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Flood Frequency Analysis,
Case Study: The River Gash

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الآيـــة

بسم الله الرحمن الرحيم

أَوَلَمْ يَرَوْا۟ أَنَّا نَسُوقُ ٱلْمَآءَ إِلَى ٱلْأَرْضِ ٱلْجُرُزِ فَنُخْرِجُ (٢٦)
بِهِۦ زَرْعًا تَأْكُلُ مِنْهُ أَنْعَٰمُهُمْ وَأَنفُسُهُمْ ۖ أَفَلَا يُبْصِرُونَ (٢٧)

صدق الله العظيم

سورة السجدة

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…

Those humble but graceful individuals that thankfully made us what we are now.

Our best friends…

For the support when things were up and mostly when there were down.

Finally, to everyone who stood by us; and helped by any means to bring this work to its final form.

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ABSTRACT

This study is concerned with application of statistics in hydraulic engineering. Most hydrologic events follow a certain pattern; statistics and statistical modeling help describe the behavior of the matter of study with ease.

The study area was described thoroughly, listing all aspects regarding hydrologic properties of the river Gash and its catchment area and its recorded history and behavior.

In addition, hydrologic statistics and frequency analysis methods which are frequently used in hydraulic engineering were illustrated in the study.

Used in this research approach for data fitting, were both, the Normal Distribution and the Extreme Value Type one. Add to that, the return period of the data was plotted to predict the flood water levels using the cumulative distribution function.

التجـــريــد

تناولت هذه الدراسة تطبيق علم الإحصاء في هندسة الري و المياه. معظم الظواهر الهيدروليكية تتبع نمط معين؛ فبالتالي يمكن بإستخدام النماذج الإحصائية توصيف هذه الظواهر و أنماطها المختلفة.

إشتملت الدراسة على مقدمة عن مشكلة البحث المتمثلة في الفيضانات بصورة عامة و الفيضانات في نهر القاش - منطقة الدراسة – بصورة خاصة بالإضافة لوصف حوض النهر و خواصه المختلفة و تاريخه المسجل. البيانات المستخدمة هي أقصى مناسيب سنوية مسجلة لنهر القاش في ثلاث عقود متضمنة في ذلك أقصى فيضانات حدثت لنهر القاش في تلك الفترة.

بالإضافة، تم شرح و تفصيل الطرق الإحصائية و طرق التحليل الترددي المستخدمة في هندسة الري و المياه.

إستُخدم في عرض بيانات الدراسة طريقتي التوزيع الطبيعي و توزيع القيمة القصوى(النوع الأول)، أما بالنسبة لتوقع زمن الحدوث **Return Period** لمناسيب الفيضانات إستخرجت النتائج برسم منحنى التوزيع التكراري للبيانات و إيجاد المعادلة المناسبة للعلاقة بين الزمن و منسوب الفيضان، وبإستخدام هذه المعادلة إستخلصت النتائج المتعلقة بإحتمالات الحدوث المستقبلية.

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Chapter One

Introduction

1.Introduction

1.1 General:

The River Gash is a seasonal river that descends from the highlands of neighboring Eritrea a few miles to the south of Asmara and its catchment is partly in that country and partly in Ethiopia. It runs generally westwards until it debouches on to the great central plains of the Sudan near the political frontier between Eretria and the Sudan, here it swings northwards and after passing the town of Kassala - which the River divides into eastern and western parts - , fans out into an inland delta where the water is finally lost by percolation and evaporation. (Swan, 1956)

Gash is the main source of water supply for all purposes to Kassala and its surroundings. It is the only recharge source to Gash ground water basin. Gash also is the source that created the delta (about 300 thousand Feddans), that has the most fertile land for agriculture on which most of the Kassala socio economic activities depend. People say that without Gash there will be no Kassala. (Bashar, 2005)

In its lower reaches the Gash is an ephermal stream, normally flowing during the months of July, August and September. During this period the discharge is very variable and the water carries a heavy load of silt; as a result its course, particularly in the delta, has been subject to many and often violent fluctuations and Floods.

Flooding in El- Ghash affects Kassala and its surroundings badly, the River has reportedly flooded numerous times within the past four decades in 1975, 1988, and most recently July 2003 –, and both Floods and Flash Floods cause damage to housing, Agriculture, and Infrastructure.

Floods can be defined in general as "an unusual high stage of a river due to runoff from rainfall and/or melting of snow, in quantities too great to be confined in the normal water surface elevations of the river, as the result of unusual meteorological combination." (Hydrology, H.M Raghunah).
Flooding is a longer term event than flash flooding; it may last days or weeks, whereas flash floods are caused by heavy or excessive rainfall in a

short period of time, flash floods are usually characterized by raging torrents that rip through the river beds, urban streets or mountain canyons, sweeping everything before them. They can occur within minutes or a few hours of excessive rainfall.

The maximum flood discharge (peak flood) in a river may be determined by the following methods:

(i) Physical indications of past floods—flood marks and local enquiry.
(ii) Empirical formulae and curves.
(iii) Concentration time method.
(iv) Overland flow hydrograph.
(v) Rational method.
(vi) Unit hydrograph.
(vii) Flood frequency studies.

Flood frequency analysis is "The calculation of the statistical probability that a flood of a certain magnitude/level for a given river will occur in a certain period of time. Each flood of the river is recorded and ranked in order of magnitude with the highest rank being assigned to the largest flood. The return period here is the likely time interval between floods of a given magnitude/level and can be calculated as:

"Number of years of river records + 1rank of a given flood."

1.2 Objectives and Scope of Study:

Having in mind that flooding has become an issue in the past decades due to its severe effect on one's life and countries as whole. Plus, the useful kits and methods provided by the science of statistics and probability; the study is basically concerned with different aspects regarding the following:

- Prediction of Floods that may occur in the future in Gash River.
- Exercising the application and practice of Applied Statistics packages.
- Practicing the analysis of hydrological data related to rivers.

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 the Hydraulic Research Station (Wad Medani), to obtain previous studies and reports related to the topic.

Studying theories related to project proposal, and collecting related references and papers. Plus, exercising how to describe and analyze data.

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 Statement of the Problem (Floods)

2.1.1 Definition:

A flood is an overflow of water that submerges land which is usually dry. The European Union (EU) Floods Directive defines a flood as a covering by water of land not normally covered by water. In the sense of "flowing water", the word may also be applied to the inflow of the tide. Flooding may occur as an overflow of water from water bodies, such as a river or lake, in which the water overtops or breaks levees, resulting in some of that water escaping its usual boundaries, or it may occur due to an accumulation of rainwater on saturated ground in an areal flood. While the size of a lake or other body of water will vary with seasonal changes in precipitation and snow melt, these changes in size are unlikely to be considered significant unless they flood property or drown domestic animals.

Floods can also occur in rivers when the flow rate exceeds the capacity of the river channel, particularly at bends or meanders in the waterway. Floods often cause damage to homes and businesses if they are in the natural flood plains of rivers. While riverine flood damage can be eliminated by moving away from rivers and other bodies of water, people have traditionally lived and worked by rivers because the land is usually flat and fertile and because rivers provide easy travel and access to commerce and industry.

Some floods develop slowly, while others such as flash floods, can develop in just a few minutes and without visible signs of rain. Additionally, floods can be local, impacting a neighborhood or community, or very large, affecting entire river basins.

The maximum flood that any structure can safely pass is called the 'design flood' and is selected after consideration of economic and hydrologic factors. The design flood is related to the project feature; for example, the spillway design flood may be much higher than the flood control reservoir design flood or the design flood adopted for the temporary coffer dams. A design flood may be arrived by considering the cost of constructing the structure to provide flood control and the flood control benefits arising directly by prevention of damage to structures downstream, disruption communication, loss of life and property, damage to crops and underutilization of land and indirectly, the money saved under insurance and workmen's compensation laws, higher yields from intensive cultivation of protected lands and elimination of losses arising from interruption of business, reduction in diseases resulting from inundation of flood waters. The direct benefits are called tangible benefits and the indirect benefits are called intangible benefits. The design flood is usually selected after making a cost-benefit analysis and exercising engineering judgment. (Hydrology, H.M Raghunah)

2.1.2 Principal types and causes:

2.1.2.1 Areal (rainfall related):

Floods can happen on flat or low-lying areas when the ground is saturated and water either cannot run off or cannot run off quickly enough to stop accumulating. This may be followed by a river flood as water moves away from the floodplain into local rivers and streams.

Floods can also occur if water falls on an impermeable surface, such as concrete, paving or frozen ground, and cannot rapidly dissipate into the ground.

Localized heavy rain from a series of storms moving over the same area can cause areal flash flooding when the rate of rainfall exceeds the drainage capacity of the area. When this occurs on tilled fields, it can result in a muddy flood where sediments are picked up by run off and carried as suspended matter or bed load.

2.1.2.2 Riverine:

River flows may rise to floods levels at different rates, from a few minutes to several weeks, depending on the type of river and the source of the increased flow.

Slow rising floods most commonly occur in large rivers with large catchment areas. The increase in flow may be the result of sustained rainfall, rapid snow melt, monsoons, or tropical cyclones. Localized flooding may be caused or exacerbated by drainage obstructions such as landslides, ice, or debris.

Rapid flooding events, including flash floods, more often occur on smaller rivers, rivers with steep valleys or rivers that flow for much of their length over impermeable terrain. The cause may be localized convective precipitation (intense thunderstorms) or sudden release from an upstream impoundment created behind a dam, landslide, or glacier.

2.1.2.3 Estuarine and coastal:

Flooding in estuaries is commonly caused by a combination of sea tidal surges caused by winds and low barometric pressure, and they may be exacerbated by high upstream river flow.

Coastal areas may be flooded by storm events at sea, resulting in waves over-topping defenses or in severe cases by tsunami or tropical cyclones. A storm surge, from either a tropical cyclone or an extratropical cyclone, falls within this category.

2.1.2.4 Urban flooding:

Urban flooding is the inundation of land or property in a built environment, particularly in more densely populated areas, caused by rainfall overwhelming the capacity of drainage systems, such as storm sewers. Although sometimes triggered by events such as flash flooding or snowmelt, urban flooding is a condition, characterized by its repetitive and systemic impacts on communities, that can happen regardless of whether or not affected communities are located within formally designated floodplains or near any body of water. There are several ways in which stormwater enters properties: backup through sewer pipes, toilets and sinks into buildings; seepage through building walls and floors; the accumulation of water on property and in public rights-of-way; and the overflow from water bodies such as rivers and lakes.

2.1.2.5 Catastrophic:

Catastrophic flooding is usually associated with major infrastructure failures such as the collapse of a dam, but they may also be caused by damage sustained in an earthquake or volcanic eruption.

2.1.3 Effects

2.1.3.1 Primary effects:

The primary effects of flooding include loss of life, damage to buildings and other structures, including bridges, sewerage systems, roadways, and canals.

Floods also frequently damage power transmission and sometimes power generation, which then has knock-on effects caused by the loss of power. This includes loss of drinking water treatment and water supply, which may result in loss of drinking water or severe water contamination. It may also cause the loss of sewage disposal facilities. Lack of clean water combined with human sewage in the flood waters raises the risk of waterborne diseases, which can include typhoid, giardia, cryptosporidium, cholera and many other diseases depending upon the location of the flood.

Damage to roads and transport infrastructure may make it difficult to mobilize aid to those affected or to provide emergency health treatment.

Flood waters typically inundate farm land, making the land unworkable and preventing crops from being planted or harvested, which can lead to shortages of food both for humans and farm animals. Entire harvests for a country can be lost in extreme flood circumstances. Some tree species may not survive prolonged flooding of their root systems.

2.1.3.2 Secondary and long-term effects:

Economic hardship due to a temporary decline in tourism, rebuilding costs, or food shortages leading to price increases is a common after-effect of severe flooding. The impact on those affected may cause psychological damage to those affected, in particular where deaths, serious injuries and loss of property occur.

Urban flooding can lead to chronically wet houses, which are linked to an increase in respiratory problems and other illnesses. Urban flooding also has significant economic implications for affected neighborhoods. In the United States, industry experts estimate that wet basements can lower property values by 10-25 percent and are cited among the top reasons for not purchasing a home. According to the U.S. Federal Emergency Management Agency (FEMA), almost 40 percent of small businesses never reopen their doors following a flooding disaster.

2.1.4 Flood forecasting:

Anticipating floods before they occur allows for precautions to be taken and people to be warned so that they can be prepared in advance for flooding conditions. For example, farmers can remove animals from low-lying areas and utility services can put in place emergency provisions to re-route services if needed. Emergency services can also make provisions to have enough resources available ahead of time to respond to emergencies as they occur.

In order to make the most accurate flood forecasts for waterways, it is best to have a long time-series of historical data that relates stream flows to measured past rainfall events. Coupling this historical information with real-time knowledge about volumetric capacity in catchment areas, such as spare capacity in reservoirs, ground-water levels, and the degree of saturation of area aquifers is also needed in order to make the most accurate flood forecasts.

Radar estimates of rainfall and general weather forecasting techniques are also important components of good flood forecasting. In areas where good quality data is available, the intensity and height of a flood can be predicted with fairly good accuracy and plenty of lead time. The output of a flood forecast is typically a maximum expected water level and the likely time of its arrival at key locations along a water-way, and it also may allow for the computation of the likely statistical return period of a flood. In many developed countries, urban areas at risk of flooding are protected against a 100-year flood - that is a flood that has a probability of around 63% of occurring in any 100 year period of time.

According to the U.S. National Weather Service (NWS) Northeast River Forecast Center (RFC) in Taunton, Massachusetts, a general rule-of-thumb for flood forecasting in urban areas is that it takes at least 1 inch (25 mm) of rainfall in around an hour's time in order to start significant ponding of water on impermeable surfaces. Many NWS RFCs routinely issue Flash Flood Guidance and Headwater Guidance, which indicate the general amount of rainfall that would need to fall in a short period of time in order to cause flash flooding or flooding on larger water basins.

2.1.5 Control:

In many countries around the world, waterways prone to floods are often carefully managed. Defenses such as detention basins, levees, bunds, reservoirs, and weirs are used to prevent waterways from overflowing their banks. When these defenses fail, emergency measures such as sandbags or portable inflatable tubes are often used to try and stem flooding. Coastal flooding has been addressed in portions of Europe and the Americas with coastal defenses, such as sea walls, beach nourishment, and barrier islands.

In the riparian zone near rivers and streams, erosion control measures can be taken to try and slow down or reverse the natural forces that cause many waterways to meander over long periods of time. Flood controls, such as dams, can be built and maintained over time to try and reduce the occurrence and severity of floods as well. In the USA, the U.S. Army Corps of Engineers maintains a network of such flood control dams.

In areas prone to urban flooding, one solution is the repair and expansion of man-made sewer systems and storm-water infrastructure.

Another strategy is to reduce impervious surfaces in streets, parking lots and buildings through natural drainage channels, porous paving, and wetlands (collectively known as green infrastructure or sustainable urban drainage systems [SUDS]). Areas identified as flood-prone can be converted into parks and playgrounds that can tolerate occasional flooding. Ordinances can be adopted to require developers to retain storm-water on site and require buildings to be elevated, protected by floodwalls and levees, or designed to withstand temporary inundation. Property owners can also invest in solutions themselves, such as re-landscaping their property to take the flow of water away from their building and installing rain barrels, sump pumps, and check valves.

2.1.6 Benefits:

Floods (in particular more frequent or smaller floods) can also bring many benefits, such as recharging ground water, making soil more fertile and increasing nutrients in some soils. Flood waters provide much needed water resources in arid and semi-arid regions where precipitation can be very unevenly distributed throughout the year. Freshwater floods particularly play an important role in maintaining ecosystems in river corridors and are a key factor in maintaining floodplain biodiversity. Flooding can spread nutrients to lakes and rivers, which can lead to increased biomass and improved fisheries for a few years.

For some fish species, an inundated floodplain may form a highly suitable location for spawning with few predators and enhanced levels of nutrients or food. Fish, such as the weather fish, make use of floods in order to reach new habitats. Bird populations may also profit from the boost in food production caused by flooding.

Periodic flooding was essential to the well-being of ancient communities along the Tigris-Euphrates Rivers, the Nile River, the Indus River, the Ganges and the Yellow River among others. The viability of hydropower, a renewable source of energy, is also higher in flood prone regions.

2.1.7 Computer modeling:

While flood computer modeling is a fairly recent practice, attempts to understand and manage the mechanisms at work in floodplains have been made for at least six millennia. Recent developments in computational flood modeling have enabled engineers to step away from the tried and tested "hold or break" approach and its tendency to promote overly engineered structures. Various computational flood models have been developed in recent years; either 1D models (flood levels measured in the channel) or 2D models (variable flood depths measured across the extent of a floodplain). HEC-RAS, the Hydraulic Engineering Centre model, is currently among the most popular computer models, if only because it is available free of charge. Other models such as TUFLOW combine 1D and 2D components to derive flood depths across both river channels and the entire floodplain. To date, the focus of computer modeling has primarily been on mapping tidal and fluvial flood events, but the 2007 flood events in the UK have shifted the emphasis there onto the impact of surface water flooding.

In the United States, an integrated approach to real-time hydrologic computer modeling utilizes observed data from the U.S. Geological Survey (USGS), various cooperative observing networks, various automated weather sensors, the NOAA National Operational Hydrologic Remote Sensing Center (NOHRSC), various hydroelectric companies, etc. combined with quantitative precipitation forecasts (QPF) of expected rainfall and/or snow melt to generate daily or as-needed hydrologic forecasts.

2.2 Hydrometry:

2.2.1 General:

Hydrometry means literally water measurement. In the past hydrometric engineers were particularly involved in stream-flow measurements. Today many aspects of water measurements are included.

Hydrometry is defined as "*the measurement of flow in open watercourses, supported or complemented by the measurements of water levels, bed levels and sediment transport.*" (Hydrometry, W. Boiten, 2005)

2.2.2 Water Levels:

2.2.2.1 Purpose:

Water levels may be considered the basis for any river study. Most kinds of measurements – such as discharges – have to be related to river stages. However, in reality the discharge of a river is a better basic information than the water level, and if it could be possible to measure this discharge daily or even several times a day at many places, this would be preferable. Water levels are obtained from gauges, either by direct observation or in recorded form. The data can serve several purposes:

- By plotting gauge readings against time over a hydrological year the hydrograph for a particular gauging station is obtained. Hydrographs of a series of consecutive years are used to determine duration curves, indicating either the probability of occurrence of water levels at the station considered, or, by applying rating curves, indicating the probability of occurrence of discharges.
- Combining gauge readings with discharge values, a stage discharge relation can be determined, resulting in a rating curve for the particular station under consideration.
- From the readings of a number of gauges, observed under steady flow conditions and at various stages, stage relation curves can be derived.
- Apart from use in hydrological studies and for design purposes, the data can be of direct value for other purposes such as, for instance, navigation, flood prediction, water management and waste water disposal. (Hydrometry, W. Boiten, 2005)

2.2.2.2 The Water Level Gauging Station:

The stage of a stream or lake is the height of the water surface above an established datum plane. The water surface elevation referred to some gauge datum is called the gauge height or stage. Stage or gauge height is usually expressed in metres and centimetres.

A record of stage may be obtained by systematic observations of a staff gauge or with an automatic water level recorder. The advantages of the staff gauge are the low initial cost and the ease of installation.

The disadvantages are the need for an observer and the low accuracy. For long term operation the advantages of the recording gauge are far more than those of the staff gauge.

Hence the use of the non-recording gauge as a base gauge is not recommended.

To obtain accurate and reliable stage data, the station gauge and bench-marks must be referred to a fixed datum.

The datum may be a recognized datum plane, such as mean sea level, MSL, or an arbitrary datum plane chosen for convenience. An arbitrary datum plane is selected for the convenience of using relatively low numbers for gauge heights, or to eliminate minus values of gauge heights.

