Sudan University of Science & Technology

College of Engineering

School of Civil Engineering

DEPT. OF HYDRAULIC ENG.

A DESSERATION SUBMITTED IN PARTIAL FULFILLMENT OF OBTAINING A BACHELOR OF SCIENCE DEGREE (HONORS) IN CIVIL ENGINEERING (HYDRAULICS)

Flash Floods Control Measures & Mechanisms,
Case Study: East Nile Locality, Khartoum

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بسم الله الرحمن الرحيم

﴿۱٦﴾ أَنزَلَ مِنَ السَّمَاءِ مَاءً فَسَالَتْ أَوْدِيَةٌ بِقَدَرِهَا فَاحْتَمَلَ السَّيْلُ زَبَدًا رَّابِيًا وَمِمَّا يُوقِدُونَ عَلَيْهِ فِي النَّارِ ابْتِغَاءَ حِلْيَةٍ أَوْ مَتَاعٍ زَبَدٌ مِّثْلُهُ كَذَٰلِكَ يَضْرِبُ اللَّهُ الْحَقَّ وَالْبَاطِلَ فَأَمَّا الزَّبَدُ فَيَذْهَبُ جُفَاءً وَأَمَّا مَا يَنفَعُ النَّاسَ فَيَمْكُثُ فِي الْأَرْضِ كَذَٰلِكَ يَضْرِبُ اللَّهُ الْأَمْثَالَ ﴿۱۷﴾

صدق الله العظيم

سورة الرعد : الآية (17)

I

Dedication

We humbly dedicate this effort to:

Our parents …

For enriching our lives with wisdom, knowledge; care for others and passion to make change.

Our Teachers…

To All Who seek knowledge in this life…

We dedicate this work in hope of success in this life and hereafter.

ACKNOWLEDGEMENTS

ABSTRACT

This study presents flash floods control measures and mechanisms in East Nile Locality, Khartoum.

The study area was described thoroughly, listing all aspects regarding hydrologic properties. Data records obtained from various rainfall records, climate and stream flow measurements were used in the study.

In addition, Hydraulic structures and underground filters used in maintaining floods were illustrated listing the different types and their method of design.

Finally, the recommended measures and mechanisms for controlling flash floods have been applied in the area of study. The hydraulic structures were designed and their proposed locations were shown.

التجـــــريــد

تناولت هذه الدراسة موضوع الفيضانات المفاجئة و طرق و آليات التحكم فيها في منطقة محلية شرق النيل، الخرطوم.

إشتملت الدراسة على شرح مفصل لمنطقة الدراسة، و ذلك بتفصيل الخصائص الهيدرولوجية المتعلقة بالمنطقة. تم الحصول على بيانات الدراسة من معطيات بيانات المناخ و معدلات هطول الأمطار بالإضافة لمعدلات الإنسياب لمجاري السيول و الخيران.

بالإضافة لما سبق، تم التعرض لأنواع المنشآت الهيدروليكية و الفلاتر تحت الأرضية ، و التي تساعد في التحكم في الفيضانات، أضف إلى ذلك طرق تصميمها.

أخيراً، تم تطبيق الآليات و الطرق التي ينصح بإستخدامها في حالات الفيضانات المفاجئة على منطقة الدراسة. أيضاً، صُممت المنشآت الهيدروليكية اللازمة مع إقتراح و توضيح أفضل مناطق لإنشائها.

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CHAPTER ONE

INTRODUCTION

1. Introduction

1.1 General:

In the last years the urgent demand for flash floods control caused by storm water events of different return periods has increased markedly. Flash floods occur when urban runoff fill the major creeks and valleys after a storm event. The practically instantaneous occurrence of flash floods together with their capacity of transport renders flash floods which is one of the most significant weather-related hazards in many parts of the world, causing considerable economic and human losses every year.

Discharge of flash floods is to transport storm water from roads and residential areas in urban areas, to avoid environmental and health disasters by maintaining of storm water in specific area.

The basic issues of urban flash floods start to appear approximately about 30 to 50 years ago, when the construction in the rural areas and areas bounded by the Nile begun in separated areas in Khartoum , with the aim of urbanization, land reclamation, and the establishment of new residential areas, to accommodate the increasing of population , the construction of these residential areas has happened randomly, with concept that contrary to the natural of the area e.g. (East Nile locality).

The competent authorities didn't have any vision about the stream's paths, seasons, volume and return periods this led to environmental disasters, the effects of the disasters are represented below:

1- Construction of buildings in the direction of streams and valleys.
2- Land reclamation and settlement for agriculture led to diversion of the flood which brought floods to flow in residential neighborhoods (storm water catchment areas).

A flood can be determined as a mass of water that produces runoff on land that is not normally covered by water. Urban flooding is a phenomena that occurs where there are been man-made developments within the existing floodplains or drainage areas. The risk of floods does not need a disclaimer, like natural disaster when it occurs for the following reasons:

1- The sudden occurrence of flood
2- Flow velocity of storm water.

This leads to serious and major materialism damages of sabotage and destruction in specific areas , there is no doubt that the urgent need for every drop of water in the desert areas and semi-desert where it doesn't often rain , this require the use of storm water as one of available water resources .

There are many factors that affect the development of flash floods:

- Bridging the flood plain due to weather factors.
- lack of data in the study area.

1.2 Objectives of the Study:

The study objectives are to design a committable flood control system in East Nile locality to achieve the following purposes:

1- Study of the size and volume of rainfall in East Nile region and Khartoum in general.

2- Flood control technology settlement to avoid the effects and risk of floods.

3- Delay the time of runoff floods into the outlet zone by building dams.

4- Reduce the problems due to floods.

5- Inventory of all the valleys that flow through the region.

1.3 Study Methodology:

Phase one:

Determination of project proposal and study area, and collecting necessary information about the topic and gathering related literature.

Phase Two:

Visiting East Nile Locality office, Department of Urban Planning, to obtain documents and data related to the topic.

Studying theories related to project proposal, and collecting related references and papers.

Phase Three:

Analysis and process of the data, hence, building up conclusions and recommendations based upon results.

Phase Four:

Preparation of final project Thesis.

CHAPTER TWO

LITERATURE REVIEW

2. Literature review

2.1 General:

The impact of rapid urbanization has resulted in serious flood problems in Khartoum. In this respect, the East Nile locality has been no exception. Increased runoff rates, associated with urbanization, have contributed to dramatic increases in flood damages. To alleviate these problems the authorities must make serious efforts to control floods by carefully addressing and resolving the storm water problems that exist in urban areas, several flash floods control projects have been done to control flash floods but the mechanism of action didn't provide the expected result, in addition to the high cost of the projects. (*Just peace Forum*)

2.2 Flash Floods Control Systems:

The system contains different main parts:

1- Drainage system which contains Pipes of different diameters to collect storm water depends upon gravity in cities.

2- Tanks to collect storm water.

3- Drainage channels to transport water.

4- Culverts.

5- flood control dams in rural areas

The construction of the system depends on the topography of the area (exist of valleys & creeks) to construct flood control dams, drainage channels and roadway culverts. The drainage system must be divided into areas for rainfall harvesting, and also determining the details of the control

system which include (channels, pipes, inspection chambers ...etc.), regular repair according to main plan certified by the City department, the capacity of the drainage system estimated by rainfall records.

2.3 Hydrology:

2.3.1 Introduction:

Hydrology is the science, which deals with the occurrence, distribution and disposal of water; it is the science which deals with the various phases of the hydrologic cycle. The most important phases of the hydrologic cycle namely evaporation, precipitation and runoff, it is the 'runoff phase', which is important to a civil engineer since he concerned with the storage of surface runoff in tanks and reservoirs for the purposes of irrigation, municipal water supply hydroelectric power …etc.

The study of hydrology helps us to know:
1- The maximum probable flood that may occur at a given site and its frequency; this is required for the safe design of drains and culverts, dams and reservoirs, channels and other flood control structures.
2- The water yield from a basin its occurrence, quantity and frequency, etc.; this is necessary for the design of dams, municipal water supply, water power, river navigation, etc.
3- The ground water development for which a knowledge of the hydrogeology of the area.
4- The maximum intensity of storm and its frequency for the design of a drainage project in the area.

2.3.2 Hydrological Data:

For the analysis and design of any hydrologic project adequate data and length of records are necessary. A hydrologist is often posed with lack of adequate data. The basic hydrological data required are:

1- Climatological data
2- Precipitation records
3- Stream flow records
4- Hydro-meteorological characteristics of basin (long term precipitation, space average over the basin)
5- Geomorphologic studies of the basin, like area, shape and slope of the basin, mean and median elevation, mean temperature (as well as highest and lowest temperature recorded) and other physiographic characteristics of the basin; stream density and drainage density; tanks and reservoirs.

2.3.3 Hydrological Data Analysis:

Hydrological data analysis can be represented in form of statistical and probability distribution. The science of hydrology involves in major cases in monitoring, analysis and design.

2.4 Probability of Hydrological Events:

2.4.1 Normal probability distribution:

The density function of normal probability distribution is given by:

$$f(x) = \frac{1}{\sigma_p \sqrt{2\pi}} \exp\left(-\frac{(x-\mu)^2}{2\sigma_p^2}\right), \quad -\infty < x < \infty \qquad (2.1)$$

Where: σ_p and μ are the two parameters, which affect the distribution.

2.4.3 Lognormal probability distribution:

The density function of lognormal distribution is given by

$$P(x)= exp \ (\ exp(-(x-c)/a)) \qquad (2.2)$$

2.4.3 Return Period:

The value of X_t of a return time period is

$$R_t = X_t + Y_t \qquad (2.3)$$

Where :

Y_t= standard deviation, X_t= mean

A return period of five years is recommended in which the probability
of the event will occur. (*Ministry of Water resources & Electricity*)

2.5 Hydraulic Structures

2.5.1 Embankment Dams:

2.5.1.1 Introduction:

The primary purpose of a dam may be defined as to provide for the safe retention and storage of water. As a corollary to this every dam must represent a design solution specific to its site circumstances. The design therefore also represents an optimum balance of local technical and economic considerations at the time of construction. Reservoirs are readily classified in accordance with their primary purpose, e.g. irrigation, water supply, hydroelectric power generation river regulation, flood control, etc. Dams are of numerous types, and type classification is sometimes less clearly defined. An initial broad classification into two generic groups can be made in terms of the principal construction material employed.

1. Embankment dams are constructed of earth fill and/or rock fill. Upstream and downstream face slopes are similar and of moderate angle, giving a wide section and a high construction volume relative to height.

2. Concrete dams are constructed of mass concrete. Face slopes are dissimilar, generally steep downstream and near vertical upstream, and dams have relatively slender profiles dependent upon the type.

Embankment dams can be of many types, depending upon how they utilize the available materials.

1. *Earthfill embankments.* An embankment may be categorized as an earthfill dam if compacted soils account for over 50% of the placed volume of material. An earthfill dam is constructed primarily of selected engineering soils compacted uniformly and intensively in relatively thin layers and at a controlled moisture content.

2. *Rockfill embankments.* In the rockfill embankment the section includes a discrete impervious element of compacted earthfill or a slender concrete or bituminous membrane. The designation 'rockfill embankment' is appropriate where over 50% of the fill material may be classified as rockfill.

The following are the important factors which affect the choice of the type of dam:

 (i)   Topography,
 (ii)   Geology and foundation conditions,
 (iii) Material available, and
 (iv)   Size and location of spillway.

In this project we will discuss the structures of flood control, small dams are constructed for many purpose especially in irrigation and flood control .

2.5.1.2 Selection of Small Dams Types:

Dams may be classified into a number of different categories, depending upon the purpose of the classification.

Dams are classified according to their use, their hydraulic design, or the materials of which they are constructed.

1- Classification According to use:

Dams may be classified according to the broad function they serve, such as storage, diversion, or detention. Refinements of these classifications can also be made by considering the specific functions involved. Detention dams are constructed to retard flood runoff and minimize the effect of sudden floods. Detention dams consist of two main types. In one type, the water is temporarily stored and released through an outlet structure at a rate that does not exceed the carrying capacity of the channel downstream. In the other type, the water is held as long as possible and allowed to seep into pervious banks or into the foundation. The latter type is sometimes called a water-spreading dam or dike because its main purpose is to recharge the underground water supply.

2- Classification by Hydraulic design:

  a) Over flow dam

  b) Non over flow dam

3- Classification by materials:

 a) Earth fill dams

 b) Rock fill dams

Earth fill dams are further divided into the following types:

 a) Homogeneous earth dam

 b) Zoned earth dam.

