.1\ m$$

The second (and final) approximation:

$$U''c = 1.7\sqrt{9.81.0.00005}\left(\frac{3.1}{0.0005}\right)^{0.2} = 0.69\ m/s$$

$$h''1 = \frac{100}{50.0.69} = 2.9\ m = h1$$

2. The drop in water level for an arbitrarily taken distance x=500 m reads:

$\Delta y=(i0-i\square)x$ where;

$i = \frac{1}{2}(i0 + \frac{n^2 Uc^2}{h1^{\frac{4}{3}}})$……….. (4.19)

For n= 0.035 one has:

$$i = \frac{1}{2}\left(0.0004 + \frac{0.035^2 0.69^2}{2.9^{\frac{4}{3}}}\right) = 0.00027$$

Hence:
$\Delta y=(0.0004 -0.00027).500=0.07$ m

3. The scoured volume of bed material:

$\Delta V = \frac{50}{2}[(2.9 - 2.1)(500 - 200) + 500.0.07] = 6875\ m^3$

4. The rate of sediment transport (by Levi):

$St = \frac{0.002.50}{(9.81.0.0005)^{1.5}}(\frac{0.0005}{2.1})^{1.25}(\frac{100}{50.2.1})^{4}\frac{2.9-2.1}{2.1}$=2.5.10^{-3} m^3/s

5. Time needed for scouring the volume ΔV:

Δt=ΔV/St=6875/0.0027= 2.5.10^6 seconds=29 days

6. The total current time:
t= TΔt/tbf =1x29/11=2.6 years

Local scour due to a long constriction
Long constrictions in river channels appear on construction of groins, bridge crossings, etc. Longitudinal and cross-sectional equilibrium profiles for scour in long contractions follows two theories:

Static (or tractive-force) equilibrium theory when the rate of sediment supply into the contracted zone is zero or nearly zero;
Dynamic (or live-bed) equilibrium theory if the rate of sediment supply is greater than zero.

A. Static state scour:
Simplified morphological computation due to channel width constriction suggested by Jansen and de Vries. Equilibrium situations are compared, existing with subscription 0 and the new one with a subscription 1. Figure (4.14)

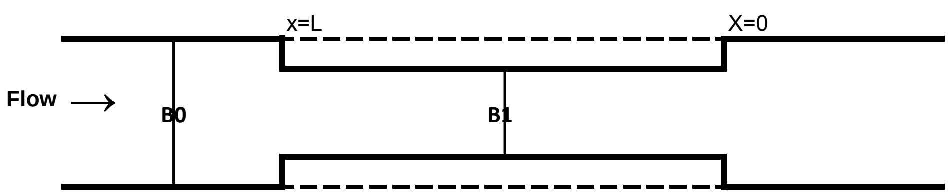


Figure 4.14: Change of river width

Water continuity:

$$Q_1 = Q_0 \text{ ....} (4.20)$$

Water motion:

$$Q = CBh^{3/2} i^{1/2} \text{ ....} (4.5)$$

Sediment continuity:

$$S_1 = S_0 \text{ ....} (4.21)$$

Sediment motion:

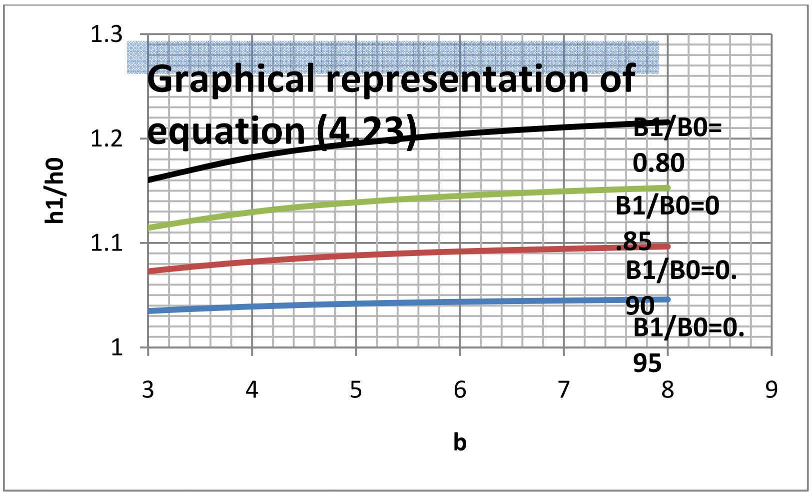
$$S = BmU^{b} \text{ ....} (4.22)$$

Assumptions, $C_1= C_0$, ripple factor $\mu_1=\mu_0$, particle diameter $D_1=D_0$, and sediment coefficients $m_1=m_0$, $b_1=b_0$. The mean depth h1 and mean energy slope i_1 after the constriction of the river width are:

$$\frac{h_1}{h_0}=\left(\frac{B_0}{B_1}\right)^{\left(\frac{b-1}{b}\right)} \qquad \text{.......(4.23)}$$

$$\frac{i_1}{i_0}=\left(\frac{B_1}{B_0}\right)^{\left(\frac{b-3}{b}\right)} \qquad \text{.......(4.24)}$$

A graphical representation of these equations is shown in Figure (4.15), below. Note that for b=3 the new slope equals the old one. This is the case for large bed load sediment transport.



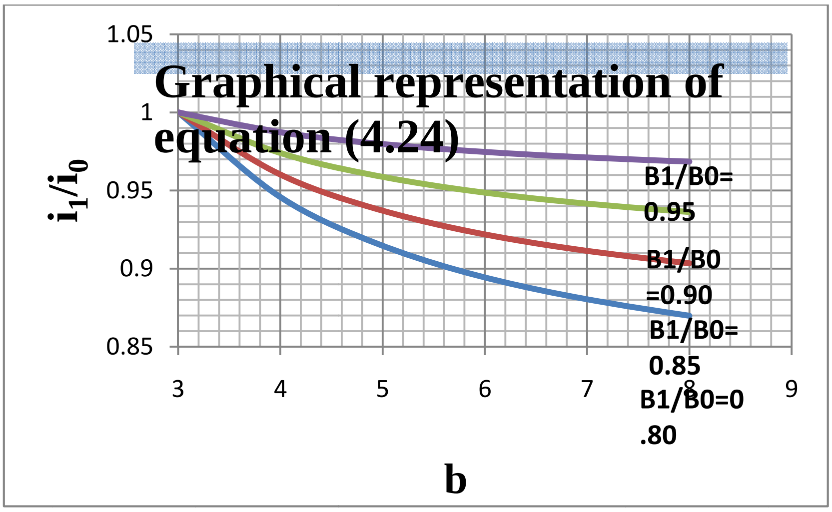


Figure 4.15: Results of river width constriction

B. Dynamic (live-bed) state scour:

Several equations are available for calculating the general scour at live-bed, the relative depth of scour along the constricted reach with the width of B1 and water depth h1. Figure (4.16).

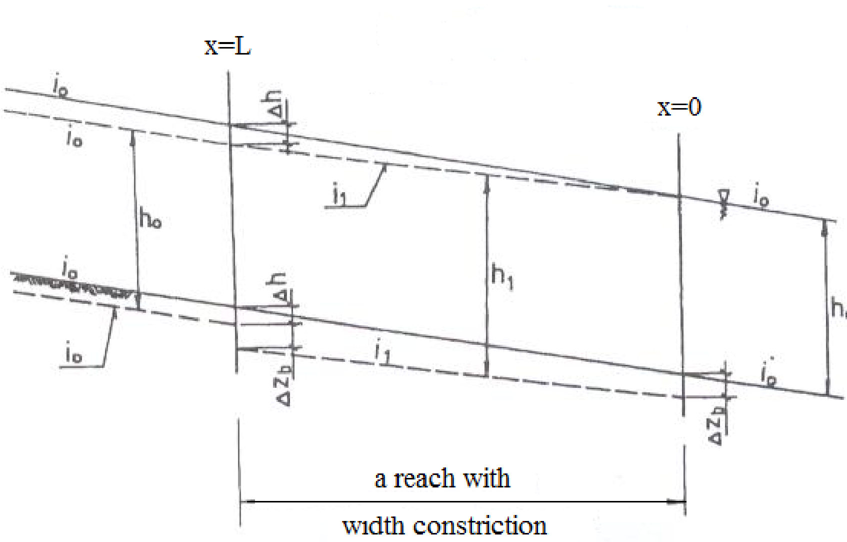


Figure 4.16: Longitudinal profile of a reach with constriction of width

Komura Equation:

$$\frac{\Delta z}{h} = \left(1 + 1.2Fr^2\right)\left[\left(\frac{B}{B_1}\right)^{2/3} - 1.0\right] \quad (4.25)$$

Michiue Equation:

$$\frac{\Delta z}{h} = \left[\left(\frac{B_1}{B}\right)^{-4/7} - 1.0\right] + \left(0.5Fr^2\right)\left[\left(\frac{B1}{B}\right)^{-6/7} - 1.0\right] \quad (4.26)$$

Where, Fr is the Frude number given by

$$Fr = \frac{U}{\sqrt{gh}} \quad (4.27)$$

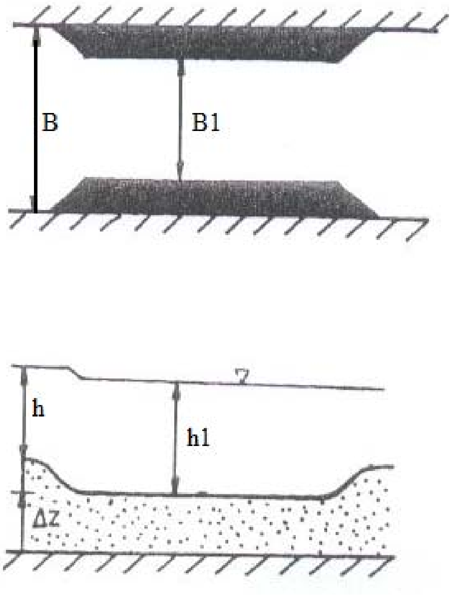


Figure 4.17: Scour along a long constriction

5 Geotechnical Considerations

The geotechnical characteristics and behavior of the embankment and its foundations are key factors affecting their performance. This chapter shows the geotechnical investigations to be carried out before the design and construction of flood embankment to evaluate the geologic, seismologic, and soils conditions that affect the safety, cost effectiveness, design, and execution of a proposed engineering project, in general.

5.1 Soils of Sudan

The country's soils (figure 5.1) can be divided geographically into four categories. These are the sandy soils of the northern and west central areas, the clay soils of the central region, the laterite soils of the south, and alluvial soils found along the lower reaches of the White Nile and Blue Nile rivers, along the main Nile to Lake Nubia, in the delta of the Qash River in the Kassala area, and in the Baraka Delta in the area of Tawkar near the Red Sea in Ash Sharqi State.

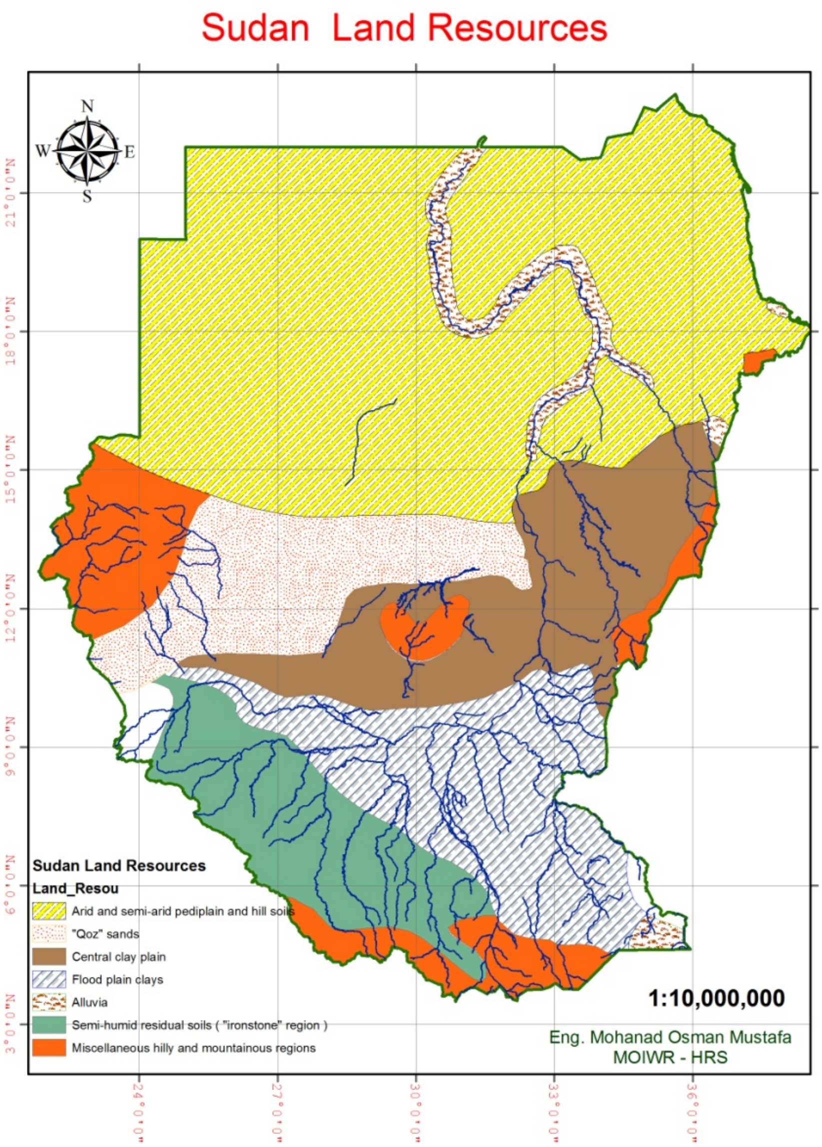


Figure 5.1: Sudan Soil map

the most important soils related to flood plain in the Sudan are the clays in central Sudan that extend from west of Kassala through central and southern Kordofan Known as cracking soils due to cracking during the dry months .The characteristics of black clays are very plastic, expansive, generally of low permeability, and of moderate to low density. As a general conclusion, these clays could be used for small earth dams successfully if attention is paid to compaction, placement water content, shrinkage fissures and filter design.