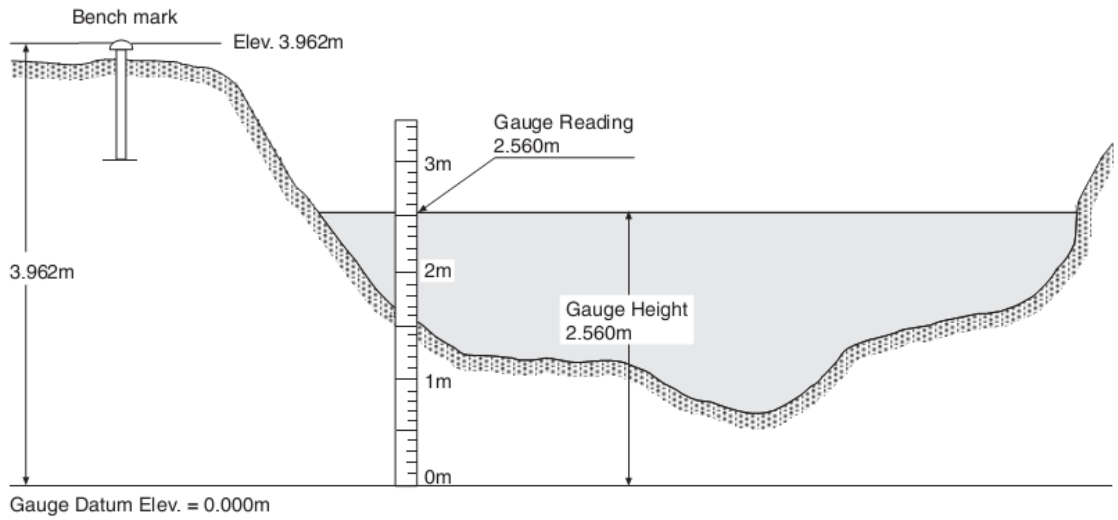
When a weir or flume is used, the gauge datum is usually set at the elevation of zero flow (crest level).

Each gauging station requires at least two bench-marks or reference marks; that is, permanent points of known elevation, that are independent of the gauge structure. The gauge datum is periodically checked by levelling from the bench-marks to the gauges at the station, at least once a year as a routine. A universal datum plane does not exist.

Most countries have their own horizontal datum plane. As a consequence, direct comparison of gauged water levels in neighbouring countries is not always possible.

The national datum plane in the Netherlands is called NAP (= Normaal Amsterdam Peil), which is about mean see level in Amsterdam, years ago.

The Belgium datum plane is 2.34 metres lower. The Danish and French datum planes are 0.14 m and 0.13 m higher than NAP, the German Normall Null is 0.02 m higher than NAP. In many countries sloping datums are used for main river systems: in general SLW and SHW, where SLW = 95% exceedance level (= 5% subceedance). (Hydrometry, W. Boiten, 2005)



Figure(2.1): Definition sketch, gauge datum and gauge reading (*Source Hydromemtry, W.Boiten*).

2.2.2.3 Selection of Gauge Sites:

The network of gauges along a river should be arranged so that water level information at any place along that river can be gathered by means of inter-polation of the gauge records. Gauges should be placed where a change occurs in the water level gradient, the discharge, or in general, in the character of the river. At the same time the river sections between two gauges should not be too long. The selection of a gauge site and the installation of a gauge requires thorough knowledge about the hydraulic and morphological phenomena in a river. It also requires knowledge about ship movements, etc. For the gauge site and installation of the gauge, the following requirements should be met:

- the site should be accessible to a gauge reader, who can easily read the gauge, and (unless an automatic recording gauge is used) a gauge reader should be available at all time;
- even during extreme low water levels the gauge should still be in open connection with the river and not be dried;
- even during extreme high water levels the gauge should not be overflowed;
- damaging of the gauge by ships, floating debris or slidings off the river bank should be prevented;
- the location of the gauge site must be so that no influence is felt of backwater effects due to confluences etc. Preferably the location should be chosen just upstream of a control section so as to avoid the influence of local scour and sedimentation;
- one or two leveled bench-marks should be near, for a regular check of the gauge datum.

The required frequency of gauge readings depends on the fluctuation of the levels. When these fluctuations are small, one reading a day can be sufficient, but if great fluctuations occur, three or more readings a day are required. At places with very rapid changes in water levels, hourly readings should be taken, but it is preferable that continuous readings be taken by an automatic gauge.

Gauges on lakes and reservoirs are normally located near the outlet, but upstream of the zone where an increase in velocity causes a drawdown in water level, which is the case with weirs and flumes.

Gauges in large bodies of water should also be located so, that the effects of strong winds do not generate to much uncertainty. (Hydrometry, W. Boiten, 2005)

2.2.2.4 Different Types of Gauges:

2.2.2.4.1 Overview of water level gauges:

Most water level gauging stations are equipped with a sensor or gauge and a recorder. In many cases the water level is measured in a stilling well, thus eliminating strong oscillations. (Hydrometry, W. Boiten, 2005)

2.2.2.4.2 The staff gauge:

The staff gauge is the simplest type and very popular. It usually consists of a graduated gauge plate, resistant to corrosion: cast aluminum or enameled steel. This plate is fixed vertically onto a stable structure, such as a pile, a bridge pier or a wall. Sometimes the gauge is placed in an inclined position, for instance upon a sloping river bank. Inclined staff gauges are not exposed to damage by ships or floating material. (Hydrometry, W. Boiten, 2005)

They are provided with a graduation making allowance for the slope m. In wavy conditions however, accurate reading of the inclined gauge is difficult. Another disadvantage is that adjustment of the gauge is usually not very easy.

The staff gauge is the only gauge which can be read directly, at any time and without preceding manipulations.

Where the range of water levels exceeds the capacity of a single gauge, additional gauges may be installed on the line of the cross section normal to the direction of flow. The scales on such a series of stepped gauges should have adequate overlap. (Hydrometry, W. Boiten, 2005)

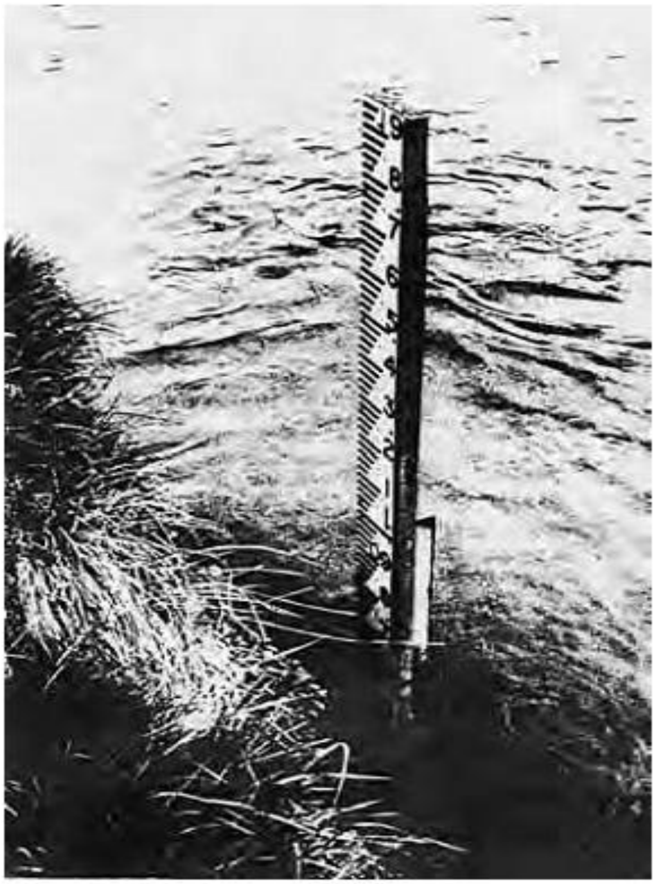


Figure 2.2 : Shows a vertical staff gauge fixed onto a pile along a river.

Figure 2.3 Gives the so-called E type vertical Staff Gauge.(*Source Hydrometry, W.Boiten)*

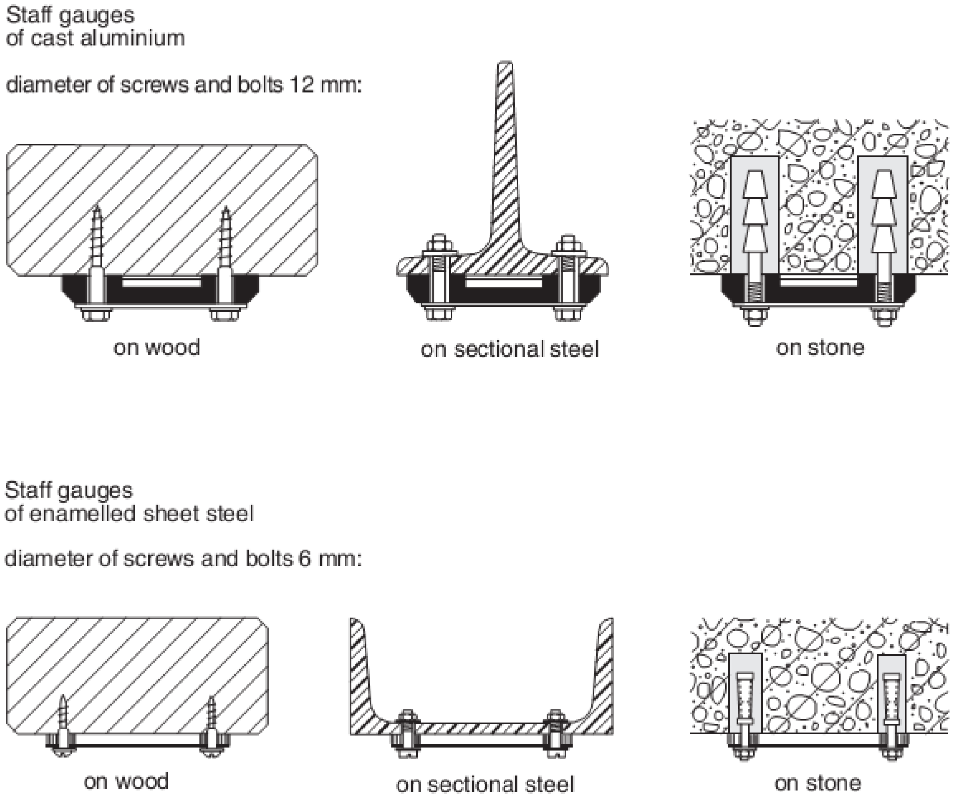


Figure 2.4 Shows examples of mounting Staff Gauges.

4.2.2.4.3 The float operated gauge:

The principle of the automatic recording float operated gauge is as follows (Figure 2.5). A float inside a stilling well, which is connected with the river by an intake pipe, is moved up or down by the water level. (Fluctuations caused by short waves are almost eliminated.)

The movement of the float is transmitted by the float wheel to a mechanism which records these movements on paper (mechanically), or in a fixed memory (electronically). It is strongly recommended to operate the float in a stilling well.

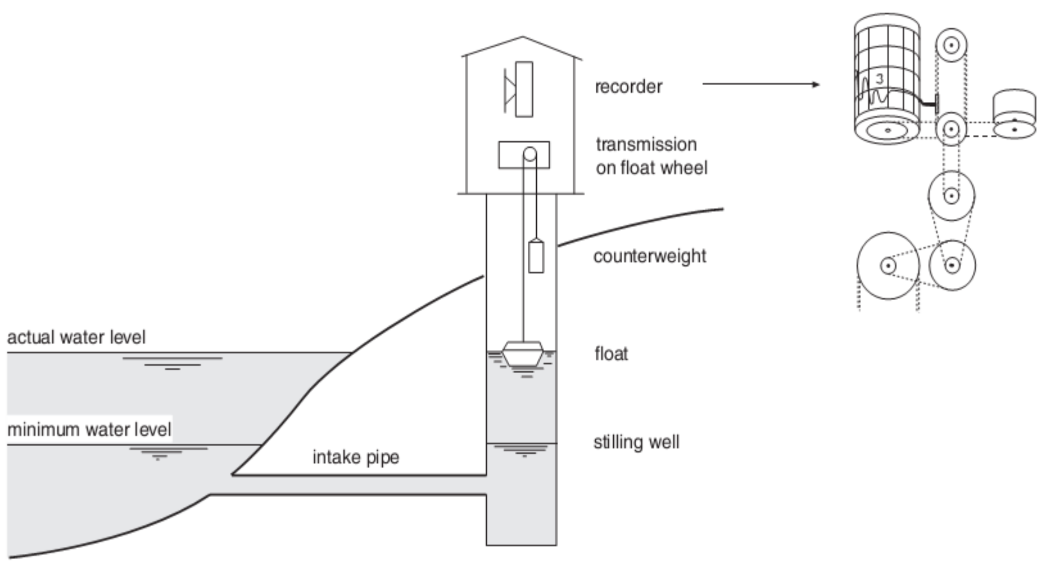


Figure (2.5): The Float operated recorder.

The functional requirements are:

a) A float operated gauge should permit measurement of stage to be made at all levels from the lowest to the highest level expected.

b) Float and counterweight dimensions and the quality of the elements of the mechanical device for remote indication should be selected so that there is a sufficiently high accuracy.

c) The float should be made of durable corrosion resistant and antifouling material. It should be leakproof and function in a truly vertical direction. Its density should not change significantly.

d) The float should float properly and the tape or wire should have no twists or kinks.(Hydrometry, W. Boiten, 2005)

2.2.2.4.4 Pressure transducers:

Pressure transducers are also referred to as pressure sensors, pressure probes and pressure transmitters.

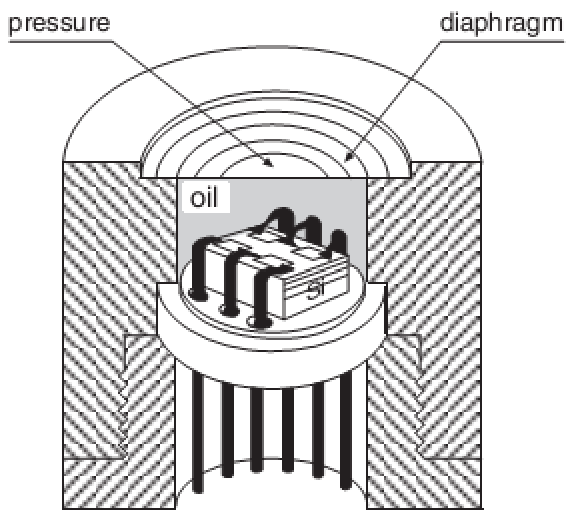
The water level is measured as a hydrostatic pressure and transformed into an electrical signal, in most cases with a semi-conductor sensor (piezo resistive pressure transducer). The measured value corresponds to the actual water level above the sensor.

In some cases errors are generated due to the varying weight of the water-column (salinity, temperature and sediment content) and fluctuations of the atmospheric pressure.

Pressure transducers are used for the measurement of water levels in open water, as well as for the continuous recording of groundwater levels.

– Open water:

Pressure transducers are mostly installed in stilling wells. These wells can be attached to an existing wall, or placed in the embankment in the same way as for float-operated recorders. The stilling well may not be closed totally airtight, so as to maintain the atmospheric pressure in it.



Figure(2.6) Piezo resistive pressure transducer.

– Ground water:

The transducer is installed in a pipe or a borehole of small diameter and to large depths (up to 200 m). In this way it is also possible to measure water levels in not permanently water carrying riverbeds, like wadis.

Preferably pressure transducers are compensating for changes in the atmosphere pressure. However, air vented cables (combined with the signal cable) are expensive. In case of submerged self recording systems, there is no cable at all and air pressure needs to be measured separately. (Hydrometry, W. Boiten, 2005)

Some characteristics of pressure transducers:

– Diameter 10 mm < D < 45 mm.

– Measuring range: from 0 – 1.25 m to 0 – 40 m and more.

– Accuracy 0.1% of the full range. It is recommended to check the output of the transducer, using a reference plate connected with the stilling well.

– Power supply: Accu 12 V or small batteries.

– Output: volts, or milli-amperes (0 – 20 mA in many cases)

Most pressure transducers are equipped with a data logger, having a storage capacity of at least 10.000 measured values as a standard. Other transducers are equipped with analog paper-written drum recorders or with digital water level indicators (display).

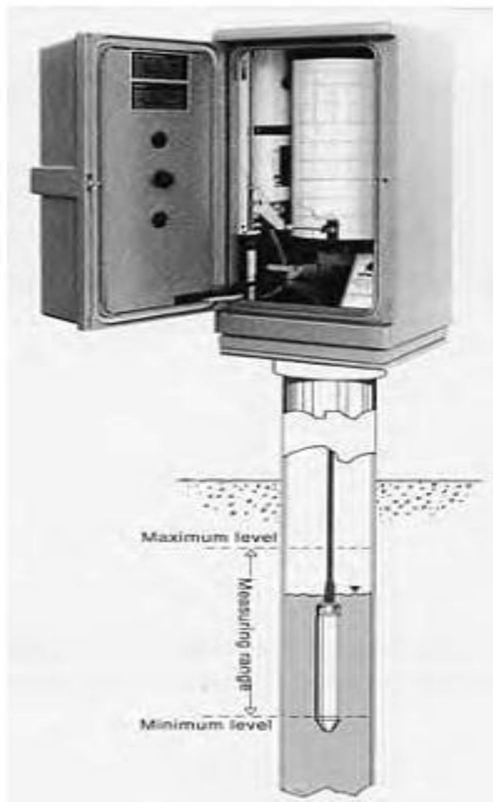


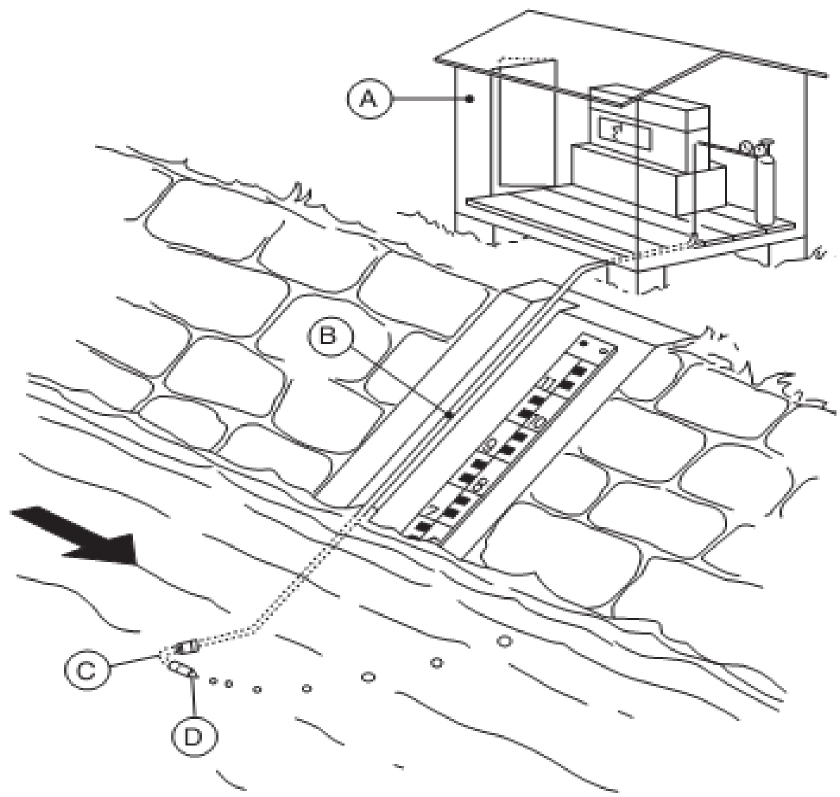
Figure (2.7) Vertical water level gauge (drum recorder) with pressure transducer

2.2.2.4.5 Bubble gauges:

The bubble gauge – also called Pneumatic Water Level Gauge – is a pressure actuated system, based on the measurement of the pressure which is needed to produce bubbles through the bubble orifice against the water pressure. The gauges are primarily used at sites where it would be expensive or difficult to install a float operated recorder or a pressure transducer.

From a pressurized gas cylinder or a small compressor, gas (nitrogen or compressed air) flows over a pressure reducer (for instance a proportioning valve) through the measuring pipe. At the end of this pipe, gas bubbles constantly flow out through the bubble orifice in the water. The pressure in the measuring tube corresponds to the static pressure of the water column above the orifice. This static pressure of the water column is transmitted to a manometer.

Short time variations, like wind waves, should not in any way affect the instrument. Therefore, a damping device is installed at the end of the measuring tube.



A: Shelter
B: Protective pipe with measuring tube
C: Protective pipe laid right-angled (bubble orifice in direction of flow)
D: Bubble orifice

Figure (2.8) Bubble gauge at a riverbank (*Source: Hydrometry, W.Boiten*).