Homogeneous earth dams are constructed entirely or almost entirely of one type of earth material. A zoned earth dam, however, contains materials of different kinds in different parts of the embankment. A homogeneous earth dam is usually built when only one type of material is economically available and/or the height of the dam is not very large. A homogeneous earth dam of height exceeding about 6 to 8 m should always have some type of drain constructed of material more pervious than the embankment soil. Such drains reduce pore pressures in the downstream portion of the dam and thus increase the stability of the downstream slope. Besides, the drains control the outgoing seepage water in such a manner that it does not carry away embankment soil.

Zoned earth dams, The most common type of a rolled earthfill dam section is that in which a central impervious core is flanked by zones of materials considerably more pervious, called shells. These pervious zones or shells enclose, support, and protect the impervious core; the upstream pervious zone affords stability against rapid drawdown; and the downstream pervious zone acts as a drain to control seepage and lower the phreatic surface.

Design criteria and typical features for small dams are generally different from those for high dams, because the construction methods focus upon economy. So the risk may increase and corresponding accidents may cause significant victims.

Small Dams are defined as having the following characteristics:

- 2.5 m < H < 15 m

- H is height in meters above river bed level to maximum crest level

- V is storage volume in million m^3 at maximum operating level = full supply level.
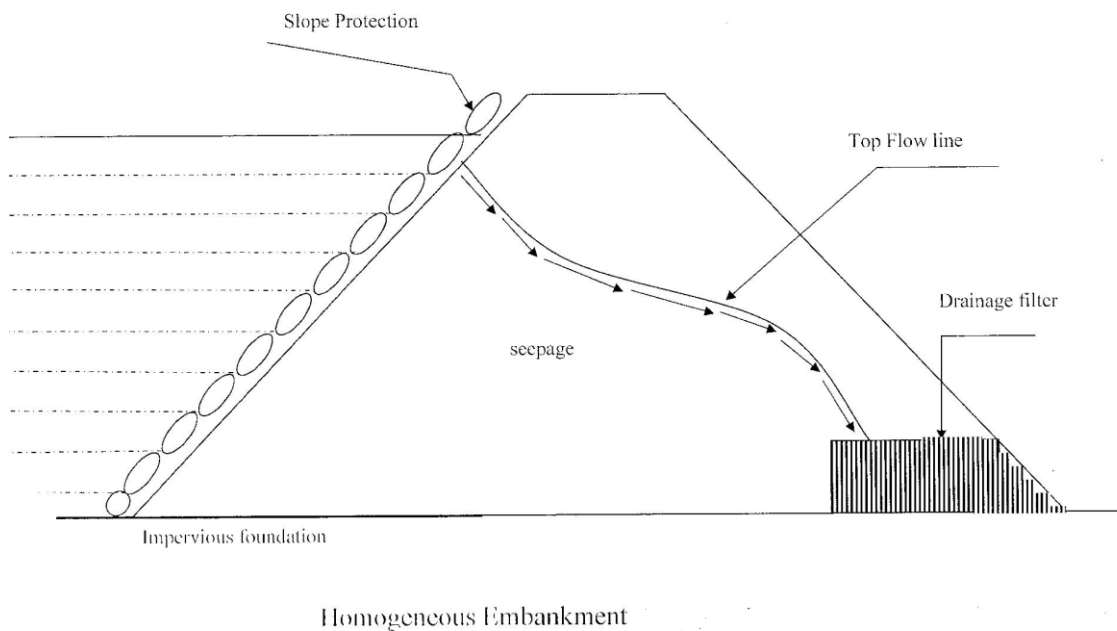
Figure (2.1): Homogeneous Embankment Dam

2.5.2 Culverts:

2.5.2.1 Introduction:

Culvert is the second hydraulic structure used in this project it is a conduit which conveys stream flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

2.5.2.2 Shapes:

Numerous cross-sectional shapes are available for both closed conduit and open-bottom culverts. The most common closed conduit shapes are circular, box (rectangular), elliptical, and pipe-arch. These typical manufactured culvert shapes have the same material on the entire perimeter. Shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance.

2.5.2.3 Materials:

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability (corrosion and abrasion resistance), and constructability. The most commonly used culvert materials are concrete (both reinforced and non-reinforced), corrugated metal (aluminum or steel) and plastic (high – density polyethylene (HDPE) or polyvinyl chloride (PVC)) Less commonly used materials

include clay, stone and wood, as might be found in historic culvert structures.   Most common culvert materials used in Sudan are concrete and steel.

2.5.3 Open Drainage Channels:

2.5.3.1 Introduction:

The most basic way to drain off rain and storm water is via open channels. A more developed but also more expensive solution is a separate sewer system .Open channels drainage system is ideal for projects where open channel drainage is required.

Storm water drainage systems with open channels for the discharge of rainfall exist in most urbanized areas. The channels usually drain off rain water into rivers or sometimes into agriculture canals.

Unauthorized discharge of domestic wastewater leads to surface water pollution. Solid waste is also commonly disposed of in these open channels. This is a particularly a problem in many middle – to low income countries e.g. Sudan.

2.5.3.2 Basic design principles:

An open channels or drain system generally consists of a secondary drainage system, with a network of small drains attached. Each serves a small catchment area that ranges from a single property to several blocks of houses. These small drains bring the water to a primary drainage system, composed of main drains (also called interceptor drains), which serve large areas of the city. These drains are generally connected with natural drainage channels such as streams or rivers. (WHO, 1991)

2.5.3.3 Cost considerations:

Compared to underground sewer system open channels are a less expensive solution. The precise cost depends on local conditions. Consideration of community participation could have a positive offset on the overall cost.

2.5.3.4 Operation and maintenance

The main duties and responsibilities for operations and maintenance of an open channel drainage system are:

- Routine drain cleaning

- Reporting of defects and blockages

- Semiannual inspection Repairs.

- Payment for maintenance

- Passing of by-laws regarding the use of drains

- Enforcement of by-laws

However, the most important issues to optimize existing open drain systems are: preventing overflow, Table (2.1) below describe the advantages of open channels drainage system.

Table (2.1): Advantages of Open Drainage System

ADVANTAGES	COMMENT
Working Principle	The open drains collect storm and sometimes *sewage*, then drain it off into *rivers, lakes* or to underground storm drainge system and sometimes irrigation canals.
Capacity/Adequacy	To be found in smaller and larger urban as well as rural areas, in coastal areas often influenced by tide level of sea. Furthermore they are prone to flooding or clogging in rainy season.
Performance	Good performance if designed correctly and kept free from *solid waste*.
Costs	Still cheaper than advanced *sewer* systems, but terrain, labour and material could raise the investment costs.
Self-help Compatibility	High
Reliability	Open drains are prone to blockage caused by garbage and solids.
Main strength	Simple to construct with locally available materials
Main weakness	It is prone to be used as a disposal for all kinds of waste and can cause health risks for residents.

2.5.4 Storm water Filters:

2.5.4.1 Introduction:

Storm water filters are designed to either infiltrate or to treat and convey runoff to a disposal point.

Filters may be visible from the surface, or completely subsurface. They may be designed as a single large chamber (often with a smaller chamber for pretreatment. Sand and other media filters remove constituents from storm water runoff primarily through a physical process of filtering out particulates from the water.

2.5.4.2 Materials

Stone Storage used to provide additional storage shall be uniformly-graded, crushed, washed stone meeting the specifications of AASHTO No. 3 or AASHTO No. 5.

Stone shall be separated from filter medium by a non-woven filter fabric or a gravel filter.

2.6 Description of the Study Area:

East Nile locality is in the eastern part of Khartoum state it extends from the eastern bank of the Blue Nile , and bordered to the west of the blue Nile , there are several creeks and valleys in the region that ravaged by floods in the rainfall seasons. (see figure 2.1)

2.6.1 Valleys and Creeks in East Nile Locality:

East Nile region exposed to Natural phenomena that are closely linked to stormwater and floods , and to know the paths and formation of these phenomena study must be made to the area , the study involve knowledge of valleys paths and outlets see figure (5.3).

Main valleys in east Nile Locality:

1- Keriyab Creek which flows in omdom

2- Marabee Creek which flows in al-marabee

3- Soba valley which flows in al-droshab

4- Alfadal valley

5- Altmada valley

Storm water gathers in main valleys about 100 km from the outlet point, the most known valleys are wadi-alhessen which flows in Aljazeera state, soba valley which flows in keriyab and al-marabee areas. Storm water estimated in three days is about 185 mm in the year 2013, which exceeded the storm water record 135mm in 1988.

The amount of rainfall that swept the valleys are 12 millon m^3 per 100 km of the Nile. (*Department of urban planning east nile locality*)

2.7 Flooding issues statement:

2.7.1Primary effects:

The effects of flood include the following:

1- loss of life

2- Damage to buildings in residential areas and other structures

3- Loss of drinking water supply

4- Damage to roads and infrastructure that affect mobility in the region.

2.7.2 Flooding issues in Khartoum:

Khartoum is exposed to three natural phenomena (river flood, flash floods, storm water). However it is not difficult to maintain the expected phenomena, if the authorities take precautions crisis.

2.8 Previous studies

In previous studies in Europe flash floods have been associated to orography; flash floods are often associated to complex orography.

Relief is important since it may affect flash flood occurrence in specific catchments by combination of two main mechanisms:

- Orographic effects augmenting precipitation and anchoring convection, and topographic relief promoting rapid concentration of streamflow. However, major flash floods were also observed in areas either completely flat – such as the flood which impacted the metropolitan area of Venice in September or only marginally influenced by orography such as the major flash flood occurred in the Gard region in September 2002.

In previous studies about flood control in different countries, the floods have been classified as under.

(i) *Low flash floods:* If water level in the creeks or valleys during monsoon rises higher than usual in other seasons of the year and results in over flowing of bank once in every two years; submerges the adjoining fields but generally doesn't prevent flow of drainage of fields; also doesn't create drainage congestion in the nearby populated area, it is termed as low flood situation. In such situation, the water level always remains at least 1 m below plinth level of township as fixed by the Civil Authorities for civil construction of industrial Complexes and residential areas.

(ii) *high flash floods:* When the water level in the creeks or valleys rises to the extent that populated areas are encircled with flash floods waters and the flood waters overflow the or valleys bank, with flood frequency of 1 in 10 years.

Precipitation records and flow data have been widely used for flood studies however; the use of systematic data on flash floods presents several challenges in valleys catchment, as representative instrumental records are not normally available in these environments. (C*how, 1959*)

The occurrence of flash floods in complex terrain represents an important geomorphic agent. These floods are usually associated with widespread slope failures and flood power is sufficient to cause significant erosion and sedimentation in the floodplains. (*Marchi et al., 2009, 2010*).

CHAPTER THREE

THE STUDY METHODOLOGY

3. The Study Methodology

For the continuity of literary narrative and in order to achieve the desired objectives of this project, therefore we supply the methodology of flood control in East Nile locality in the following:

3.1 Strategy of the project:

1- Routing the main and sub- floods passing through the East Nile region see figure (4.2)

2- The study of various alternative of the valleys paths which penetrate the residential blocks (diversion of the valley and the establishment of drainage channels) see figure (4.3)

3- Construction of floods control dams.

4- Construction of culverts.

5- Design of a drainage channels for each valley.

6- Develop a structural plan to drain storm water underground.

There are many optional criteria to design a storm water drainage system, in this project the design of discharge system will follow design of small dams plus, design of cross-drainage facilities such as culverts, bridges, subsurface drainage design(open channels), and underground filter system.

3.2 Benefits from the results of project:

a) Determination of urban development axes away from risks.

b) Identify hazards that maybe exposed to some of the existing urban areas.

c) Classification according to potential risks in urban areas.

d) Determine expected disasters size that may occur due to floods.

3.3 Information Sources:

1. Rainfall data & meteorological data from rain monitoring stations of the general authority of meteorology in Khartoum.

2. Data of dams built on the valleys stream from the (ministry of water resources and electricity).

3. Images of the topography of the region from the department of urban planning

4. Topographic maps from the geological survey authority (contour maps)

3.4 Data analysis:

The most important aspect of the project is the analysis of hydrological data e.g. (the amount of rainfall in the region), therefore the analysis of rainfall data and determination of rainfall amount for different return period (5, 10, 20, 50,100 years)

3.5 Routing valleys:

A study of catchment areas of different valleys in the region is the most important case in flash floods control project , previous   study has been done to different valleys in Khartoum, where a detailed study has been done which   include   analysis and classification of soil , description of climate and storm water flow in the rain season.