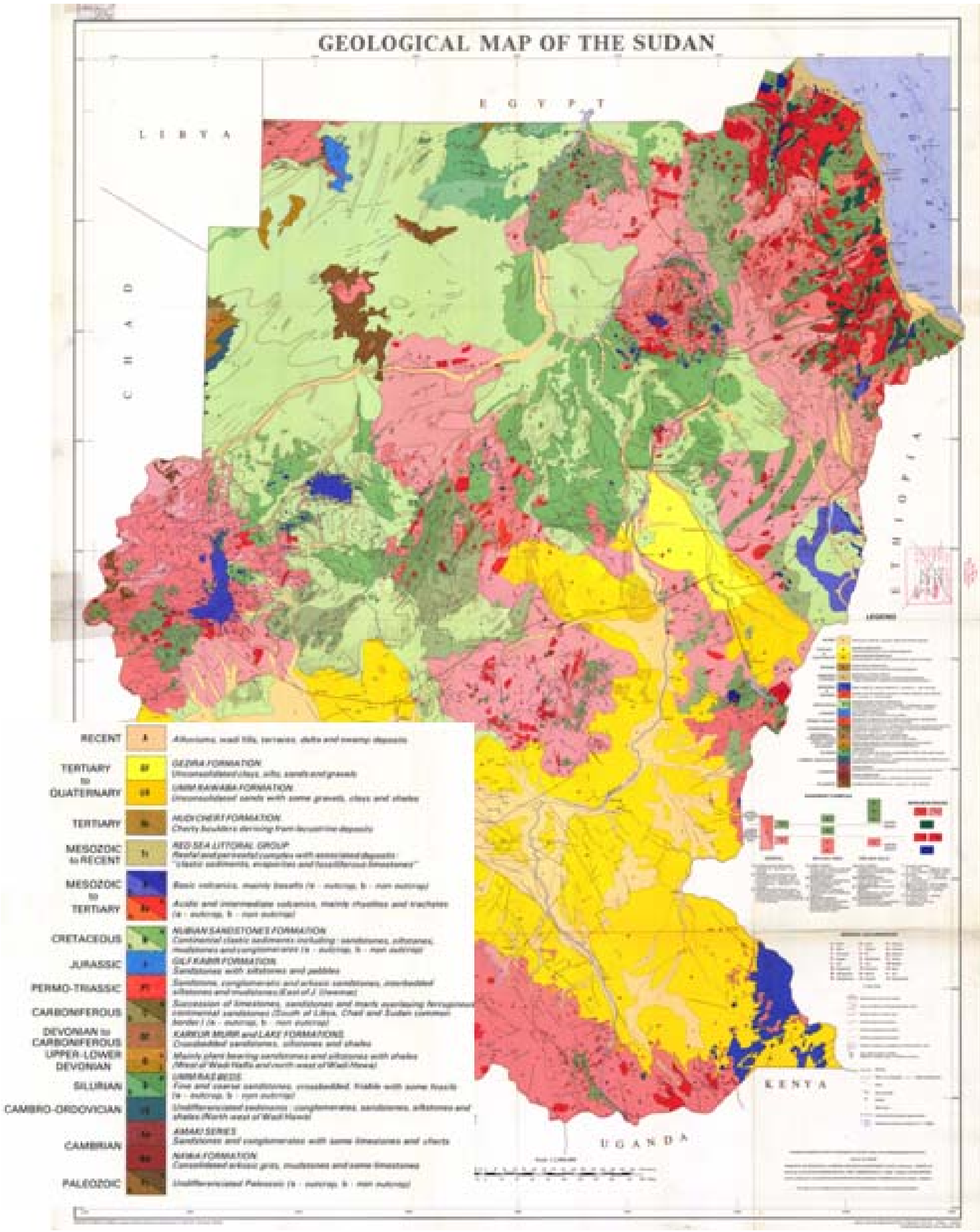


Figure 5.2: Sudan Geology map

Embankment dams are constructed of all types of geologic materials, with the exception of organic soils and peats. Most embankments are designed to utilize the economically available on-site materials for the bulk of construction. Special zones such as filters, drains, and riprap, may come from off-site sources. Soil materials used in embankment dams commonly are obtained by mass production from local borrow pits, and from required excavations where suitable. The geological map of Sudan is shown in figure 5.2

5.2 Geotechnical Investigations

Insufficient geotechnical investigations, may contribute to inappropriate designs, costly construction modifications, and even failure of a structure. Investigations performed to determine the geologic, seismologic, and soil conditions that influence selection of the project site; the characteristics of the foundation soils and rocks; geotechnical conditions which influence project safety, design, and construction; critical geomorphic processes; and sources of construction materials.

Some of the geotechnical investigation tests related to flood embankments design are listed below:

- **The cone penetration test** (CPT): is an in situ testing method used to determine the geotechnical engineering properties of soils and delineating soil stratigraphy. Today, the CPT is one of the most used and accepted in situ test methods for soil investigation worldwide.

  The test method consists of pushing an instrumented cone tip first into the ground at a controlled rate (usually 2 centimeters/second). The resolution of the CPT in delineating stratigraphic layers is related to the size of the cone tip, with typical cone tips having a cross-sectional area of either 10 or 15 cm^2, corresponding to diameters of 3.6 and 4.4 cm.

- **The standard penetration test** (SPT): is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil. The test uses a thick-walled sample tube, with an outside diameter of 50 mm and an inside diameter of 35 mm, and a length of around 650 mm.

  The main purpose of the test is to provide an indication of the relative density of granular deposits, such as sands and gravels from which it is virtually impossible to obtain undisturbed samples. The great merit of the test and the main reason for its widespread use is that it is simple and inexpensive.

- **Exploration geophysics** is the applied branch of geophysics which uses surface methods to measure the physical properties of the subsurface earth, in order to detect or infer the presence and position of concentrations of core minerals and hydrocarbons.

  Exploration geophysics is the practical application of physical methods (such as seismic, gravitational, magnetic, electrical and electromagnetic) to measure the physical properties of rocks, and in particular, to detect the measurable physical differences between rocks that contain ore deposits or hydrocarbons and those without.

- **Reflection seismology** (or **seismic reflection**) is a method of exploration geophysics that uses the principles of seismology to estimate the properties of the Earth's subsurface from reflected seismic waves.

Seismic refraction is a geophysical principle (see refraction) governed by Snell's Law. Used in the fields of engineering geology, geotechnical engineering and exploration geophysics, seismic refraction traverses (seismic lines) are performed using a seismograph(s) and/or geophone(s), in an array and an energy source. The seismic refraction method utilizes the refraction of seismic waves on geologic layers and rock/soil units in order to characterize the subsurface geologic conditions and geologic structure.

5.3 Soil Compaction and testing

Soil Compaction is a significant part of the embankment building process, and requires mechanical compaction techniques such as rammers, compactors, and rollers. If compacted improperly, will cause unnecessary repair costs and structural damages. Many embankments in Kasala (Eastern Sudan), and Kurdofan (Western Sudan) failed when subjected to high flows because of improper compaction. The following section presents methods and techniques for proper soil compaction.

The selection of each method or a technique is mainly depends on the type of soil dealt with and the design requirements. Almost all types of construction projects utilize mechanical compaction techniques. Therefore, the soil compaction could be defined as the method of mechanically increasing the density of soil.

There are five principle reasons to compact soil:

- Increases load-bearing capacity
- Prevents soil settlement and frost damage
- Provides stability
- Reduces water seepage, swelling and contraction
- Reduces settling of soil

Soil testing accomplishes the following:

- Measures density of soil for comparing the degree of compaction vs. specs
- Measures the effect of moisture on soil density vs. specs
- Provides a moisture density curve identifying optimum moisture.

5.3.1 Soil Density Tests

The tests to determine optimum moisture content are done in the laboratory. The most common is the Proctor Test, or Modified Proctor Test. A particular soil needs to have an ideal (or optimum) amount of moisture to achieve maximum density. This is important not only for durability, but will save money because less compaction effort is needed to achieve the desired results.

Table 5.1: Optimum Soil Moisture to attain maximum Density.

No.	Soil Type	Optimum Soil moisture (%)
1	Sand	06 – 10
2	Sand-silt mixture	08 – 12
3	Silt	11 – 15
4	Clay	13 – 21

The Hand Test
A quick method of determining moisture. Pick up a handful of soil, squeeze it in your hand, open your hand. If the soil is powdery and will not retain the shape made by your hand, it is too dry. If it shatters when dropped, it is too dry. If the soil is moldable and breaks into only a couple of pieces when dropped, it has the right amount of moisture for proper compaction. If the soil is plastic in your hand, leaves traces of moisture on your fingers and stays in one piece when dropped, it has too much moisture for compaction.

Proctor Test
A small soil sample is taken from the jobsite. A standard weight is dropped several times on the soil. The material is weighed, and then oven dried for 12 hours in order to evaluate water content. The **Modified Proctor Test** is similar to the Proctor Test except a hammer is used to compact material for greater impact, the test is normally preferred in testing materials for higher shearing strength.

Field Tests
It is important to know and control the soil density during compaction. The common field tests to determine the compaction densities are Sand Cone Test and Nuclear Density

5.3.2 Soil Compaction Techniques

There are four types of compaction effort on soil; vibration, impact, kneading and pressure. The compactors deliver a rapid sequence of blows (impacts) to the surface, thereby affecting the top layers as well as deeper layers. Many types of equipment are used during compaction, the famous are listed below:

- **Rammers**
  Rammers deliver a high impact force (high amplitude) making them an excellent choice for cohesive and semi-cohesive soils. Rammers get compaction force from a small gasoline or diesel engine powering a large piston set with two sets of springs. The rammer is inclined at a forward angle to allow forward travel as the machine jumps. Rammers cover three types of compaction: impact, vibration and kneading.

- **Vibratory Plates**
  Vibratory plates are low amplitude and high frequency, designed to compact granular soils and asphalt. Gasoline or diesel engines drive one or two eccentric weights at a high speed to develop compaction force. The resulting vibrations cause forward motion. The engine and handle are vibration-isolated from the vibrating plate.

- **Reversible Vibratory Plates**
  In addition to some of the standard vibratory plate features, reversible plates have two eccentric weights that allow smooth transition for forward or reverse travel, plus increased compaction force as the result of dual weights. Due to their weight and force, reversible plates are ideal for semi-cohesive soils.

- **Rollers**
  Rollers are available in several categories: walk-behind and ride-on, which are available as smooth drum, padded drum, and rubber-tired models; and are further, divided into static and vibratory sub-categories. Criterias of selecting the type of roller are listed below

  - Sheepsfoot rollers are most suitable for compaction of impervious-type material with no particles larger than gravel sizes.

  - Lifts are usually placed at about 20 to 30 cm (loose measure) and compacted with 8 to 12 passes of the roller.

  - When the impervious material is well graded from 10 to 15 mm maximum to silt and clay sizes and of low plasticity, rubber-tired rollers are used for compaction.

  - The rollers may be up to 50-ton rubber-tired rollers. 100 ton rollers have been used also.

  - The compacted lift thickness of 30 cm is commonly used. The minimum number of passes of a rubber tired roller is usually 4.

5.4 Foundation of flood Embankment

A sound rock foundation is the best possible type of foundation for an earth dam from both stability and seepage considerations. Where an embankment dam is founded upon a pervious-soil stratum the main requirement of foundation treatment generally is the control of under seepage. The preferred foundation treatment for control of under-seepage is a cutoff (impervious fill cutoffs, concrete wall cut-offs and grouted cutoffs) through the pervious stratum to impervious material.

Impervious-soil foundations may consist of alluvial silts and clays or well-graded glacial tills or residual soils. These soils generally present little difficulty from the standpoint of seepage but where silt or clay is present require careful investigation in connection with stability and settlement. Although the quantity of leakage is not a problem, careful control of exit gradients by drain wells and other means is required for cohesionless soils which are susceptible to piping.

A plastic clay foundation in an earth dam is usually the type of foundation which requires the greatest amount of study and investigation in order to obtain unquestionable safety. Frequently extremely flat slopes must be used for the earth dam built on such a foundation in order to keep the stresses in the foundation sufficiently less than the strength of the material to provide a suitable factor of safety.

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Preparation of foundation
To insure proper embankment foundation the following preparations are required (1) Clearing and (2) grubbing. Depending on site conditions stripping and the disposal of products may also be required.

1. Clearing
   Removal of trees, fallen timber, brush, vegetation, loose stone, abandoned structures, fencing, and other matters above the ground surface.
2. Grubbing
   Removal of all stumps, roots, buried logs, old piling, old paving, drains, and other objectional matter. It is not necessary beneath stability berms.
3. Stripping
   Stripping is the removal of low growing vegetation and organic topsoil. The depth of stripping is determined by local conditions and normally varies from 15 to 30 cm. Stripped material maybe suitable for use as topsoil on the slopes of the embankment and berms.
4. Disposal of debris
   Debris from clearing, grubbing, and stripping operations can be disposed of by burning or any other environmentally approved manner.
5. Dewatering
   Dewatering embankment foundations for the purpose of excavation and back filling in the dry.

5.5 Stability Analysis

The slope stability analyses are performed to assess the safe and economic design of a human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. The aims of slope stability analyses are:

- To understand the development and form of natural slopes and the processes responsible for different natural features.
- To assess the stability of slopes under short-term (often during construction) and long-term conditions.
- To assess the possibility of landslides involving natural or existing engineered slopes.
- To analyze landslides and to understand failure mechanisms and the influence of environmental factors.
- To enable the redesign of failed slopes and the planning and design of preventive and remedial measures, where necessary.
- To study the effect of seismic loadings on slopes and embankments.

If the forces available to resist movement are greater than the forces driving movement, the slope is considered stable. A factor of safety is calculated by dividing the forces resisting movement by the forces driving movement. In earthquake-prone areas, the analysis is typically run for static conditions and pseudo-static conditions, where the seismic forces from an earthquake are assumed to add static loads to the analysis.

Method of slices

The method of slices is a method for analyzing the stability of a slope in two dimensions. The sliding mass above the failure surface is divided into a number of slices. The forces acting on each slice are obtained by considering the mechanical equilibrium for the slices.

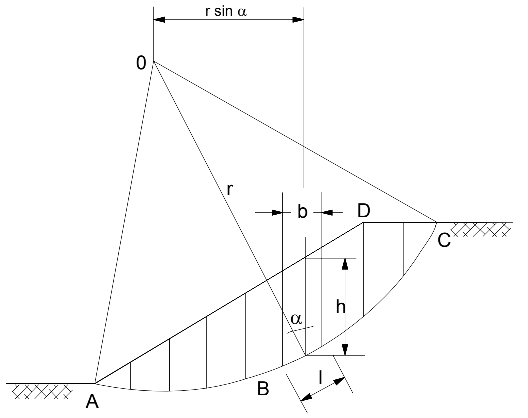


Fig. (5.2): Stability analysis – The method of slices.

Bishop's method

The Modified (or Simplified) Bishop's Method is a method for calculating the stability of slopes. It is an extension of the Method of Slices. The method has been shown to produce factor of safety values within a few percent of the "correct" values.

$$F = \frac{\sum \left(\dfrac{c'b + (W - ub)\tan\phi'}{\cos\alpha\left(1 + \dfrac{\tan\alpha \tan\phi'}{F}\right)} \right)}{\sum W \sin\alpha} \qquad \text{..} \quad (5.1)$$

Where;

c' is the effective cohesion
Φ' is the effective internal angle of internal friction
b is the width of each slice, assuming that all slices have the same width
W is the weight of each slice
u is the water pressure at the base of each slice

Lorimer's method

Lorimer's Method is a technique for evaluating slope stability in cohesive soils. It differs from Bishop's Method in that it uses a clothoid slip surface in place of a circle. This mode of failure was determined experimentally to account for effects of particle cementation.

6 Construction Aspects

Flood embankments essentially act as low-level dams for short retention periods. For the majority of the time, most embankments are exposed to no or to low, hydraulic head and remain largely unsaturated. Knowledge about the type of fill material used to construct embankments and the method of the construction does allow the performance of the embankment to be considered in a rational manner and, if appropriate, analyzed using principles of soil mechanics.

This section provides a review of embankment construction along with information about typical foundation performance with reference to historic breaches and general experience of embankment performance. To minimize the cost of construction, the know-how and local experiences need to be explored before starting the construction. The main target group are the skilled labourer and builders to be contracted on daily-paid bases.

6.1 Construction Materials

The construction engineer has to explore the availability of the embankment construction materials. These may include earth, sand, gravel, rock, geotextile, gabions, etc. The location and the cost of such materials and their properties are very essential. Alluvial clays and silts are a common source of fill material. One of the most popular examples to prevent the flood during the emergency is filling bag with sand. The local people and the Ministry of Civil Defence normally do this kind or work. They fill bags of sand and used it as a barrier in the low areas. Another example is a specialized unit to build, maintain and monitor the embankment, like the unit in Gash River. Dikes and flood embankments are not uncommon in Sudan. However, the majority of the flood prone areas depend as mentioned above on sand bags. e.g., the local people in Kalakla in Khartoum and Toti Island depend on filling the sad bag during high floods. The Gabions has been used in Gezira scheme and Gash River and other important embankment as shown in photo 6.1.

Photo 6.1: Gabions embankment in Gezira scheme

6.2 Crossing structures and accessibility requirements

6.2.1 Road passages

Passages are constructed to connect the land side area of the levee with the cut-off inter-levee floodplain. The slope and width of such passages depend on the class of the road are 2 - 2.5 m wide, the slope of two-sided approaches are 8-10%. The riverside approach has to be parallel to the levee with the inclination in the direction of river flow (Figure 6.1).