Some characteristics of bubble gauges:

- measuring range: from 0-8.00 m to 0-30.00 m
- accuracy: error less than 1 cm over the total range
- power supply: 12V battery (sufficient for 1 year) or connection to 220/110V supply
- output depends on equipment combination
- use of potentiometer 0-20 mA or volts or a shaft encoder, for connection of data transmission systems
- connection with an analog strip-chart recorder (several months) or a drum recorder (several days).

(Hydrometry, W. Boiten, 2005)

2.2.2.4.6 Ultrasonic sensor:

Ultrasonic (or acoustic) sensors are used for continuous non-contact level measurements in open channels. The sensor emits ultrasonic pulses at a certain frequency. The inaudible sound waves are reflected by the water surface and are received by the sensor. The roundtrip time – i.e the time elapsed between transmitting and receiving the echo – is measured electronically and appears as an output signal proportional to level.

In most of the ultrasonic sensors a built-in temperature probe compensates automatically roundtrip time errors which are mainly caused by the temperature coefficient of the speed of sound in air.

Average propagation speeds of sound waves in air and in water:

C_{air} = 330 to 340 m/s

C_{water} = 1450 to 1480 m/s

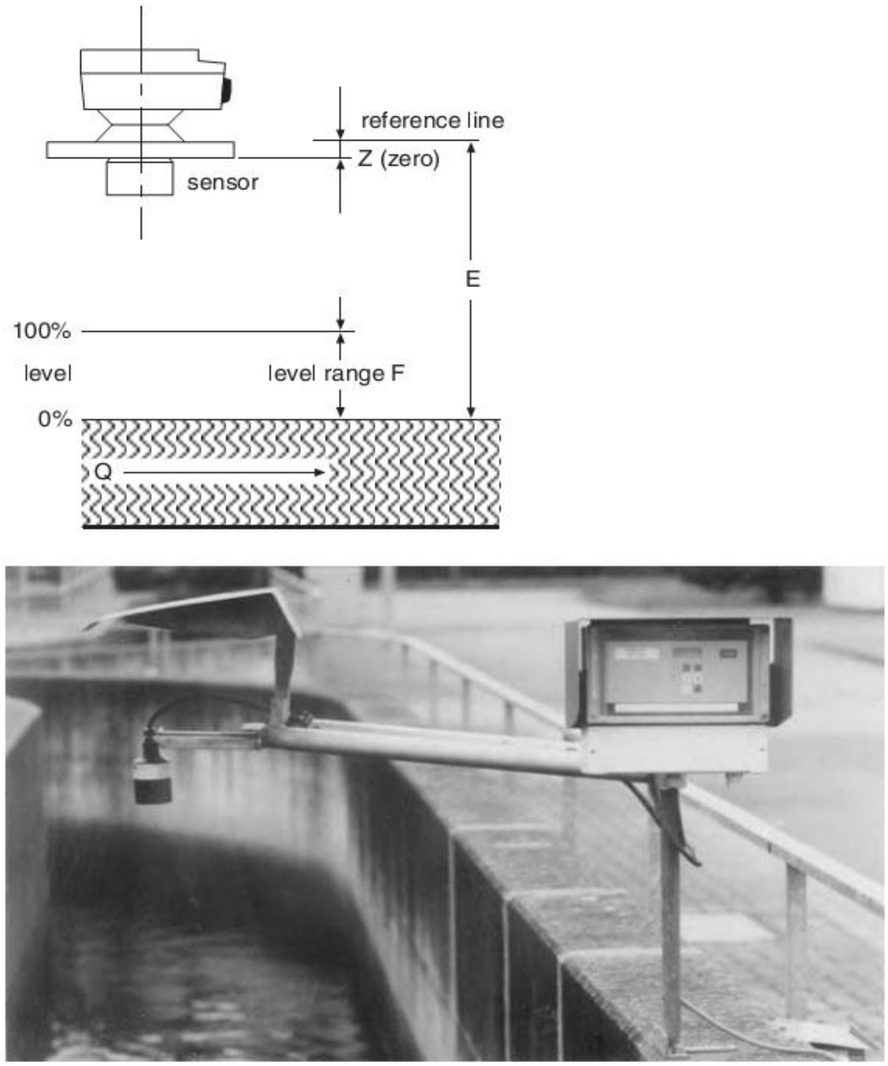


Figure (2.9): Ultrasonic sensor

For open channel flow measurement, the proportional level signal can be modified by a lineariser programmed with the head discharge relation of the particular structure (weir of flume).

The accuracy of the level measurement depends on the measuring conditions (stable water level) and on the mounting conditions (factor E/F). The latter indicates the meter accuracy, which is a function of the E/F factor. (Hydrometry, W. Boiten, 2005)

2.2.2.4.7 Peak level indicators:

After floods a marking line is printed along the banks of rivers and lakes by floating debris, and sediments on the walls of houses, bridges, quaywalls, etc. which have been subject to high water levels.

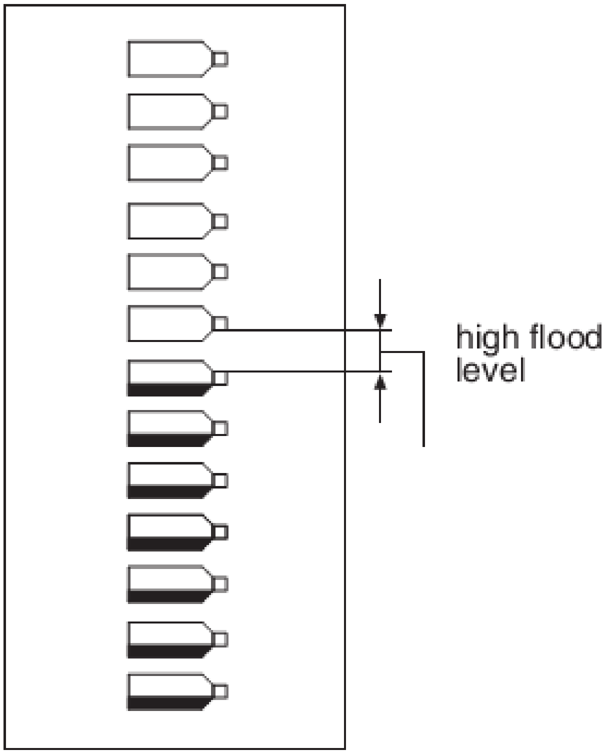


Figure (2.10): Bottle gauge.

Apart from a flood survey after an extreme water level, use can be made of special peak level indicators.

Flood crest gauges are very simple instruments to determine top stages during floods at remote or unaccessible locations along rivers.

They are read during an inspection after floods. The variety of types depends on local conditions and possibilities and on the inventiveness of the hydrometrist.

2.3 Introduction to Frequency Analysis:

In examining water supply schemes, river regulation and the construction of dams, it is essential to form a reliable estimate of the great flood discharge and level that a structure is likely to encounter.

The design of spillways can be critical to the integrity of the dam and safety of those in the path of flood water, should it fail catastrophically.

Frequency analysis isn't limited to river flow data, but can be used with any series of recorded observation. For example, frequency analysis could be conducted on precipitation data to determine the probability of a storm of given magnitude. This storm could be then be used with a rainfall-runoff model to determine the probable magnitude of flood. (see chapter4, section 4.2)

2.4 Previous Studies:

2.4.1 Flood Frequency Analysis – Dinder River, Sudan (*by Assad Yahia Shams Aldeen and Abu Obieda Babiker Ahmed, Hydraulic Research Station)* (see Appendix B).

2.4.1.1 General:

Dinder river is on of tributaries of the Blue Nile and joins the Blue Nile in the reach between Wad – Medani and Sennar. The head stream of Dinder rises on the Ethiopian Platue about 30Km west of Lake Tana. It is a seasonal River being dry during the period January to June during which it reduces to pools separated by dry sandy beds. The total length of Dinder river is about 750Km while the Effective Catchment area is 1600 Km^2 (Shahin, 1985).

The data used in this study, which is the daily annual discharge of Dinder river, is available to the Water Resources directorate, Ministry of irrigation and Water Resources. The Name of gauging station at which the data was measured is Giwasi.

2.4.1.2 Objectives:

The report deals with flood frequency analysis for Dinder river, in which the main task is to estimate a flood magnitude (Q) corresponding to any return period (T) of occurrence for Dinder river i.e establish the Q –T relationship.

2.4.1.3 Methodology of Analysis:

In the study, the annual maximum method is used. The series of annual daily maximum flood peaks is obtained by taking the maximum daily discharge occurring within the year.

Two statistical models are used to model the annual maximum series of Dinder river. The first one is the extreme value distribution type(I), based upon the theory of extreme value developed by Gumbel(1941). The second distribution is the lognormal distribution.

Chapter Three

Study Area Overview,
Hydrologic Statistics & Frequency Analysis

3. Study Area Overview

3.1 General description of The Gash River:

3.1.1 Background:

3.1.1.1 Early History:

The earliest history of the reach is to be found in a legend of the Halenga quoted by Mr. B. Kennedy – Cooke, M.C.

This tribe which now occupies that part to the delta just to the north of Kassala, came originally from south of Asmara. They followed the Gash until it came to "a rocky barrier which held the river up. (Presumably Tessenei gap). This they split by pouring boiling water on the rocks. The river burst through and they followed it down past Jebel Kassala and onward".

Passing from legend to fairly well authenticated fact, the earliest record of the reach is that of the building of a dam across the river. It is alleged to have been built between July and September, 1840, so the Gash must have been late, or running very spasmodically. Its construction was ordered by Ahmed Pasha Abu Udan with the idea of bringing the Hadendowa in the Gash delta to heel by depriving them of water for their crops and well centers and causing them to pay a heavy sum for its release. The idea is said to have been put into his head by Mohammed Eila, the sheikh of the Halenga, who had old scores to settle with the Hadendowa; certainly the Halenga helped extensively in its construction. The site was upstream from Kassala where the river comes closest to the Jebel. The trunks of dom palms were sunk in the bed of the river and laced together; earth was then piled behind and reinforced with baskets and palm leaf mats provided by the Halenga. The dam was 1613 meters long and was 5 meters wide across the top, the earthwork slopes being 1:1.

A detachment of troops was told off to guard the dam but failed to prevent its being cut by a raiding party of two hundred Hadendowa one night during a thunderstorm. It was said to be holding three meters of water when breached and the whole construction collapsed rapidly and completely.

The intention of the dam is said to have been to divert the waters of the Gash down Khor Kwenti and eventually into the Atbara. It should be noted here that there is no water-shed between the Gash and the Atbara, so the idea was not so far – fetched as may at first appear. It is significant that the first attempt at major irrigation work was a canal dug in 1841 at Khor Kwenti to water the Kwenti – Kalahote area of the Western Gash; possibly it was intended that the dam would divert water for this scheme in addition to its main purpose of curbing the Hadendowa. In any case the canal was not dependent on the weir and continued to function for about thirty years before it silted up. There is no further evidence of any major irrigation scheme until after the Mahdia. (Swan, 1956)

3.1.1.2 The Kassala Reach before 1930:

The earliest actual surveys available of the Gash in this area date from 1926 when a series of cross-sections of the river were taken south of Kassala weir. They show the river varying in width from 170m to 330m and up to just over 1m deep. As a comparison the river in the 1950's had a fairly constant width of 700m for three kilometers upstream of the weir site.

The first accurate, leveled survey of the reach that were made after the 1926 flood. The Kassala Cotton Company's Annual Flood report for

1926 says: “The Gash at Kassala is yearly widening its bed and the matter of river training works will have to be considered. The recent flood has eaten away the east and west river banks at places and has don considerable damage to native quarters on both side of the river”.

The damage refered to above took place downstream, but the widening tendency could be noted as beginning just downstream the weir where agricultural plots were being eroded on the west side.

The flood of 1927 was a high one and erosion was increased. Just south of Gharb el Gash canal head the west bank was eaten away on a frontage of 500m and to a maximum depth of 150m. Various spurs, both of brushwood and earthworks, were built out into the river before the flood to try and guide the river down a central channel; they were soon washed away.

Further training spurs, built before the 1928 flood, were effective in keeping the Gash in one channel about 400m in certain kilmoterage. These spurs, were mainly downstream of Kassala but the flood report for 1928 pointed out that it would be necessary to extend the training system upstream as far as the weir to ensure stability.

The 1929 flood was the greatest that has ever been recorded in that period; major changes were to be expected. The weir was short –circuited and the system of training spurs largely swept away. A survey made after the flood showed very deep erosion on the west bank just downstream the weir as far as the off-take of Gharb el Gash canal. This erosion carried away valuable agricultural land. The bed of the river was very wide, the deepest channels being, in general, at the extreme east and west sides. It was realized that plain earth or sand spurs were incapable of giving the necessary protection during heavy floods.

There was practically no further erosion downstream the weir site during the much smaller flood of 1930. The Gash bed at Kassala was nearly a kilometer wide and the river ran at medium discharges. In three main channels, one to the east, one to the west and one, the largest, in the middle. Further heavy erosion occurred south of the weir, presumably on the west side. Unfortunately no survey for 1930 remains.

It does not appear from the surveys available of this period that the Gash bed was rising in the area. Indeed the reverse is the case after the great 1929 flood. However, evidence was too slight and over too short a period for any definite conclusions to be drawn. (Swan, 1956)

2.1.1.3 Gharb El Gash Canal:

This channel was dug in 1913. It took off the Gash about 2.5Km below the weir on the west side and watered an area of land behind the village of Gharb el Gash.

No record is available of the early history of this canal, nor is it known whether it had head-works or not. It was very dependent on the vagaries of the Gash and is reported, in a good year, to have watered up to 7,000 feddans. There is almost no trace of it now though the line of it can be detected along the way from Gharb el Gash village to Ankora head. It watered an area which would far better have been irrigated from wells. (Swan, 1956)

2.1.1.4 Kassala weir:

After the re-occupation, the first major irrigation work, installed in 1905 by the Egyptian Irrigation Service, was a diversion weir across the Gash some three kilometers upstream of Kassala, with a canal called the Khatmia canal taking off the right hand side of the river to irrigate rather inferior land lying between Kassala and the Jebels behind it. The weir was designed to pass a discharge of $300m^3/sec$ –a grave underestimation of possible Gash flows. The dry bed level of the Gash was 122.00 at the time the weir crest was set at 123.00. the design as a core-wall 1m thick built 2m below river bed level; rubble slopes of 3:1 upstream and 10:1 downstream were built on either side, the downstream slope ending in lines of blocks of $1m^3$ each. The length of the weir was 100m. unfortunately no drawing of this weir could be found, but from the above description it appears that apart from the underestimation of probable discharge in the river, the designer did not realize the extent to which the bed of the Gash degrades during a spate.

The 1906 flood was a high one, though not an abnormal one as thought at the time. A spate, estimated at between 500 and $550m^3/sec$, wrecked the weir on 26th July. With anything more than 40cm of water over the crest a standing wave formed over the footing blocks; when this depth exceeded 80cm the wave formed downstream of the blocks. Actually a depth over the crest of as much as 1.28m occurred, the bed downstream degraded and footing blocks moved, followed in quick succession by downstream pitching and core-wall.

The weir was entirely destroyed but unfortunately much of its material appeared to have remained on site and the river over it was described as a cataract. This possibly helped, by preventing normal bed erosion, in the next disaster three days later upstream the weir site and breached the Jebel Tie bank.

Efforts were made to hold up the flood, but these efforts were unavailing. The flood broke through behind Kassala town and swept through it in a stream over 100m wide and 60cm deep; it was estimated to be flowing at at least a velocity of 1 m/sec.

All irrigation works were more or less destroyed and damage in the town was considerable though no lives were lost. The Gash discharge at the time of this second spate was estimated at 750m^3/sec.

When the weir was reconstructed after the 1906 flood, it was lengthened by 20m and its crest level reduced from 123.00 to 122.25. Fortunately a continuous wall was built at the toe of the downstream rubble slope; the foundation level of this wall was 121.00 and it rose to 122.00. Thus the crest of the weir was only 25cm above the footing and it looked as though it would offer no appreciable obstruction to the flow of the river. Clearly the extent to which the river degraded its bed while in spate was still not appreciated. As usual, after the 1907 flood the dry bed level appeared at much same level as before the flood, at 122.00, but the footing blocks had disappeared leaving the footing wall exposed.

When the sand was cleared away the blocks were found lying any how, some having sunk as low as 120.00. The footing wall saved the day; had it failed, and it must have been very near failing, the weir would again have been wrecked. After the 1907 flood, the footings were extended and stepped down to 121.50 (with foundation at 120.50) and a continuous wall, 50cm high, was built along their downstream faces. As thus built, no further trouble was experienced.

The original area watered in the Kassala irrigation scheme is not known but by 1910 it was 1920 feddans. The middle third of the weir was removed in 1921 to scour the Gash bed and reduce the danger of Gash taking a westerly course down Khor Kwenti. The weir was finally out flanked in 1929.

No trace was left of the weir except its left hand side which was incorporated as a part of a spur in the Kassala training works. The site, however, remains as the zero of kilometrages measured along the Gash. The possibility of reinstalling Kassala weir has never been seriously considered because a major off-take is not suitable in that area; on the right bank the land is poor and on the left bank ground-water supplies are so abundant that the best line for future development in irrigation lies in tube-wells. The danger of a westerly break-away is still with us. (Swan, 1956)

3.1.2 Catchment:

a) Upper reaches:

The catchment covers an area of approximately 21,000 square kilometers and it is delineated between latitudes 36.5 – 39.5 East and longitudes 14.0 – 15.5 North, shared with Eretria, Ethiopia and Sudan (Figure 1). Elevation in the catchment varies between 1100 m and 2000 m. In its upper reaches the Gash is known as the river Mareb. It starts from Eretria, about 20Km south of Asmara. From here to Sudan frontier the river basin is long and relatively narrows (Swan, 1956).

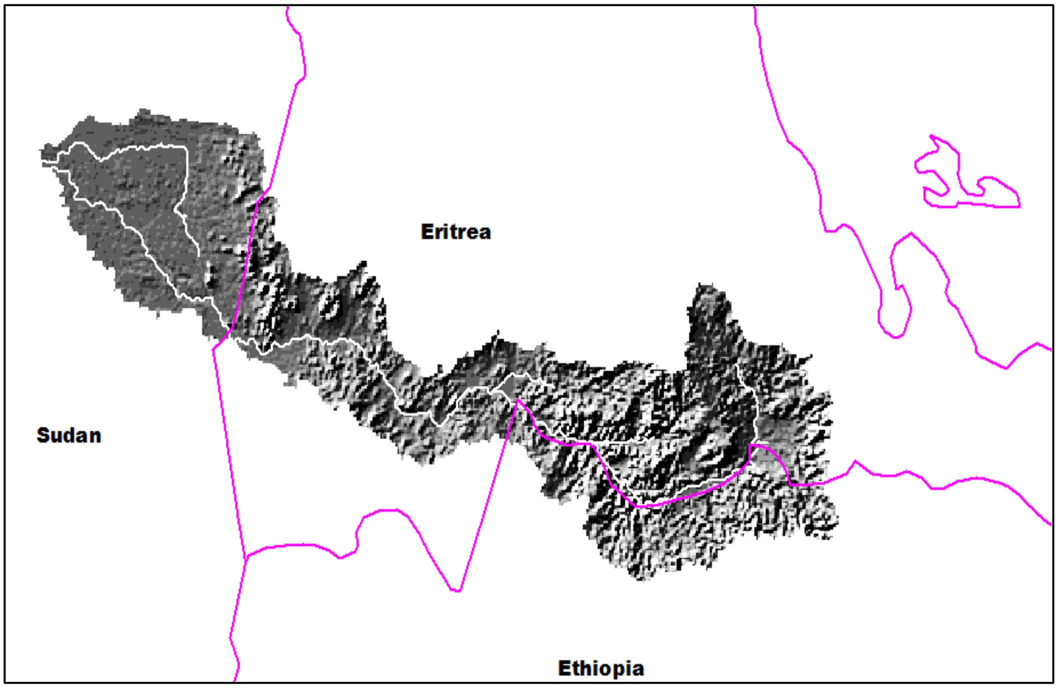


Figure (3.1): Location of Gash river catchment

b) Lower reaches:

After passing the narrow rocky gap at Tessenei, a wide shallow stream with a sandy bed and extensive flood plains flows slightly north-west for some 20 Km; and then runs north for about 5 Km before crossing the Sudanese frontier close to Jebel Gulsa. Thereafter, it runs to North-

northwest for about 20 Km to the southern bastion of Jebel Kassala. Here it is joined by a major tributary from the east, Khor Abu Alga. From the frontier to Jebel Kassala the Gash varies in width between 100 m and 800 m, with an average about 300 m. The slope of the bed is relatively constant throughout at 1.3 m/km. (Swan, 1956)

3.1.3 Rainfall:

Rainfall is sparse over the whole of the Gash delta and decreases sharply from south to north, as is shown by the following ten-year average rainfall figures for the period 1943 to 1952:- (Swan,1956)

Table(3.1): Rainfall Figures for the period 1943 to 1952(*source: Swan,1956)*

Kassala	269 mm
Mekali	185 mm
Aroma	188 mm
Degein	165 mm
Tendelai	165 mm
Metateib	161 mm
Hadaliya	142 mm

Normally the rain falls in violent storms of short duration and, owing to its intensity, produces considerable run-off.