CHAPTER FOUR

DATA ANALYSIS, HYDRAULIC & STRUCTURAL DESIGN

4. Data Analysis, Hydraulic and Structural Design

4.1 Climatological data in Khartoum:

The monthly average temperature in Khartoum is shown in Figure (4.1) Source (MOI). The maximum temperature in the last forty years was 35℃ and the minimum was23℃.
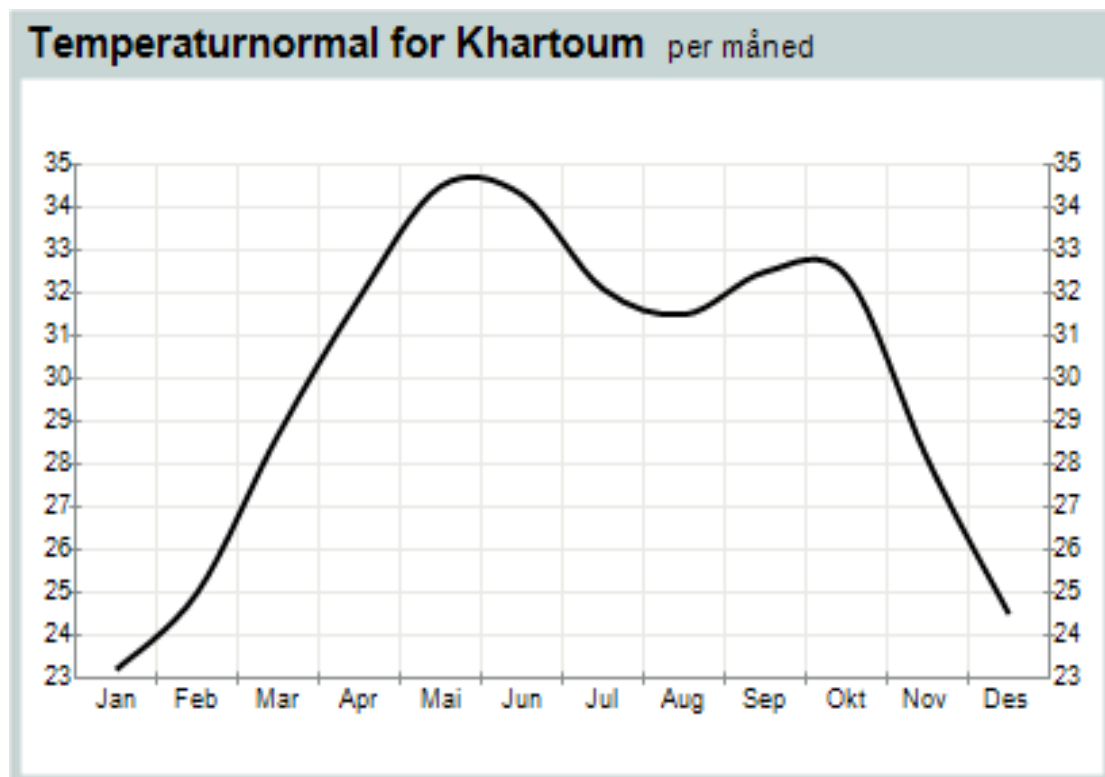

Temperaturnormal for Khartoum per måned

Figure (4.1): Shows the monthly average temperature in Khartoum

4.2 Precipitation Records in Khartoum:

Data of rainfall records in Khartoum is shown in Table (4.1), the table shows the monthly average of rainfall in (mm/hr) between the years (1970- 2010). The rainfall data records of the last century between the years (1913-2000) reached the average of 128 mm, and reached the minimum and maximum of 2mm and 443 mm respectively. However the daily rainfall reached the average of 38 mm and reached the minimum and maximum of 2mm and 147 mm respectively. (Ministry of water resources and Electricity)

Table (4.1) shows the monthly average of rainfall data

YEAR	1	2	3	4	5	6	7	8	9	10	11	12	TOTAL
KHARTOUM MONTHLY RAIN FALL (MM) FOR (1971-2010)													
1971	0.0	0.0	0.0	0.0	0.0	0.0	53.8	33.2	31.9	1.6	0.0	0.0	120.5
1972	0.0	0.0	0.0	0.0	0.0	6.5	6.0	110.5	10.0	0.6	0.0	0.0	133.6
1973	0.0	0.0	0.0	0.0	14.2	0.0	74.6	0.0	34.3	51.6	0.0	0.0	174.7
1974	0.0	0.0	0.0	0.0	0.0	13.9	55.4	12.0	4.7	0.0	0.0	0.0	86.0
1975	0.0	0.0	0.0	0.0	4.2	0.0	0.9	50.1	47.8	0.0	0.0	0.0	103.0
1976	0.0	0.0	0.0	0.0	0.0	0.0	43.0	80.1	25.7	7.8	22.2	0.0	178.8
1977	0.0	0.0	0.0	0.0	0.0	0.0	37.4	125.6	2.9	0.8	0.0	0.0	166.7
1978	0.0	0.0	0.0	0.0	0.0	4.0	60.3	46.3	14.5	8.5	0.0	0.0	133.6
1979	0.0	0.0	0.0	0.0	3.2	12.1	2.7	62.3	19.8	0.0	0.0	0.0	100.1
1980	0.0	0.0	0.0	0.0	0.7	4.2	60.0	19.2	11.0	1.2	0.0	0.0	96.3
1981	0.0	0.0	1.4	0.0	1.2	1.7	86.9	30.7	12.9	6.3	0.0	0.0	141.1
1982	0.0	0.0	1.6	0.0	5.0	0.0	9.4	46.9	39.8	0.0	0.0	0.0	102.7
1983	0.6	0.0	0.0	0.0	1.1	46.5	23.2	6.8	5.8	0.0	0.0	0.0	84.0
1984	0.0	0.0	0.0	0.0	1.4	0.0	0.0	0.0	3.3	0.0	0.0	0.0	4.7
1985	0.0	0.0	0.0	0.0	9.1	3.1	16.4	0.3	9.9	0.0	0.0	0.0	38.8
1986	0.0	0.0	0.0	0.0	0.0	0.0	9.1	21.9	16.3	10.4	0.0	0.0	57.7
1987	0.0	0.0	0.0	0.0	21.1	23.0	22.0	48.9	0.0	0.6	0.0	0.0	115.6
1988	0.0	0.0	0.0	0.0	0.0	0.0	65.9	301.4	46.3	1.9	0.0	0.0	415.5
1989	0.0	0.0	0.0	0.0	5.8	1.3	0.0	24.7	48.0	0.0	0.0	0.0	79.8
1990	0.0	0.0	0.0	0.0	0.0	0.0	2.0	0.0	2.4	0.0	0.0	0.0	4.4
1991	0.0	0.0	0.0	0.0	2.3	0.0	4.3	37.7	0.0	0.6	0.0	0.0	44.9
1992	0.0	0.0	0.6	0.0	10.8	4.0	6.9	89.0	38.6	0.0	0.0	0.0	149.9
1993	0.0	0.0	0.0	0.0	3.8	0.0	1.0	33.4	0.0	1.6	0.0	0.0	39.8
1994	0.0	0.0	0.0	0.0	12.7	0.0	51.3	48.0	101.2	19.4	0.0	0.0	232.6
1995	0.0	0.0	0.0	0.0	0.0	0.3	83.9	52.9	46.5	11.4	0.0	0.0	195.0
1996	0.0	0.0	0.0	0.0	19.5	0.4	32.0	55.0	91.9	0.5	0.0	0.0	199.3

1997	0.0	0.0	0.0	0.0	0.0	3.1	50.9	35.0	9.2	42.8	0.0	0.0	141.0
1998	0.0	0.0	0.0	0.0	0.0	0.0	0.0	21.3	85.4	4.0	0.0	0.0	110.7
1999	0.0	0.0	0.0	0.0	0.0	1.9	26.9	47.3	25.5	29.0	0.0	0.0	130.6
2000	0.0	0.0	0.0	0.0	0.0	0.0	1.4	8.5	15.3	34.8	0.0	0.0	60.0
2001	0.0	0.0	0.0	0.0	0.0	0.0	37.4	73.7	13.5	3.2	0.0	0.0	127.8
2002	0.0	0.0	0.0	0.0	0.0	0.0	7.7	82.6	13.7	3.5	0.0	0.0	107.5
2003	0.0	0.0	0.0	0.0	23.5	44.3	27.2	39.4	11.0	8.4	7.6	0.0	161.4
2004	0.0	0.0	0.0	0.0	0.0	0.3	6.0	88.8	9.6	5.0	0.0	0.0	109.7
2005	0.0	0.0	0.0	0.0	32.8	0.0	77.6	30.3	0.0	0.0	0.0	0.0	140.7
2006	0.0	0.0	0.0	0.0	8.0	1.8	1.5	37.2	85.2	0.0	0.0	0.0	133.7
2007	0.0	0.0	0.0	0.0	0.0	0.0	25.8	145.9	0.5	5.8	0.0	0.0	178.0
2008	0.0	0.0	0.0	13.0	0.0	8.1	8.4	42.3	7.4	3.0	0.0	0.0	82.2
2009	0.0	0.0	0.0	0.0	0.0	0.0	29.2	101.2	4.8	0.5	0.0	0.0	135.7
2010	0.0	0.0	0.0	0.0	0.2	1.4	13.9	43.0	18.0	0.0	0.0	0.0	76.5
TOTAL	0.6	0.0	3.6	13.0	180.6	181.9	1122.3	2133.4	964.6	264.8	29.8	0.0	4894.6

4.3 Routing the main valleys in East Nile locality:

Main valleys and sub- valleys paths are been determined, also the slopes and length of valleys, down to the catchment area. A set of Valleys in East Nile region are given below:

4.3.1 Soba valley:

Figure (4.2) shows the path of soba valley, catchment area, maximum flow slope, maximum flow distance, Basin slope, and length of the valley.

Figure (4.2): shows the path of soba valley, catchment area, maximum flow slope, maximum flow distance, Basin slope, and length of the valley

MFS= 0.0018m/m

MFD= 93986.40m

L= 64958.24 m

BS= 0.0072 m/m

A= 1350.49 km^2

4.3.2 Alfadal Valley:

Figure (4.3) shows the path of Alfadal valley, catchment area, maximum flow slope, maximum flow distance, Basin slope, and length of the valley.

MFS=0.0007 m/m
MFD=95501.53 m
L=53511.41 m
BS=0.0058 m/m
A=829.27 km^2

East Nile Valley

MFS=mAXIMUM fLOW sLOPE
MFD=Maximum Flow Diatance
L=Length
BS=Basin Slope
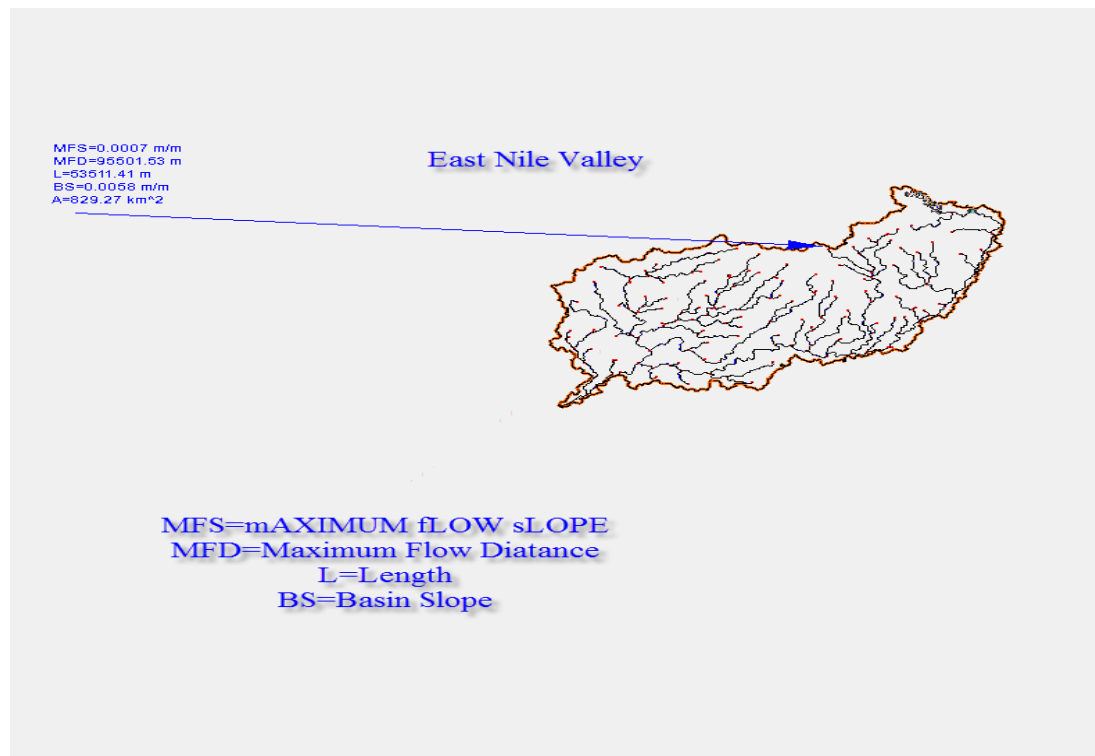
Figure (4.3) shows the path of Alfadal valley, catchment area, maximum flow slope, maximum flow distance, Basin slope, and length of the valley.