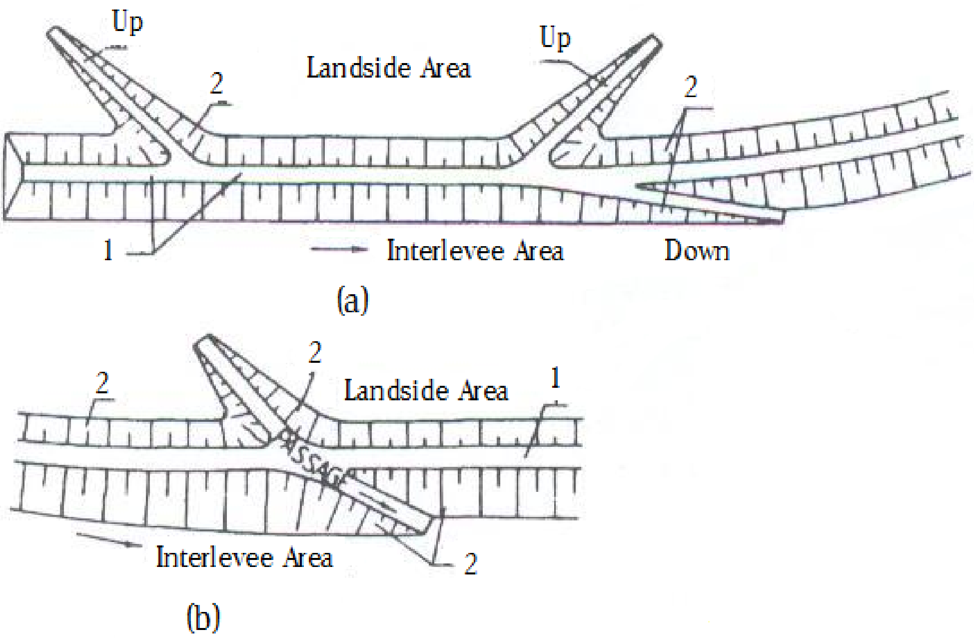


Figure 6.1: Levee passages a) Two-way levee passage b) One-way levee passage (Source: Przedwojski, 1995)

6.2.2 Drainage Culverts

Drainage culverts are designed for passing smaller streams and drainage canals through the levee. They have gates from the riverside that are closed during the floods. The cross sections of the embankment where the culverts are located are not safe. They should be built carefully and their foundation should meet all requirements. There are open and closed levee culverts:

- Closed culverts are longer and its wings are low and short and the upper part of the levee runs continually above the culvert. They are the most appropriate in terms of levee safety.

- An open culvert is set in a break in the levee with high abutments, high and long wings. They are seldom built and with appropriate gates.

Pumping units are used to pump the accumulated flood water on the protected flood plain or village when the construction of culverts is not economically feasible.

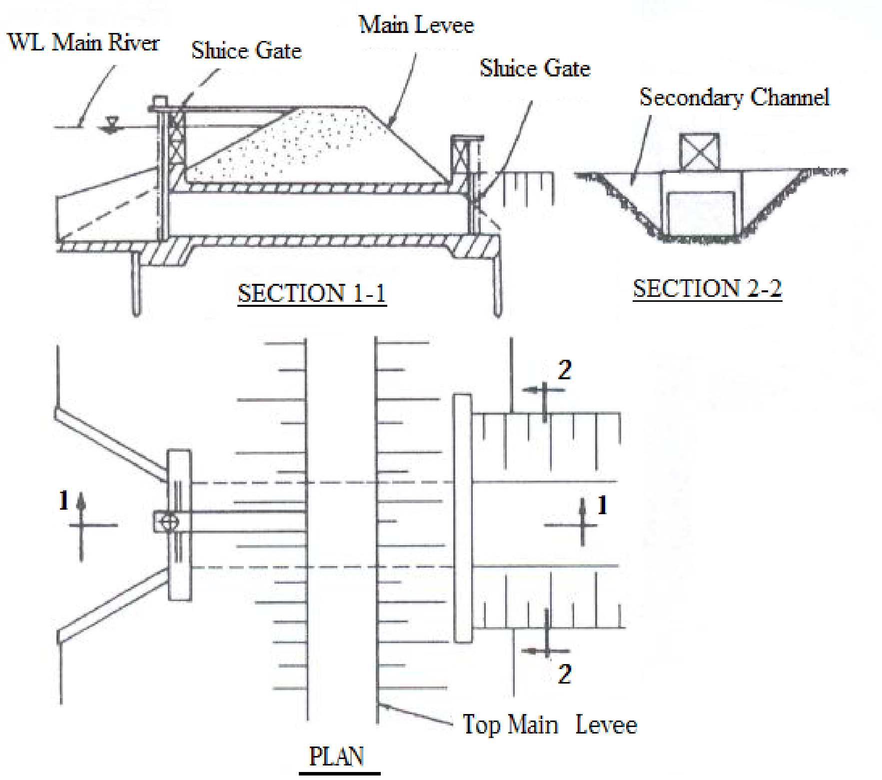


Figure 6.2: Culvert outlet through levee with sluice gates. (Source: Przedwojski, 1995)

Culvert design items
The following should be considered for all culvert designs where applicable:

1. Engineering aspects
   a. flood frequency
   b. velocity limitations
   c. buoyancy protection

2. Site criteria
   a. length and slope
   b. debris and siltation control
   c. culvert barrel bends
   d. ice buildup

3. Design limitations
   a. headwater limitations (see Section 2A-1)
   b. tailwater conditions
   c. storage – temporary or permanent

4. Design options
   a. culvert inlets
   b. inlets with headwalls
   c. wingwalls and aprons
   d. improved inlets
   e. material selection

f. culvert skews
g. culvert sizes and shapes
h. twin pipe separations (vertical and horizontal)
i. culvert clearances

5. Related designs
a. weep holes
b. outlet protection
c. erosion and sediment control
d. environmental considerations

Hydraulic Design

- The design of culverts is based on an increased velocity of flow through the culvert, and to create that the water is allowed to head up at the inlet end of the culvert.
- By fixing the upstream level up to which the water will head up while passing maximum design discharge, the downstream water level is determined (usually taken equal to the surface level of the natural unobstructed flow at the site before embankment construction.
- The operating head causing the flow H_L is the difference of the upstream and downstream levels.
- The area of the opening should then be decided, so that it is sufficient to pass the design discharge.
- Design data
  Q = culvert design discharge (m^3/s)
  L = culvert length (m)
  S = culvert slope (m/m)
  H_L= Total Head loss
  K_e = inlet loss coefficient

H_L= Entrance loss + Friction loss + Velocity Head in barrel

$$= K_e \frac{V^2}{2g} + \frac{n^2 \times L \times V^2}{R^{4/3}} + \frac{V^2}{2g}$$

Where K_e = 0.505 for square edged entrance,
K_e = 0.05 for bell mouthed entrance

$$H_L = K_e \times \frac{V^2}{2g} + \frac{V^2}{2g} + \frac{n^2 \times L}{R^{4/3}} \times 2g\left(\frac{V^2}{2g}\right)$$

$$= \frac{V^2}{2g}\left[1 + K_e + \frac{n^2 \times L \times 2g}{R^{4/3}}\right] = \frac{V^2}{2g}\left[1 + K_e + K_f\right]$$

Where; $K_f = \dfrac{n^2 \times L \times 2g}{R^{4/3}}$

Or

$$V = \sqrt{\frac{2g \times H_L}{1 + K_e + K_f}} \; ; \; \therefore \; Q = A \times V = A \times \sqrt{\frac{2g \times H_L}{1 + K_e + K_f}}$$

Hence, knowing the design discharge (Q) the required area (A) of the culvert can be computed. For more details please refer to Chapter 7 of MOIWR Design Sheet File "Hydraulic design of energy dissipaters"

6.3 Access for Cattle Watering in Dry Seasons

Channeling by animals crossing the embankment is very dangerous for levees built on silt and clay which is eroded easily and cause a collapse of the embankment. Ramps for cattle crossing with flatter slopes should be provided. Ramps having slopes 1V:6H and 4 - 8 m width are appropriate.

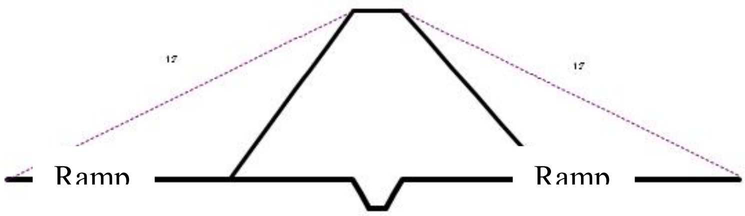


Figure 6.3: Cattle crossing ramps on both sides of embankment

6.4 Know-How and Local Experience

The local people through years of different floods ranging from maximum to normal, they get some experience and knowledge about how to deal with flood. This knowledge has been accepted through different generations. The local people know where are the low areas located, and the most likely flood prone sites. They know these places without satellite images or levelling or contour map. In addition, they know the route of the flash flood and the draining areas of the catchment. Moreover, they know the suitable locations for future expansion of their buildings. They have good knowledge about duration of floods, and how long it takes before it precedes. They also know with good accuracy, where to put flood barriers to prevent the flood entering their town or neighbourhood.

In order to do any permanent embankment, professional engineers should select the best location for the embankment through site visit and surveys. They should involve the local people through all the phases of design and consider their experience and ideas. They should select the materials from the surrounding areas as much as possible to reduce the cost. The final step is the preparation of drawings and the cost estimate.

The following local conditions are of high importance to be considered before construction:

- Awareness and trust building among beneficiaries;
- Requirements for construction camp, with estimated population and quarters required supervisory and construction employees, required water supply and sanitation facilities, local laws regarding sanitation, stream pollution, etc.

- Additional transportation facilities required for construction, including access roads;
- Availability of electric power for construction purposes;
- Safety, insurance and health requirements.

6.5 Site Preparation

Before starting construction the followings need to be considered:

- Foundation should be excavated to a depth of at least 1 m and possibly deeper depending on soil conditions (Topsoil is not the optimal foundation to build on).
- Clearing the site from any debris and pushes, if exist;
- A good route between the borrow pit and the embankment site needs to be prepared.

6.5.1 Surveying consideration

Staking out the embankment site before construction begins will ensure that the bottom layers are placed correctly and that all outlet structures are in the right position.

Two reference stakes should be placed first. These will define the centreline of the embankment. They should be placed close enough to the construction to be useful but far enough away as not to be disturbed. These will provide a reference point during construction. When the other stakes are removed or disturbed during construction, their location can be resurveyed from these points, more information on topographic surveying is given in Appendix 5.

6.5.2 Traditional methods of construction

Some of the flood embankments are generally old structures that have built before decades (Gash embankment) or even centuries from original constructions. In contrast with the modern construction of flood embankments, heavy earth compaction machine has been used; flood embankments have been built using low cost traditional techniques.

These traditional methods of construction have often evolved to suit local sources of fill material, which have been excavated from surface deposits or retrieved from river sediments. As a result, the construction of flood embankments can be highly variable across the country, and this can affect the performance and potential failure mechanism for embankments.

Some Remarks:

Embankment geometry varies according to type of material used and construction history. Ideally, an embankment should have a crest width of greater than 2.5m to allow access along the crest for operations and maintenance vehicles.

Whilst the slope of inward and outward embankment faces might sometimes exceed 1 in 2 (according to construction material), stability problems will be encountered as the face is steepened. Poorly controlled maintenance activities can result in bank steepening through excessive removal of soil when cutting vegetation.

Changing the slope of an embankment affects the way in which waves run up the face and potentially overtop the embankment. There is also anecdotal evidence that the grazing habits of sheep are affected by the gradient of embankments.

Recent embankments are typically constructed in layers using standard compaction specification akin to highway construction. In cases where the fill material is considered to be too permeable, a less permeable core could be incorporated into the construction. In practice, an impermeable core is not often used, even where highly permeable fill materials such as quarry waste or silty sand is used. Nevertheless it would be feasible to design the core or cut off to control unacceptable internal seepage and inundation of water behind an embankment that could otherwise pose a threat to the long-term stability. For example, the core may be built from a more impervious local material, probably with higher clay content, or could be formed by steel sheet piling or construction of a concrete, asphaltic concrete or cement bentonite cut-off wall.

One of the most common methods for construction of flood embankments is to excavate soil from the adjacent ground, especially the neighbouring ditch For shallow embankments. The excavated soil would normally be placed in layers using light compaction equipment without stick to engineering specification to achieve a target density or permeability (Morris et al., 2007).

River dredging material can also be used for flood embankment. In this method, the sediment has been dredged directly from the riverbed to raise existing embankments. The river dredging are predominantly silts, which allow water to freely drain away. On the other hand, consideration must also be given to the excavating material, because it may not be appropriate for the use in embankments (Morris et al., 2007).

6.6 Contractual Issues

For smooth construction the following should be considered before construction phase takes place

- The location of the embankment (path) and borrow bit areas have to be acquired legally before starting construction so as to avoid any disputes.
- Negotiations and awareness raising among owners may help a lot in accomplishing a legal status if disputes arise.
- This issue may also be extended to the selection of the camp location and availing of water and electric power supply.
- Local contracting (supply of materials, water, food ..etc.) is another issue need to be considered while dealing with the cost/benefit analysis.

6.7 Procurement Procedures

Procurement of any good or service needed for establishment of flood embankment, has to be contracted upon. Therefore, it is important for the planner to be acquainted with reasonable ideas about the procurement of goods and services. The following steps are the common ones for such procurement:

- Preparation of the Terms of References (TOR);

- Estimation of the Budget;
- Advertising for Expression of Interest (EOI);
- Prequalification and Preparation of Shortlist;
- Determination of Selection Procedure;
- Preparation and Issuing of Request for Proposals;
- Evaluation of technical Proposals;
- Opening and Evaluation of Financial Proposals;
- Negotiation and Signing of Contract;
- Notification of Unsuccessful Applicants.

Contracts have to be prepared carefully. FIDIC guidelines may be used as one of the available supporting reference in the preparation of the contracts. During the implementation session, the signed contract has to be managed properly and carefully. Miss-management of the contract may lead to disputes and some unrecoverable negative impacts. For more detailed information refer to procedures and Guidelines for World Bank Financed Projects.

7 Social and Environmental Considerations

As we have seen in the previous chapters, flood disasters result from the interaction between extreme hydrologic events and environmental, social and economic processes. Floods not only have negative consequences but positive impacts as well. They provide valuable natural resources, thereby supporting livelihoods and economic activities. But floods frequently have destructive nature that tremendously affects the economic, social, environment, and even loss of lives. This section briefly discusses the social and environmental considerations for flood mitigation measures particularly the food embankments.

7.1 Land Tenure and ownership systems

Since any flood protection structure would use the land, it is important for the Engineer in charge to be familiar with the land tenure system in the area. Land tenure refers to the group of rights of individuals, households or communities with respect to land. Land is a central issue for both rural and urban communities in Sudan. It is not just a means of livelihood and basic survival, but also has profound cultural and socio-political dimensions.

Land tenure legislation is only superficially different from the colonial legacy. It was in 1970 when the Unregistered Lands Act, was introduced and implemented over the country. The Act declared that all waste, forest, and unregistered lands were government land. It even entitled the Government to use force in safeguarding its “land”; this has further been strengthened by the 1991-1993 amendments to the 1984 Civil Transactions Act, which states that: “No court of law is competent to receive a complaint that goes against the interest of the state”.

The current devolution of powers between the Central Government and the regions under the present Federal (decentralization) system, has also given rise to land claims with conflicting sources of legitimacy and contradictory outcomes regarding who can establish access to and control over land. Therefore it is of utmost important for the Engineer in charge of constructing a flood embankment to conduct the necessary communications at different levels of governance and at the local communities who have customary rights.

7.2 Social considerations

Flooding in an area may incur substantial social and economic costs due to the loss of lives, injuries, damages to assets, disruption to livelihood, education, medical services, and dislocation of families and communities. Enhanced protection from floods provides substantial social benefits for the population of the flood prone areas. This is particularly true for the people living in the inner flood plains between the flood embankments, where flooding is frequent. The Blue Nile, Main Nile and Gash River (Kasala area) flood plains are good examples in the case of Sudan. Negative social impacts may arise due to the land acquisition and resettlement required for construction activities under the project. These, however, should be kept to a minimum in accordance with the appropriate land tenure system. Therefore, the following social consideration should be regarded when designing and operating flood control structures.

7.2.1 Structural and Non-Structural Components of a Project

Construction of physical structures such as embankments and water diversion systems is highly effective in preventing or mitigating floods. However, people's over-reliance on physical structures sometimes results in a lack of preparedness for floods. It is important to keep in mind that non-structural measures such as the establishment of warning systems and community-based water management associations should go hand in hand with the structural component of the project.