In the southern half of the delta the local inhabitants often hold up the run-off by building low banks and the water so impounded percolates into the soil sufficiently to produce crops of millet. In the neighborhood of

Aroma and to the north this is only possible in particularly favored spots or in years of exceptionally heavy rainfall.

3.1.4 Climate:

Like the central part of the Sudan the climate is conditioned by two main air streams, the dry northerlies in winter and the moist southerlies in summer. These give rise to a climate of a tropical continental type. Winter weather is very pleasant and differs slightly from the weather further west by its greater humidity which is shown up by quite heavy dews from December to March. High summer in April, May and June is intensely hot and during that season the Gash delta is particularly prone to “haboobs” or violent dust – storms.

These storms are connected with frontal conditions; they can be seen approaching some time before they break, looking like a great wave rearing up, dark brown and menacing. Once the “haboob” has broken visibility is reduced to a few metres and it becomes quite dark; a howling wind carries a thick dust, so fine that it can penetrate anywhere.

The storm may last for two or three hours; it is usually accompanied by a perceptible drop in temperature. High summer is not a pleasant time in the Gash delta. In July, August and September, that is during the rains and the Gash floods, the weather is far cooler and is usually quite pleasant.

The rain falls in a few intense storms and the rest of the time it is usually sunny and fine. At this time the delta transforms itself completely; dust is a thing of the past, new grass shows up and soon the country takes on a fresh greenness which seems almost incredible after months of aridity. The drawback to this season is that it brings out myriads of flying and buzzing and creeping things, particularly at night. The rains are followed by another hot spell, rather shorter than in central plains of the Sudan, without dust but with insects, until the north wind sweeps both heat and creatures away and winter is back again. (Swan, 1956)

3.1.5 Soil:

There are two main soils to be found in the Gash delta. The best, known locally as "lebad", is a rich silt which is highly permeable. Moisture penetration, after normal flooding, may extend more than five meters below the surface of the ground. The other soil, known as "badobe", is a heavy, cracking clay, a form of cotton soil. It absorbs moisture more slowly (after initial wetting has sealed the cracks) and normally produces a smaller crop of cotton. All gradations between "lebad" and "badobe" occur. Nowhere is any appreciable quantity of material to be found with a particle size greater than 0.1 mm. (Swan, 1956)

3.1.6 Sedimentation in Gash River:

The Gash River although is seasonal, however, it has high impact on its valley. Gash River seems to carry an exceedingly large charge of sediment, Abdalla A. A. (1994). Recent measurements and estimation of the sediment load showed enormous variations where the highest value exceeded 70,000 ppm. This is compared to the highest recorded sediment concentration of 9,722 ppm during the fifties, Abdalla A. A. (1994). The sharp increase in the sediment (8 times) shows the serious effect of the recent drought years (during the eighties) on the Gash River catchment area. At the same time, it has a negative impact on the magnitude of the flood and its damaging capability.

Assessment of sediment in Gash River course indicates that a huge amount of sediment has been deposited along the bed with variation in its thickness (1 to 3 meters), especially near and under the Kassala Bridge, which acts as a bottleneck obstructing the flow. Some observers attribute the last flood impact on Kassala town to the obstruction of this bridge to the river flow. The Author of this paper agrees that existence of the bridge with its low deck is one of the reasons aggravating the situation during the flood of 2003. (Bashar, 2005)

3.1.7 Population:

The indigenous population of the Gash delta is Beja, of whom the main tribe in this area is the Hadendowa. They are primarily pastoral people and nomads; they still live in matting tents and move from one grazing ground to another as one season is succeeded by the next. The men are nearly always armed with sword and knife. They have taken to cotton cultivation unenthusiastically, if at all. Though they have first priority to the land very few of them, even yet, have settled down in permanent villages, preferring to leave the cultivation to which they are entitled to be worked by agents and hired labour.

Other Beja tribes to be found in the delta are small sections of the Amarar and Beni Amer; near Kassala as the Halenga who have taken more kindly to agriculture than the others. From the point of the Agriculturist or engineer it is probably just as well that there is a large number of West Africans, largely Haussa and Fellata, settled in the delta.

These people were originally transient piligrims on their way to and from Mecca, but many of them have now settled in villages scattered about the delta. They are cheerful and colorful people, excellent workers whom the main reliance must be placed for running of cotton scheme, both for engineering and agricultural labor.

One other tribe to be seen in the delta which is worthy of mention is the Rashaida. These people are Arabs, one of the latest tribes to enter the Sudan. They are light skinned, pastoral people living in tents. They water their animals at the well centers but play little part in the cotton growing, though they sometimes engage in the picking. They also are colorful people. (Swan, 1956)

3.2 Gash River Hydrology:

3.2.1 General:

The River originates from the Eritrean Highlands and the Ethiopian Plateau. It is a seasonal and flashy river with high variation in flow during the wet season of July, August and September.

The minimum annual total discharge of The Gash River is 140 million m^3, which was recorded in 1921, while the maximum annual discharge is 1430 million m^3 recorded in 1983. The maximum annual flow is almost 10 times the minimum one, indicating the high variability of the flows in Gash River. The annual average flow is 680 million m^3 (Bashar, 2005).

The aquifer in the area is unconfined; the water level in the aquifer rises during the River Gash flow period and declines when the river is dry. Kassala drinking water depends on it. The percolation amount is about 250 million cubic meters.(Meki, 2008).

3.2.2 Gash River Behavior:

In Eritrea 20 km south Asmara, the Gash River which is called Mareb has a catchment area ranges from 30 to 90 Km in width and approximately

250 Km in length. The first 175Km is perennial while it becomes ephemeral before it enters Sudan. The catchment area is estimated to be about 21,000 km2. The river characteristics in this area are sandy bed with varying width and well-defined banks (one to two meters high). The bed slope of the river is varying with average of 2m per km. Before entering Sudan, the river passes a narrow rocky course while the flood plain is populated with dense palm trees. From morphological point of view, the Gash River is considered braided river. Thus, it becomes wider and shallower. In such rivers, the flow takes many directions resulting in unstable river that changes its course.

There is no reliable rainfall data available in the catchment.

Generally, in the Kassala area the climate is a tropic continental type. It is governed by the dry north wind in the winter and moist south wind in the summer.

The temperature ranges from 16 to 42 c^o and relative humidity ranges from 40 to 60 %. Geologically the area is Precambrian basement complex, while Kassala and Mokram hills represent the main out crops of the basement complex. Furthermore, the other formation is the clays of the plain, consisting of a maximum of 18meters thickness on the east and west sides of the river. The alluvial deposits formed by the Gash River are the third type of formation with a thickness ranging from 17 to 34 meters.

The discharge measurements of the Gash River, which go back to 1907, are showing high fluctuation. (Bashar, 2005)

3.3 Flooding in El-Gash River:

3.3.1 General:

The River Gash is considered a source of frequent terror to the inhabitants on both sides of its banks especially the Kassala town (the capital of the Kassala State). Kassala town (more than ½ million population) had been attacked by several damaging high floods from the Gash River most of which were very severe. Some of the records are 1975, 1983, 1988, 1993, 1998 and 2003. (Bashar, 2005)

The most damaging one occurred in year 2003, where almost half the city was washed out. The total loss, which constitutes Buildings, Roads, Drinking Water Plants, Agriculture….etc., is estimated to amount to 150 Million US$. (Bashar, 2005)

3.3.2 Causes of Flooding in Kassala Town:

A study carried out by the Gash Protection Committee 2003, concluded that flooding of Kassala town could strictly happen if the embankment failed and attributed the failure to:

- Increase in water levels of the river.
- Cavitations.
- Direct scour of the embankment by water currents and waves.

Rise in water levels can be caused by increase in the river discharge attributable to climate change, which lead to land use change, and eventually to high floods and sediment transport. Human activities on the riverbanks led to control the river from natural expansion in its flood plain.

Dam failure in the upper catchment area also contributes to rise in water levels of the river.

Another prime cause of rise in water levels in the river is the reduction of its channel carrying capacity caused by the structures obstructing flows such as bridges, spurs and dykes. In addition, some morphological changes such as rise of riverbed and change in slope have its contribution in river water level rise.

Cavitations happen due human and animal activities or weakening of the embankment. The direct scour of embankment can happen due to the steep nature of the Gash River which leads to high-speed flows bearing in mind that the embankments are sandy and have no resistance to such high waves.

In the 2003 flood, there are many factors worked collectively to produce the catastrophic flood of 2003. Some of these factors have cumulative nature while the others have instantaneous effects. The river flow was tremendously increased following high rainfall over the upper catchment. Around Kassala the existence of the two bridges constituted obstruction to the flow. The channel carrying capacity under the bridges is estimated as 400 m^3/s compared to an estimated flood wave of 700 m^3/s. This lack of carrying capacity led to fast water levels on the back of the bridges to overtop the embankment, scour it from the back and demolish it with aid of the failure of the spurs that allowed the water current to directly strike the embankment. The difference in pressure on the two sides of the embankment contributed greatly in the failure of the embankment. (Bashar, 2005)

3.3.3 Early Warning System:

As stated above the catchment of the Gash River is completely outside Sudan and that the floods wave needs only four hours from the border to reach Kassala and may be only two hours from the first gauging station inside Sudan (Algira). This makes the early warning systems that depend on real time data ineffective. This fact also paves the way for warning systems that depend on remotely sensed data and calibrated CCD-RFE models. No early warning system is in place so that people get prepared to the flood, in addition to the bad drainage system in the town the accumulated Gash water is not drained at the required speed. (Bashar, 2005)

3.3.4 History of the Protection Works in the Gash River:

The protection works in Gash River around Kassala town goes back to early last century. A protection embankment is constructed on both sides of the riverbanks on a length of 10km to protect Kassala town. Also spurs at 500 m spacing were constructed to combat lateral movement of the river flows and force it to transport its sediment downstream the protected zone. These spurs and dykes were very efficient in keeping the river channel in the Kassala reach since 1931 as compared to other unprotected reaches.

As mentioned above, the main purpose of these set of structures is to direct the flow towards the center of the river course and hence force it to scour the bed i.e. direct the flow away from the bank. This will allow the water stored between the spurs to slow down and deposit its sediment, which at the end strengthen the banks of the river. The disadvantages of this system:

i. It is expensive.

ii. It requires very high engineering experience. Especially when determining the length, direction and spacing between the spurs.

iii. It requires regular maintenance.

In case there is any damage or breakage in any of the spurs the whole system will be affected and may totally collapse. The later might cause negative impact on the entire system of the river course. Therefore, riverbank failure may follow, leading to what has happened in Gash River during the flooding of 2003.

The sequence in which these spurs and dykes were constructed is as follows:

Seven spurs were constructed downstream the Kassala bridges in 1931 to 1937 some of which were completely silted. Another seven spurs were constructed during the period 1976-1984 with some spurs silted. Seventeen spurs were constructed in period 1984-1998.

Thus, 31 spurs were in place however, Kassala town was hit severely by the 2003 flood. (Bashar, 2005)

3.3.5 Living with Floods:

There are many ways to reduce flooding risks. Choice among these methods depends on the cost, the community's vulnerability to flooding, the extent of existing development and available funding. There are two types of risk reduction methods commonly adopted worldwide which can be applied to the Gash River namely structural measures and nonstructural measures.(Bashar,2005)

i- Structural measures: involve constructing physical works designed to contain floods and limit erosion from the river-such as stopbanks, rock linings, revetments, gabions, groynes and vegetation buffers. This was practiced in the protection works existing in the Gash River. This category also includes building proofing against floods. (Bashar, 2005)

ii- Nonstructural measures: include land use planning regulations and voluntary actions, and steps that could be taken to prepare for floods. These measures aim to keep people out of the flood prone areas and improve the community's ability to respond to and recover from floods. Nonstructural measures also include early warning system complete with communication system and emergency preparedness. Non-structural measures enable a community to be more resilient to flooding through flood awareness, preparation and sensible land use.(Bashar,2005)

Structural and non-structural measures are equally important management approaches for the Gash River. The ultimate should consider both (structural and non-structural) in an integrated way to reduce flood hazard effects. (Bashar, 2005).

3.4 Data Collection:

3.4.1 Gash River Monitoring Program:

The regular Gash river monitoring program has been implemented in period from 1970 until the year of 2004. During the period, the monitoring of the river was the responsibility of Kassala Research office (KRO). Following the flood disaster of July 2003, the responsibility of the river monitoring has been transferred to Gash River Training Unit (GRTU), which was established in January 2004 (Saied, 2010).

3.4.2 Gauging stations along Gash River:

The Gash River has six gauging stations along the river (Figure 2) where the water level and velocity (by float method) are regularly measured. These stations are: New Geera gauging station, Old Geera gauging station, Kilo 1.5 gauging station, Kassala Bridge gauging station, Fota gauging station and Salamalekum gauging station. The regular water level measurement started in 1970, except for the new Geera that was established in 2007. (Hydrological Data Analysis in Gash River, Yasir Mohamed Omer, Jan 2013**)**

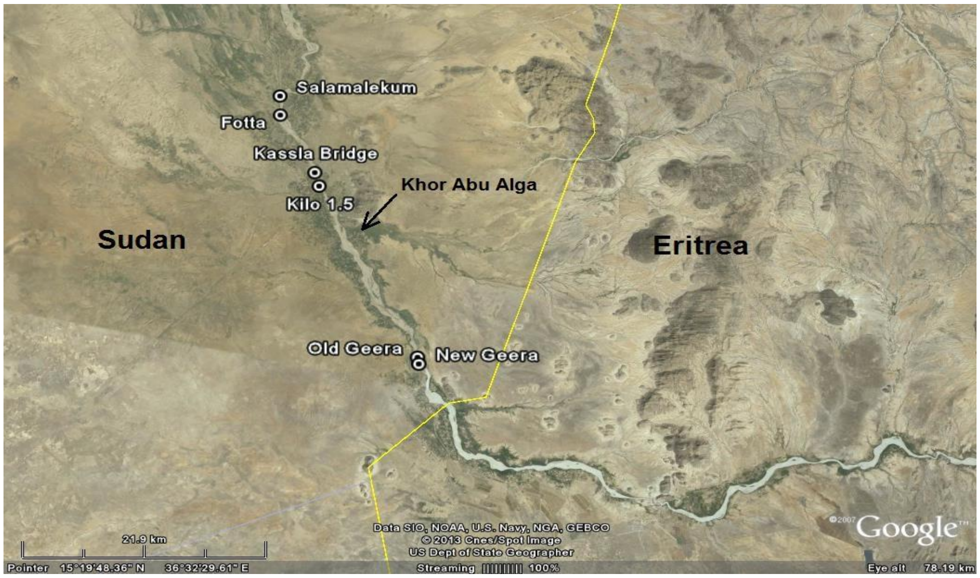


Figure (3.2): Location of Gauging Stations along the Gash

1) New Geera Gauging station:

This gauging station was built in 2007 near the right bank of Gash River to measure the water level and velocity by float method (see figure). New Geera is located at 15.262N latitude and 36.4783E longitude. It is the first upstream gauging station on The Gash River inside Sudan, about five

Kilometers from Sudanese Eritrean border. The velocity measurement near the left bank has recently been added.

New Geera station is located just after the river bend and the cross-section at this site is relatively stable about 260m width, but there is a little scouring in the outer bend of the river. (Hydrological Data Analysis in Gash River, Yasir Mohamed Omer, Jan 2013**)**

Figure (3.3): Inclined gauge staff placed upon the right bank of New Geera gauging station

2) Old Geera Gauging station:

It is located at 15.2682N, 36.4773E, about 700m downstream of New Geera. The measurement in old Geera station started from 1970 on the right bank of Gash River, but over time the gauging station suffered from extreme erosion at the left bank of River (Figure 4), also the width of river section in this site is very wide, about 450m, thus the Old Geera readings

are considered to be less reliable compared with the New Geera gauging station. (Hydrological Data Analysis in Gash River,
Yasir Mohamed Omer, Jan 2013**)**

Figure (3.4): Erosion on the left bank of Old Geera gauging Station

3) Kassala Bridge gauging station:

It is located in Kassala town at 15.4477N latitude and 36.389E longitude, about 22 Km downstream of the Old Geera. In the past Kassala Bridge gauging station was only used for water level measurements, but since 2005 both velocity methods float and current-meter have been used. The study is mainly focused on this station because it is the only possible location for current-meter and measurement of bed level during floods. The cross-section at this site is relatively narrow compared with the previous upstream stations, about 120m (See Figure 5).
(Yasir Mohamed Omer, Jan 2013**)**

Figure (3.5): Right bank of Gash River at Kassala Bridge station

4) Salamalekum gauging station:

Salamalekum station is located at latitude and longitude of 15.5249N, 36.358E, respectively, about 9 Km downstream Kassala bridge. This station is used for measurement of water level and velocity by float method near the left bank of river. The measurement near the right bank has recently been added. The width of river cross-section at this station is just about 95m. (Yasir Mohamed Omer, Jan 2013**)**

3.5 Description of Study Data:

This study was based on annual maximum water levels data of the river Gash measured at the Kassala Bridge gauging station. Data record period is from 1970 to 2004, as shown in Table (3.1), during which as mentioned in Section(3.3) the river reportedly flooded over 6 times.

Table(3.2): Annual Maximum series For Gash

N	Years	Water Level	N	Years	Water Level
1	1970	504.97	17	1986	506.8
2	1971	505	18	1987	505.9
3	1972	505.7	19	1988	506.45
4	1973	505.3	20	1989	505.4
5	1974	505.95	21	1990	505.9
6	1975	505.22	22	1991	505.9
7	1976	505.98	23	1992	506.1
8	1977	505.58	24	1996	505.95
9	1978	505.68	25	1997	506.4
10	1979	505.1	26	1998	506.3
11	1980	505.79	27	1999	506.1
12	1981	505.7	28	2000	506.45
13	1982	505.91	29	2001	506.75
14	1983	506.2	30	2002	506.3
15	1984	505.9	31	2003	507
16	1985	505.7	32	2004	506.7

3.6 Hydrologic Statistics:

3.6.1 General:

Hydrologic processes evolve in space and time in a manner that is partly predictable, or deterministic and partly random. Such a process is called a *stochastic process*. In some cases, the random variability of the process is so large compared to its deterministic variability that the hydrologist is justified in treating the process as purely random. As such, the value one observation of the process is not correlated with the values of adjacent observations, and the statistical properties of all observations are the same. (Ven Te Chow and others, Applied Hydrology)

When there is no correlation between adjacent observations, the output of a hydrologic system is treated as stochastic, space-independent, and time independent. This type of treatment is appropriate for observations of extreme hydrologic events, such as floods or droughts, and for hydrologic data averaged over long time intervals, such as annual precipitation. Statistical methods are based on mathematical principles that describe the random variation of a set of observations of a process, and they focus attention on the observations themselves rather than on the physical processes which produced them. Statistics is a science of description, not causality.(Ven Te Chow and others, Applied Hydrology)

3.6.2 Elements of Statistics:

1. *Population and sample*. Observed value of *x* (variate) for a finite number of years is known as 'sample' of *x*. Say annual flood peaks or annual rainfall for 75 years gives the sample; on the other hand, population consists of the values of annual flood peaks from time immemorial to eternity. The 'population parameters' can be estimated by means of parameters obtained from the sample, known as 'sample parameters'. Each phenomena is characterized by a certain value, which varies in time and space. This characterization is called 'variable' and its particular value is a 'variate'.