MFS= 0.0007m/m

MFD= 95501.53 m

L= 53511.41 m

BS= 0.0058

A= 829.27 km^2

4.3.3 Al-Tamada Valley

Figure (4.4) shows the path of Al-tmada valley, catchment area, maximum flow slope, maximum flow distance, Basin slope, and length of the valley.
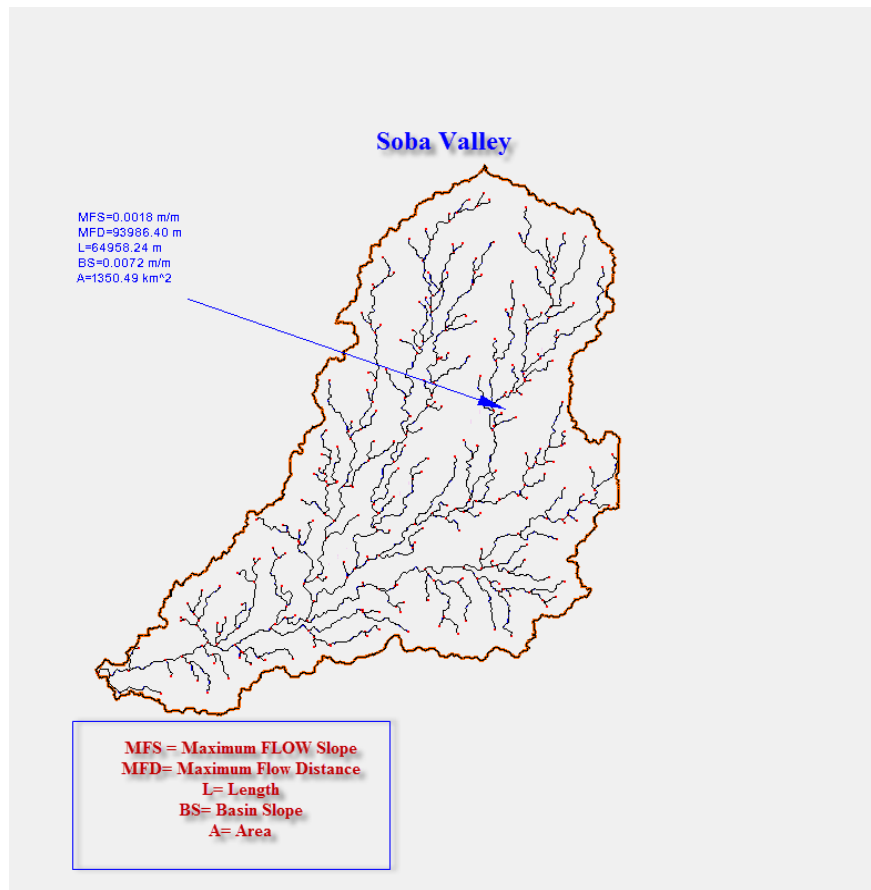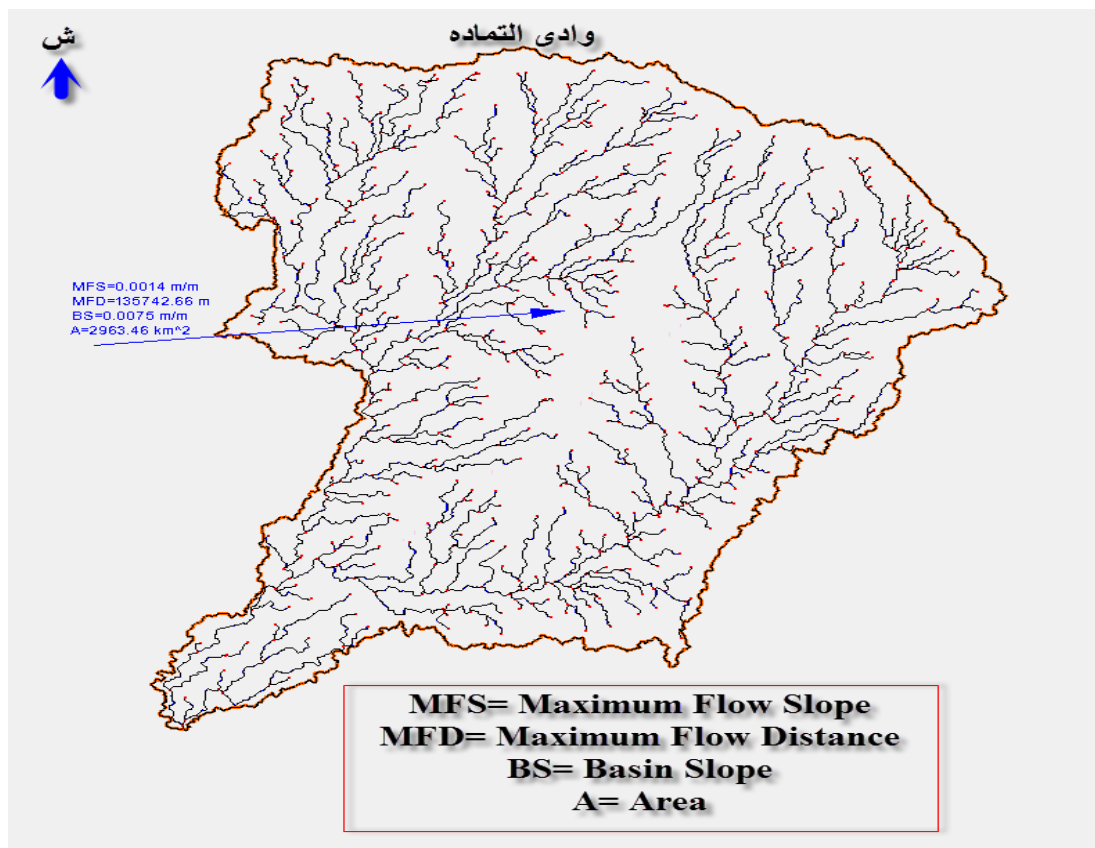

Figure (4.4): shows the path of Al-tmada valley, catchment area, maximum flow slope, maximum flow distance, Basin slope, and length of the valley.

MFS=0.0014m/m

MFD=135742.66 m

BS= 0.0075 m/m

A= 2963.46 km^2

4.4 Analysis of rainfall data:

4.4.1 Estimating the Probability of Rainfall data:

If there are rainfall records for 30 to 40 years, the various storms during the period of record may be arranged in the descending order of their magnitude (of maximum depth or intensity). When arranged like this in the descending order, if there are a total number of n items and the order number or rank of any particular storm (maximum depth or intensity) is m, then the recurrence interval T (also known as the return period) of the storm magnitude is given by the following equations:

$$T = \frac{n+1}{m} \tag{4.1}$$

The probability that a T- year storm may occur is equal to :

$$F = \frac{1}{T} \times 100\% \tag{4.2}$$

The probability (p) and return period rainfall data are shown in table (4.2) below. The main discharge of valleys and return periods are shown in table (4.3)

Table (4.2) shows the probability of Rainfall data and return period T

YEAR	RAINFALL (MM)	T	P%
1971	120.5	2.05	48.70
1972	133.6	2.56	39.02
1973	174.7	5.85	17.07
1974	86.0	1.41	70.70
1975	103.0	1.64	60.97
1976	178.8	8.2	12.19
1977	166.7	5.12	19.51
1978	133.6	2.41	41.46
1979	100.1	1.52	65.85
1980	96.3	1.46	68.29
1981	141.1	3.73	26.82
1982	102.7	1.57	63.41
1983	84.0	1.36	73.17
1984	4.7	1.05	95.12
1985	38.8	1.08	92.68
1986	57.7	1.17	85.36
1987	115.6	1.95	51.22
1988	415.5	40.98	2.44
1989	79.8	1.28	78.05
1990	4.4	1.02	97.56
1991	44.9	1.15	87.80
1992	149.9	4.1	24.39
1993	39.8	1.11	90.24
1994	232.6	20.53	4.87
1995	195.0	10.25	9.75

1996	199.3	13.68	7.31
1997	141.0	2.93	34.14
1998	110.7	1.86	53.66
1999	130.6	2.27	43.90
2000	60.0	1.2	82.93
2001	127.8	2.16	46.34
2002	107.5	1.71	58.53
2003	161.4	4.55	21.95
2004	109.7	1.78	56.10
2005	140.7	3.15	31.71
2006	133.7	2.73	36.58
2007	178.0	6.83	14.63
2008	32.2	1.32	75.61
2009	137.7	2.93	34.15
2010	76.5	1.24	80.48

4.4.2 Distribution charts

Figure (4.5), (4.6) shows the logarithmic distribution of rainfall data below

Statistical distribution of Rainfall

$$y = -76.074 Ln(x) + 399.08$$
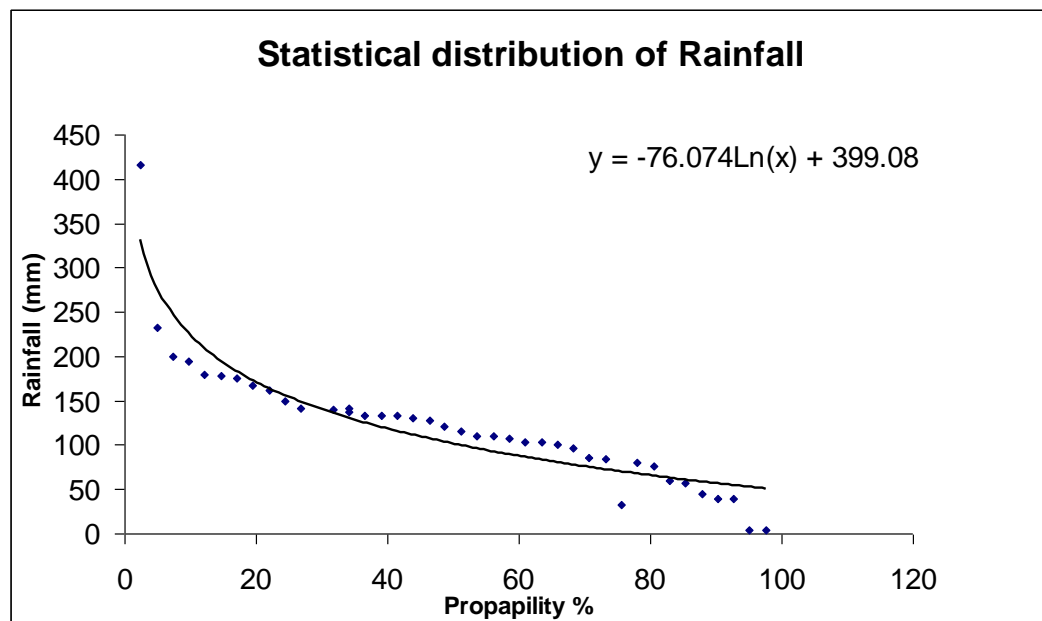
Figure (4.5) shows probability distribution curve

Distribution of rainfall data

$$y = 76.074 Ln(x) + 48.751$$
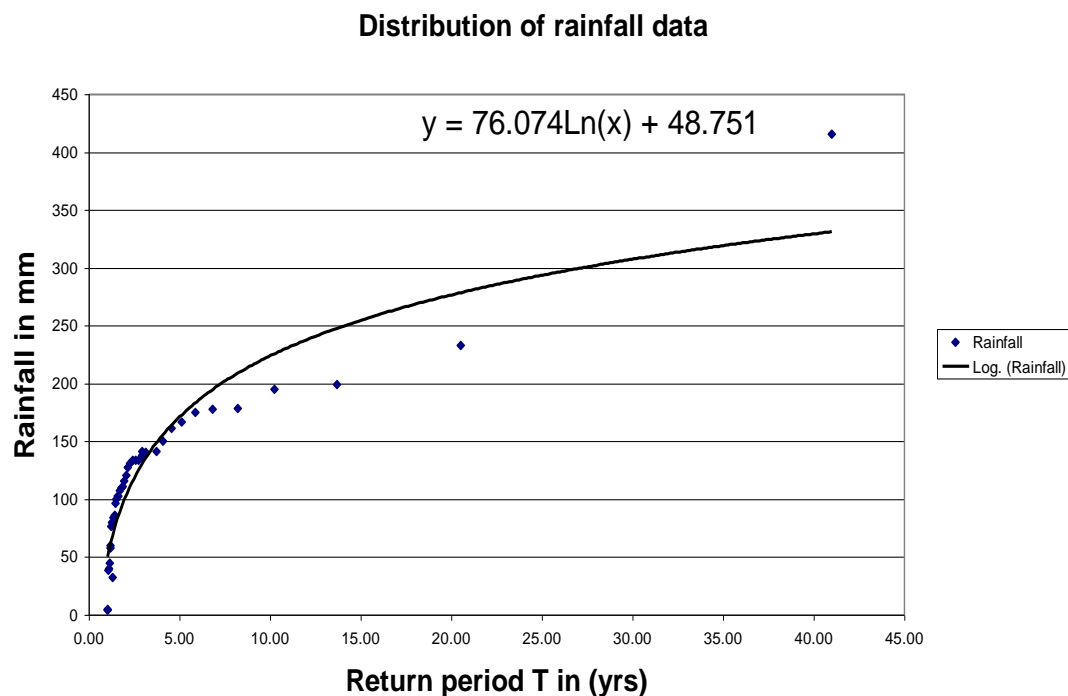
Figure (4.5) shows rainfall, return period distribution curve

Table (4.3): Daily rain fall data and discharges of valleys in East Nile region

DISCHARGES M^3/SEC	RAINFALL(MM)	RETURN PERIOD
1.81 x 10²	87	10
2.53 **x** 10²	122	25
3.08 x 10²	148	50
3.62 x 10²	174	100

4.5 Estimating flash flood discharge in catchment area:

4.5.1 Determination of peak flow and flood

The flood analysis typically consists of two components: hydrologic analysis (determination of peak flows and flood hydrographs) and a hydraulic analysis (determination of flood depths, extents and conceptual design of hydraulic structures).