In this regard, effective communication among project beneficiaries is highly relevant to securing a positive outcome. The project team needs to ensure that all people in the project area become aware of the warning system and know how to react under sudden disaster situations. Effective communication, for instance, can help avoid the often-cited problem of underutilized evacuation due to lack of awareness and poor communication channels among community members.

7.2.2 Appropriate technology vs. advanced technology

Engineers and project planners should carefully consider the choice of technology because cutting-edge technologies may not be the most appropriate. The adoption of technology that is too advanced to be maintained and managed by communities often reduces sustainability. Applying local knowledge and using locally available resources will help in the selection of the right level of technology for the community.

7.2.3 Protection of Physical cultural resources

The physical cultural resources, are defined as movable or immovable objects, sites, structures, groups of structures, and natural features and landscapes that have archaeological, historical, religious, or other cultural significance. El Bigrawia pyramids and other sites of religious interest such as El Mahdi Dome are good examples to mention for the physical cultural resources may be located in urban or rural settings, and may be above or below ground, or under water. Their cultural interest may be at the local, provincial or national level, or within the international community.

7.2.4 Capital and running costs

Construction, operation and maintenance of flood control structures and warning systems often generate costs to be borne by beneficiaries. Even when projects satisfy the demands of communities, high costs can diminish the level of beneficiary involvement and in turn affect the overall sustainability of the project. Care should be taken to set the costs within the reach of beneficiaries.

7.2.5 Community participation in decision-making

Community participation in flood control projects can take many different forms, such as construction and operation of structures, design of warning systems, or attending training for an active water management association. An active promotion of community participation by project designers and implementers is encouraged, particularly in decision-making processes where local knowledge and technology can be shared with the project team. "Nafeer" which is a form of mutual collective group work, is traditionally practiced in Sudanese rural communities, could be organized by the Engineer in charge to enhance community participation. Limiting community participation to a labour contribution may result in a low sense of project ownership.

7.2.6 Social dimensions checkpoints

The main points that must be put in mind by the planners or designers of a project to be socially acceptable, could be summarized in the answer to the following questions:

- Have the intended beneficiaries been identified?
- Are users willing and able to contribute?
- Is there any group who may experience negative impacts from the project? If so, have appropriate measures been taken to minimize those impacts?

7.3 Environmental Impacts and considerations

The environmental impacts of flooding can be quite wide-ranging, from the dispersion of low-level household wastes into the fluvial system to contamination of community water supplies and wildlife habitats with extremely toxic substances. During a flood, variables such as depth of water, velocity of flows, and duration of inundation, in combination with land-use attributes, all contribute to the relative severity of flood impact. Floods of greater depth are likely to result in greater environmental damage than floods of lesser magnitude, in part because more area has been flooded. Long duration floods will exacerbate environmental problems because clean-up will be delayed and contaminants may remain in the environment for much longer time.

Flood control structures including embankments have impacts on the physical, ecological, and the human environments. Impacts on the physical environment include the surface water and groundwater hydrology, the land and soil resources. The ecological impacts include the changes in the bio-mass production through reduced crop fish production in the flood damaged areas. While the impacts on the human environment include loss of land to embankments and other project works and resultant population displacement is one of the major consequences. Flood control projects require acquisition of substantial land for embankments causing economic hardship to the displaced population.

Environmental effects of flow control structures are overshadowed by the need for protection. Structural flood controls, especially levee systems and flood control reservoirs, deprive river systems of natural disturbance and stimulation. Environmental effects of utmost importance include separation of main channel and natural floodplain, desiccation and destruction of wetland areas, alteration of the hydrologic regime, depression of geomorphic influence of floods, and masking of natural cues.

Wetlands are dynamic centres of biologic activity. Seasonally inundated wetlands are exceptionally productive land areas. Separation of channel and floodplain is an unnatural state that destroys key aquatic habitat and depresses the productive potential of aquatic systems. Structural flood control is generally a deterrent to the health of natural systems, but remains a valuable tool in flood management.

To mitigate the environmental impacts discussed above, it is important to integrate environmental considerations into the planning, design, and implementation of flood protection structures to ensure that potentially negative impacts will be avoided or reduced to an acceptable limit, and to enhance the long term sustainability of all such measures. There is also a need to increase the awareness throughout the community of the value of environmental safeguards as opposed to short-term solutions to isolated problems.

Conducting an Environmental Impact Assessment (EIA) on the project area, will result on identifying the potential environmental impacts. There are guidelines and manuals in practice for environmental impact assessment (EIA) of flood control projects. These guidelines and manuals have been developed to enable the local engineers and other technical staff to better appreciate the environmental issues in all the stages of the flood protection projects. Some manuals deal with small-scale projects undertaken at the local level and others deal with medium- and large-scale projects.

The guidelines constitute simple procedures and formats to guide Initial Environmental Examination (IEE) and Environmental Impact Assessment (EIA) of proposed projects and draw up plans for environmental management. The guidelines may also be used to conduct IEE and EIA of ongoing and implemented projects to identify potential negative impacts, and to design environmental protection measures and appropriate monitoring programs.

7.4 Assessing the Environmental Impacts of a project

7.4.1 Legal and Institutional Aspects of EIA

In Sudan the environment protection issue is covered under the High Council for the Environment and Natural Resources Act. This is further supported by the environmental requirements of the international financial bodies such as the World Bank. In general these legislations require the provision of the general definition of the effects to be considered, in what projects should be applied, and submission of an EIA report.

Internationally there are significant developments in the international environmental law and policy which are relevant to or applicable by the EIA systems of all countries. These can be divided into:

- The Rio Declaration, which is non-binding instrument that establish important principles for sustainable development, including those which need to be reflected in EIA.
- Legal conventions and treaties related to environmental protection at the global or regional level, which carry obligations for signatory countries that may be met through EIA arrangements; and;
- Legal conventions and protocols that apply specifically to EIA arrangements (eg. the Espoo Convention adopted in 1997).

Consideration flood embankments projects in Sudan, it is not upnormal for the project to be small affecting some local community. Hence it is important for those in charge of the project to consider the characteristics of EIA policy and legislation at the local level, and to draw up three lists relating to the EIA requirements as follows:

1. Descriptive characteristics

2. Strengths and positive aspects
3. Weaknesses and/or areas which may be further strengthened

7.4.2 Public and stakeholders involvement

The main objectives of public and stakeholders involvement are to:

- obtain local and traditional knowledge that may be useful for decision-making;
- consider alternatives, mitigation measures and tradeoffs;
- ensure important impacts are considered and benefits are maximised;
- reduce conflict between interested groups through the early identification of issues;
- provide an opportunity for the public to participate in project design in a positive manner;
- ensure transparency and accountability of decision-making; and
- increase public confidence in the EIA process.

The range of stakeholders that are to be involved in an EIA typically includes:

- The people individuals, groups and communities who are affected by the project.
- Any other project beneficiaries;
- Government agencies;
- NGOs and interest groups; and
- Others, such as donors, the private sector, academics etc.

When conducting a public involvement technique, the following points should be considered:

- the required degree of interaction between participants;
- the ability of participants to influence decisions;
- the suitability of the EIA stage to involve stakeholders.
- the time available for involvement;
- the likely number of participants and their interests;
- the complexity and controversy of the issues under consideration; and
- the consideration of cultural norms which may influence the content of discussions, for example relating to gender, religion, etc.

7.4.3 Main steps of the EIA

The major steps in conducting an EIA are:

a/ Screening

Is the process undertaken to decide which level of environmental assessment the project requires. It is simply a decision whether conduct an EIA or not using prescribed lists or criteria. The criteria depends on the nature of the project (infrastructure, heavy industries, resource extraction..etc.), the environmental sensitivity of the project area (eg national parks, watershed reserves, wildlife reserves), and threat to habitats and indigenous wildlife;

The international financial agencies such as the World Bank, categorizes the projects in three groups depending on their environmental impacts. The categorization procedure in use at the World Bank is explained in the World Bank's Operational Manual under Operational

Directive 4.01 (original OD 4.00 is reproduced in Volume 1 of World Bank, 1991 and updated to OD 4.01 in World Bank, 1993). The categories are defined below.

Project Category A: Projects in this category typically require an EIA. The potential significant environmental issues for these projects may lead to significant changes in land use, as well as changes to the social, physical, and biological environment.
Project Category B: This category is for projects that usually require an environmental review but at a level of effort less than that of an EIA study.
Project Category C: This category is for projects that typically do not require an environmental assessment. These projects are unlikely to have adverse environmental impacts.

b/ Scoping

Is the process of determining the issues to be addressed, the information to be collected, and the analysis required to assess the environmental impacts of a project. The primary output of scoping is the terms of reference (TOR) required to conduct an EIA and to prepare the EIA report. The TOR are usually prepared by an EIA expert for approval by the EIA administrative agency.

Scoping is often conducted to serve as an I*nitial Environmental Examination (IEE)*. An IEE is undertaken to determine the probable environmental impacts associated with the project and ascertain whether a full-scale EIA is required. The IEE is usually conducted with a limited budget, and is based on existing information and the professional judgment of people who are knowledgeable about impacts from similar projects. The three primary objectives of the IEE are to:

1. identify the nature and severity of specific environmental issues associated with the project;
2. identify easily implementable mitigative measures for the identified environmental issues. If the IEE shows no significant environmental issues which need further investigation, then the IEE serves as the final EIA Report; and
3. develop the TOR for the full-scale EIA study if needed.

c/ Impact identification and its Significance:

The aim is to take account of all of the important environmental/project impacts and interactions, making sure that indirect and cumulative effects, which may be potentially significant, are not inadvertently omitted. This process begins during screening and continues through scoping, which identifies the key issues and classifies them into impact categories for further study.

The most common formal methods used for impact identification is checklists. Checklists provide a systematized means of identifying impacts. They also have been developed for application to particular types of projects and categories of impacts (such as dams or road building). Sectoral checklists often are useful when proponents specialise in one particular area of development. However, checklists are not as effective in identifying higher order impacts or the inter-relationships between impacts.

The checklist of environmental parameters of a flood control embankment project is shown in Table (7.1) below. If there are checked items in moderate and severe impact columns, then mitigation measures will need to be identified which will lower the adverse effects to a satisfactory level. If no definite mitigation measure can be identified without further analysis, full scale follow-up EIA will be required.

Table (7.1): Checklist of Environmental parameters of small scale projects in EIA

	Positive Impact	No Impact	Adverse Impact		
			Low	Moderate	Severe
I. Ecological					
Tree plantation					
Wetland/Wetland habitat					
Fisheries					
II Physico-Chemical					
Erosion and Siltation					
Soil fertility					
Obstruction to waste water flows					

d/ Impact Significance:

A test of significance can be applied by asking three questions:

- Are there residual environmental impacts?
- If yes, are these likely to be significant or not?
- If yes, are these significant effects likely to occur e.g. is the probability high, moderate or low.

Once the impacts have been analysed, they are evaluated to determine their significance. A systematic process should be followed in evaluating significance, distinguishing between as “predicted” and “residual” impacts. First evaluate the significance of the “predicted” impacts to define the requirements for mitigation and other remedial actions. Second evaluate the significance of the “residual” impacts, i.e. after mitigation measures are taken into account. This test is important to decide whether or not a proposal is likely to cause significant impacts. It is determined by the joint consideration of its characteristics (magnitude, extent, duration etc.) and the importance (or value) that is attached to the resource losses, environmental deterioration. However, the environmental acceptability of a proposal and the terms and conditions to be attached to its implementation must be weighed against other economic and social factors by the decision-maker.

Evaluation of significance should take place against a framework of criteria and measures established for the purpose. EIA guidelines related to significance fall into two main categories: *emissions based* and *environmental quality based*. Flood embankments fall under the second category which comprise significance criteria for valued ecosystem components or similar attributes. Environmental quality based criteria or thresholds are qualitative, broadly drawn and require interpretation. Some countries and international agencies have established environmental sustainability criteria and environmental acceptability rules against which evaluation can be conducted. For example, the World Bank input and output guidelines are meant to ensure that each project does not exceed the regenerative and assimilative capacities of the receiving environment (see Annex 6.B, *World Bank 1991).*

Principles for evaluating significance include:

1) Key reference points for evaluating significance:

- environmental standards, guidelines and objectives;
- level of public concern (particularly over health and safety);
- scientific and professional evidence for:
    - loss/disruption of valued resource stocks and ecological functions;
    - negative impact on social values, quality of life and livelihood; and
    - foreclosure of land and resource use opportunities.

2) Guiding principles for determining significance:

- use procedure and guidance established by the jurisdiction;
- adapt other relevant criteria or identify points of reference from comparable cases;
- assign significance in a rational, defensible way;
- be consistent in the comparison of alternatives; and
- document the reasons for the judgments made.

Significance Criteria include:

1) Criteria to evaluate whether or not adverse impacts are significant:

- environmental loss and deterioration;
- social impacts resulting directly or indirectly from environmental change;
- non-conformity with environmental standards, objectives and guidelines; and
- likelihood and acceptability of risk.

2) Criteria to evaluate adverse impacts on natural resources, ecological functions or designated areas:

- reductions in species diversity;
- depletion or fragmentation on plant and animal habitat;
- loss of threatened, rare or endangered species;
- impairment of ecological integrity, resilience or health e.g.
    - disruption of food chains;
    - decline in species population;
    - alterations in predator-prey relationships.

3) Criteria to evaluate the significance of adverse social impacts that result from biophysical changes:

- threats to human health and safety e.g. from release of persistent and/or toxic chemicals;
- decline in commercially valuable or locally important species or resources e.g. fish, forests and farmland;
- loss of areas or environmental components that have cultural, recreational or aesthetic value;
- displacement of people e.g. by dams and reservoirs;
- disruption of communities by influx of a workforce e.g. during project construction; and
- pressures on services, transportation and infrastructure.

4) Environmental standards, objectives and targets to evaluate significance:

- prescribed limits on waste/emission discharges and/or concentrations;
- ambient air and water quality standards established by law or regulations;
- environmental objectives and targets contained in policy and strategy; and
- approved or statutory plans that protect areas or allocate, zone or regulate the use of land and natural resources.

A project must undergo a *full scale EIA* if it is explicitly prescribed by law (or regulation) or if the IEE results indicate that an EIA is required. A full-scale EIA normally involves a rigorous study whereby new environmental information is collected. A number of environmental experts are generally required. A full-scale EIA may also undergo or involve elaborate review procedures and requirements for public consultation. A detailed EIA report is required as part of a full-scale EIA.

An ideal Terms of Reference TOR is shown in Annex (6.A). This comprehensive framework TOR for environmental assessment is prepared for use by desk officers and environmental specialists of bilateral aid agencies, other operational staff of in-country units, and implementing agencies within developing countries. The framework TOR is also applicable to the environmental assessment requirements of multilateral institutions (for example, the ADB and the World Bank). These TOR are a useful standard. The framework TOR outlines the requirements for two qualitatively different types of information: 1) detailed project justification; and 2) detailed environmental assessment information.

Detailed environmental assessment information includes:
The a project description;

- a description of the environment;
- information quality;
- positive impacts;
- negative impacts on:
  - natural resources and human resources
  - resettlement and compensation
  - cumulative impacts
  - trans-boundary impacts
  - significance of impacts;
  - mitigation measures;
  - an environmental management plan; and
  - an environmental monitoring program/plan.

The above information is to be provided by the project proponent or the EIA practitioners who are responsible to prepare the environmental assessment report. The report should have the following contents:

Executive Summary

1. Introduction
2. Description of the Project
3. Description of the Environment
4. Anticipated Environmental Impacts and Mitigation Measures
5. Alternatives
6. Environmental Monitoring

7. Additional Studies
8. Environmental Management Plan and Environmental Management Office
9. Summary and Conclusions
10. Annexes

e/ ***The EIA report is then*** *Reviewed* by a review agency or by a special "Standing Committee" or "Commission" established to review projects in a given sector. In most cases, a technical evaluation of the EIA report is made by specialists. The output of the review is either a rejection of the project, or an approval report outlining terms and conditions under which the project may proceed. The World Bank and other international financing agencies also use experts for the review and evaluation of EIA reports submitted to them as part of their environmental assessment requirements.

f/ ***The Approval*** of the EIA report rests on the agency that is responsible for ultimately approving the proposed project. In many jurisdictions, project approval also depends on approval from the EIA agency. The high council for environment and natural resources is official body in case of Sudan. One output of the EIA review process is the terms and conditions that are attached to approvals. These terms and conditions define the environmental protection measures that must be integrated into a project. The terms and conditions may also specify environmental monitoring that must be undertaken in conjunction with the project.

g/ ***Environmental management Plan*** One of the goals of the EIA process is to develop an implementable set of environmental protection measures. These measures are normally outlined in an environmental managemnent plan which are to be taken to: 1) mitigate environmental impacts; 2) provide compensation for lost environmental resources; or 3) enhance environmental resources.
The environmental management plan outlines the mitigations and other measures that will be undertaken to ensure compliance with environmental laws and regulations, to reduce or eliminate adverse impacts, and to promote feasible environmental enhancement measures.