If the value of one variate is independent of any other, the variable in question is a 'random variable' the hydrological processes are mostly random and hence the respective variables are equally random. All the time series (as well as other series) may be characterized by statistical parameters.

2. *Central tendency*. Three types of parameters are generally used to represent measures of central tendency.

(a) *Expected value*. This value of the random variable *x* is given by:

$$\mu = \int_{-\infty}^{\infty} x.f(x).dx \qquad (3.1)$$

This population parameter can be estimated by the sample parameters as:-

(i) Arithmetic mean

$$\bar{x} = \frac{\Sigma x}{n} \tag{3.2}$$

For grouped data

$$\bar{x} = \frac{\Sigma f(x)}{n} \tag{3.2 a}$$

(ii) Geometric mean

$$\bar{x}_{\mathrm{g}} = (x_1 . x_2 . x_3 . \dots\ x_n)^{1/n} \tag{3.3}$$

(iii) Harmonic mean

$$\bar{x}_h = \frac{n}{\Sigma \frac{1}{x}} \tag{3.4}$$

Where n= the size of the sample, say the number of annual flood peaks.

For any set of variates $\bar{x} > \bar{x}_{\mathrm{g}} > \bar{x}_h$

In most cases, the arithmetic mean gives the best estimate of the expected value, *i.e.*, $= \bar{x}$.

(*b*) *Median*. The value of the variate such that half of the variates are below it and the other half above it, is called the *median* of the series, *i.e.*, it is the value of the variate having a 50% cumulative frequency.

(*c*) *Mode*. The value of the variate having the highest frequency is called the mode. For unimodel curves, which are moderately skewed, the empirical relation is

$$\text{Mean} - \text{mode} = 3(\text{mean} - \text{median}) \tag{3.5}$$

3. *Variability*. The measures of variability or dispersion of a probability distribution curve are given by the following parameters.

(*a*) *Mean deviation*. The mean of the absolute deviations of values from their mean is called mean deviation (MD).

$$\text{MD} = \frac{\sum|x - \bar{x}|}{n} \tag{3.6}$$

(*b*) *Standard deviation*. It is the square root of the mean-squared deviation of the variates from their mean, and the standard deviation for the population (σ_p) is given by

$$\sigma_p = \sqrt{\frac{\sum(x - \mu)^2}{n}} \tag{3.7}$$

And this is estimated from the standard deviation for the sample (σ) given by

$$\sigma = \sqrt{\frac{\sum(x - \bar{x})^2}{n - 1}} \tag{3.8}$$

$$= \sqrt{\frac{\sum x^2 - \bar{x}\sum x}{n - 1}} \tag{3.8 a}$$

$$= \sqrt{\frac{\sum x^2 - \frac{(\sum x)^2}{n}}{n - 1}} \tag{3.8 b}$$

$$= \sqrt{\left(\overline{x^2} - \bar{x}^2\right)\frac{n}{n - 1}} \tag{3.8 c}$$

where $\overline{x^2} = \frac{\sum x^2}{n}$

for grouped data,

$$\sigma = \sqrt{\frac{\sum f.(x-\bar{x})^2}{n-1}} \tag{3.8 d}$$

The dispersion about the mean is measured by the standard deviation, which is also called the root mean square of the departures from the mean.

(*c*) *Variance*. The square of the standard deviation is called variance, *i.e.*, given by σ_p^2 for the population and σ^2 for the sample.

(*d*) *Range*. The range (*R*) denotes the difference between the largest and smallest values of the sample and is given by Hurst (1951, 1956) and Klemes (1974) as

$$\mathrm{R} = \sigma({}^{n}/_{2})^{k} \quad , \quad 0.5 < k < 1 \tag{3.9}$$

$$= 1.25\sigma_p\sqrt{n} \tag{3.9 a}$$

for a random normally distributed time series.

(*e*) *Coefficient of variation*. The standard deviation divided by the mean is called the coefficient of variance (C_v) and is given by

$$C_v = \frac{\sigma_p}{\mu} \approx \frac{\sigma}{x} \tag{3.10}$$

4. *Skewness (asymmetry)*. The lack of symmetry of a distribution is called skewness or asymmetry. The population skewness (α) is given

$$\alpha = \frac{\sum(x-\mu)^3}{n} \tag{3.11}$$

This is estimated from the sample skewness (α) given by

$$\alpha = \frac{\sum(x-\bar{x})^3}{n-1} \tag{3.12}$$

For grouped data

$$\alpha = \frac{\sum f(x-\bar{x})^3}{n-1} \tag{3.12 a}$$

The degree of skewness of the distribution is usually measured by the 'coefficient of skewness' (C_s) and is given by

$$C_s = \frac{\alpha}{\sigma_p^3} \approx \frac{a}{\sigma^3} \tag{3.13}$$

Another measure of skewness often used in practice is Pearson's skewness (S_k) given by

$$S_k = \frac{\mu - mode}{\sigma_p} \approx \frac{\bar{x} - mode}{\sigma} \tag{3.14}$$

From Eq. (5.5)

$$S_k = \frac{3(\bar{x} - median)}{\sigma} \tag{3.14 a}$$

3.6.3 Fitting a Probability Distribution:

3.6.3.1 General:

A probability distribution is a function representing the probability of occurrence of a random variable. By fitting a distribution to a set of hydrologic data, a great deal of probabilistic information in the sample can be compactly summarized in the function and its associated parameters. Fitting distributions can be accomplished by the *method of moments* or the *method of maximum likelihood*. (Ven Te Chow and others, Applied Hydrology)

3.6.3.2 Method of Moments:

The method of moments was first developed by Karl Pearson in 1902. He considered that good estimates of the parameters of a probability distribution are those for which moments of the probability density function about the origin are equal to the corresponding moments of the sample data. As shown in Fig.(4.1), if the data values are each assigned a hypothetical "mass" equal to their relative frequency of occurrence $(1/n)$ and it is imagined that this system of masses is rotated about the origin $x = 0$, then the first moment of each observation x_i about the origin is the product of its moment arm x_i and its mass $(1/n)$, and the sum of these moments over all the data is

$$\sum_{i=1}^{n} \frac{x_i}{n} = \frac{1}{n} \sum_{i=1}^{n} x_i = \bar{x} \qquad (3.15)$$

the sample mean. This is equivalent to the centroid of a body. The corresponding centroid of the probability density function is

$$\mu = \int_{-\infty}^{\infty} x\, f(x)\, dx \qquad (3.16)$$

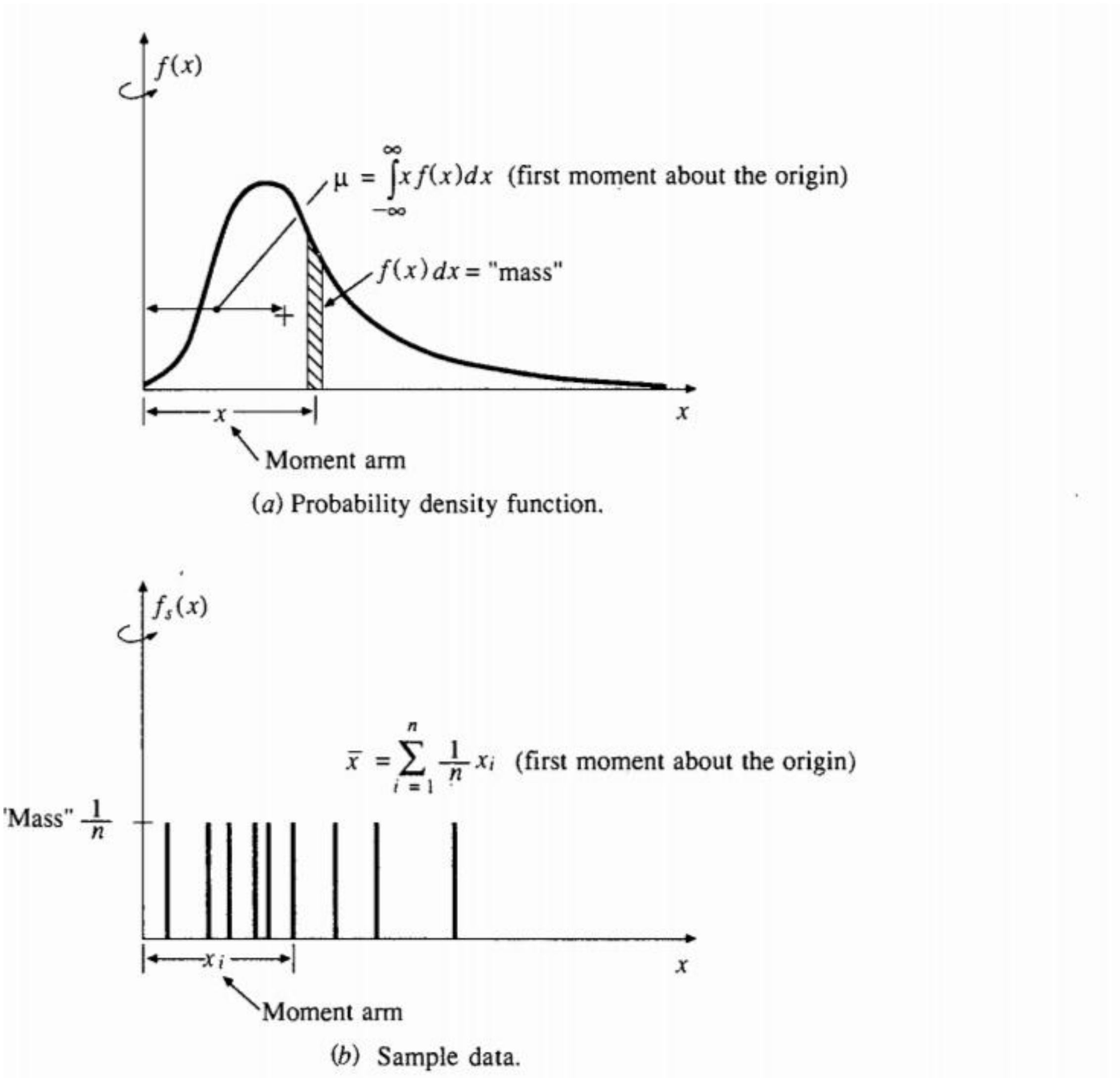


Figure (3.6): The method of moments selects values for the parameters of the probability density function so that its moments are equal to those of sample data.

Likewise, the second and third moments of the probability distribution can be set equal to their sample values to determine the values of parameters of the probability distribution. Pearson originally considered only moments about the origin, but later it became customary to use the variance as the second central moment, $\sigma^2 = E[(x-\mu)^2]$, and the coefficient of skewness as the standardized third central moment, $\gamma = E\,[(x-\mu)^3]/\sigma^3$, to determine second and third parameters of distribution if required.(Ven Te Chow and others, Applied Hydrology)

3.6.3.3 Method of Maximum Likelihood:

The method of maximum likelihood was developed by R. A. Fisher (1922). He reasoned that the best value of a parameter of a probability distribution should be that value which maximizes the likelihood or joint probability of occurrence of the observed sample. Suppose that the sample space is divided into intervals of length *dx* and that the a sample of independent and identically distributed observations $(x_1, x_2, \ldots, x_n)$ is taken. The value of the probability density for $X = xi$ is $f(x_i)$, and the probability that the random variable will occur in the interval including x_i is $f(x_i)\,dx$. Since the observations are independent, their joint probability of occurrence is given from $P(A \cap B) = P(A)P(B)$ as the product $(x_1)dxf(x_2)dx \ldots f(x_n)dx \; = \; [\prod_{i=1}^{n} f(x_i)]dx^n$, and since the interval size *dx* is fixed, maximizing the joint probability of the observed sample is equivalent to maximizing the *likelihood function*

$$L = \prod_{i=1}^{n} f(x_i) \tag{3.17}$$

Because many probability density functions are expo-nential, it is sometimes more convenient to work with the log-likelihood function

$$\ln L = \sum_{i=1}^{n} \ln[f(x_i)] \tag{3.17}$$

The method of maximum likelihood is the most theoretically correct method of fitting probability distributions to data in the sense that it produces the most *efficient* parameter estimates – those which estimate the population parameters with the least average error. But, for some probability distributions, there is no analytical solution for all parameters in terms of sample statistics, and the log-likelihood function must then be numerically maximized, which may be quite difficult. In general, the method of moments is easier to apply than the method of maximum likelihood and is more suitable for practical hydrologic analysis. (Ven Te Chow and others, Applied Hydrology).

3.6.3.4 Testing the Goodness of Fit:

The goodness of fit of a probability distribution can be tested by comparing the theoretical and sample values of the relative frequency or the cumulative frequency function. In the case of relative frequency function, the χ^2 test is used. The sample value of the relative frequency of interval i is, from $f_s(x_i) = n_i/n$; the theoretical value is $\mathrm{p}(x_i) = F(x_i) - F(x_{i-1})$. The χ^2 *test statistic* χ_c^2 is given by

$$\chi_c^2 = \sum_{i=1}^{m} \frac{n[f_s(x_i) - p(x_i)]^2}{p(x_i)} \tag{3.18}$$

where m is the number of intervals. It may be noted that $nf_s(x_i) = n$, the observed number of occurrences in interval i, and $np(x_i)$ is the corresponding expected number of occurrences in interval i; so the calculation of Eq.(4.18) is a matter of squaring the difference between the observed and expected numbers of occurrences, dividing by expected number of occurrences in the interval, and summing the result over all intervals.

To describe the χ^2test, the χ^2 probability distribution must be defined. A χ^2 distribution with ν *degrees of freedom* is the distribution for the sum of squares of ν independent standard normal random variables z_i; this sum is the random variable

$$\chi_\nu^2 = \sum_{i=1}^{\nu} z_i^2 \qquad (3.19)$$

The χ^2 distribution function is tabulated in many statistics texts (e.g. Haan,1977). In the χ^2 test, $\nu = m - p - 1$, where m is the number of intervals as before, and p is the number of parameters used in fitting the proposed distribution. A *confidence level* is chosen for the test; it is often expressed as $1 - \alpha$, where α is termed the *significance level*. A typical value for the confidence level is 95 percent. The *null hypothesis* for the test is that the proposed probability distribution fits the data adequately. This hypothesis is rejected (i.e., the fit is deemed inadequate) if the value of χ_c^2 is larger than a limiting value, $\chi_{\nu,1-\alpha}^2$, determined from the χ^2 distribution with ν degrees of freedom as the value having cumulative probability $1 - \alpha$. (Ven Te Chow and others, Applied Hydrology).

3.6.4 Probability Distributions for Hydrologic Variables:

3.6.4.1 General:

Observations of hydrologic variables follow different distributions. Since most hydrologic events are represented by continuous random variables, their density functions denote the probability distribution of the magnitudes. Some of the frequently used density functions used in hydrologic analysis are Normal distribution, Gamma distribution, lognormal distribution, Extreme value distribution and others.

3.6.4.2 Normal Distribution:

The normal distribution arises from the *central limit theorem*, which states that if a sequence of random variables X_i are independently and identically distributed with mean μ and variance σ^2, then the distribution of the sum of *n* such random variable, $Y = \sum_{i=1}^{n} X_i$, tends towards the normal distribution with mean $n\mu$ and variance $n\sigma^2$ as n becomes large. The important point is that this is true no matter what the probability distribution function of *X* is. So, for example, the probability distribution of the sample mean $\bar{x} = 1/n \sum_{i=1}^{n} x_i$ can be approximated as normal with mean μ and variance $(1/n)^2 n\sigma^2 = \sigma^2/n$ no matter what the distribution of x is. Hydrologic variables, such as annual precipitation, calculated as the sum of the effects of many independent events tend to follow the normal distribution. The main limitations of the normal distribution for describing hydrologic variables are that it varies over a continuous range$[-\infty, \infty]$, while most hydrologic variables are nonnegative, and that it is symmetric about the mean, while hydrologic data tend to be skewed. (Ven Te Chow and others, Applied Hydrology).

The density function of normal probability distribution is given by

$$f(x) = \frac{1}{\sigma_p\sqrt{2\pi}} exp\left(-\frac{(x-\mu)^2}{2\sigma_p^2}\right) \qquad (3.20)$$
$$-\infty < x < \infty$$

where σ_p and μ are the two parameters, which affect the distribution,

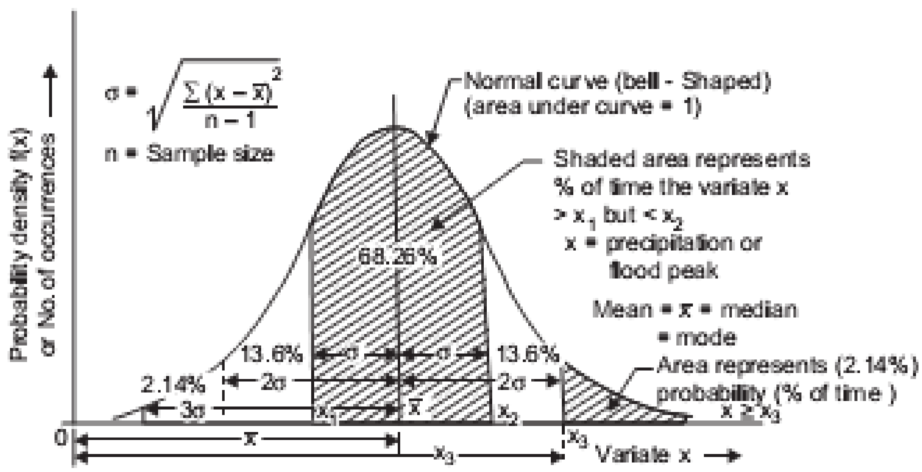


Figure (3.7): Normal Distribution Curve

In this distribution, the mean, mode and median are the same and the area under the curve is unity.

3.6.4.3 Lognormal Distribution:

If the random variable $Y = \log X$ is normally distributed, then X is said to be lognormally distributed. Chow(1954) reasoned that this distribution is applicable to hydrologic variables formed as the products of other variables since if $X = X_1X_2X_3 \ldots X_n$, then $Y = \log X = \sum_{i=1}^{n} \log X_i = \sum_{i=1}^{n} Y_i$, which tends to the normal distribution for large *n* provided that the

X_i are independent and identically distributed. The lognormal distribution has been found to describe the distribution of hydraulic conductivity in a porous medium (Freeze, 1975), the distribution of raindrop size in storm, and other hydrologic variables. The lognormal distribution has the advantages over the normal distribution that it is bounded $(X > 0)$ and that the log transformation tends to reduce the positive skewness commonly found in hydrologic data, because taking logarithms reduces large numbers proportionately more than it does small numbers. Some limitations of the lognormal distribution are that it has only two parameters and that it requires the logarithms of the data to be symmetric about their mean.

The density function in this distribution is given by

$$f(x) = \frac{1}{\sigma_y\sqrt{2\pi}} exp\left(-\frac{(y-\mu_y)^2}{2\sigma_y}\right) \tag{3.21}$$

where $= \ln x$, x=variate, μ_y=mean of y, σ_y= standard deviation of y. This is a skew distribution of unlimited range in both directions. Chow has derived the statistical parameters for x as follows

$$\mu = exp(\mu_y + \sigma_y^2/2)\,,\ \sigma = \mu\ \sqrt{exp\sigma_y^2 - 1} \tag{3.21 a}$$

$$C_v = \sqrt{exp\sigma_y^2 - 1}\,,\ C_s = 3C_v + C_v^3 \tag{3.21 b}$$

3.6.4.4 Exponential Distribution:

Some sequences of hydrologic events, such as the occurrence of precipitation, may be considered *Poisson processes*, in which events occur instantaneously and independently on a time horizon, or a long line. The time between such events, or *interarrival time*, is described by the exponential whose parameter λ is the mean rate of occurrence of the events. The exponential distribution is used to describe the interarrival times of random shocks to hydrologic systems, such as slugs of polluted runoff entering streams as rainfall washes the pollutants off the land surface. The advantage of the exponential distribution is that it is easy to estimate λ from observed data and the exponential distribution lends itself well to theoretical studies, such as a probability model for the linear reservoir ($\lambda = 1/k$, where k is the storage constant in the linear reservoir). Its disadvantage is that it requires the occurrence of each event to be completely independent of its neighbors, which may not be a valid assumption for the process under study – for example, the arrival of a front may generate many showers of rain – and this has led investigators to study various forms of *compound Poisson processes*, in which λ is considered a random variable instead of a constant(Kavvas and Delleur,1981; Waymire and Gupta, 1981). (Ven Te Chow and others, Applied Hydrology).