4.5.2 Factors Affecting the Quantity of Storm water:

The surface run-off resulting after precipitation contributes to the storm water.

The factors affecting the quantity of storm water flow are as below:

i. Area of the catchment

ii. Slope and shape of the catchment area

iii. Porosity of the soil

iv. Obstruction in the flow of water as trees, fields, gardens, etc.

v. Initial state of catchment area with respect to wetness.

vi. Intensity and duration of rainfall

vii. Atmospheric temperature and humidity

   Methods used in estimating the quantity of storm water are

1- Rational Method

2- Empirical formulae method

4.5.3 Rational Method:

In this project rational method is been used to determine the quantity of storm water, storm water is considered as function of intensity of rainfall, Coefficient of runoff and area of catchment, rational method equation is as following:

$$Q = 0.278 \; C.I.A \hspace{4cm} (4.3)$$

Where,

Q = Quantity of storm water, m3/sec

C = Coefficient of runoff

I = intensity of rainfall (mm/hour) for the duration equal to time of concentration

A = Drainage area in km

The typical runoff coefficient for the different ground surface is provided in Table (4.4)

Table (4.4) Runoff coefficient for different type of surface in catchment

Type of surface	Factor C
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%	0.13-0.22
Heavy soil, average, 2-7%	0.18-0.22
Heavy soil, steep, 7%	0.25-0.35
Asphaltic pavements	0.80-0.95
Concrete pavements	0.70-0.95
Gravel or macadam pavements	0.35-0.70

Runoff coefficient in Sudan is taken between (0.1- 0.15)

4.5.4 Time of Concentration:

Is the period after which the entire catchment area will start contributing to the runoff is called as the time of concentration.

5-5-3-2 Rainfall Intensity Duration frequency curve (IDF)

IDF curve is used to determine the intensity of rainfall in the study area; it does provide information about heavy rainfall events of various amounts and durations. IDF curve are critical to determining the appropriate design standards. Figure (4.7) below shows the IDF curve.
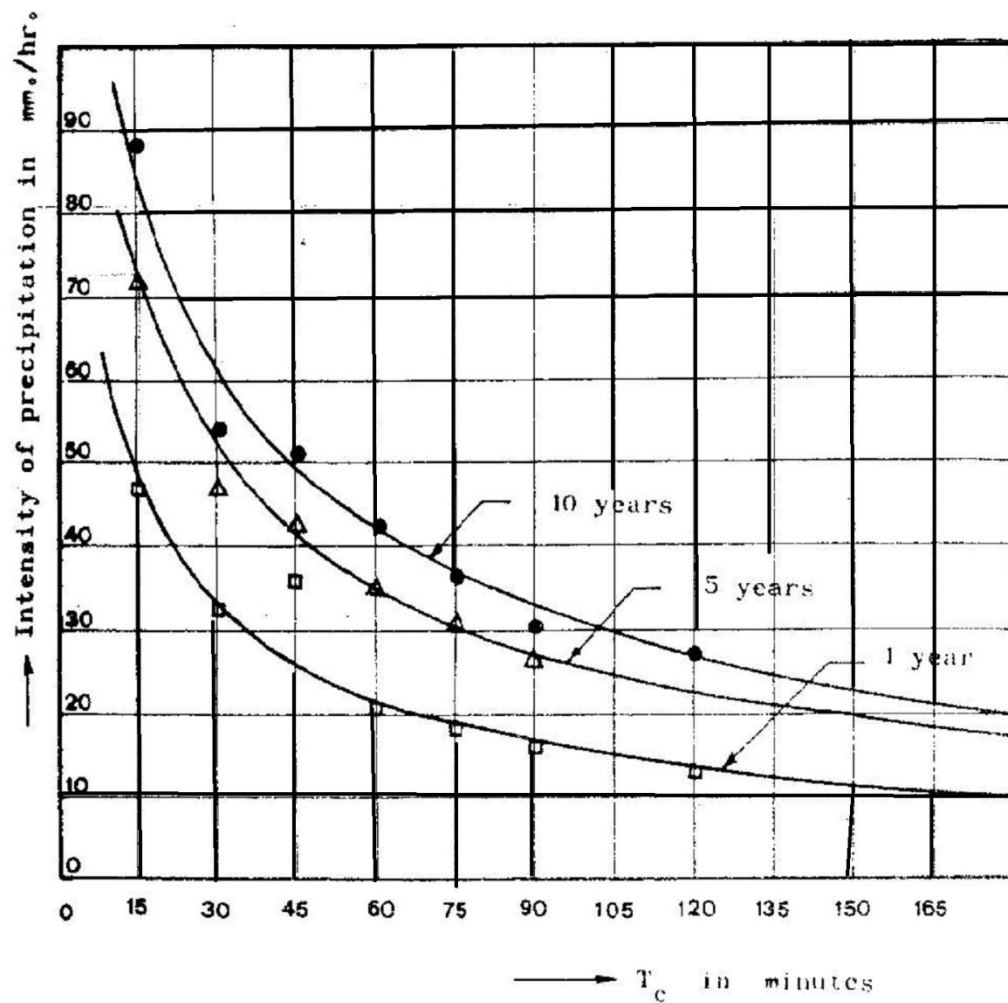
Figure (4.7): Shows the intensity – Duration – Frequency curve

4.6 Case study:

The case study focuses on three catchments located to the Northeast of Blue Nile; the return period of storm water shall be 5 years.

In Khartoum state Duration of rainfall is between (10 to 45) for east Nile region duration of storm water shall be taken 15 minutes.

4.6.1 Determination of peak flow:

4.6.1.1 Soba Valley:

From: $Q = 0.278$ CIA

And from table (4.2) coefficient of runoff in sandy soil ground is 0.1, Catchment area = 1350.49 km2

For computing the intensity of rainfall, from the IDF curve the intensity of rainfall is given by 72 mm/hr.

Then

$Q = 0.278$ x 0.1 x72 mm/hr x 1350.49 x10^6 = 2.7x10^6 m^3/hr

$Q = 750.8$ m^3/sec

4.6.1.2 Al-fadal valley:

From table (4.2) coefficient of runoff in sandy soil ground is 0.15

Catchment area = 829.27 km^2

For computing the intensity of rainfall, from the IDF curve the intensity of rainfall is given by 72 mm/hr.

Then

$Q = 0.278$ x 0.15 x72 mm/hr x 829.77 x10^6 = 2.4x10^6 m^3/hr

$Q = 692$ m^3/sec

4.6.1.3 Al-tamada valley:

From table (4.2) coefficient of runoff in sandy soil ground is 0.1

Catchment area = 2963.46 km^2

For computing the intensity of rainfall, from the IDF curve the intensity of rainfall is given by 72 mm/hr.

Then

$$Q = 0.278 \times 0.1 \times 72 \text{ mm/hr} \times 2963.46 \times 10^6 = 5.9 \times 10^6 \text{ m}^3/\text{hr}$$

$$Q = 1647.68 \text{ m}^3/\text{hr}$$

4.6.2 Analysis of maximum daily discharge for design of spillway:

Statistical analysis has been done to daily discharges data, and the best model for the distribution of data is exponential distribution.

In Khartoum University in the year 1995 a study of characteristic of rainfall in Khartoum has been done for computing the IDF curve, the best distribution model of daily rain fall data from the curve is exponential distribution; the equation of probability of daily rainfall is in the following:

$X_t = X \ln T$

Where

T = Return period

X_t = annually rainfall of T

X = Daily rainfall average

4.6.2.1 Statistical analysis of daily rainfall data of shambat station in 84 years is shown in table (4.5) below.

Table (4.5): Statistical analysis of daily rainfall data of Shambat station in 84 years

84 YEARS	N
37.76	Median
2.84	Coefficient of error
26.07	Standard deviation
679.63	Variation
2.00	Maximum level
147.00	Minimum level

4.6.2.2 Statistical analysis of annual rainfall data of shambat station is shown in table (4.6) below.

Table(4.6): Statistical Analysis of annual rainfall data

84 YEARS	N
37.76	Median
2.84	Coefficient of error
26.07	Standard division
679.63	Variation
2.00	Maximum level
147.00	Minimum level

4.7 Open channels Hydrological Analysis:

A *return period* of five years is widely used to design primary drainage systems in cities, but shorter periods (three years or less) are more suitable for micro-drainage within residential areas, where an occasional overflow is less likely to cause serious damage. If it is designed too big, it may never be fully amortized within its lifetime. In this case the money for the big construction could have been used for other constructions.

4.8 Hydraulic Design:

4.8.1 Hydraulic design of Drainage channels

4.8.1.1 Design concepts

The design method is based on the concept of maximum permissible tractive force. The method has two parts, computation of the flow conditions for a given design discharge and determination of the degree of erosion protection required. The flow conditions are a function of the channel geometry, design discharge, channel roughness, channel alignment and channel slope. The erosion protection required can be determined by computing the shear stress on the channel lining (and underlying soil, if applicable) at the design discharge and comparing that stress to the permissible value for the type of lining/soil that makes up the channel boundary.

4.8.1.1 Type of Flow

Open-channel flow can be classified according to three general conditions:

1. Uniform or non-uniform flow

2. Steady or unsteady flow.

3. Subcritical or supercritical flow.

In uniform flow, the depth and discharge remain constant along the channel. In steady flow, no change in discharge occurs over time. Most natural flows are unsteady and are described by runoff hydrographs. It can be assumed in most cases that the flow will vary gradually and can be described as steady, uniform flow for short periods of time. Subcritical Flow is distinguished from super critical flow by a dimensionless number called the Froude number (Fr), which is defined as the ratio of inertial forces to gravitational forces in the system. Subcritical flow (Fr < 1.0) is characterized as tranquil and has deep, slower velocity flow. Supercritical flow (Fr > 1.0) is characterized as rapid and has shallow, high velocity flow.

4.8.2 Design Criteria:

For design purposes, uniform flow conditions are usually assumed with the energy slope approximately equal to average bed slope. This allows the flow conditions to be defined by a uniform flow equation such as Manning's equation. Supercritical flow creates surface waves that are approaching the depth of flow.

Open channels having a design flow rate less than five hundred (500) cfs may be designed assuming uniform flow conditions in conjunction with computed headwater depths at culverts and other hydraulic structures or reservoir stages at detention and sediment basins. Water surface profiles using techniques for gradually varied flow may be required for design flow rates less than five hundred (500) cfs where accurate determination flooding depths is necessary to ensure flood safety.