Example Environmental Management Plan:
As required by the Sudanese High Council for Environment and Natural Resources, the Ministry of Irrigation and Water resources commissioned an environmental management plan as part of the EIA for the Flood Embankment of ---------------------------------
This plan covers The full description of the embankment, construction areas such as excavation, borrow and spoil disposal sites and access roads, base camps, etc. The management plan included sections on project approach, preconstruction phase issues, construction phase issues, operation and maintenance phase issues, and implementation of environmental management requirements. In each section, measures relating to environmental management were detailed. Technical, economic, and institutional approaches were used for environmental management. These approaches were described in great detail in the management plan and are summarized below:
Technical approaches included:

- *Selection of the embankment alignment to minimize impacts associated with inundated area.*
- *Hydraulic design and drainage works to minimize upstream and downstream flooding;*
- *Design of embankment and drainage works to minimize erosion and land sliding;*
- *Design to maintain access to schools, mosques, agricultural land, and other communities together with design of appropriate structures, taking into account such future needs as could reasonably be foreseen.*

h/ **The** ***environmental monitoring plan*** is concerned with the systematic collection of data to determine 1) the actual environmental effects of a project; 2) the compliance of the project with regulatory standards; or 3) the degree of implementation of environmental protection measures and their success as environmental protection measures. The information generated by monitoring programs provides the feedback necessary to ensure that environmental protection measures have been effective in helping achieve an environmentally sound project.

Example Environmental Monitoring Plan:

The Ministry of Irrigation and Water Resources of Sudan commissioned an EIA for the Flood Embankment of --Project.

An environmental monitoring plan was prepared as part of an overall quality control measure to ensure that environmental protection measures as detailed in the environmental management plan were adopted, and to make sure that any enforcement measures needed were carried out. In addition, the plan aimed to assess the effectiveness of the environmental management measures in practice; to provide information on which to base additional environmental protection measures; and to provide feedback on the magnitude and nature of actual impacts. Monitoring was to include all areas covered in the environmental management plan as well as communities within the vicinity of the project who could experience social impacts as a result of the movement of displaced people.

The monitoring plan included sections on impacts from each of the four project phases (preconstruction, construction, operation, and maintenance), and on the implementation of environmental monitoring.

The monitoring plan recognized that adverse social impacts associated with the land-take were likely to be the most important category of impacts associated with the project. Detailed social studies of eight sample settlement areas were the basis for the assessment of these impacts. Project implementation, however, had already proceeded to the point that many of the recommendations for mitigating these impacts through land acquisition, resettlement, and restoration of incomes were already underway before the environmental management plan was prepared. Monitoring was unlikely to significantly improve environmental management in this project,

During the preconstruction phase notice boards were constructed to provide affected communities with information concerning the project. These notice boards were monitored to establish whether or not people were receiving up-to-date and relevant information about the project. The success in replacing public facilities that will be drowned by the flood embankment, such as mosques, schools, and health facilities that had to be demolished was also monitored.

Construction phase monitoring was extensive and detailed. In addition to a plan for monitoring related to the deployment of heavy plant, equipment, and materials, the deployment of the contractor's work force, the base camp, land clearance, construction of earthworks, construction of embankment base and surfacing, excavation and borrow areas, spoil disposal, and other major spillway structures.

Monitoring in the operation and maintenance phase was to be focused on social aspects, erosion and slope stability, debris waste, and traffic.

Implementation of the monitoring plan was organized by construction phase. Organizational aspects, executive responsibilities, procedures, and financial aspects are also determined.

i/ Roles and Responsibilities of Groups Involved in the EIA System: There are many actors in the EIA process. Each has an important role to play (see Annex 6.C). An effective EIA system gives each actor ample opportunity for participation. The following is a brief description of those roles:

The *EIA administrative agency* has responsibility for efficient operation of the EIA process. This encompasses a number of tasks, including screening of projects and provision of general procedural advice to the project proponents throughout the EIA process. In cases where an IEE or full-scale EIA is required, the EIA agency will approve the TOR for the EIA report. The EIA agency manages the review of the EIA report and is responsible for any approvals or recommendations associated with the EIA. In most jurisdictions, the EIA agency is responsible for verifying that environmental protection measures are properly implemented.

The *project proponent* is the entity with overall responsibility for the project. The proponent may be a private sector developer, a government agency, a joint venture, or some combination of these. The proponent is responsible for providing the scientific and technical information necessary at all stages of the EIA process. Proponents usually contract outside experts skilled in EIA to assist them in this task. The proponent is also responsible for providing access to information about the project activities and the environmental setting of those activities.

Environmental practitioners act for the proponent, the EIA agency, and governmental project implementing agencies. Environmental practitioners can be drawn from private consultancy practices, project proponent personnel, government utilities and infrastructure development agencies, scientific and technical institutes, and academia. They have considerable influence on the scientific and technical aspects of the EIA review process.

Other government agencies: EIA is usually conducted in conjunction with the project approval process. Responsibility for granting final project approval may lie with a planning agency or an economic development agency. This agency normally is involved throughout the EIA process. At the beginning of the project approval process, the agency ensures that the project proponent is aware of the requirements of the EIA process, and may refer the proponent to the EIA administrative agency.

The Public Most development projects affect a wide range of people with varied interests. Public participation is required to allow the affected people to identify significant environmental and social issues. An effective EIA process takes issues raised by the public into account in the project design, or addresses the issues through appropriate environmental protection measures. Many development projects have failed because their designs did not address local needs or were not appropriate to the socioeconomic context of the locality.

International Assistance Agencies: Most projects funded by loans from International Assistance Agencies, must undergo an EIA. These agencies operate on the principle that responsibility for the preparation and review of the EIA rests with the recipient country. In some cases, however, they provide technical assistance for the EIA pursuant to local EIA laws and regulations.

Academic Institutions: Universities and other academic organizations can assume several roles in the EIA process. They may assemble teams to perform EIAs because they have access to different disciplines on their faculties. The same advantage gives them a role in reviewing EIA drafts; more importantly, they usually have an independence from the project that is difficult to find in other sources of reviewers.

7.4.4 Environmental Impacts and Mitigation Measures

The recommended mitigation measures for the identified significant environmental impacts are the core of a successful EIA. It is preferable to present this information in terms of the various stages of the project: preliminary design, final design, construction and operation. Addressing impacts through the associated project stage indicates which aspects of the project will require mitigative actions in the form of design changes, and matches the decisions regarding mitigation with the project implementation schedule. For example, in flood embankments it is good to:

- Design the embankment alignment to minimize inundated areas.
- Design the outlet works to minimize erosion and land sliding.
- Design for easy access to public facilities if any.

8 Maintenance and Inspections

This chapter describes the maintenance and routine inspection procedures needed for safe and sustainable use of flood embankments. However, first, different causes of embankment failures are briefly outlined.

8.1 Causes of embankments failures

The principal causes of earth embankments failures are:

8.1.1 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.

8.1.2 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 (8.1).
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 (8.2).

8.1.3 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 (8.3).
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, Figure (8.4).

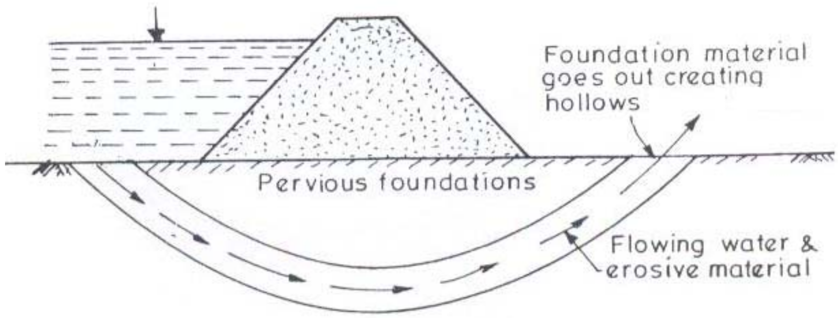


Figure 8.1: Piping through embankment foundation

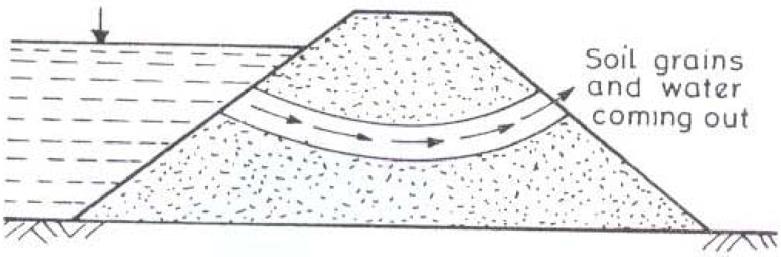


Figure 8.2: Piping through the embankment body

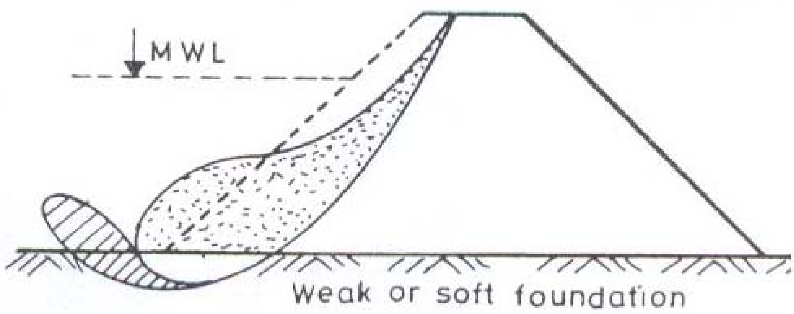
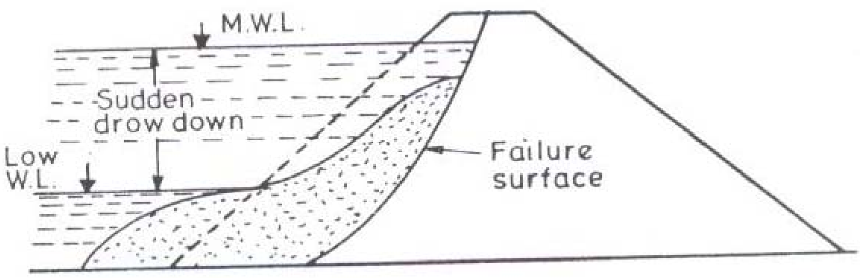
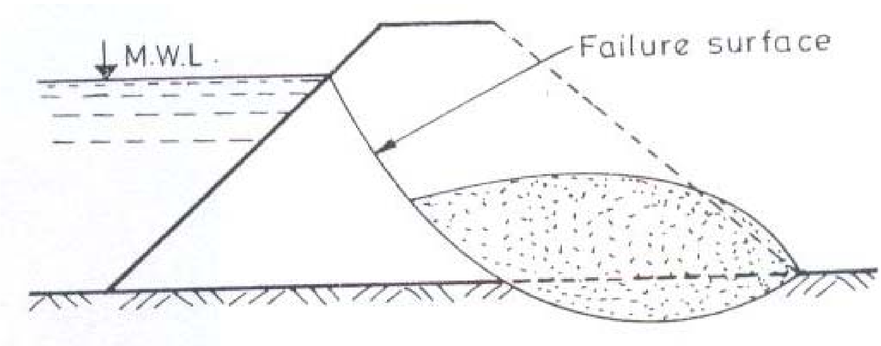


Figure 8.3: Sliding due to soft or weak foundation



(a)



(b)

Figure 8.4: Slope slide a) u/s due to sudden draw-down b) d/s slide due to lengthy flood wave

8.2 Safety Requirements

When the crest of an embankment is used as a highway, cable or beam-type guardrails are usually constructed along both shoulders of the crest. If little or no traffic will use the crest, treatment may not be necessary.

The measures to prevent batter failure by erosion, land-side, gully formation, piping, etc are:

- proper maintenance, filling the cuts from time to time;
- Grassing the slopes (batter)
- Provide proper berms at suitable heights
- Provide proper drainage arrangement for the removal rain water collected on the crest and the horizontal berms.
- Protection of the slopes by riprap, stone pitching, masonry,.etc.

Measures to prevent piping through the embankment body would be through:

- Compacting the soil near the inlet and outlet of pipes and culverts passing through the embankment.

- Protection of the slopes by geotextile; grouted stones, sandbags, stone asphalt, etc.

8.3 Communal Involvement and Managerial Issues

Community participation in flood control projects such as protection embankments is vital to the success of the project since they contribute in planning, design and construction phases and this gives them a sense that they are part of the project. More important is that they are the real managers; they operate, inspect, maintain and keep the project in good shape for their own benefit.

8.4 Maintenance

8.4.1 Critical conditions

The following conditions are critical, call for immediate repair or maintenance, and should trigger a response from the community:

- Erosion, slope failure, or other conditions that are endangering the integrity of the earth dam
- Piping and internal erosion as evidenced by increasingly cloudy seepage or other symptoms
- Spillway blockage or restriction
- Excessive or rapidly increasing seepage appearing anywhere near the embankment site

Critical items such as these should be detected during routine inspection. Other items require maintenance either routinely or at the earliest possible date.

8.4.2 Schedule of maintenance

Whenever the yearly inspection or more frequent informal inspections identify items requiring maintenance, they should be noted in the Operations Log and added to the work schedule. The following maintenance items should be completed as soon as possible after identification:

- Removal of underbrush and trees
- Repair of erosion gullies
- Repair of defective gates
- Repair of deteriorated metal components
- Maintenance of riprap covers if there is any

In addition, continued maintenance should be performed for the following items:

- Testing, cleaning, of gates
- Removal of debris from the outlet intake structure
- Removal of debris from embankment

8.4.3 Embankment Maintenance

- Fill erosion gullies with properly compacted, cohesive soil, material. Seed or riprap the surface.
- Remove rodents. Rodent burrows can provide paths for seepage. Fill rodent burrows with slurry of soil, cement, and water.
- Maintain grass cover and spray for weeds.
- Remove brush, bushes, and trees from the embankment. Remove tree roots. Compact and seed these areas. Clear areas are necessary in order to properly inspect the embankment.
- If necessary add or replace riprap to the upstream slope.
- Remove debris from outlets. Debris can seriously endanger the embankment by plugging outlets.

8.4.4 Outlet Works Maintenance

For the hydraulic structures, such gates or locks attached to a flood embankment, the following maintenance items are of importance.

- Test gate annually.
- Lubricate gate annually if it has moving parts
- Repair the gate if it becomes defective. Ensure smooth operation.

- Repair any deteriorated concrete on the inlet structure, outlet pipe, outlet structure, and gate house.
- Repair any deteriorated metal on the gate controls.

8.5 Inspections

The inspection is a preliminarily stage which comes before the maintenance. The most important stage is the monitoring stage. Because, if defect has been monitored early, then it could be inspected urgently, and finally apply the required maintenance for that defect as mentioned below. The order of these steps may save a lot of life and goods in the affected area. Inspection will be scheduled and completed:

- Yearly for routine maintenance inspections
- Periodically (not to exceed five years) for comprehensive inspections and engineering reviews
- After critical events including severe rain or wind storms, earthquakes, or periods of extremely high storage

8.5.1 The importance of flood embankment inspections:

- Ensure the flood embankment system will perform as expected.
- Identify deficiencies or areas that need monitoring or immediate repair.
- Continuously assess the integrity of the flood embankment system to identify any changes over time.
- Collect information to help inform decisions about future actions.
- Provide the public with information about the flood embankment on which they rely.

8.5.2 Inspection Areas of Interest:

The following areas of flood embankment needs special consideration during inspection.