The density function in this distribution is given by

$$f(x) = \lambda e^{-\lambda x} \tag{3.22}$$

where the distribution range is $(x \geq 0)$, and the parameter $\lambda = \frac{1}{\bar{x}}$.

3.6.4.5 Gamma Distribution:

The time taken for a number β of events to occur in a Poisson process is described by the gamma distribution, which is the distribution of a sum of β independent and identical exponentially distributed random variables. The gamma distribution has a smoothly varying form like the typical probability density function. it is useful for describing skewed hydrologic variables without the need for log transformation. It has been applied to describe the distribution of depth of precipitation in storms, for example. The gamma distribution involves the *gamma function* $\Gamma(\beta)$, which is given by $\Gamma(\beta) = (\beta - 1)! = (\beta - 1)(\beta - 2) \ldots 3 \cdot 2 \cdot 1$ for positive integer β, and in general by

$$\Gamma(\beta) = \int_0^\infty u^{\beta-1} e^{-u} du \qquad (3.23)$$

(Abramowitz and Stegun, 1965). The two-parameter gamma distribution (parameters β and λ) has a lower bound at zero, which is a disadvantage for application to hydrologic variables that have a lower bound larger than zero. (Ven Te Chow and others, Applied Hydrology).

The density function of this distribution is given by

$$f(x) = \frac{\lambda^\beta x^{\beta-1} e^{-\lambda x}}{\Gamma(\beta)} \qquad (3.24)$$

where Γ=gamma function, the Range is $x \geq 0$, and the parameters

$$\lambda = \frac{\bar{x}}{s_x^2} \quad , \ \beta = \frac{\bar{x}^2}{s_x^2} = \frac{1}{CV^2} \qquad (3.25)$$

Or,

$$f(x) = \frac{x^a e^{-x/b}}{a! b^{a+1}}, \ \text{for } 0 < x < \infty \qquad (3.26)$$

$$= 0, \qquad \text{elsewhere}$$

Where *a* and *b* are the two parameters, which affect the distribution (Figure 4.3), change of the parameter *b* merely changes the scale of the two axes.

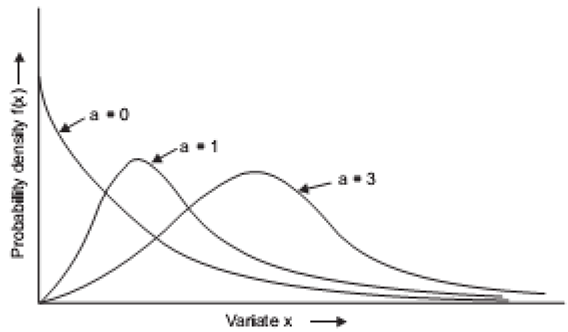


Fig (3.8): Gamma Distribution

Here $\mu = b(a+1)\ and\ \sigma_p^2 = b^2(a+1)$

3.6.4.6 Pearson Type III Distribution:

The Pearson Type III distribution, also called the *three-parameter gamma distribution*, introduces a third parameter, the lower bound ϵ, so that by the method of moments, three sample moments (the mean, the standard deviation, and the coefficient of skewness) can be transformed into the three parameters $\lambda, \beta,$ and ϵ of the probability distribution. This is a very flexible distribution, assuming a number of different shapes as $\lambda, \beta,$ and ϵ vary (Bobee and Robitaille,1977). (Ven Te Chow and others, Applied Hydrology).

The Pearson system of distributions includes seven types; they are all solutions for *f(x)* in an equation of the form

$$\frac{d[f(x)]}{dx} = \frac{f(x)(x-d)}{C_0+C_1x+C_2x^2} \qquad (3.27)$$

where *d* is the *mode* of the distribution (the value of *x* for which *f(x)* is a maximum) and C_0,C_1, and C_2 are coefficients to be determined. When $C_2 = 0$, the solution of (4.27) is a Pearson Type III distribution, having a probability density function of the form shown in Eq.(4.28). For $C_1 = C_2 = 0$, a normal distribution is the solution of (4.27). Thus, the normal distribution is a special case of the Pearson Type III distribution, describing a nonskewed variable. The Pearson Type III distribution was first applied in hydrology by Foster(1924) to describe the probability distribution of annual maximum flood peaks. When the data are positively skewed, a log transformation is used to reduce the skewness. (Ven Te Chow and others, Applied Hydrology).

The probability density function of this distribution is given by

$$f(x) = \frac{\lambda^{\beta}(x-\epsilon)^{\beta-1}e^{-\lambda(x-\epsilon)}}{\Gamma(\beta)} \qquad (3.28)$$

where the range is $x \geq \epsilon$, and the parameters in terms of the sample moments is given by

$$\lambda = \frac{s_x}{\sqrt{\beta}} \quad , \; \beta = \left(\frac{2}{C_s}\right)^2 \quad , \epsilon = \bar{x} - s_x\sqrt{\beta} \qquad (3.29)$$

3.6.4.7 Log – Pearson Type III Distribution:

If log *X* follows a Pearson Type III distribution, then *X* is said to follow a log – Pearson Type III distribution. This distribution is the standard distribution for frequency analysis of annual maximum floods in the United States (Benson,1968). As a special case, when log *X* is symmetric about its mean, the log – Pearson Type III distribution reduces to the lognormal distribution. The location of the bound ϵ in the log-Pearson Type III distribution depends on the skewness of the data. If the data are positively skewed, then $\log X \geq \epsilon$ and ϵ is a lower bound, while if the data are negatively skewed, $\log X \leq \epsilon$ and ϵ is an upper bound. The log transformation reduces the skewness of the transformed data and may produce transformed data which are negatively skewed from original data which are positively skewed. In that case, the application of the log-Pearson Type III distribution would impose an artificial upper bound on the data. Depending on the values of the parameters, the log-Pearson Type III distribution can assume many different shapes, as shown in Table(4.1) (Bobee,1975).

Table(3.3): Shape and mode location of the log-Pearson Type III distribution as a function of its parameters

Shape parameter $\boldsymbol{\beta}$	$\boldsymbol{\lambda < -\ln 10}$	$\boldsymbol{-\ln 10 < \lambda < 0}$	$\boldsymbol{\lambda > 0}$
$\boldsymbol{0 < \beta < 1}$	No mode J-shaped	Minimum mode U-shaped	No mode Reverse J-shaped
$\boldsymbol{\beta < 1}$	Unimodal	No mode reverse J-shaped	Unimodal

*Source:*Bobee,1975.(Ven Te Chow, Applied Hydrology)

As described previously, the log-Pearson Type III was developed as a method of fitting a curve to data. Its use is justified by the fact that it has been found to yield good results in many applications, particularly good for flood peak data. The fit of the distribution to data can be checked using the χ^2test, or by using probability plotting. This distribution has the advantage of providing a skew adjustment. If the skew is zero, the Log-Pearson distribution is identical to the log-normal distribution.

The probability density function for type III is given by

$$f(x) = \frac{\lambda^{\beta}(y-\epsilon)^{\beta-1}e^{\lambda(y-\epsilon)}}{x\Gamma(\beta)} \quad ,(where\ y = \log x) \tag{3.30}$$

where the Range is ($\log x \geq \epsilon$), and the parameters in terms of the sample moments are given by

$$\lambda = \frac{s_y}{\sqrt{\beta}} \ , \ \beta = \left[\frac{2}{C_s(y)}\right]^2 \ , \ \epsilon = \bar{y} - s_y\sqrt{\beta} \tag{3.31}$$

(assuming $C_s(y)$ is positive)

Or, the density function for type III (with origin at the mode) can also be given by

$$f(x) = f_0\left(1 - \frac{x}{a}\right)^c \exp(-cx/2) \tag{3.32}$$

Where

$$c = \frac{4}{\beta} - 1, \ \ a = \frac{c}{2}\frac{\mu_3}{\mu_2} \ , \ \beta = \frac{\mu_3^2}{\mu_2^2} \tag{3.33}$$

$$f_0 = \frac{n}{a}\frac{c^{c+1}}{e^c\Gamma(\mathrm{c+1})} \tag{3.34}$$

μ_2= the variance,

μ_3= third moment about the mean = σ^6g

e= the base of the natural (napierian) logarithm

Γ= the gamma function

n= the number of years of record

g= the skew coefficient

σ= the standard deviation

The US water Resources Council (1967) adopted the Log-Pearson Type III distribution (to achieve standardization of procedure) for use by federal agencies. The procedure is to convert the data series to logarithms and compute.

Mean:

$$\overline{\log x} = \frac{\sum \log x}{n} \tag{3.35}$$

Standard deviation:

$$\sigma_{\log x} = \sqrt{\frac{\sum\left(\log x - \overline{\log x}\right)^2}{n-1}} \tag{3.35 a}$$

Skew coefficient:

$$\mathrm{g} = \frac{n\sum\left(\log x - \overline{\log x}\right)^3}{(n-1)(n-2)\left(\sigma_{\log x}\right)^3} \tag{3.35 b}$$

The values of *x* for various recurrence intervals are computed from

$$\log x = \overline{\log x} + K\sigma_{\log x} \tag{3.36}$$

and the frequency factor *K* is obtained from Tables for the computed value of ‘g’ and the desired recurrence interval.

3.6.4.8 Extreme Value Distribution:

Extreme values are selected maximum or minimum values of sets of data. For example, the annual maximum discharge at a given location is the largest recorded discharge value during a year, and the annual maximum discharge values for each year of historical record make up a set of the extreme values that can be analyzed statistically. Distributions of the extreme values selected from sets of samples of any probability distribution have been shown by Fisher and Tippet(1928) to converge to one of three forms of *extreme value distributions*, called Types I, II, and III, respectively, when the number of selected extreme values is large. The properties of the three limiting forms were further developed by Gumbel (1941) for the Extreme Value Type I (EVI) distribution, Frechet (1927) for the Extreme Value Type II (EVII), and Weibull (1939) for the Extreme Value Type III (EVIII).

The three limiting forms were shown by jenkinson (1955) to be special cases of a single distribution called the *General Extreme Value* (GEV) distribution. The probability distribution function for the GEV is

$$F(x) = \exp\left[-\left(1 - k\frac{x-u}{\alpha}\right)^{1/k}\right] \qquad (3.37)$$

where $k, u,$ and α are the parameters to be determined.

The three limiting cases are

(1) for $k = 0$, the Extreme Value Type I distribution.

(2) for $k < 0$, the Extreme Value Type II distribution for which Eq.(4.37) applies for $(u + \alpha/k) \leq x \leq \infty$, and

(3) for $k > 0$, the Extreme Value Type III distribution, for which Eq.(4.37) applies for $-\infty \leq x \leq (u + \alpha/k)$.

In all three cases, α is assumed to be positive.

For the EVI distribution *x* is unbounded, while for EVII, *x* is bounded from below by $(u + \alpha/k)$, and for the EVIII distribution, *x* is similarly bounded from above. The EVI and EVII distributions are also known as the *Gumbel* and *Frechet* distributions, respectively. If a variable *x* is described by the EVIII distribution, then $-x$ is said to have a *Weibull distribution*. (Ven Te Chow,1988)

The probability distribution function for the three types can be given as follows:

1. EVI distribution:

$$f(x) = \frac{1}{c} \exp\left[\left(-\frac{a+x}{c}\right) - \exp\left(-\frac{a+x}{c}\right)\right] \qquad (3.38)$$
$$-\infty < x < \infty$$

where *x* is the variate and *a, c* are the parameters. By the method of moments the parameters have been evaluated as

$$a = \gamma c - \mu \, , \;\; c = \frac{\sqrt{6}}{\pi}\sigma \tag{3.38 a}$$

where $\gamma = 0.57721 \ldots$ Euler's constant. The distribution has a constant $C_s = 1.139$. The **Gumbel distribution** used in flood frequency analysis is an example of this type.

2. EVII distribution: the cumulative probability is given by

$$F(x) = \exp[-(\theta/k)^k] \, , -\infty < x < \infty \tag{3.39}$$

   where θ and k are the parameters.

3. EVIII distribution: the cumulative probability is given by

$$F(x) = \exp\left[-\left(\frac{x-\epsilon}{\theta-\epsilon}\right)^k\right] , -\infty < x \le \epsilon \tag{3.40}$$

where θ and k are the parameters. **Weibull distribution used in drought – frequency analysis** is an example of this type.

3.6.4.9 Poisson Distribution:

If n is very large and y is very small, such that $y \cdot n = m$ is a positive number, then the probability density function which is in the Poisson distribution is given by

$$y = f(x) = \frac{m^x - e^{-m}}{x!} \tag{3.41}$$

Under abnormal skewness, the Poisson distribution is useful. The statistical parameters are $\sigma_p = \mu$ and $skewness = \frac{1}{\sqrt{\mu}}$.

3.7 Frequency Analysis:

3.7.1 General:

Hydrologic systems are sometimes impacted by extreme events, such as severe storms, floods, and droughts the magnitude of an extreme event is inversely related to its frequency of occurrence, very severe events occurring less frequently than more moderate events. The objective of frequency analysis of hydrologic data is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions. The hydrologic data analyzed are assumed to be independent and identically distributed, and the hydrologic system producing them (e.g., a storm rainfall system) is considered to be stochastic, space-independent, and time-independent. The hydrologic data employed should be carefully selected so that the assumptions of independence and identical distribution are satisfied, in practice, this is often achieved by selecting the annual maximum of the variable being analyzed (e.g., the annual maximum discharge, which is the largest instantaneous peak flow occurring at any time during the year) with the expectation that successive observations of this variable from year to year will be independent.

Flood data are analyzed to gain an understanding of a river's past behavior and to provide guidance on expected future floods. The results of flood flow frequency analysis can be used for many engineering purposes: for the design of dams, bridges, culverts, and flood control structures; to determine the economic value of flood control projects and to delineate flood plains and determine the effect of encroachments on the flood plain.

3.7.2 Return Period:

Suppose that an extreme event is defined to have occurred if a random variable X is greater than or equal to some level x_T. The *recurrence interval* $\boldsymbol{\tau}$ is the time between occurrences of $X \geq x_T$.

The *return period* T of the event $X \geq x_T$ is the expected value of $\boldsymbol{\tau}$, $E(\tau)$, its average value measured over a very large number of occurrences. Thus the return period of an event of a given magnitude may be defined as the *average recurrence interval* between events *equaling* or *exceeding* a specified magnitude.

The probability $p = P(X \geq x_T)$ of occurrence of the event $X \geq x_T$ in any observation may be related to the return period in the following way. For each observation, there are two possible outcomes: either "success" $X \geq x_T$(probability p) or "Failure" $X < x_T$ (probability$1 - p$). Since the observations are independent, the probability of a recurrence interval of duration τ is the product of the probabilities of $\tau - 1$ failures followed by one success, that is, $(1-p)^{\tau-1}p$, and the expected value of τ is given by

$$\begin{aligned} E(\tau) &= \textstyle\sum_{\tau=1}^{\infty} \tau(1-p)^{\tau-1}p \\ &= p + 2(1-p)p + 3(1-p)^2p + 4(1-p)^3 + \cdots \\ &= p[1 + 2(1-p) + 3(1-p)^2 + 4(1-p)^3 + \cdots] \end{aligned} \tag{3.42a}$$

The expression within the brackets has the form of the power series expansion

$(1 + x)^n = 1 + nx + [n\,(n-1)/2]x^2 + [n(n-1)(n-2)/6]x^3 + \cdots$

with $x = -(1-p)$ and $n = -2$, Eq.(5.2a) may be written

$$\begin{aligned} E(\tau) &= \frac{p}{[1-(1-p)]^2} \\ &= \frac{1}{p} \end{aligned} \tag{3.42 b}$$

Hence $E(\tau) = T = 1/p$; that is, the probability of occurrence of an event in any observation is the inverse of its return period:

$$P(x \geq x_T) = \frac{1}{T} \tag{3.43}$$

In addition, to calculate the probability that a *T*-year return period event will occur at least in *N* years, first consider the situation where no *T*-

year event occurs in N years. This would require a sequence of N successive "failures," so that

$$P(X < x_T \; each \; year \; for \; N \; years) = (1-p)^N$$

The complement of this situation is the case required, given by

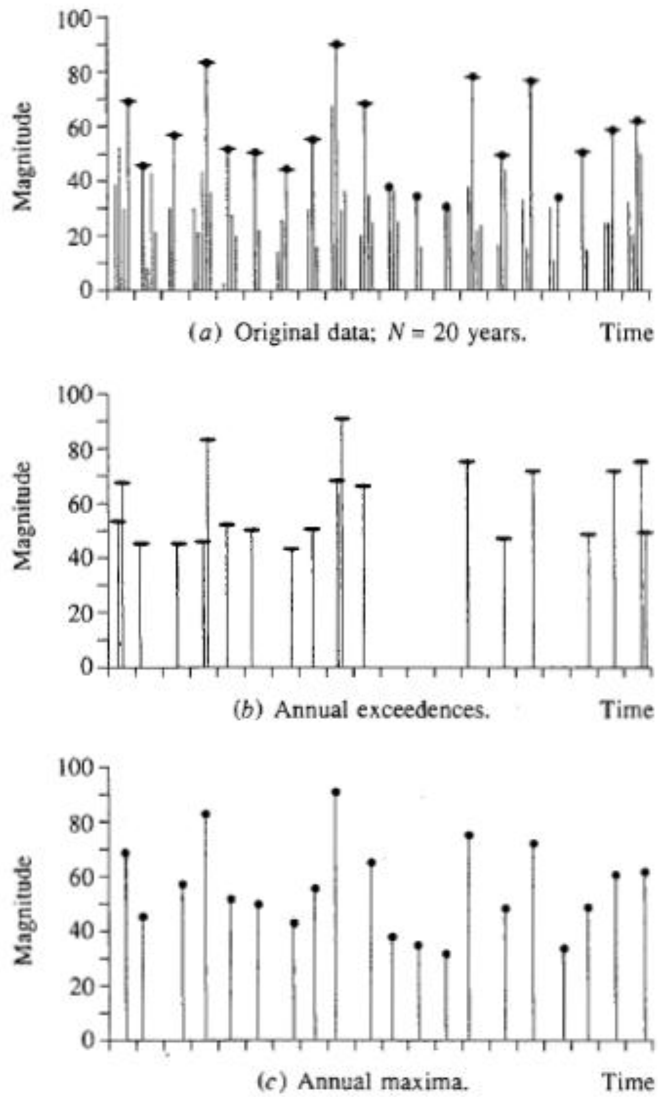
$$P(X \geq x_T \; at \; least \; once \; in \; N \; years) = 1 - (1-p)^N \tag{3.44}$$

Since $p = 1/T$,

$$P(X \geq x_T \; at \; least \; once \; in \; N \; years) = 1 - \left(1 - \frac{1}{T}\right)^N \tag{3.45}$$

3.7.3 Hydrologic Data Series:

A complete duration series consists of all the data available as shown in Fig.4.4(a). A *partial duration series* is a series of data which are selected so that their magnitude is greater than a predefined *base value*. If the base value is selected so that the number of values in the series is equal to the number of years of the record, the series is called an *annual exceedence series*; an example is shown in Fig.4.4(b). An *extreme value series* includes the largest or smallest values occurring in each of the equally-long time intervals of the record. The time interval length is usually taken as one year, and a series so selected is called an *annual series*. Using largest annual values, it is an *annual maximum series* as shown in Fig.12.1.2(c). Selecting the smallest annual values produces an *annual minimum series*.