Under steady state, uniform flow conditions channel capacity shall be computed using Manning's Equation:

$$Q = \frac{1.49}{n} A \, R^{2/3} S^{1/2} \tag{4.4}$$

Where

Q = rate of flow, cubic feet per second

n = Manning's roughness coefficient

A = cross sectional area of flow, square feet

P = wetted perimeter, feet

R = hydraulic radius = A/P, feet

S = S f = friction slope

For SI units Manning's equation shall be

$$Q = \frac{1}{n} . A . R^{2/3} . S^{1/2} \tag{4.5}$$

Table (4.7): gives the values of roughness coefficient for various channels lining

MANNING'S ROUGHNESS COEFFICIENT FOR VARIOUS CHANNEL LININGS

Type of Surface	Manning's n Value
Concrete, float finish	0.013-0.015
Gunite, shotcrete	0.016-0.019
Concrete bottom, with pre-cast masonry unit sides	0.020-0.030
Gravel bottom with pre-cast masonry unit sides	0.023-0.033
Riprap, Note 3	0.050-0.060
Grouted riprap, Note 3	0.023-0.030
Grass Channels, VR>10	0.030-0.035
VR<10	0.030-0.200 (See Section 111.4.2)
Stream Channels: gravel, cobbles, few boulders	0.040-0.050
Floodplains	
Short grass pasture, hayfield	0.030-0.035
Tall grass pasture, hayfield	0.035-0.050
Mature field crops	0.040-0.050
Heavy weeds, scattered brush	0.050-0.070
Light brush and trees	0.060-0.080
Medium to dense brush	0.100-0.160
Heavy stand of timber, a few down trees, little undergrowth	0.100-0.120

(Source: Chow, open channels Hydraulics)

4.8.3 Resistance to Flow

For rigid channel lining types, Manning's roughness coefficient, n, is approximately constant. However, for very shallow flows the roughness coefficient will increase slightly. (Very shallow is defined where the height of the roughness is about one-tenth of the flow depth or more.)

For a riprap lining, the flow depth in small channels may be only a few times greater than the diameter of the mean riprap size. In this case, use of a constant n value is not acceptable and consideration of the shallow flow depth should be made by using a higher n value. Table (4.8) below shows typical roughness coefficient for selected linings.

Table (4.8) shows typical roughness coefficient for selected linings.

TYPICAL ROUGHNESS COEFFICIENTS FOR SELECTED LININGS				
		Manning's n		
Lining Category	Lining Type	Maximum	Typical	Minimum
Rigid	Concrete	0.015	0.013	0.011
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.025	0.020	0.016

	Rock Cut uniform	0.045	0.035	0.025
RECP	Open-weave textile	0.028	0.025	0.022
	Erosion control blankets	0.045	0.035	0.028
	Turf reinforcement	0.036	0.030	0.024

4.8.4 Design Parameters:

4.8.4.1 Design Discharge Frequency:

Design flow rates for permanent roadside and median drainage channel linings usually have a 5 or 10-year return period. A lower return period flow is allowable if a transitional lining is to be used, typically the mean annual storm approximately a 2-year return period (50 percent probability of occurrence in a year). Transitional channel linings are often used during the establishment of vegetation. The probability of damage during this relatively short time is low, and if the lining is damaged, repairs are easily made.

4.8.4.2 Channel Cross Section Geometry

Most highway drainage channels are trapezoidal or triangular in shape with rounded corners. For design purposes, a trapezoidal or triangular representation is sufficient. Design of roadside channels should be integrated with the highway geometric and pavement design to insure

proper consideration of safety and pavement drainage needs. If available channel linings are found to be inadequate for the selected channel geometry, it may be feasible to widen the channel. Either increasing the bottom width or flattening the side slopes can accomplish this.

4.8.4.3 Channel Slope:

The slope of a roadside channel is usually the same as the roadway profile and so is not a design option. If channel stability conditions are below the required performance and available linings are nearly sufficient, it may be feasible to reduce the channel slope slightly relative to the roadway profile.

Channel slope is one of the major parameters in determining shear stress. For a given design discharge, the shear stress in the channel with a mild or subcritical slope is smaller than a channel with a supercritical slope.

4.8.4.4 Freeboard:

The freeboard of a channel is the vertical distance from the water surface to the top of the channel at design condition. The importance of this factor depends on the consequence of overflow of the channel bank. At a minimum, the freeboard should be sufficient to prevent waves or fluctuations in water surface from washing over the sides. In a permanent roadway channel, about 0.15 m (0.5 ft) of freeboard should be adequate, and for transitional channels, zero freeboard may be acceptable.

4.8.4.5 Design computation:

The design discharge assumption of open channels is 15 m^3 per sec distributed among the West Nile district.

- Assume:

$Q = 15$ m^3/s

- Channels Type and lining:

Trapezoidal channel

Stone Masonry $n = 0.042$

Bed width $= 1m$

Bed slope $S = 0.008$

$Z = 3$

- Assume that the depth of flow equal 1.5m

- Manning's formula

$A = Bd + Zd^2 = 1 \times 1.5 + 3 \times 1.5^2 = 8.3$ m

$P = 10.5$ m

$R = A/P = 8.3/10.5 = 0.8$ m

$$Q = \frac{1}{n} AR^{2/3} S_f^{1/2} = \frac{1}{0.042}(8.3)(0.8)^{2/3}(0.008)^{1/2} = 15.2 \ m^3/s$$

Since this value is different from the design flow, we need to go back to step 4 to estimate a new flow depth.

Estimate a new depth:

This can be estimated from the following equation:

$$d_{i+1} = d_i \left(\frac{Q}{Q_i} \right)^{0.4}$$

$$d_{i+1} = d_i \left(\frac{Q}{Q_i} \right)^{0.4} = 1.5 \left(\frac{15}{15.2} \right)^{0.4} = 1.49 \quad m$$

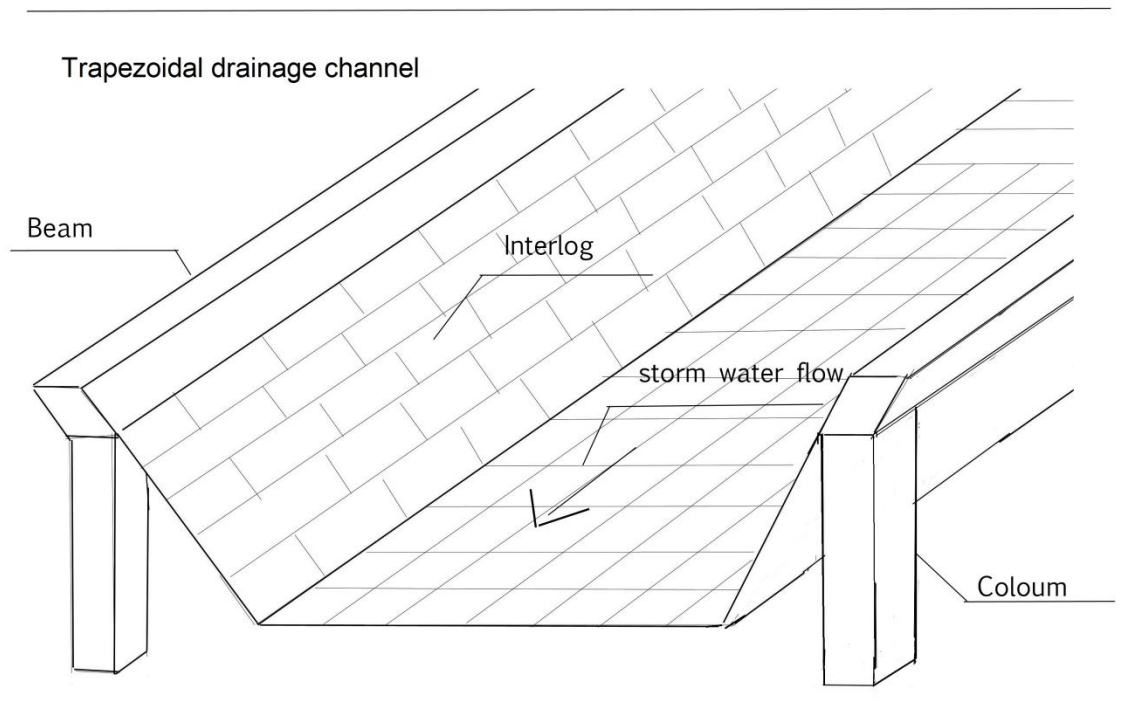The trapezoidal open channel in figure (4.8) shows a lining masonry stone.

Figure (4.8): Trapezoidal open channel

4.8.2 Design of Filter

4.8.2.1 Components of a Storm water Filter System

Storm water filters can be designed to infiltrate all or some of the flow. Components of storm water filter system include:

- Excavation or container

- Pretreatment

- Flow entrance/inlet

- Filter media

- Surface storage (ponding area)

- Underground drain, if required (septic tanks, wells)

4.8.2.2 Excavation or Container:

The filter media may be contained in a simple trench lined with a geotextile, or it may be contained in a more structural facility such as concrete. In either case, the container may be designed either to allow infiltration or to collect flow in an under drain system.

4.8.2.3 Surface Storage:

The filter allows water to pond during intense storms as water flows slowly through the filter media.

4.8.2.4 Filter Media:

Storm water flows onto filter media where sediments and other pollutants are separated from the storm water. Filter materials such as sand, peat, granular activated carbon leaf compost, pea gravel and others are used for water quality treatment. In addition to determining the degree of

filtration, media particle size determines travel time in the filter and plays a role in meeting release rate requirements.

4.8.2.5 Underground Sand Filter:

This system is placed underground but maintains essentially the same components as the surface sand filter. The practice consists of a three chamber vault. A three feet deep wet sedimentation chamber is hydraulically connected by an underwater opening with the second chamber. This element is designed to dissipate energy and to provide pretreatment by trapping grit and floating organic material. The second chamber contains an 18" - 24" sand filter bed and an under drain system including inspection/cleanout wells The third chamber collects the flow from the under drain system and directs flow to the downstream receiving drainage system .

4.8.2.6 Design Hydrology:

Because of the stochastic nature and temporal variability of storm water runoff, any storm water media filter will need a detention storage volume upstream of it. This detention volume permits the capture of rapid runoff so as to buffer the flows that have to be processed through the filter.

A filter without such a buffer would have to be very large to keep up with the instantaneous runoff rates during rainstorms. The amount of this detention volume is determined by local runoff characteristics. To deal with the stochastic nature of the runoff process, typically a *design storm* is selected. Also, the rate at which the runoff from this *design storm* is allowed to drain through the filter determines its size. This detention

capture volume needs to be emptied out in a reasonable amount of time to provide volume for the next storm runoff event that may follow.

4.8.2.5 Design procedure:

A conceptual design for storm water filter must maintain the storm water discharge system in residential areas the concept of design is to construct underground filters in fields which include filter chambers and wells, figure (4.9) shows a plan of filter chambers, and figure (4.10) shows a profile of filter.
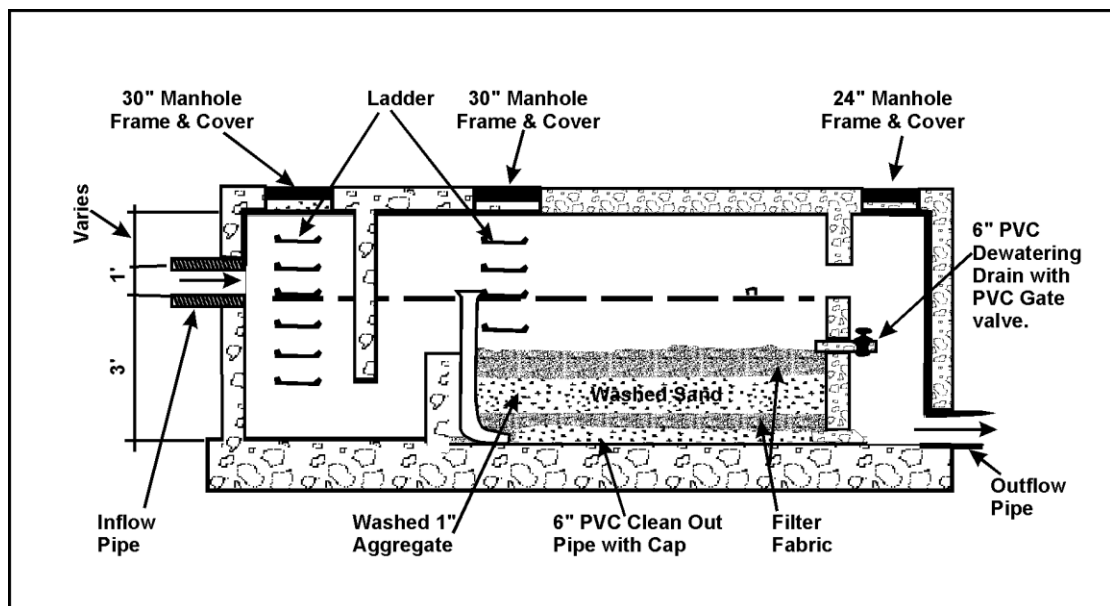
Source: Reference 3

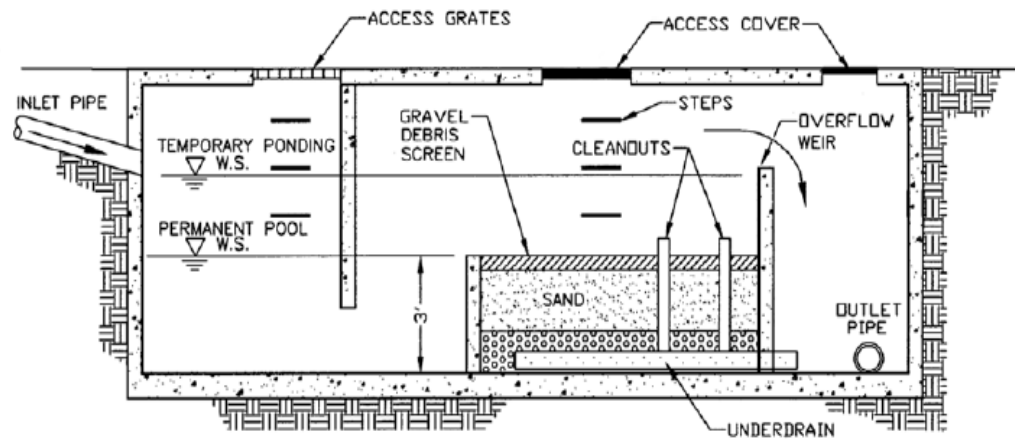**Figure(4.9):** shows a plan of filter chambers

Figure (4.10) shows a profile of filter.