- Depressions/Settlement.
- Seepage/Saturation.
- Surface Erosion.
- Animal Burrows.
- Encroachments.
- Animal Crossing.
- Ramps/Cross over.
- Slope Stability.
- Revetments/Riprap.
- Toe Drainage System.
- Building / Structure.

8.5.3 Periods of Inspection

The community, as the embankment owner, will assign personnel either from the community or outside the community to conduct the annual maintenance inspection. Inspections of the embankment should occur when the river is full, while inspections of the outlet works should occur while it is empty. However, a qualified professional engineer will conduct the periodic inspection and engineering review every five years at most. For critical event inspection, a qualified professional engineer should conduct these inspections to determine the impact of the critical event on the embankment's performance. All inspections should be documented, including documentation with regard to actions taken to correct recommendations resulting from these inspections.

Inspection during flood time is very important because there is little time to do the maintenance. It needs many people to thoroughly monitor the flood embankment. In this respect, community involvement is very useful. Maintenance using local means (sacks, tree branches, earth or stones dumping ... etc.) could be very helpful to ensure embankment safety during the flood season.

8.6 Effect of Budget flow on Maintenance Scheduling:

Usually the periodic maintentance of the flood embankment should be done during the dry season. In practice this is not the case. Hence, negative impacts arise during severe flood embankment failures. Much amount of money need to be spent, which is more larger than the accumulated amount of money supposed to be paid for periodic maintenance up to the time of the event. Figure (8.5) below shows an example (Kassala protection work) of such comparison.

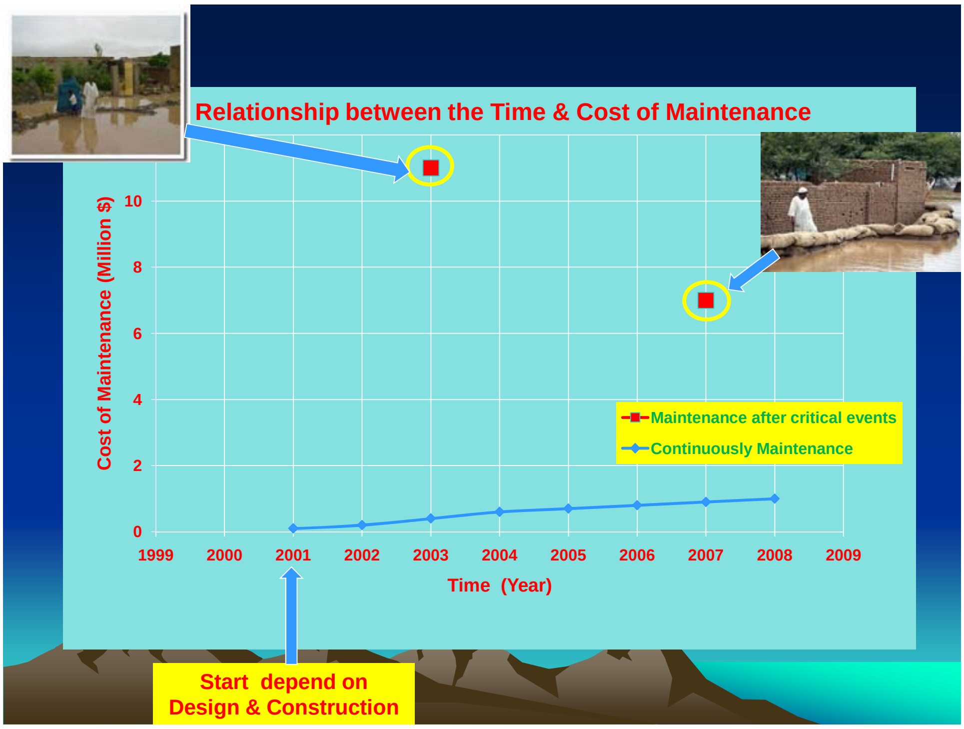


Fig. (8.5): Comparison of Periodic and Causal Maintenance Budget (Kassala Protection works).

9 Conclusions and Recommendations

High riverine floods causes massive damages to property along the Eastern Nile of the Sudan. Flash floods in the wadis (khors) and in the urban areas is a second source of flooding, which causes extensive damages, in particular if synchronizes with riverine flooding. It seems that, the frequency of high river floods has increased in the last 2 to 3 decades. This has exacerbates the suffering of the poor farmers and residents living along the rivers.

The possibilities for effective flood management in the Sudan are confined to early warning, and flood embankments near urban areas. There are limited opportunities to control floods by storage reservoirs because of inadequate capacities.

It has been recognized that the ability of the government engineers and private sector is lacking to design, construct and maintain proper flood embankments in the Sudan. In this regard the FPWP of ENTRO has requested this consultancy assignment from the HRS of the Sudan to prepare a flood embankment manual, and train young engineers of the government and private sector to properly design, construct and maintain flood embankments.

This manual includes main steps needed to design, construct, and maintain a flood embankment manual. Procedure to assess possible impacts has been discussed as well. The steps has been listed in sequential order and formulated in simple format for easy use by young engineers from the Sudan. It starts with key hydrological variables, e.g., rainfall and flow time series analysis, estimation of maximum probable flood level, and flood discharge. The hydraulic design covers key aspects of flood hydraulics and river morphology, such as height of embankment, layout of embankment, and stability of slopes. It also addresses scouring depth and possible sedimentation effects. Third, the manual covers geotechnical and soil issues of flood embankments, such as fill material, soil compaction, and foundation requirements. The stability of slopes being critical for safety of flood embankment has been addressed in more details, and provided design options for stable slopes. The construction aspects has been outlined, including construction material, contractual and procurement, crossing structures and accessibility requirements, preparation of the construction site, and involvement of local communities. The impacts of floods embankments on environment and the socio-economic life of local people have been discussed in brief. A check list required for Environmental Impact Assessment has been prepared. Finally, the causes of possible embankment failures has been reviewed (hydraulic, structural, and seepage), to recommend for routine maintenance and inspection. Guidelines for routine maintained and safety measures have been prepared.

The manual includes data from existing experience in the Sudan, and local information collected from various departments. These are valuable information, in addition to the theoretical background for the design of flood embankment. As much as possible these examples have been simplified to serve the required applications and targeted users (young engineers from Sudan). The discussion and feedback from the training workshop will be used to further improve the manual, and make it more case specific.

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11 APPENDIX 1: RELEVANT TABLES OF RUNOFF ANALYSIS.

Table (A1.1): Runoff Coefficients for Urban Watersheds (Source: Urban Hydrology for Small Watershed TR-55, NRCS,1986)

Type of Drainage Area	**Runoff Coefficient**
Business:	
downtown areas	0.70-0.95
neighbourhood areas	0.30-0.70
Residential:	
single-family areas	0.30-0.50
multi-units, detached	0.40-0.60
multi-units, attached	0.60-0.75
Suburban	0.35-0.40
apartment dwelling areas	0.30-0.70
Industrial:	
light areas	0.30-0.80
heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.30-0.40
Railroad yards	0.30-0.40
Unimproved areas:	
sand or sandy loam soil, 0-3%	0.15-0.20
sand or sandy loam soil, 3-5%	0.20-0.25
black or loessial soil, 0-3%	0.18-0.25
black or loessial soil, 3-5%	0.25-0.30
black or loessial soil, >5%	0.70-0.80
deep sand area	0.05-0.15
steep grassed slopes	0.70
Lawns:	
sandy soil, flat 2%	0.05-0.10
sandy soil, average 2-7%	0.10-0.15
sandy soil, steep 7%	0.15-0.20
heavy soil, flat 2%	0.13-0.17
heavy soil, average 2-7%	0.18-0.22
heavy soil, steep 7%	0.25-0.35
Streets:	
Asphaltic	0.85-0.95
concrete	0.90-0.95
brick	0.70-0.85
Drives and walks	0.75-0.95
Roofs	0.75-0.95

Table (A1.2): Runoff Coefficient for Rural Watersheds. (Source: Urban Hydrology for Small Watershed TR-55, NRCS, 1986)

	Extreme	**High**	**Normal**	**Low**
Relief - Cr	0.28-0.35 steep, rugged terrain with average slopes above 30%	0.20-0.28 hilly, with average slopes of 10-30%	0.14-0.20 rolling, with average slopes of 5-10%	0.08-0.14 relatively flat land, with average slopes of 0-5%
Soil Infiltration - Ci	0.12-0.16 no effective soil cover either rock or thin soil mantle of negligible infiltration capacity	0.08-0.12 slow to take up water, clay or shallow loam soils of low infiltration capacity or poorly drained	0.06-0.08 normal; well drained light or medium textured soils, sandy loams	0.04-0.06 deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover - Cv	0.12-0.16 no effective plan cover, bare or very sparse cover	0.08-0.12 poor to fair; clean cultivation, crops or poor natural cover, less than 20% of drainage area over good cover	0.06-0.08 fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	0.04-0.06 good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover
Surface - Cs	0.10-0.12 negligible; surface depression few and shallow, drainage ways steep and small, no marshes	0.08-0.10 well defined system of small drainage ways, no ponds or marshes	0.06-0.08 normal; considerable surface depression storage lakes and ponds and marshes	0.04-0.06 much surface storage, drainage system not sharply defined; large floodplain storage of large number of ponds or marshes

Table (A1.3): Runoff Coefficient Adjustment Factors for Rational Method.

Recurrence Intervals (years)	**Cf**
25	1.10
50	1.20
100	1.25

Table (A1.4): Runoff Curve Numbers for Urban Areas (Source: NRCS National Engineering Handbook,1993)

Cover Type and Hydrologic Condition	Average Percent Impervious Area	A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.)					
Poor condition (grass cover < 50%)		68	79	86	89

Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas:					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

Notes: Values are for average runoff condition, and Ia = 0.2S. The average percent impervious area shown was used to develop the composite RCNs. Other assumptions are: impervious areas are directly connected to the drainage system, impervious areas have a RCN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition

Table(A1.5): Runoff Curve Numbers for Cultivated Agricultural Land1

Cover Type	Treatment2	Hydrologic Condition3	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
Close-seeded	SR	Poor	66	77	85	89
or broadcast		Good	58	72	81	85
Legumes or C		Poor	64	75	83	85
Rotation		Good	55	69	78	83
Meadow	C&T	Poor	63	73	80	83

		Good	51	67	76	80

Source: NRCS National Engineering Handbook, 1993

Notes:
1 Values are for average runoff condition, and Ia = 0.2S.
2 Crop residue cover applies only if residue is on at least 5 percent of the surface throughout the year.
3 Hydrologic condition is based on a combination of factors affecting infiltration and runoff: density and canopy of vegetative areas, amount of year-round cover, amount of grass or closed-seeded legumes in rotations, percent of residue cover on land surface (good > 20 percent), and degree of roughness.
Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better infiltration and tend to decrease runoff.

Table (A1.6): Runoff Curve Numbers for Other Agricultural Lands

Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Meadow – continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush – brush-weed-grass mixture, with brush the major element	Poor Fair Good	48 35 30	67 56 48	77 70 65	83 77 73
Woods – grass combination (orchard or tree farm)	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods	Poor Fair Good	45 36 30	66 60 55	77 73 70	83 79 77
Farmsteads – buildings, lanes, driveways, and surrounding lots		59	74	82	86

Source: NRCS National Engineering Handbook, 1993

Notes: Values are for average runoff condition, and Ia = 0.2S.

Pasture: Poor is < 50% ground cover or heavily grazed with no mulch, Fair is 50% to 75% ground cover and not heavily grazed, and Good is >75% ground cover and lightly or only occasionally grazed.
Meadow: Poor is <50% ground cover, Fair is 50% to 75% ground cover, Good is >75% ground cover.
Woods/grass: RCNs shown were computed for areas with 50 percent grass (pasture) cover. Other combinations of conditions may be computed from RCNs for woods and pasture.

Woods: Poor is forest litter, small trees, and brush destroyed by heavy grazing or regular burning. Fair is woods grazed but not burned and with some forest litter covering the soil. Good is woods protected from grazing and with litter and brush adequately covering soil.
Meadow: Poor is <50% ground cover, Fair is 50% to 75% ground cover, Good is >75% ground cover.
Woods/grass: RCNs shown were computed for areas with 50 percent grass (pasture) cover. Other combinations of conditions may be computed from RCNs for woods and pasture.
Woods: Poor is forest litter, small trees, and brush destroyed by heavy grazing or regular burning. Fair is woods grazed but not burned and with some forest litter covering the soil. Good is woods protected from grazing and with litter and brush adequately covering soil.

Table (A1.7): Runoff Curve Numbers for Arid and Semi Arid Rangelands

Cover Type	Hydrologic Condition	A	B	C	D
Herbaceous—mixture of grass,	Poor		80	87	93
weeds, and low-growing brush,	Fair		71	81	89
with brush the minor element	Good		62	74	85
Oak-aspen—mountain brush	Poor		66	74	79
mixture of oak brush, aspen,	Fair		48	57	63
mountain mahogany, bitter brush,	Good		30	41	48
maple, and other brush					
Pinyon-juniper—pinyon, juniper,	Poor		75	85	89
or both; grass understory	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
saltbush, greasewood, creosote-	Poor	63	77	85	88
bush, blackbrush, bursage, palo	Fair	55	72	81	86
verde, mesquite, and cactus	Good	49	68	79	84

Source: NRCS National Engineering Handbook,1993

Notes. Values are for average runoff condition, and Ia = 0.2S.

Hydrologic Condition: Poor is <30% ground cover (litter, grass, and brush overstory), air is 30% to 70% ground cover, Good is >70% ground cover.
Curve numbers for Group A have been developed only for desert shrub

Table (A1.8): Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons.

Antecedent Condition	Description	Growing Season 5-Day Antecedent Rainfall	Dormant Season 5-Day Antecedent Rainfall
Dry AMC I	An optimum condition of watershed soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes pace	Less than 1.4 in. or 35 mm	Less than 0.05 in. or 12 mm
Average AMC II	The average case for annual floods	1.4 in. to 2 in. or 35 to 53 mm	0.5 to 1 in. or 12 to 28 mm
Wet AMC III	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2 in. or 53mm	Over 1 in. or 28 mm

Source: NRCS National Engineering Handbook,1993

Table (A1.9): Coefficients for Graphical peak discharge procedure (Source: Hydraulic Design Manual, 2009)

Rainfall Type	Ia/P	C0	C1	C2
II	0.1	2.5532	-0.6151	-0.164
	0.3	2.4653	-0.6226	-0.1166
	0.35	2.419	-0.6159	-0.0882
	0.4	2.3641	-0.5986	-0.0562
	0.45	2.2924	-0.5701	-0.0228
	0.5	2.2028	-0.516	-0.0126
III	0.1	2.4732	-0.5185	-0.1708
	0.3	2.3963	-0.512	-0.1325
	0.35	2.3548	-0.4974	-0.1199
	0.4	2.3073	-0.4654	-0.1109
	0.45	2.2488	-0.4131	-0.1159
	0.5	2.1777	-0.368	-0.0953

Table (A1.10): Ponding Adjustment Factor (Source: Hydraulic Design Manual, 2009)

% Ponded/Swamp Area	Factor (F)
0	1
0.2	0.97
1	0.87
3	0.75
5	0.72

Table(A1-11): Roughness coefficients for sheet flow

Surface Description	**n**
Hot Mix Asphalt	0.011- 60.01
concrete	0.012 0.014
Brick with cement mortar	0.014
Cement rubble	0.024
Fallow (no residue)	0.05
Grass Short grass prairie Dense grass Bermuda Grass	 0.15 0.24 0.41
Woods(2) Light underbrush Dense underbrush	 0.40 0.80

Table(A1-12): Coefficients for shallow concentrated flow

Land cover/ Flow regime	**K (m/s)**
Forest with heavy ground litter : hay meadow	0.076
Trash fallow or minimum tillage cultivation : contour or strip cropped: woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled – alluvial fans	0.305
Grassed waterway	0.457

12 Appendix 2: Applications examples in hydrology

This Appendix gives further details on the hydrological considerations, starting with rainfall analysis of the gauging station of El Managil town by identifying the suitable probability distribution. The rational method calculated based on a catchment data from Gedaref area. Unit hydrograph computation are given based on generic data.