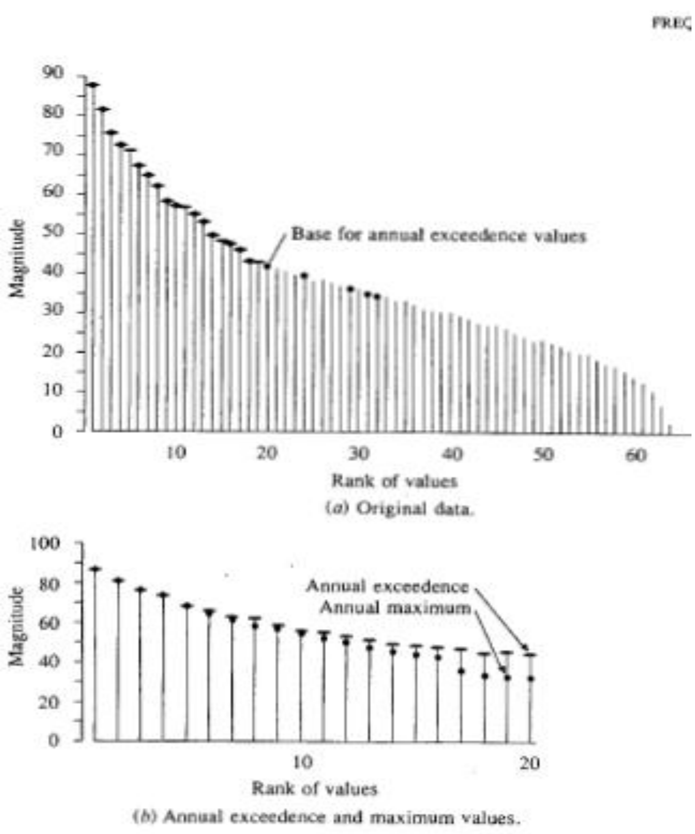
Figure(3.9): Hydrologic Data arranged by time of occurrence (*Source: Ven Te Chow &others, Applied Hydrology)*

The annual maximum values and the annual exceedence values of the hypothetical data in Fig.4.5(a) are arranged graphically in Fig.4.5(b) in order of magnitude. In this particular example, only 16 of 20 annual maxima appear in the annual exceedence series; the second largest value in several years outranks some annual maxima in magnitude. However, in the annual maximum series, these second largest values are excluded, resulting in the neglect of their effect in the analysis.

The return period T_E of event magnitudes developed from an annual exceedence series is related to the corresponding return period T for magnitudes derived from an annual maximum series by (Chow, 1964)

$$T_E = \left[ln\left(\frac{T}{T-1}\right)\right]^{-1} \tag{3.46}$$

Although the annual exceedence series is useful for some purposes, it is limited by the fact that it may be difficult to verify that all the observations are independent – the occurrence of a large flood could well be related to saturated soil conditions produced during another large flood occurring a short time earlier. As a result, it is usually better to use the annual maximum series for analysis. In any case, as the return period of the event being considered becomes large, two such events will occur within any year is very small.



Figure(3.10): Hydrologic Data arranged in order of magnitude(*Source: Ven Te Chow&others, 1988)*

3.7.4 Extreme Value Distributions:

The study of extreme hydrologic events involves the selection of a sequence of the largest or smallest observations from sets of data. For example the study of peak flows uses just the largest flow recorded each year at a gauging station out of the many thousands of values recorded. In fact, water level is usually recorded every 15minutes, so there are $4 \times 24 = 96$ values recorded each day, and $365 \times 96 = 35{,}040$ values recorded each year; so the annual maximum flow event used for flood flow frequency analysis is the largest of more than 35,000 observations during that year. And this exercise is carried out for each year of historical data.

Since those observations are located in the extreme tail of the probability distribution of all observations from which they are drawn (the parent population), it is not surprising that their probability distribution is different from that of the parent population. As mentioned in section 4.1, there are three forms of the distributions of extreme values, named Type I, Type II, Type III, respectively.

The Extreme Value Type I (EVI) probability distribution function is

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\alpha}\right)\right], -\infty \leq x \leq \infty \tag{3.47}$$

The parameters are estimated, as given

$$\alpha = \frac{\sqrt{6}s}{\pi} \tag{3.48}$$

$$u = \bar{x} - 0.5772\alpha \tag{3.49}$$

The parameter u is the mode of the distribution (point of maximum probability density). A *reduced variate y* can be defined as

$$y = \frac{x-u}{\alpha} \tag{3.50}$$

Substituting the reduced variate into Eq.(5.7) yields

$$F(x) = \exp[-\exp(-y)] \tag{3.51}$$

Solving for *y*:

$$y = -\ln\left[\ln\left(\frac{1}{F(x)}\right)\right] \tag{3.52}$$

Let Eq.(5.13) be used to define *y* for the Type II and Type III distributions. The values of *x* and *y* can be plotted as shown in Figure (4.6). For the EVI distribution the plot is a straight line while, while for large values of *y*, the corresponding curve for the EVII distribution slopes more steeply than for EVI, and the curve for the EVII distribution slopes less steeply, being bounded from above. Figure (4.6) also shows values of the return period *T* as an alternate axis to *y*. As shown by Eq. (5.2),

$$\frac{1}{T} = P(x \geq x_T)$$

$$= 1 - P(x < x_T)$$

$$= 1 - F(x_T)$$

So

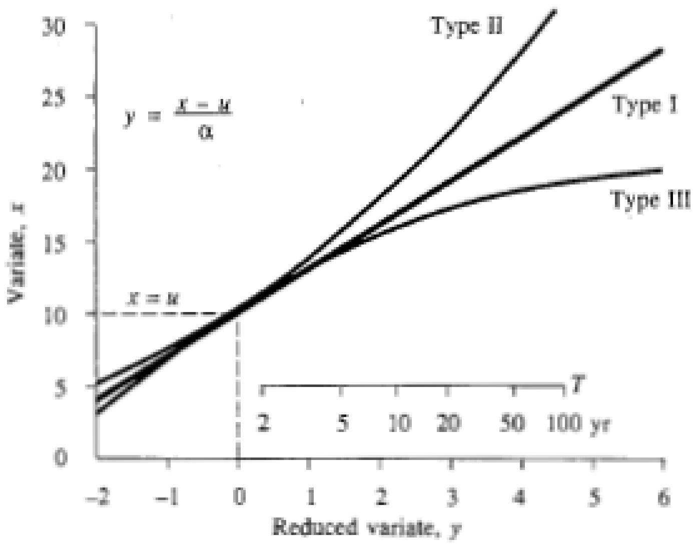
$$F(x_T) = \frac{T-1}{T}$$

and, substituting into Eq.(5.11),

$$y_T = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \tag{3.53}$$

For the EVI distribution, x_T is related to y_T by Eq.(5.9), or

$$x_T = u + \alpha y_T \tag{3.54}$$



Figure(3.11): For each of three types of extreme value *x* is plotted against a reduced variate *y* calculated for the Extreme Value Type I Dist., the Type I distribution is bounded in *x* while the TypeII distribution has alower bound and the TypeIII dist. Has an upper bound. (*Source: Chow & others,1988)*

Extreme value distributions have been widely used in hydrology. They form the basis for the standardized method of flood frequency analysis in Great Britain (Natural Environmental Research Council, 1975). Storm rainfalls are most commonly modeled by the Extreme Value Type I distribution (Chow, 1953: Tomilnson, 1980), and drought flows by the Weibull distribution, that is, the EVIII distribution applied to – *x* (Gumbel, 1954,1963). (Chow & others, 1988)

3.7.5 Probability Plots and Plotting Positions:

A distribution of known algebraic form is usually thought of in terms of its probability density (pdf) e.g. the normal distribution as symmetrical and bell shaped or the exponential distribution as unsymmetrical pdf, with peak at the origin and with a slowly decreasing tail.

Corresponding to the theoretical pdf, is the relative frequency histogram built up from a sample of observed data. In practical applications, however, the distribution function is required, e.g. for calculating exceedence and non-exceedence probabilities and the return period. The cumulative histogram, is one of the counterparts of the distribution function which can be built up from a sample of observed data. The cumulative histogram is naturally more irregular in shape than the theoretical distribution function of the parent population and it is also limited in extent on the probability scale; it ranges from $\frac{1}{N}$ to $\frac{N-1}{N}$ whereas the entire range is 0 to 1. Variate values with probability values outside the $\left(\frac{1}{N}, \frac{N-1}{N}\right)$ range are often required, but the cumulative histogram cannot easily be extrapolated because

(a) It is usually too irregular especially in small samples, and
(b) The underlying shape may not be known, and even if its algebraic form is known it is impossible to draw such a curve through the data by hand.

In the past it was generally felt that these limitations would be less serious if the shape being aimed at were a straight line.

Hence engineers began to look at the distribution function of the normal and exponential distributions on non-linear graph papers, in each of which the probability scale is chosen so as to make the distribution function appear as a straight line.

In this treatment it is usual to view the distribution function curve with variate *x* as ordinate and probability as abscissa.

This linearation is achieved by using as abscissa the standardized variate y and the straight line relation

$$x = \begin{bmatrix} \text{location} \\ \text{parameter} \end{bmatrix} \qquad + \begin{bmatrix} \text{scale} \\ \text{parameter} \end{bmatrix} y$$

$$\begin{aligned} &= \mu + \sigma y && \text{In Normal Dist.} \\ &= x_o + \beta y && \text{In Exponential Dist.} \\ &= u + \alpha y && \text{In EVI Dist.} \end{aligned}$$

Each value x and y, so related, have the same probability value and return period, that is $F(x) = G(y)$ for values of x and y related by $x = a + by$.

Hence if values of $G(y)$ are marked alongside the y scale the resulting probability scale also serves for $F(x)$.

3.7.5.1 Construction of probability Paper:

Probability plots are much used in engineering analysis particularly where the assumption that a set of data is a random sample from a distribution of specified form is being checked. The plot may also be used to estimate, by drawing a fair curve through the points, variate values of desired probability or return period, a common practice before statistical estimation theory became well known among engineers.

The probability plot may be made

(a) On special probability paper in which the abscissa is graduated in terms of probability values. The ranked values $x_1 < x_2 < x_3 < \cdots x_i < \cdots x_n$, are plotted against plotting probabilities $(F_i,\ i = 1,2, \ldots N)$ the formula for which varies from one distribution to the next, or

(b) On ordinary graph paper which the abscissa is linearly graduated in the standardized (or reduced) variate y of the distribution in question. Plotting positions y_i are used which are related to the F_i above by the

$y - F(y)$ relation in the standardized variate. A probability or return period scale is then sketched alongside the *y* scale.

3.7.5.2 Normal Distribution:

a) Plotting probability $F_i = \frac{i-3/8}{N+1/4}$ (Blom Formula), $i = 1$ for smallest.

b) Plotting $Y_i = E(y_{(i)})$= average value of $Y_{(i)}$ in $N(0,1)$ distribution. The tabulated values given are exact; the F_i values given in (a) are approximately equal to the value of *F(y)* obtained by converting Y_i in the $N(0,1)$ table.

3.7.5.2 Extreme Value Type 1 Distribution:

a) Plotting probability $F_i = \frac{i-0.44}{N+0.12}$ (Grington Formula), $i = 1$ for smallest.

b) Plotting $Y_i = E(y_{(i)})$ = average value of $Y_{(i)}$ in $\text{EV1}(0,1)$ distribution.

For $N \leq 26$ exact values have been tabulated by Lieblein and Salzer (1957) and are very close to *y* values obtained by converting the F_i values in (a) to $\text{EV1}(0,1)$ variate values, $Y_i = -\ln(-\ln F_i)$.

3.7.5.3 Exponential Distribution:

a) Plotting probability $F_i = \frac{i-0.44}{N+0.12}$ as for EVI Distribution

b) $Y_i = E\left(y_{(i)}\right) = \sum_{j=1}^{i} \frac{1}{N+1-j}$ as tabulated.

The justification of the formulae given above derives from the statistical properties of the resulting plot. Figure (4.7) shows a sample item $x_{(i)}$ plotted at $y = E(y_{(i)})$ while the distribution of all possible $x_{(i)}$ values and its mean is also shown and it is seen that the mean, $E(x_{(i)})$, lies on the population line. This is true for all values of $i = 1,2, \ldots N$ and means that the average plot of many samples from the same population would fall on the population line. For this reason the plotting position $E(y_{(i)})i$ is said to be unbiased. The value of $E(y_{(i)})$ depends not only on rank i and sample size N but also on the distribution being considered. The formula for $E(y_{(i)})$ is generally cumbersome and not easy to evaluate but the tables supplied obviate the problem. (See Appendix "A")

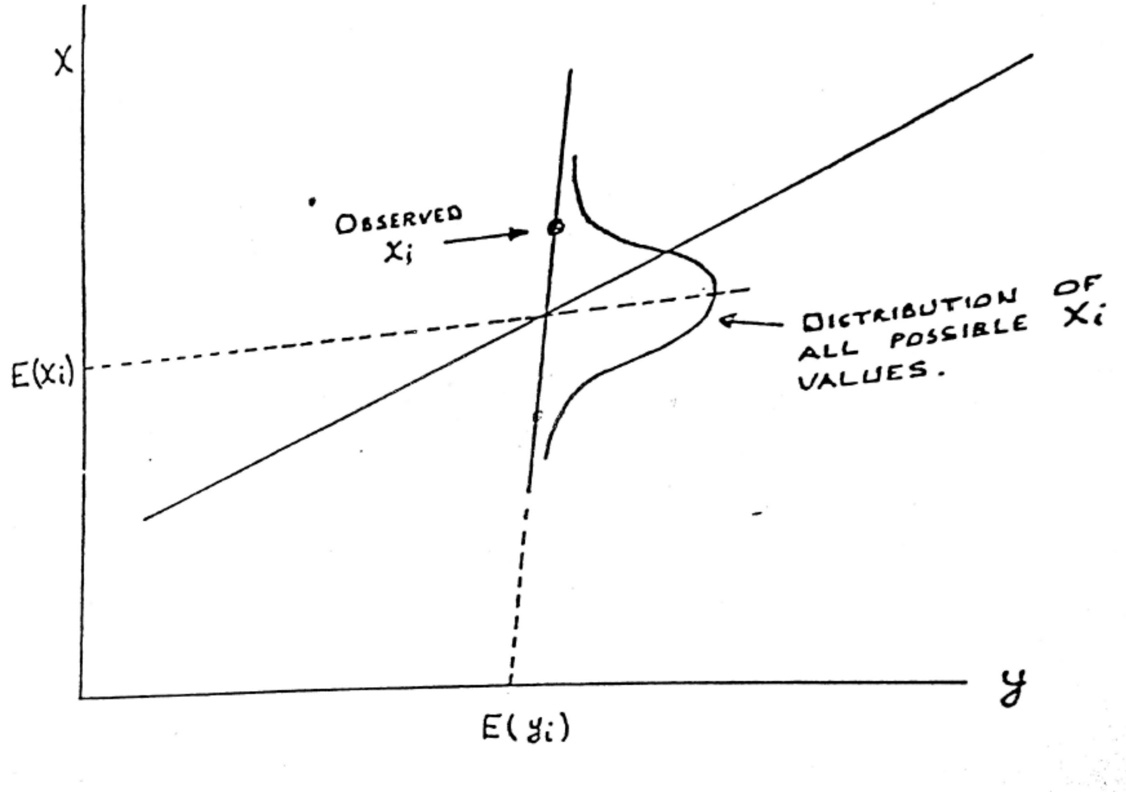


Figure (3.13): a sample item plotted at $y = E(y_{(i)})$

The values of probability corresponding to the $E(y_{(i)})$ values have been investigated and all can be approximated well by a formula with one variable α which depends on the distribution

$$F_i = \frac{i - \alpha}{N + 1 - 2\alpha} \tag{3.55}$$

Normal:

$$\alpha = {}^{3}/_{8},\ F_i = \frac{i-{}^{3}/_{8}}{N+{}^{1}/_{4}} \text{ (Blom Formula)} \tag{3.55a}$$

Exponential and EVI:

$$\alpha = 0.44,\ F_i = \frac{1-0.44}{N+0.12} \text{ (Gringorten Formula)} \tag{3.55b}$$

The tables for sample sizes up to 50 which follow (See Appendix “A”) can be considered to fall into four groups as follows:

Type A: Tables of F_i and $E(y_{(i)})$ for the EVI distribution. The $E(y_{(i)})$ values are exact for $N \leq 35$ and are based on the Gringorten approximation for higher N.

Type B: Table of exact values of $E(y_{(i)})$ for the Exponential distribution, values of F_i if required are the same as for EV1 case (Appendix “A”: Table1).

Type C: Tables of F_i and $E(y_{(i)})$ for the Normal distribution. The latter are exact for $N \leq 30$ and are based on the Blom approximation for higher N.

Type D: Table of $F_i = \frac{i-2/5}{N+1/5}$ followed by tables giving the corresponding y_i values in the EV1, exponential and normal distributions. These three tables are based on closet available compromise to a single easily remembered plotting position formula applicable to most distributions which are not too highly skewed.

Chapter Four

Data Analysis & Results

4. Analysis & Results

4.1 Preface:

The research results are considered the base of which recommendations and conclusions regarding the objectives and problems stated are obtained, in order to come up with proper decisions and judgment for the matter at hand.

This chapter is concerned with displaying the annual maximum water level data of the Gash River provided (Chapter 3, Section 5), using the Normal distribution and Extreme value Type 1. Also, frequency analysis methods were used to find the Return Period.

4.2 The Data:

The data used for this study as mentioned earlier in Chapter 3, section 5, is the annual maximum series for the Gash, data was obtained from the Ministry of Electricity &Water Resources Records. The record is continuous from the years 1970 to 1992, and then there is a gap of record of four years, afterwards, from 1996 till 2004 is once again continuous.

Table(4.1): The annual maximum water level series for the River Gash (*Source: Ministry of Electricity and Water Resources, Sudan*)

N	Years	Water Level	N	Years	Water Level
1	1970	504.97	17	1986	506.8
2	1971	505	18	1987	505.9
3	1972	505.7	19	1988	506.45
4	1973	505.3	20	1989	505.4
5	1974	505.95	21	1990	505.9
6	1975	505.22	22	1991	505.9
7	1976	505.98	23	1992	506.1
8	1977	505.58	24	1996	505.95
9	1978	505.68	25	1997	506.4
10	1979	505.1	26	1998	506.3
11	1980	505.79	27	1999	506.1
12	1981	505.7	28	2000	506.45
13	1982	505.91	29	2001	506.75
14	1983	506.2	30	2002	506.3
15	1984	505.9	31	2003	507
16	1985	505.7	32	2004	506.7

By plotting the annual maximum series versus the years of record the Time series is obtained as follows in Figure (4.1). The series trendline with the equation provided in the figure and R^2 of 0.6332, shows that the water level tends to increase with time.

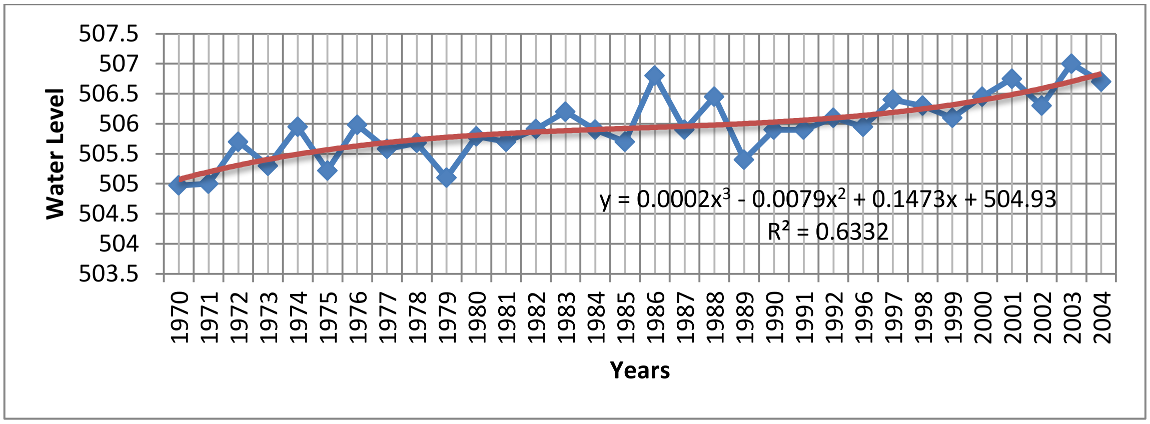


Figure (4.1): The River Gash Time Series for recorded data from 1970 to 2004

In this study, the annual maximum method is used. The series of annual daily maximum flood peaks is obtained by taking the maximum daily water level occurring within the year.

Two statistical models are used to model the annual maximum series of Gash river. The first one is the Extreme Value distribution type (I) (EVI). The second distribution is the Normal Distribution.