4.8.2.6 Design values for ponding and media depth:

Table (4.8): Design values for ponding and media depth

AVERAGE PONDING DEPTH	3-6 INCHES
Filter media depth	18-30 inches

4.8.2.7 Construction Guidelines:

- Areas for storm water filters shall be clearly marked before any site work begins to avoid soil disturbance and compaction during construction.

- Permanent filters should not be installed until site is stabilized. Excessive sediment generated during construction can clog filter and prevent its function prior to post-construction benefits.

- Structures such as inlet boxes, reinforced concrete boxes, inlet controls, and outlet structures should be constructed in accordance with manufacturer's guidelines or Engineer's guidance.

- Excavated filters or structural filters that infiltrate should be excavated in such a manner as to avoid compaction of the sub-base. Structures should be set on a layer of clean, lightly compacted gravel specified as AASHTO No. 57.

- A layer of permeable non-woven geotextile should underlie infiltration filters.

- Place underlying gravel/stone in minimum 6 inch lifts and lightly compact. Place under drain pipes in gravel during placement (if applicable).

- Wrap and secure non-woven geotextile to prevent gravel/stone from clogging with sediments.

In figure (4.10) shown below a plan view of underground drain system constructed in central field in residential areas , the system contain well and large subsurface filter, this system can also be designed to infiltrate directly into the soil .

Underground stormwater drain
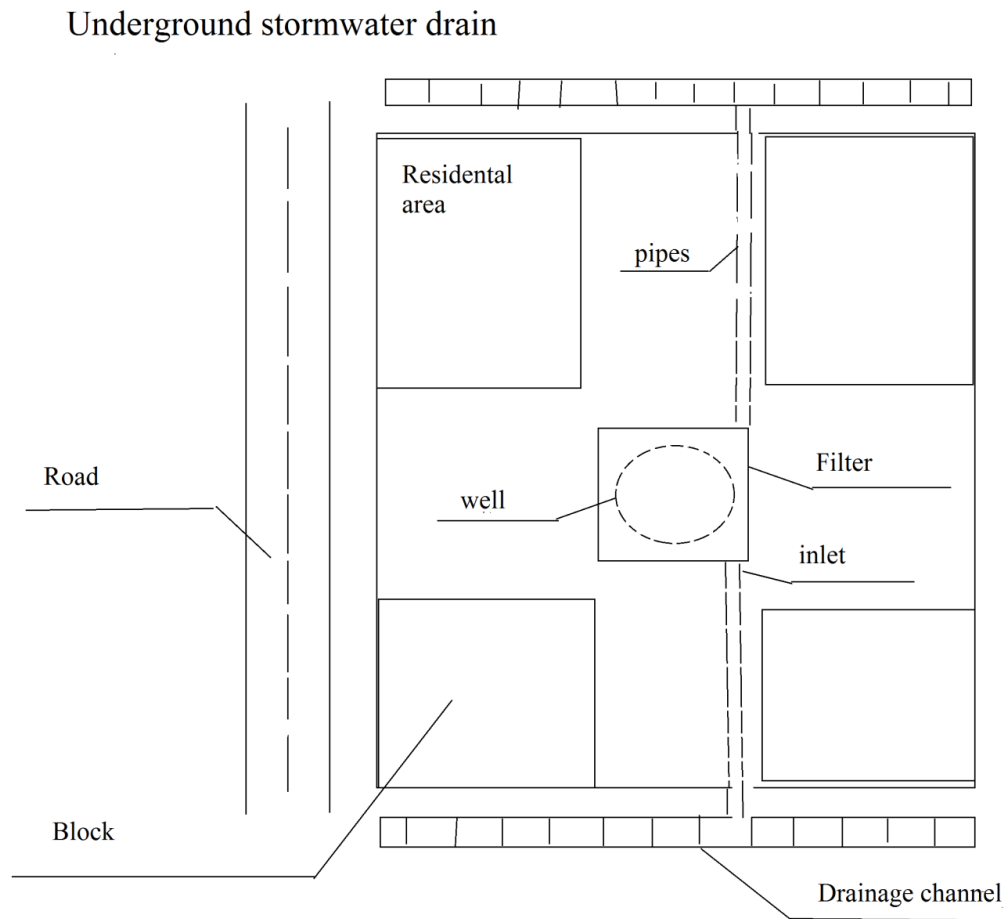
Figure(4.10): a plan view of underground drain system constructed in central field in residential areas

4.7 Structural Design:

4.7.1 Structural design of Flood control Dams:

The principal design considerations of embankment dams are:

1. *Overtopping and freeboard.* Spillway and outlet capacity must be sufficient to pass the design maximum flood without overstopping and risk of serious erosion and possible washout of the embankment.eg. (Freeboard) the difference between maximum water hight level and minimum crest level of the dam, must also be sufficient to accept the design flood plus wave action without overtopping, and must include an allowance for the predicted longterm settlement of the embankment and foundation.

2. *Stability.* The embankment, including its foundation, must be stable under construction and under all conditions of reservoir operation. The face slopes must therefore be sufficiently flat to ensure that internal and foundation stresses remain within acceptable limits.

3. *Control of seepage.* Seepage within and under the embankment must be controlled to prevent concealed internal erosion and migration of fine materials, e.g. from the core, or external erosion and sloughing. Hydraulic gradients, seepage pressures and seepage velocities within and under the dam must therefore be contained at levels acceptable for the materials concerned.

4. *Upstream face protection.* The upstream face must be protected against local erosion as a result of wave action, ice movement, etc.

5. *Outlet and ancillary works.* Care must be taken to ensure that outlet or other facilities constructed through the dam do not permit

unobstructed passage of seepage water along their perimeter with risk of soil migration and piping.

In designing the crest of an earthfill dams, the following items should be considered:

- width
- drainage
- camber
- surfacing
- safety requirements
- zoning

4.7.1.1Crest width of Dam:

The crest width of an earthfill depends on considerations such as

(1) nature of embankment materials and minimum allowable percolation distance through the embankment at normal reservoir water level,

(2) height and importance of structure,

(3) possible roadway requirements

(4) Practicability of construction.

The minimum crest width should provide a safe seepage gradient through the embankment at the level of the full reservoir. Because of practical difficulties in determining this factor, the crest width is, as a rule, determined empirically and largely by precedent.

The following formula is suggested for the determination of crest width for small earthfill dams:

$$w = \frac{z}{5} + 10 \tag{4.6}$$

where:

w = width of crest, in feet

z = height of dam, in feet , above the streambed.

For ease of construction with power equipment the minimum width should be at least 12 feet. For many dams, the minimum crest width is determined by the requirements for the roadway over the dam.

For practical reasons it is recommended that the minimum crest to be 4 to 5 meters, the crest width is given by the equation below;

$$W_c = (H/3) + 3 \tag{4.7}$$

Where :

W_c = Crest width

H = Height of dams in meters

The equation above is used to determine the crest width of flood control dams.

H= 4.5 m

$W_c = (4.5/3) + 3 = 4.5m$

4.7.1.2Freeboard

Freeboard is the vertical distance between the crest of the embankment (without camber) and the reservoir water surface. The more specific term "normal freeboard" is defined as the difference in elevation between the crest of the dam and the normal reservoir water level as fixed by design requirements. The term "minimum freeboard" is defined as the difference in elevation between the crest of the dam and the maximum reservoir water surface that would result should the inflow design flood occur and the outlet works and spillway function as planned.

4.7.1.2.1Freeboard Design Considerations:

Freeboard must be sufficient to prevent overtopping of the dam that could result from reasonable combinations of a number of factors listed above:

1- Wind-generated wave action, wind setup, and wave runup.

2- Earthquake and/or landslide-generated waves and runup;

3- Post-construction settlement of embankment dams and foundations;

4- Provision for malfunction of spillways (especially gated structures) and outlet works, and

5- Site-specific uncertainties including flood hydrology.

4.7.1.3 Evaluate Effective Fetch (Fe):

Effective fetch (Fe) is defined as the average horizontal distance in the general direction of the wind over water, over which a wind acts to generate waves; table (4.9) bellow shows the effective fetch.

Table (4.9): shows the effective fetch.

Effective fetch (Fe) KM	Wind velocity Ratio Over water / over land
0.8	1.08
1.6	1.13
3.2	1.21
4.8	1.26
6.4	1.28
>8	1.30

The deep water length (L) is computed as follows :

$$L = 5.12 \, T^2$$

There is a number of approximate equations that gives the value of wave height

$$H_w = 0.032 \, V.F + 0.763 - 0.271 \, (F) \qquad \text{for } F < 32 \text{ km}$$

$$H_w = 0.032 \, V.F \qquad \text{for } F > 32 \text{ Km}$$

Where:

H_w = height of water from top of crest to bottom of trough in meters

V = wind velocity in Km/hr

F = fetch in Km

(design of small dams)

Table (4.9) below shows height of dams and freeboard for different spillway types

Spill way Type	Height of Dam (m)	Max free board (m)
Uncontrolled spillway	Any height	2-3 m
Controlled spillway	H < 60	2.5 m
Controlled spillway	H > 60	3 m

F for small dam is between (4.5 - 7.5) km

Free board height is between (1.5– 3) m

Free board is equal to 1.5

4.7.1.4 Up stream and downstream slopes

Recommended slopes for small homogeneous earthfill dams on stable foundation is given in table (4.10)

Table (4.10): Recommended slopes for small homogeneous earthfill dams on stable foundation

Type	Subject to rapid drawdown	Soil classification	Upstream Slope	Downstream slope
Homogeneous		GW,GPSW,SP	Previous , unsuitable	
		GC,GM,SC,SM	2.5:1	2:1

or modified -	No	CL,ML	3:1	2.5:1
homogeneous		CH,MH	3.5:1	2.5:1
Modified - homogeneous	Yes	GW,GP,SW,SP	Previous , unsuitable	
		GC,GM,SC,SM	3:1	2:1
		CL,ML	3.5:1	2.5:1
		CH,MH	4:1	2.5:1

Upstream and downstream slopes are designed to protect the body of dams from continually seepage and settlement. Table (4.10)

In our design of small dam the preferred slopes are:

U/S slope (2.5:1)

D/S slope (1: 2)

Selection of dam slopes depend on the dam type is given in table (4.11) by Streling

Table (4.11): Selection of dam slopes depend on the dam type

Height	Freeboard	Top width	U/S slope	D/S slope
Up to 4.5	1.2 – 1.5	1.85	2:1	1.5:1
4.5 – 7.5	1.5 – 1.8	1.85	2.5:1	1.75-2:1
7.5 – 15	1.85	2.5	3:1	2:1
15 – 25.5	2.1	3.0	3:1	2:1

(Irrigation engineering & Hydraulic structures)

The suggested height is 4.5 m , then from table (7-3)