Rainfall of El Managil Town

Rainfall information is basic to the design of all storm facilities. Rainfall is a natural event and precise projections of its frequency and intensity cannot be made. However, useful information can be obtained by analysis of past storms. Reasonable predictions of the frequency of occurrence (recurrence interval), the duration, the amount, the distribution of the amount with respect to area, the distribution of the intensity with respect to time, and the seasonal probability of an occurrence can be made. Rainfall Intensity - Duration – Frequency the most familiar presentation of rainfall data is a set of curves representing different frequencies of occurrence of rainfall events with the intensity of the rainfall plotted against its duration.

The average annual rainfall in Managil (1961-2009) is about 299 mm with a range from 504 mm in 1967 to 115 mm in 1990. The highest amount of rainfall which occurred in a 24-hour period was 150 mm which fell on July25, 2009. The year-to-year variation in total rainfall is a function of the showery nature of the rainfall which is characteristic of the region. July and August are the wettest months accounting for about 66.2% of the average rainfall.

Rainfall characteristics

Precipitation in arid and semi-arid zones results largely from convective cloud mechanisms producing storms typically of short duration, relatively high intensity and limited areal extent.

Rainfall intensity is defined as the ratio of the total amount of rain (rainfall depth) falling during a given period to the duration of the period It is expressed in depth units per unit time, usually as mm per hour (mm/h).

The statistical characteristics of high-intensity, short-duration, convective rainfall are essentially independent of locations within a region and are similar in many parts of the world. Analysis of short-term rainfall data suggests that there is a reasonably stable relationship governing the intensity characteristics of this type of rainfall. Studies carried out in Saudi Arabia suggest that, on average, around 50 percent of all rain occurs at intensities in excess of 20 mm/hour and 20-30 percent occurs at intensities in excess of 40 mm/hour. This relationship appears to be independent of the long-term average rainfall at a particular location.

Table A2.1 - ANNUAL RAINFALL,MANAGIL (Sudan).

Year	R mm	Year	R mm	Year	R mm	Year	R mm	Year	R mm	Year	R mm
1919	346	1936	455	1953	376	1970	326	1987	268	2004	194
1920	556	1937	458	1954	463	1971	367	1988	340	2005	294
1921	341	1938	356	1955	350	1972	205	1989	286	2006	316

1922	578	1939	550	1956	441	1973	235	1990	115	2007	365
1923	437	1940	300	1957	326	1974	291	1991	128	2008	206
1924	339	1941	279	1958	440	1975	443	1992	251	2009	409
1925	279	1942	374	1959	556	1976	273	1993	295		
1926	240	1943	316	1960	271	1977	251	1994	244		
1927	362	1944	477	1961	367	1978	346	1995	278		
1928	319	1945	252	1962	436	1979	237	1996	335		
1929	707	1946	425	1963	319	1980	305	1997	321		
1930	402	1947	250	1964	422	1981	321	1998	382		
1931	221	1948	447	1965	312	1982	221	1999	373		
1932	424	1949	247	1966	270	1983	235	2000	199		
1933	353	1950	416	1967	504	1984	147	2001	222		
1934	381	1951	238	1968	349	1985	439	2002	253		
1935	328	1952	238	1969	262	1986	249	2003	355		

Probability analysis

To determine the probability or frequency of occurrence of yearly or seasonal rainfall the first step is to obtain annual rainfall totals from the area of concern. In where rainfall records do not exist, figures from stations nearby may be used with caution. It is important to obtain long-term records. As explained, the variability of rainfall in arid and semi-arid areas is considerable. An analysis of only 5 or 6 years of observations is inadequate as these 5 or 6 values may belong to a particularly dry or wet period and hence may not be representative for the long term rainfall pattern.

In the following recording rain gauge, 90 annual rainfall totals from Managil were used for an analysis (Table A2.1). The next step is to rank the annual totals from Table A2.2 with m = 1 for the largest and m = 90 for the lowest value and to rearrange the data accordingly. The probability of occurrence P (%) for each of the ranked observations can be calculated from the equation:

$$P(\%) = \frac{m - 0.375}{N + 0.25} \times 100$$

where:
P = probability in % of the observation of the rank m
m = the rank of the observation
N = total number of observations used

Table A2.2: Ranked annual Rainfall Data (MANAGIL Sudan).

Year	R	m	P	Year	R	m	P
	mm		%		mm		%
1967	504	1	2.1	1974	291	16	51.7
1975	443	2	5.4	1989	286	17	55.0
1985	439	3	8.7	1976	273	18	58.3
1962	436	4	12.0	1966	270	19	61.6
1964	422	5	15.3	1987	268	20	64.9
1961	367	6	18.6	1969	262	21	68.2

1971	367	7	21.9	1977	251	22	71.5
1968	349	8	25.2	1986	249	23	74.8
1978	346	9	28.5	1979	237	24	78.1
1988	340	10	31.8	1973	235	25	81.4
1970	326	11	35.1	1983	235	26	84.7
1981	321	12	38.4	1982	221	27	88.0
1963	319	13	41.7	1972	205	28	91.3
1965	312	14	45.0	1984	147	29	94.6
1980	305	15	48.3	1990	115	30	97.9

In the above table, the annual rainfall with a probability level of 68 percent of exceedance is 262 mm i.e. on average in 68 percent of time (2 years out of 3) annual rain of 262 mm would be equalled or exceeded. For a probability of exceedance of 35 percent, the corresponding value of the yearly rainfall is 326 mm.

The return period T (in years) can easily be derived once the exceedance probability P (%) is known from the equations.

$$T = \frac{100}{P}\text{(years)}$$

From the above examples the return period for the 68 percent and the 35 percent exceedance probability events would thus be:

T 68% = 100/ 68 = 1.5 years.

i.e. on average an annual rainfall of 262 mm or higher can be expected in 2 years out of 3;
T 35% = 100/ 35 ≈ 3 years.

Respectively i.e. on average an annual rainfall of 326 mm or more can only be expected in 1 year out of 3.

Table A2.3 - RANKED ANNUAL RAINFALL DATA (Jul. & Aug.), MANAGIL

Year	R	M	P	Year	R	m	P
	mm		%		mm		%
2009	394.50	1	2.1	1980	149.00	16	51.7
1995	354.00	2	5.4	1987	149.00	17	55.0
1998	304.00	3	8.7	1986	148.00	18	58.3
1985	291.00	4	12.0	1989	141.00	19	61.6
1999	281.00	5	15.3	1988	140.00	20	64.9
2003	254.00	6	18.6	1982	138.00	21	68.2
2007	234.30	7	21.9	2002	137.40	22	71.5
2005	219.60	8	25.2	2008	130.50	23	74.8
1997	215.00	9	28.5	2004	113.50	24	78.1
1996	212.00	10	31.8	1993	111.00	25	81.4
2006	209.20	11	35.1	1983	107.00	26	84.7
1981	191.00	12	38.4	1991	96.00	27	88.0
1992	187.00	13	41.7	2000	67.10	28	91.3
1994	167.00	14	45.0	1984	47.00	29	94.6

2001	167.00	15	48.3	1990	39.30	30	97.9

Example For Rational Method

The following is an example problem that illustrates the application of the Rational method to estimate peak discharges. Estimates the peak design discharge needed to design an earth dam at the outlet of khor Abu Garae between El Gadarif and Alshowek, eastern part of Sudan. Design for a 50- year peak runoff

Table (1): Maximum Daily Rainfall Data

Year	Gedarif	Alshowak	Abu Gara
1960	45	61	53
1961	55.4	61	58.2
1962	55.5	45	50.25
1963	59.5	45	52.25
1964	60.3	94	77.15
1965	56.2	90	73.1
1966	89	75	82
1967	71	60	65.5
1968	67	42	54.5
1969	32	60	46
1970	45	65	55
1971	66	37	51.5
1972	78	53	65.5
1973	89	93	91
1974	56	30	43
1975	77	56	66.5
1976	45.7	71	58.35
1977	68	63	65.5
1978	74	75	74.5
1979	73	45	59
1980	89	68	78.5
1981	89.7	37	63.35
1982	89.8	40	64.9
1983		65	65
1984	39.5	32	35.75
1985	65	83	74
1986	55	117	86
1987	37	44	40.5
1988	58.2	76	67.1
1989	73	53	63
1990	54		

catchment data

Using a topographic map of the catchment and a field survey , the area of the drainage basin upstream from the dam site location is found to be 50 km^2. In addition the following data were measured:

Average overland(Sheet flow) slope= 3%
Length of overland flow = 3km
Length of main basin channel= 10 km
Slope of the channel = 8%
Roughness coefficient of the channel was estimated as n= 0.22
Length of shallow concentrated flow= 3 km

Land use

Land use of the catchment was estimated to be:
Grass area 50% of area, silt loam sandy soil ,slope 5%
Land use of overland flow area (sheet flow area) estimated as lawn- silt-loam soil, ,good grass land
Surface consist of well defined drainage system overland flow
A runoff coefficient C for the overland flow area is determined from Table (A1-2) to be

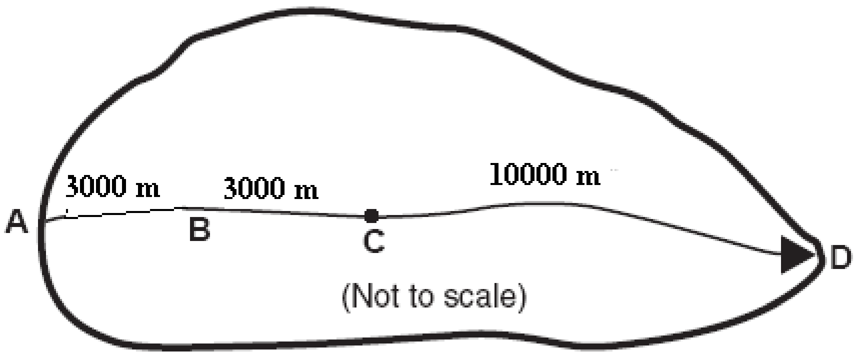
$$C = C_r + C_i + C_v + C_s$$

Cr=0.14, Ci=0.12, Cv =0.08, Cs= 0.08

C= 0.14+0.12 +0.08+0.08=0.42 (42%)

Flow Path

Sheet flow across lawn	3000 m
Shallow concentrated flow	3000 m
Open channel flow	10000 m



Time of Concentration

Sheet flow

Surface description	Lawn Dense grass
Manning's roughness, n (Table A1-11)	0.24
Flow length	3 000

Land slope 0.05

i= 50-yr 24 hr rainfall(Table1) 115 mm/hr

$T_t = \frac{6.92 L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}}$ Tt= 132min

Shallow concentrated flow

Surface description (paved or unpaved) unpaved, short grass

Flow length L 3 000

Water course slope, S 0.07

Average velocity $V = KS^{0.5}$, K=0.213 (Table A2-12) , V=0.052 m/s

$T_t = \frac{L}{60V}$ = 887.2 min

Open channel flow

Knowing hydraulic channel characteristics: cross sectional area, A, Wetted perimeter, Elevation difference between the maximum remote point and the outlet (m) Manning's roughness coefficient and length of water way

Compute the velocity (assuming bankful condition) $V = \frac{1}{n} R^{2/3} S^{1/2}$

Roughness n=0.022,

$T_t = \frac{L}{60V}$, or

Use Kirpich equation

$T_t = 0.01947 L^{0.77} S^{-0.385}$

S=elevation difference/L

Channel slope 0.08

Channel length 10 000

Tt= 71.8 min

Time of concentration Tt=131 +887.2+71.8= 1081 min= 18 hr

Determine rainfall intensity

Use a rainfall duration IDF, for a duration Tc=1081 min and a 50 year return period.

i= 0.58in/hr (1 inch=25.4 mm) i=14.7 mm/hr

Determine the peak discharge Q

Correction factor for T=50-year C_f =1.2

$C = C_f C$ = 0.42 *1.2=0.504

Drainage area 50 km^2

$Q_{50} = \frac{C_f CIA}{3.6}$

Q_{50}=0.504 *14.7*50/3.6= 103 m^3/s

Example: Computation of runoff using Hydrograph method

A. Obtain a Unit Hydrograph for a basin of 315 km^2 of area using the rainfall and streamflow data tabulated below.

Time (h)	Observed Hydrograph (m^3/s)
0	100
1	100
2	300
3	700
4	1000
5	800
6	600
7	400
8	300
9	200
10	100
11	100

Time (h)	Gross Precipitation (GRH) (cm/h)
0 - 1	0.5
1 - 2	2.5
2 - 3	2.5
3 - 4	0.5

Empirical Unit Hydrograph Derivation
Separate the baseflow from the observed streamflow hydrograph in order to obtain the Direct Runoff Hydrograph (DRH). For this example, use the horizontal line method to separate the baseflow. From observation of the hydrograph data, the streamflow at the start of the rising limb of the hydrograph is 100 m^3/s.

Compute the volume of Direct Runoff. This volume must be equal to the volume of the Effective Rainfall Hyetograph (ERH).

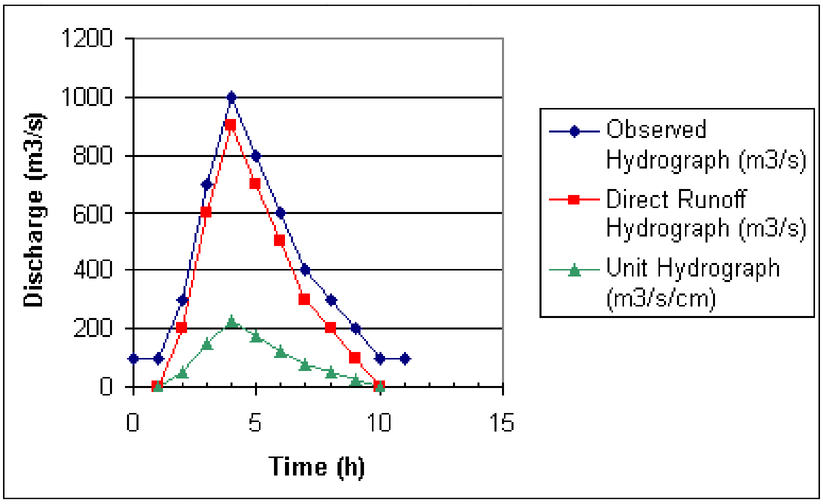
$$V_{DRH} = \int_t Q_{DRH}(t)dt \cong \sum_i Q_{DRH_i} \Delta t$$

Thus, for this example:
V_{DRH} = (200+600+900+700+500+300+200+100) m^3/s (3600) s = 12'600,000 m^3
Express V_{DRH} in equivalent units of depth:

V_{DRH} in equivalent units of depth = V_{DRH}/A_{basin} = 12'600,000 m^3/(315000000 m^2) = 0.04 m = 4 cm.

Obtain a Unit Hydrograph by normalizing the DRH. Normalizing implies dividing the ordinates of the DRH by the V_{DRH} in equivalent units of depth.

Time (h)	Observed Hydrograph (m^3/s)	Direct Runoff Hydrograph (DRH) (m^3/s)	Unit Hydrograph (m^3/s/cm)
0	100	0	0
1	100	0	0
2	300	200	50
3	700	600	150
4	1000	900	225
5	800	700	175
6	600	500	125
7	400	300	75
8	300	200	50
9	200	100	25
10	100	0	0
11	100	0	0



Determine the duration D of the ERH associated with the obtained UH. In order to do this: Determine the volume of losses, V_{Losses} which is equal to the difference between the volume of gross rainfall, V_{GRH}, and the volume of the direct runoff hydrograph, V_{DRH} .