4.3 Analysis:

4.3.1 Distribution parameters:

Necessary values needed for the distributions parameters are calculated as follows from study data given in Table (5.1):

(i) The Mean (average):

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n}$$

$$\therefore \bar{x} = 505.94$$

(ii) $\sum_{i=1}^{n} x_i^2$

$$\underline{\underline{= 262118690}}$$

(iii) Variance :

$$Var\ X = \frac{\left(\left(\sum_{i=1}^{n} x_i^2\right) - (N\bar{x})^2\right)}{n-1}$$

$$= \frac{(262118690) - 32 * (505.94)^2}{32 - 1}$$

$$\underline{\underline{= 8191209.062}}$$

(iv) $\hat{\sigma} = \sqrt{VarX}$

$$= \sqrt{8191209.062}$$

$$\underline{\underline{\therefore \hat{\sigma} = 2862.0288}}$$

The distributions parameters are as follows:

(a) Normal distribution:

$$\therefore \hat{\sigma} = 2862.0288 \qquad \mu = \bar{x} = 505.94$$

(b) Extreme value Type1:

$$\mu = U + 0.577\alpha \qquad \sigma = 1.28\alpha$$

$$\hat{U} = \bar{x} - 0.577\hat{\alpha} \qquad \bar{\alpha} = {}^{\hat{\sigma}}/_{1.28}$$

$$= \bar{x} - 0.45\hat{\sigma}$$

Hence,

$$\hat{\alpha} = \frac{2862.0288}{1.28} = 2235.96$$

$$\hat{U} = 505.94 - 0.45 * 2862.0288 = -781.9729$$

$$\mu = -781.9729 + 0.577 * 2235.96 = 508.176$$

4.3.2 Distribution Fitting and plotting positions:

1. Table (4.2) shows the distribution fitting and plotting position for the study data, using both Normal distribution and Extreme Value Type1 formulae.
2. The Data in Column (B) was ranked from the smallest value to the highest and put in Column (C).
3. Values in of (D) and (E) in table (4.2) were obtained from Appendix (A), Table (1) and Table (2).
4. Column (C) versus Column (D) in Table (4.2), were plotted in Figure(4.3) to give the Normal Distribution Fitting.
5. Column (C) versus Column (E) in Table (4.2), were plotted in Figure(4.2) to give the Extreme Value Type1 Distribution Fitting.

Table(4.2): Distribution Fitting and Plotting positions for the Normal & EVI dist.

N	Years	Water Level	Rank	plotting positions $\frac{1-0.44}{N+0.12}$ FROM TABLE 1	E(Y(1)) FROM TABLE 2
#	(A)	(B)	(C)	(D)	**(E)**
1	1970	504.97	504.97	0.017	**-1.36**
2	1971	505	505	0.49	**-1.09**
3	1972	505.7	505.1	0.8	**-0.91**
4	1973	505.3	505.22	0.111	**-0.77**
5	1974	505.95	505.3	0.142	**-0.66**
6	1975	505.22	505.4	0.173	**-0.55**
7	1976	505.98	505.58	0.204	**-0.45**
8	1977	505.58	505.68	0.235	**-0.36**
9	1978	505.68	505.7	0.267	**-0.27**
10	1979	505.1	505.7	0.298	**-0.18**
11	1980	505.79	505.7	0.329	**-0.1**
12	1981	505.7	505.79	0.36	**-0.01**
13	1982	505.91	505.9	0.391	**0.07**
14	1983	506.2	505.9	0.422	**0.16**
15	1984	505.9	505.9	0.453	**0.24**
16	1985	505.7	505.9	0.484	**0.33**
17	1986	506.8	505.91	0.516	**0.42**
18	1987	505.9	505.95	0.547	**0.51**
19	1988	506.45	505.95	0.578	**0.61**
20	1989	505.4	505.98	0.609	**0.71**
21	1990	505.9	506.1	0.64	**0.82**
22	1991	505.9	506.1	0.671	**0.93**
23	1992	506.1	506.2	0.702	**1.05**
24	1996	505.95	506.3	0.733	**1.18**
25	1997	506.4	506.3	0.765	**1.33**
26	1998	506.3	506.4	0.796	**1.49**
27	1999	506.1	506.45	0.827	**1.68**
28	2000	506.45	506.45	0.858	**1.89**
29	2001	506.75	506.7	0.889	**2.16**
30	2002	506.3	506.75	0.92	**2.51**
31	2003	507	506.8	0.951	**3.03**
32	2004	506.7	507	0.983	**4.04**

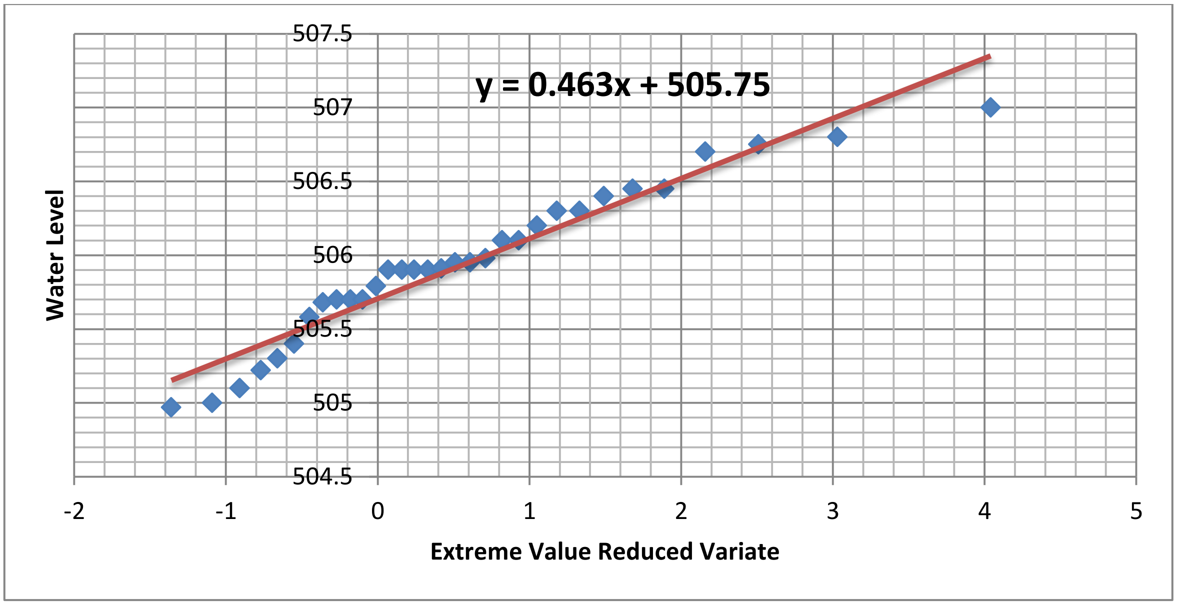


Figure (4.2): Extreme Value Type1 Distribution

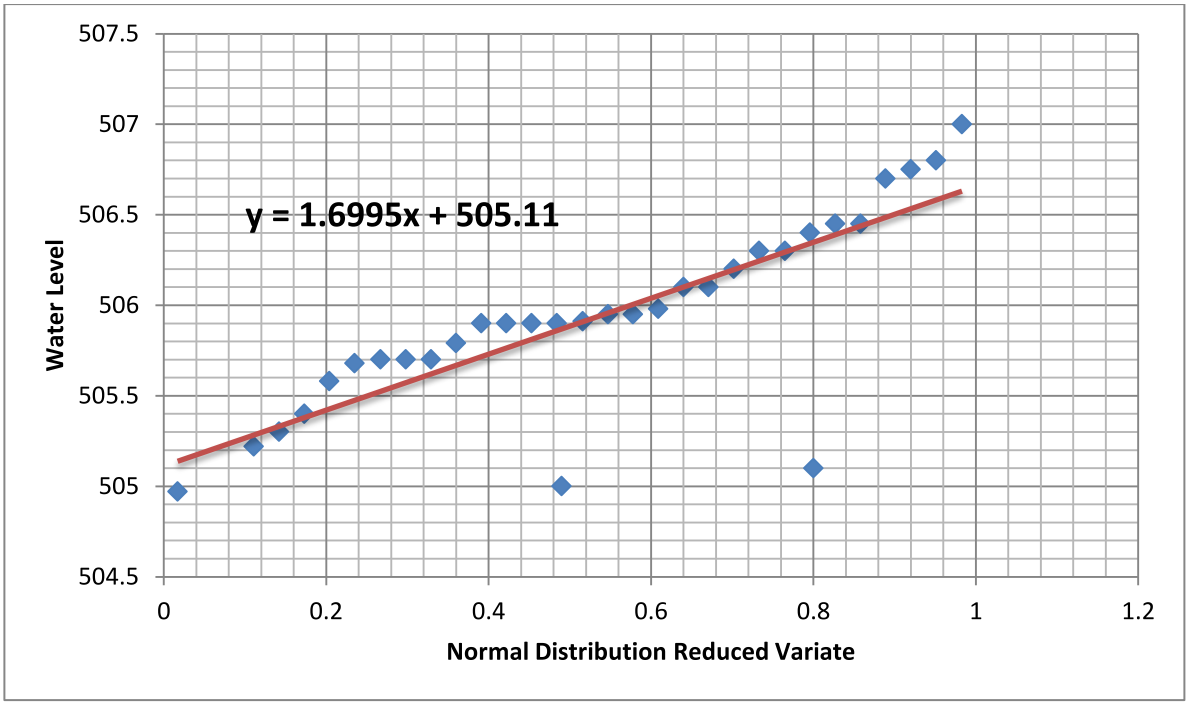


Figure (4.3): Normal Distribution

4.3.3 Return Period:

1. Table (4.3) shows the Return Period Calculations and plotting position for the study data, using Frequency Analysis formulae.
2. The Data in Column (B) was ranked from the smallest value to the highest and put in Column (C).

3. Column (C) versus Column (E) in Table (4.2), were plotted in Figure(4.4) to give the Return Period.

4. The relation was found to have a an $R^2 of\ 0.7368$

Table (4.3): Return Period Calculations and Plotting Positions

N	Years	Water Level	rank	RETURN PERIOD $P = \frac{i}{N+1}$	RETURN PERIOD T=1/P
#	(A)	(B)	(C)	(D)	**(E)**
1	1970	504.97	504.97	0.030303	**66**
2	1971	505	505	0.060606	**33**
3	1972	505.7	505.1	0.090909	**22**
4	1973	505.3	505.22	0.121212	**16.5**
5	1974	505.95	505.3	0.151515	**13.2**
6	1975	505.22	505.4	0.181818	**11**
7	1976	505.98	505.58	0.212121	**9.428571**
8	1977	505.58	505.68	0.242424	**8.25**
9	1978	505.68	505.7	0.272727	**7.333333**
10	1979	505.1	505.7	0.30303	**6.6**
11	1980	505.79	505.7	0.333333	**6**
12	1981	505.7	505.79	0.363636	**5.5**
13	1982	505.91	505.9	0.424242	**4.714286**
14	1983	506.2	505.9	0.454545	**4.4**
15	1984	505.9	505.9	0.484848	**4.125**
16	1985	505.7	505.9	0.515152	**3.882353**
17	1986	506.8	505.91	0.545455	**3.666667**
18	1987	505.9	505.95	0.575758	**3.473684**
19	1988	506.45	505.95	0.606061	**3.3**
20	1989	505.4	505.98	0.636364	**3.142857**
21	1990	505.9	506.1	7.333333	**0.272727**
22	1991	505.9	506.1	0.69697	**2.869565**
23	1992	506.1	506.2	0.727273	**2.75**
24	1996	505.95	506.3	0.757576	**2.64**
25	1997	506.4	506.3	0.787879	**2.538462**
26	1998	506.3	506.4	0.818182	**2.444444**
27	1999	506.1	506.45	0.848485	**2.357143**
28	2000	506.45	506.45	0.878788	**2.275862**
29	2001	506.75	506.7	0.909091	**2.2**
30	2002	506.3	506.75	0.939394	**2.129032**
31	2003	507	506.8	0.969697	**2.0625**
32	2004	506.7	507	1	**2**

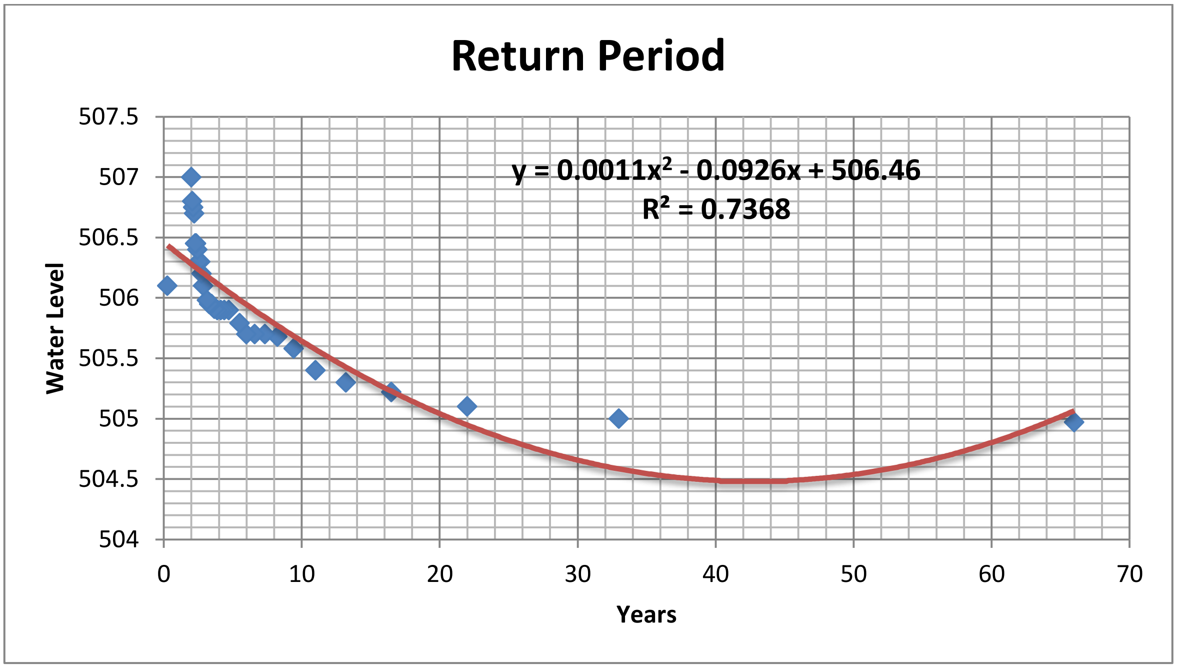


Figure (4.4): Return period Graph for the Study data.

4.4 Obtained Results:

Using the formula obtained from the Return period graph (Figure 4.4), which represents a relationship between years and annual maximum water levels for Gash river, the authors' were able to predict Flood water levels for assumed recurrence intervals (i.e., 2,10,15,25,50,80,100);(see Table 4.4and Figure 4.5).

The formula has a Coefficient of Determination "R^2" of "0.7368" , thus, it's necessary to emphasis that based on the value of "R^2" the integrity of the equation is acceptable.

Table (4.4): Expected Flood Water Levels corresponding to a Return period (T)

Return Period (Years)	Expected Flood Water Levels
2	**506.2792**
10	**505.644**
15	**505.3185**
25	**504.8325**
50	**504.58**
80	**506.092**
100	**508.2**

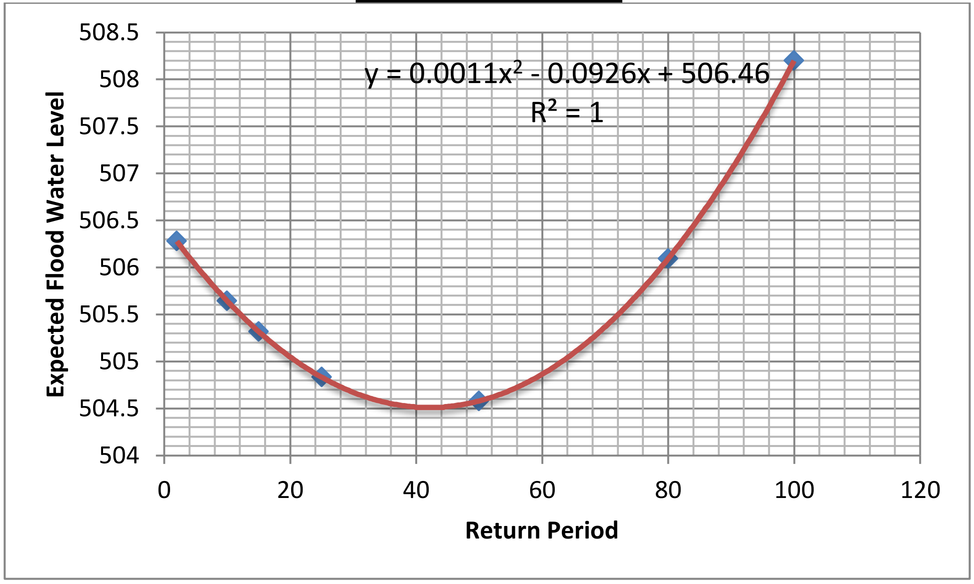


Figure (4.5): Relationship between expected flood water levels and Return period(T)

Chapter Five

Conclusions, Recommendations and Future Results

5. Conclusions, Recommendations & Future Research

5.1 Conclusions:

The conclusions fall into the main aspects described in the scope and objectives of this study. Respective conclusions were drawn as follows:

1- Prediction of Floods that may occur in the future in Gash river. The Flood frequency analysis for the River, in which the levels corresponding to any return period (T) of occurrence for Gash river were determined, after fitting the data to the proposed distributions.
The maximum predicted flood water level of "508.2" had a return period of 100 years.
Hence, the prediction flood water levels in the Gash with ten years return period from end of study record, year 2004, was 505.64; whereas, the actual record at time of submitting this study, August 2014, was 506.8 (*source: Gash river Training Unit GRTU,Kassala*), the error was found to be 0.22% as the actual amount had a higher value.

2- Exercising the application and practice of Applied Statistics packages was illustrated in detail to prove that most hydrological events can be described and measured. Data fitting was practiced using both, the Normal distribution and the Extreme Value Type1 (EVI).

3- Practicing the analysis of hydrological data related to rivers was clear in choosing the data record subjected to study, and the means that were used for collection. Add to that, the history of the study area and the river itself were helpful in understanding the future behavior and measures to be taken in order to reduce loses experienced by habitants.

Finally the authors' would like to emphasis that the length of the record isn't enough and a longer one should be used. Plus, assuring that there is no gap of record or missing data.

5.2 Recommendations:

1- **Application of more Probability distributions:**

Use of more probability distributions on the data, will lead to finding the appropriate fitting for the phenomena described, hence more precise results and conclusions could be drawn.

2- **Thorough research and study on the Gash river and its basin:**

The Gash and its behavior and some hydrologic characteristics are partially unknown, this leads to huge damage and loss caused mainly by not taking the right precautionary measures to save lives and property; such loss is clear to the reader by reviewing the Gash present and past.

More research and studies could help in the design of protective work on the river's path; add to that, the proper understanding of what's being dealt with leads to more benefits from such a major resource.

3- **The use of statistics related and river modeling computer programs to process available data:**

The application of statistical programs (i.e. SPSS: Statistical Package for the Social Sciences) will help in processing the data more accurately and dodging human error. Also, river modeling programs (i.e. HEC-RAS: The Hydrologic Engineering Center, River Analysis System) will help in the prediction of any future behavior of the river and its effect on its basin and surrounding areas.

5.3 Future Research:

From the review of this project the following research directions are suggested:

1- **Application of Frequency Analysis and Probability Distribution on different aspects regarding Civil engineering:**

In phenomena that follow a certain pattern of behavior or natural phenomena the application of frequency analysis and fitting of probability distributions could prove useful in addition to other methods used (i.e. compressive strength of concrete cubes, and time-frequency analysis for wind effects on structures).

2- Design of rainfall and flood routing systems:

Most river flooding catastrophes' are caused by excessive rainfall rates, which lead to a huge amount of runoff that flows into to the water body, resulting in rising of water levels over the safe limits, and therefore floods occur.

In the other hand, the flood routing system helps calculate control flood flows upstream and in highlands with excessive rainfall. The calculation of flood flows comes in handy when designing the various control systems.

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