U/S slope = 2.5:1

U/S slope = 2:1

The design of dam is shown in figure (4.11) where the upstream &
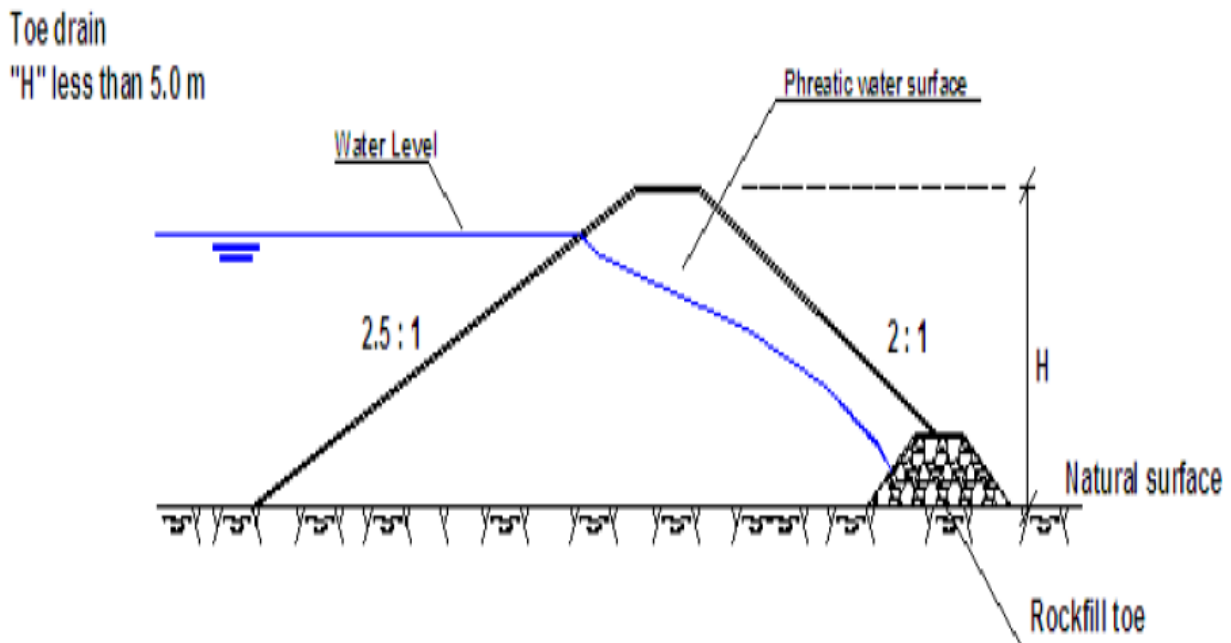downstream slopes are (2.5:1) , (2:1) respectively

Toe drain
"H" less than 5.0 m

Phreatic water surface

Water Level

2.5 : 1

2:1

H

Natural surface

Rockfill toe

Figure (4.11) the design of dam

4.7.1.5 Spillway:

In previous chapter the discharge of the valley had been identified (80% of water flow throw the spillway as suggested) the equation to determine the parameters of spillway is given by:

$$H\sqrt{2g}\,Q = 2/3\ \mu\ b\ d\ 3\backslash2 \tag{4.7}$$

Where:

b : spillway width

μ : angle depends on type and shape of spillway it is between (0.73- 0.75)

H_d : Height of water upon the top of spillway

The design of solid rock is given by:

$$X_n = KH_d^{n-1} \times y \tag{4.8}$$

Where :

n , K : safety factors depend on U/s slope , for drawing the curve take n= 1.85

K = 2

Then from given data below in table (4.11) the spillway is drawn in figure (4.12)

$$a\ =\ 0.175\ H_d$$
$$b\ =\ 0.282\ H_d$$
$$r_1\ =\ 0.5\ H_d$$
$$r_2\ =\ 0.2\ H_d$$

Table (4.11) Plotting Data

X	Y
0	0
1	0.35
2	1.27
3	2.7
3.5	3.56
4	4.59
4.5	5.57

4.7.1.5.1 Design procedure:

From the statistical characteristic of the West Nile valleys the discharge shall be:

$$Q = 308 \quad m^3/s$$

$$Q = 308 * 0.8 = 246.4 \, m^3/s$$

0.8 is a reduction factor

$$Q = \frac{2}{3}Cd \ b\sqrt{2g}(hd)^{3/2}$$
(4.9)

Where

Q = discharge over the top of spillway

Cd = discharge coefficient between $(0.56 - 0.75)$

b = effective length

Calculating h_d from equation

$$246.4 = \frac{2}{3} \times 25 \times 0.75 \sqrt{2 \times 9.81} (hd)^{\frac{3}{2}}$$

$h_d = 2.7\ m$ *not ok*

$$246.4 = \frac{2}{3} \times 30 \times 0.75 \sqrt{2 \times 9.81} (hd)^{\frac{3}{2}}$$

$h_d = 2.4\ m$ *not ok*

$$246.4 = \frac{2}{3} \times 35 \times 0.75 \sqrt{2 \times 9.81} (hd)^{\frac{3}{2}}$$

$h_d = 2.1$ *ok*

For spillway

     $B = 35\ m$

     $h_d = 2.1\ m$

     $c_d = 0.75\ m$

     $H = 3\ m$

4.7.1.6 Seepage Analysis:

     Seepage is regular movement of water of the upstream front through the body of the dam to the downstream side, this occurs due to pressure difference between two points.

     The theory of flow through porous mass is utilized for the estimation of seepage through an embankment dam and its foundation.

4.7.1.6.1Seepage damages to dam structure

1- Soil internal collapse.

2- Collapse of dams due to disintegration of soil practical and weakness of resistance.

3- Leak of water from the front of dams body cause a loss in the amount of water stored, and from previous studies 60% of dams' failures is due to seepage.

4.7.1.6.2 Seepage line in embankment dams

For drawing the seepage line the following equation is used:

(4.10)

$$S = \sqrt{b^2 + H^2} - b$$

Where :

From figure

b = horizontal length from A to F

H = water height in U/S

b = 8.9 m

H = 3 m

S= 0.5 m

For drawing the axes at a distance S/2

S/2 = 0.25

For drawing Seepage line curve the following equation is used :

$$Y = \sqrt{S^2 + 2XS}$$

Table (4.12) below shows the values of x,y in figure (4.13)

X	Y
1.0	1.12
2.0	1.5
3.0	1.8
4.0	2.06
5.0	2.3
6.0	2.5
6.5	2.6
7.0	2.7
7.5	2.78
8.0	2.87
8.5	3

4.7.1.6.3 Calculation of seepage:

To calculate the average of seepage through dam body Darcy equation below is used:

$$Q = k \, I \, A$$

(Irrigtion & hydraulic structures)

Where

i = hydraulic gradient

A = Area in m^2

K = seepage coefficient

$$A = Y*1$$

$$q = K \frac{dy}{dx} * Y$$

$$Y = \sqrt{S^2 + 2XS}$$

$$q = K/_2 * \sqrt{(S^2 + 2XS)^{-1/2}} * \sqrt{(S^2 + 2XS)^{1/2}}$$

$$q = K * S * \sqrt{\frac{S^2 + 2XS}{S^2 + 2XS}}$$

q = k * S

q = (5 x 10^(-4) x 0.5 = 0.00025 m^3/s/m

4.7.2 Structural design of Trapezoidal open channels

4.7.2.1 Shear stress:

Most drainage channels cannot tolerate bank instability and possible lateral migration. Stable channel design concepts focus on evaluating and defining a channel configuration that will perform within acceptable limits of stability. Methods for evaluation and definition of a stable configuration depend on whether the channel boundaries can be viewed as:

1- essentially rigid (static)

2- movable (dynamic)

In the first case, stability is achieved when the material forming the channel boundary effectively resists the erosive forces of the flow. Under such conditions the channel bed and banks are in static equilibrium,

remaining basically unchanged during all stages of flow. Principles of rigid boundary hydraulics can be applied to evaluate this type of system.

Chow 1979

4.7.2.2 Applied Shear Stress:

The hydrodynamic force of water flowing in a channel is known as the tractive force. The basis for stable channel design with flexible lining materials is that flow-induced tractive force should not exceed the permissible or critical shear stress of the lining materials. In a uniform flow, the tractive force is equal to the effective component of the drag force acting on the body of water, parallel to the channel bottom (Chow, 1959).

The mean boundary shear stress applied to the wetted perimeter is equal to:

$$\tau_o = \gamma \, R \, S_o \qquad\qquad (4.11)$$

Where:

τ_o= mean boundary shear stress, N/m^2 (lb/ft^2)

γ = unit weight of water, 9810 N/m^3 (62.4 lb/ft^3)

R= hydraulic radius, m (ft)

S_o= average bottom slope (equal to energy slope for uniform flow), m/m (ft/ft)

Shear stress in channels is not uniformly distributed along the wetted perimeter.

The maximum shear stress on a channel bottom, τ_d, and on the channel side, τ_s, in a straight channel depend on the channel shape. To simplify the design process, the maximum channel bottom shear stress is taken as:

$$\tau_o = \gamma \, d \, S_o$$

    :

Where

τ_d = shear stress in channel at maximum depth, N/m^2 (lb/ft^2)

d = maximum depth of flow in the channel for the design discharge, m (ft)

For trapezoidal channels where the ratio of bottom width to flow depth (B/d) is greater than 4, the above equation provides an appropriate design value for shear stress on a channel bottom. Most channels are characterized by this relatively shallow flow compared to channel width. For trapezoidal channels with a B/d ratio less than 4, equation above is conservative.

The relationship between permissible shear stress and permissible velocity for a lining can be found by considering the continuity equation:

Q= A x V

Where

Q = flow rate

V = flow velocity

A = area of flow

4.7.2.3 Permissible Shear Stress

Flexible linings act to reduce the shear stress on the underlying soil surface. Table (4.13) provides typical examples of permissible shear stress for selected lining types .Vegetative and RECP lining performance relates to how well they protect the underlying soil from shear stresses so these linings do not have permissible shear stresses independent of soil types.

Table(4.14) Typical Permissible Shear Stresses for Bare Soil and Stone Linings

Lining Category	Lining Type	Permissible shear stress	
		N/m^2	lb/ft^2
Bare Soil Cohesive (PI=10)[1]	Clayey sands	1.8-4.5	0.037-0.095
	Inorganic silts	1.1-4.0	0.027-0.11
	Silty sands	1.1-3.4	0.024-0.072
Bare Soil Cohesive[1] (PI≥20)	Clayey sands	4.5	0.094
	Inorganic silts	4.0	0.083
	Silty sands	3.5	0.072
	Inorganic clays	6.6	0.14
Bare Soil Non-cohesive[2] (PI<10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	1.0	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	5.6	0.12
	Gravel D_{75}=15 mm (0.6 in)	11	0.024
Gravel Mulch[3]	Coarse gravel D_{50}=25 mm (1 in)	19	0.4
	Very coarse gravel D_{50}=50 mm (2 in)	38	0.8
Rock Riprap[3]	D_{50}=0.15 m (0.5 ft)	113	2.4
	D_{50}=0.30 m (1.0 ft)	227	4.8

The basic comparison required in the design procedure is that of permissible to computed shear stress for a lining. If the permissible shear stressis greater than or equal to the computed shear stress, including consideration of a safety factor, the lining is considered acceptable. If a lining is unacceptable, a lining with a higher permissible shear stress is selected, the discharge is reduced (by diversion or retention/detention), or the channel geometry is modified. This concept is expressed as:

where,

$\tau_p > SF \tau_d$

τ_p = permissible shear stress for the channel lining, N/m^2 (lb/ft^2)

SF = safety factor (greater than or equal to one)

τ_d = shear stress in channel at maximum depth, N/m^2 (lb/ft^2)

4.7.2.4 Design procedure:
4.7.2.4.1 Check for stability:

$$\tau_d = \gamma dS_o = 9810(1.5)(0.008) = 117 \ N/m^2$$

From Table (4.14) the permissible shear stress, $\tau_p = .113 \ N/m^2$

For this channel, a SF = 1.0 is chosen.

$D_{50}=0.15$ m

Compare calculated shear to permissible shear using the following equation:

$\tau_p \geq SF \tau_d$

$113 \geq 117$ *not ok*

The lining is not stable from table an alternative lining type with greater permissible shear stress is selected

$D_{50}=0.30$, $227 > 117$ *ok*

CHAPTER FIVE

CONCLUSIONS & RECOMMENDATIONS

5. Conclusions & Recommendations

5.1 Conclusions:

1.   The amount and quality of hydrologic data collected in Sudan are rapidly growing, with the appropriate organization and analysis of these data becoming especially important towards gaining real benefits from many projects, such as flood control. Engineering solutions such as the Rational Method has been utilized.

2.   Flood control system is rapidly changing with time around the globe, and it is necessary to optimize the infrastructure in the locality to maintain flood control system.

5.2 Recommendations:

1.   Hydrological analysis of data to establish the design procedure of hydraulic structures is required in flash floods control project.

2.   Determine catchment area of basin by using computer applications e.g. (GIS)

3.   Construction of flood control dams in convenient location in basin area.

4.   Accuracy in culvert design prevents the occurrence of road collapse, sudden road collapses are often at poorly designed culvert crossing sites. Water passing through undersized culverts will scour away the surrounding soil over time. This can cause a sudden failure during medium sized rain events.

5.   The area needed to excavate the best hydraulic section might be larger than the area required to achieve the flow area in constructing drainage channels.

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