$V_{Losses} = V_{GRH} - V_{DRH} = (0.5 + 2.5 + 2.5 + 0.5)$ cm/h 1 h - 4 cm = 2 cm

Compute the ϕ-index equal to the ratio of the volume of losses to the rainfall duration, t_r. Thus,

ϕ-index = V_{Losses}/t_r = 2 cm / 4 h = 0.5 cm/h

Determine the ERH by subtracting the infiltration (*e.g.*, ϕ-index) from the GRH:

Time (h)	Effective Precipitation (ERH) (cm/h)
0 - 1	0.0
1 - 2	2.0
2 - 3	2.0
3 - 4	0.0

As observed in the table, the duration of the effective rainfall hyetograph is 2 hours. Thus, D = 2 hours, and the Unit Hydrograph obtained above is a 2-hour Unit Hydrograph. Therefore, it can be used to predict runoff from precipitation events whose effective rainfall hyetographs can be represented as a sequence of uniform intensity (rectangular) pulses each of duration D. This is accomplished by using the principles of superposition and proportionality, encoded in the discrete convolution equation:

$$Q_n = \sum_{m=1}^{n} P_m U_{n-m+1}$$

where Q_n is the nth ordinate of the DRH, P_m is the volume of the mth rainfall pulse expressed in units of equivalent depth (*e.g.*, cm or in), and U_{n-m+1} is the (n-m+1)th ordinate of the UH, expressed in units of $m^3/s/cm$.

B. Using the UH obtained in A., predict the total streamflow that would be observed as a result of the following ERH:

Time (h)	Effective Precipitation (ERH) (cm/h)
0 - 2	0.5
2 - 4	1.5
4 - 6	2.0
6 - 8	1.0

As observed in the table, the ERH can be decomposed into a sequence of rectangular pulses, each of 2 hours duration. Thus, we can use the 2-hour UH obtained in **A**.

Determine the volume of each ERH pulse, P_m, expressed in units of equivalent depth:

Time (h)	P_m *(cm)*
0 - 2	1.0
2 - 4	3.0
4 - 6	4.0
6 - 8	2.0

Use superposition and proportionality principles:

	1	2	3	4	5	6	7
Time(h)	UH ($m^3/s/cm$)	P_1*UH (m^3/s)	P_2*UH (m^3/s)	P_3*UH (m^3/s)	P_4*UH (m^3/s)	DRH (m^3/s)	Total (m^3/s)
1	0	0				0	100
2	50	50				50	150
3	150	150	0			150	250

4	225	225	150			375	475
5	175	175	450	0		625	725
6	125	125	675	200		1000	1100
7	75	75	525	600	0	1200	1300
8	50	50	375	900	100	1425	1525
9	25	25	225	700	300	1250	1350
10	0	0	150	500	450	1100	1200
11			75	300	350	725	825
12			0	200	250	450	550
13				100	150	250	350
14				0	100	100	200
15					50	50	150
16					0	0	100

Columns 2 - 5: Apply the proportionality principle to scale the UH by the actual volume of the corresponding rectangular pulse, P_m. Observe that the resulting hydrographs are lagged so that their origins coincide with the time of occurrence of the corresponding rainfall pulse.

Column 6: Apply the superposition principle to obtain the DRH by summing up Columns 2 - 5.

Column 7: Add back the baseflow in order to obtain the total streamflow hydrograph.

13 APPENDIX 3: HYDROLOGICAL PARAMETERS of NILE flows

Table A3.1: 10-day average discharges of gauging sites on the Blue Nile (1965-2000) - (Mm3/day).

Period	Eddeim			Roseires Dam		
	$\bar{x}$	s	CV	$\bar{x}$	s	CV
JI	26.42	7.58	0.29	34.72	9.53	0.27
JII	22.11	5.96	0.27	31.72	7.19	0.23
JIII	18.96	5.46	0.29	30.38	6.42	0.21
FI	16.03	4.32	0.27	29.17	6.71	0.23
FII	14.15	4.13	0.29	29.80	6.20	0.21
FIII	12.32	3.63	0.29	29.93	6.52	0.22
MI	10.84	2.94	0.27	27.18	6.01	0.22
MII	10.33	3.30	0.32	26.13	6.61	0.25
MIII	10.21	4.72	0.46	24.44	7.16	0.29
AI	10.19	3.74	0.37	22.71	5.67	0.25
AII	9.61	4.23	0.44	23.12	6.65	0.29
AIII	10.73	6.12	0.57	22.92	7.12	0.31
MI	11.31	5.59	0.49	24.30	8.77	0.36
MII	17.04	9.27	0.54	25.72	10.96	0.43
MIII	25.21	15.49	0.61	28.73	11.83	0.41
JI	38.07	23.37	0.61	42.54	27.19	0.64
JII	56.00	25.60	0.46	57.81	24.59	0.43
JIII	74.77	21.45	0.29	79.57	31.61	0.40
JI	122.49	35.79	0.29	123.35	32.80	0.27
JII	201.01	62.00	0.31	199.72	57.93	0.29
JIII	328.30	98.21	0.30	311.32	83.23	0.27
AI	423.98	101.72	0.24	413.85	100.19	0.24
AII	510.63	120.08	0.24	494.91	110.90	0.22
AIII	481.94	97.25	0.20	476.05	103.28	0.22
SI	425.21	102.49	0.24	396.56	147.47	0.37
SII	363.76	82.69	0.23	320.05	105.94	0.33
SIII	304.65	77.98	0.26	246.17	89.60	0.36
OI	250.82	80.89	0.32	202.92	77.46	0.38
OII	205.00	94.61	0.46	171.89	90.57	0.53
OIII	144.91	68.70	0.47	135.03	70.23	0.52
NI	101.85	37.22	0.37	96.77	36.62	0.38
NII	78.50	26.08	0.33	75.79	25.76	0.34
NIII	62.97	20.31	0.32	61.53	20.46	0.33
DI	48.59	14.45	0.30	51.00	15.62	0.31
DII	39.19	11.38	0.29	44.44	13.06	0.29
DIII	31.66	8.95	0.28	38.07	11.10	0.29

Table A3.2: 10-day average discharges of gauging sites on the Blue Nile (1965-2000) - (Mm3/day).

Period	Sennar Dam			Khartoum		
	$\bar{x}$	s	CV	$\bar{x}$	s	CV
JI	11.81	7.29	0.62	30.03	14.35	0.48
JII	11.16	6.45	0.58	27.70	13.33	0.48
JIII	11.28	6.19	0.55	24.98	13.19	0.53
FI	11.22	5.96	0.53	21.77	12.08	0.55
FII	11.15	6.36	0.57	18.76	11.38	0.61
FIII	11.56	6.68	0.58	16.56	9.70	0.59
MI	12.05	6.77	0.56	15.08	7.73	0.51
MII	13.04	7.27	0.56	15.37	6.94	0.45
MIII	14.65	6.71	0.46	23.72	10.31	0.43
AI	18.03	6.69	0.37	32.35	12.60	0.39
AII	18.30	6.65	0.36	35.01	12.58	0.36
AIII	19.74	7.08	0.36	35.15	10.59	0.30
MI	21.36	8.58	0.40	33.01	9.39	0.28
MII	22.63	10.34	0.46	29.32	9.56	0.33
MIII	25.19	11.61	0.46	25.27	9.44	0.37
JI	33.55	22.52	0.67	25.05	11.80	0.47
JII	42.20	24.26	0.57	30.25	17.77	0.59
JIII	57.77	37.12	0.64	40.47	28.45	0.70
JI	88.10	36.14	0.41	54.92	26.41	0.48
JII	158.22	59.44	0.38	98.28	43.64	0.44
JIII	274.94	87.93	0.32	180.06	76.93	0.43
AI	385.90	102.42	0.27	315.44	102.76	0.33
AII	470.22	122.66	0.26	454.49	132.41	0.29
AIII	482.49	121.50	0.25	546.09	129.75	0.24
SI	389.73	166.31	0.43	488.90	172.86	0.35
SII	287.09	132.44	0.46	374.11	196.72	0.53
SIII	208.20	106.43	0.51	273.78	163.17	0.60
OI	159.29	62.38	0.39	191.97	78.57	0.41
OII	138.24	94.26	0.68	178.46	104.73	0.59
OIII	105.84	82.34	0.78	131.24	94.15	0.72
NI	67.28	50.80	0.76	90.25	53.76	0.60
NII	40.80	29.54	0.72	60.63	26.26	0.43
NIII	29.00	23.69	0.82	49.36	24.04	0.49
DI	20.94	16.43	0.78	42.08	18.85	0.45
DII	16.26	11.60	0.71	35.41	14.92	0.42
DIII	12.67	7.81	0.62	31.26	12.29	0.39

Table A3.3: 10-day average discharges of gauging sites on the Blue Nile (1965-2000) - (Mm3/day).

Period	Gwasi			El Hawata		
	$\bar{x}$	s	CV	$\bar{x}$	s	CV
JI	0.00	-	-	0.00	-	-
JII	0.00	-	-	0.00	-	-
JIII	0.00	-	-	0.00	-	-
FI	0.00	-	-	0.00	-	-
FII	0.00	-	-	0.00	-	-
FIII	0.00	-	-	0.00	-	-
MI	0.00	-	-	0.00	-	-
MII	0.00	-	-	0.00	-	-
MIII	0.00	-	-	0.00	-	-
AI	0.00	-	-	0.00	-	-
AII	0.00	-	-	0.00	-	-
AIII	0.00	-	-	0.00	-	-
MI	0.00	-	-	0.00	-	-
MII	0.00	-	-	0.00	-	-
MIII	0.00	-	-	0.00	-	-
JI	0.00	-	-	0.06	-	-
JII	2.27	1.44	0.63	1.73	0.75	0.43
JIII	2.89	1.66	0.58	2.19	0.19	0.09
JI	6.96	4.85	0.70	3.17	1.73	0.55
JII	8.03	5.98	0.74	5.20	2.19	0.42
JIII	12.49	6.85	0.55	7.73	3.23	0.42
AI	18.38	8.49	0.46	9.77	3.27	0.34
AII	24.32	11.29	0.46	11.00	3.09	0.28
AIII	31.48	12.70	0.40	12.23	2.74	0.22
SI	30.28	14.75	0.49	12.43	3.26	0.26
SII	28.25	16.05	0.57	12.39	3.18	0.26
SIII	22.36	13.95	0.62	11.09	3.34	0.30
OI	17.61	13.15	0.75	8.61	3.67	0.43
OII	12.19	9.89	0.81	5.84	3.77	0.65
OIII	6.07	5.94	0.98	3.54	3.65	1.03
NI	3.69	3.20	0.87	1.93	1.60	0.83
NII	1.72	1.95	1.13	1.03	0.75	0.73
NIII	1.12	1.64	1.46	0.85	0.53	0.62
DI	0.92	5.75	1.36	0.81	0.57	0.70
DII	0.00	-	-	0.77	0.49	0.63
DIII	0.00	-	-	1.08	-	-

Table A3.4: 10-day average discharges of gauging sites on the White Nile, Atbara River and Main Nile (Mm3/day)

Period	Malakal	Atbara @ K.3	Tamaniat	Dongola
JI	97.88		102.02	98.62
JII	91.35	0.63	98.20	94.76
JIII	84.37		92.71	90.80
FI	77.81		86.57	85.70
FII	72.17	0.22	80.35	79.79
FIII	68.03		75.33	74.35
MI	64.73		72.84	69.87
MII	62.05	0.03	73.49	66.59
MIII	60.01		90.92	66.70
AI	58.76		105.98	81.40
AII	57.35	0.10	109.83	95.43
AIII	56.33		109.12	99.09
MI	57.06		105.80	99.26
MII	59.01	0.31	100.12	96.46
MIII	62.81		92.71	90.29
JI	66.96		91.49	82.64
JII	72.06	3.12	97.65	80.73
JIII	77.16		108.62	89.06
JI	81.73		128.95	105.60
JII	86.14	54.10	174.20	158.40
JIII	89.99		243.37	252.24
AI	94.05		368.65	395.04
AII	97.14	180.40	476.61	551.01
AIII	100.66		552.37	671.21
SI	103.64		514.83	668.33
SII	106.40	115.47	422.68	567.33
SIII	108.80		342.73	445.36
OI	110.86		276.43	344.23
OII	112.63	27.17	265.42	287.53
OIII	113.91		220.57	257.90
NI	114.36		176.24	201.47
NII	114.17	5.37	143.39	164.20
NIII	113.29		129.49	136.02
DI	111.59		117.67	121.46
DII	108.54	1.75	108.18	111.12
DIII	103.67		102.75	102.67

Table A3.5: Min. and max. stages of gauging sites on the Blue and White Niles, Atbara River and Main Nile

Gauging Station	Min. record	Max. record
The Blue Nile River:		
Eddeim	487.210	494.900
U/S Roseires	440.60	481.51
D/S Roseires	439.40	450.80
U/S Sennar	405.00	421.97
D/S Sennar	402.47	412.73
Wad Medani	389.290	400.710
Khartoum	372.68	380.090
Gwasi	421.410	429.920
El Hawata	421.750	428.39
The White Nile River:		
Malakal	367.350	371.580
U/S J. Aulia	371.34	377.79
D/S J. Aulia	370.53	376.73
The Atbara River:		
W/ el Hilew	474.283	487.053
Kubur	482.448	493.238
U/S Kh. El Girba	437.50	474.48
D/S Kh. El Girba	405.90	452.78
The Main Nile River:		
Tamaniat	368.350	375.920
Shendi	352.380	360.410
Hassanab	342.930	349.840
Atbara	332.920	340.160
Dongola	220.830	227.940

14 Appendix 4: Requirements for embankment design

Table (4.1): Major and Minimum Requirements for embankment design (Source: US Army Corps of Engineers, 2000)

Step	Procedure
1	Conduct geological study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3	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.
4	Using the information obtained in Step 3: a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas. b. Compute rough quantities of suitable material and refine borrow area locations.
5	Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
6	Analyze each trial section as needed for: a. Under-seepage and through seepage. b. Slope stability. c. Settlement. d. Trafficability of the levee surface.
7	Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use.
8	Based on the results of Step 7, establish final sections for each reach.
9	Compute final quantities needed; determine final borrow area locations.
10	Design embankment slope protection.

15 Appendix 5: Topographic surveys

INTRODUCTION
Topography refers to the characteristics of the land surface. These characteristics include natural features**,** and artificial (or man-made**)** features. Natural features are hills, valleys, plains, peaks, depressions, and other natural features, such as trees, streams, and lakes. Man-made features are highways, bridges, dams, buildings, etc. A graphic representation of the topography of an area is called a topographic map. A topographic map is simply a drawing that shows the natural and artificial features of an area. A topographic survey is a survey conducted to obtain the data needed for the preparation of a topographic map. This data consists of the horizontal and vertical locations of the features to be shown on the map.

TOPOGRAPHIC SURVEYING
The fieldwork in a topographic survey consists principally of:

1. The establishment of a basic framework of horizontally and vertically located control points (called instrument points or stations); and
2. The determination of the horizontal and vertical locations of details approximately each instrument point.

TOPOGRAPHIC CONTROL
Topographic control consists of two parts:

1. Horizontal control, which locates the horizontally fixed position of specified control points; and
2. Vertical control, in which the elevations of specified benchmarks are established. This control provides the framework from which topographic details, such as roads, buildings, rivers, and the elevation of ground points, are located.

HORIZONTAL CONTROL
Locating primary and secondary horizontal control points or stations may be accomplished by traversing, by triangulation, or by the combined use of both methods. On an important, large-area survey, there may be both primary control, in which a number of widely separated primary control points are located with a high degree of precision; and secondary control, in which stations are located with less precision within the framework of the primary control points.

VERTICAL CONTROL
In topographic surveying, benchmarks serve as starting and closing points for the leveling operations when you are locating details. Although for some surveys the datum may be assumed, it is preferable that all elevations be tied to benchmarks, which are referred to the sea-level datum. In many areas, series of permanent and precisely established benchmarks are available. As a surveyor, you must make every feasible effort to tie in your surveys to these benchmarks to ensure proper location and identification. Often, the established horizontal control marks are used as the benchmarks because the level routes generally follow the traverse lines.

TECHNIQUES OF TOPOGRAPHY

There is a variety of approaches to studying topography. Which method(s) to use depend on the scale and size of the area under study, its accessibility, and the quality of existing surveys.

1. DIRECT SURVEY
   Surveying helps determine accurately the terrestrial or three-dimensional space position of points and the distances and angles between them using leveling instruments such as theodolites, dumpy levels and clinometers.

2. REMOTE SESNSING
   Remote sensing is a general term for geodata collection at a distance from the subject area. Even though remote sensing has greatly speeded up the process of collecting information, and has allowed greater accuracy control over long distances, the direct survey still provides the basic control points and framework for all topographic work, whether manual or GIS-based.