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COLLEGE OF ENGINEERING
FACULTY OF CIVIL AND GEOMATIC ENGINEERING

USING RESERVOIR STORAGE EFFECTS FOR URBAN FLOOD MANAGEMENT: CASE STUDY OF MAMAHUMA BASIN OF TEMA

A Thesis submitted to the School of Graduate Studies, Kwame Nkrumah University of Science and Technology, in fulfilment of the requirement for the Award of the Degree of Master of Philosophy in Water Resources Engineering and management

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CERTIFICATION

I hereby declare that this submission is my own work towards the Master of Philosophy and that, to the best of my knowledge, it contains no material previously published by another person nor material which has been accepted for the award of any other degree of the University, except where due acknowledgement has been made in the text

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Dedication

To my Children

ABSTRACT

With the increase in the public awareness of flooding and flood-related issues, many Municipalities and Regulatory Authorities are requiring a higher degree of accuracy in flood studies and analysis. Today, this approach is needed specifically in urban areas where the risks are higher for the population and the infrastructure. Annual floods have claimed several lives and property. The rapid evolution in urbanization has precipitated a change in the approach to stormwater management and therefore the existing concept for the rapid removal of stormwater runoff from developed areas by channelization, must now be combined with methods for storing stormwater runoff to prevent overloading the existing drainage systems for effective flood management. To meet the increased requirements for urban flood mitigation in Ghana, a possible strategy is to introduce the use of reservoir storage concept, which has been successfully practiced in some developed countries, in combination with the traditional channelization approach. Abandoned agricultural reservoirs could be considered for flood mitigation without having to be reclaimed as usually prevails in the country. Comprehensive study was carried out on an existing reservoir in the Mamahuma basin of Tema as a case study. The geomorphologic instantaneous unit hydrograph model was used to obtain an inflow hydrograph based on rainfall event in the study area and routed through an existing reservoir to predict the impact of the reservoir on flood peak attenuation. The result showed that with the introduction of the reservoir storage approach, a large percentage of the flood plain was protected from inundation. The reservoir storage concept is thus recommended and for use where appropriate in combination with the existing system in the management of urban floods in Ghana.

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Combining the stress of working in a heavily engaged position with studies has not been all that easy. It is not by my might or strength that I have produced this thesis. I thank the Almighty God for granting me the strength and the talent to complete my thesis successfully.

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CHAPTER ONE

1 INTRODUCTION

1.1 General Information

With the increase in the public awareness of flooding and flood-related issues, many municipalities and regulatory authorities are requiring a higher degree of accuracy in flood studies and modeling (Gourbesville and Savioli, 2002). Today, this approach is needed specifically in urban areas where the risks are higher for the population and the infrastructure. Flooding is defined as the result of runoff from rainfall and or melting snow in quantities too great to be confined in the low-water channels of streams (Linsley, et al, 1992).

Throughout history most settlements have developed along rivers and coast lines, whose presence guaranteed access to and from sea to coast, irrigation for crops, water supplies for urban communities and latterly power development and industrial water supply. In all these cases infrastructural developments start from the downstream flood plains and subsequently expanded upstream. The many advantages derived from settlements along rivers and coasts have always been counterbalanced by the dangers of floods. Admittedly, some attempt has been made in certain instances to deal with it by exploiting natural defenses such as building away from flood plains and creating flood banks. These ~~were rarely~~ sufficient. To avoid the damage from floods a second consideration had to be given to the concept of flood management. For this reason different countries have put in place different approaches like the use of reservoirs, channelization, dyke, and other flood peak reduction measures in the management of urban floods.

In Ghana, the conveyance of floods through channels to their outfalls has been the sole approach in urban flood management. This approach of flood mitigation has not yielded much result, as communities continue to expand from downstream towards upstream with corresponding increase in rainfall runoff and yet building structures are developed close to the channels in the existing communities without any room for expansion. However, with the expansion of communities there is a corresponding need to expand the existing channels downstream to accommodate the increased runoff volumes. Unfortunately due to lack of space in the built-up communities downstream, expansion of the existing channels become almost impossible. To meet the increased requirements for urban flood mitigation in Ghana, a possible strategy is to introduce Reservoir storage approach to complement the effects of the existing channelization approach.

Ghana has experienced several floods in recent history, notable among them as obtained from the Hydrological Services Department, 2007 are the:-

- Floods of Accra in the Odaw, Lafa Kpeshie, and the Densu basins in 1959, 1988, 1995, 2002 and 2007 respectively.
- Floods of Kumasi in the river basins of Sisan, Aboabo and Wiwi in 1962 and 1988
- Floods in the Tamale central business area in 1988 leading to the collapse of water supply ~~reservoir~~ for the community
- Floods in the newly developed settlements of Tema in the Sakumo basin in the years 1995, 2002 and 2007

- Major floods in Sandema, Bawku and the other parts of the three Northern Regions in the White Volta basin in the years 1988, 1998 and 2007 respectively.

It is speculated that the Global environmental changes brought about by land use modifications, desertification, green house effects and gas emissions are having ultimate impact on climates of continents leading to many unwanted hydrological events such as frequent and droughts.

The rainfall event of the 3rd and 4th July 1995, a 6 hour rainfall of 259 mm magnitude caused considerable floods in the Sakumo basin which resulted in the submerging of the Tema sewerage treatment plant. About 300 billion cedis worth of property were destroyed, 100 lives were lost either during the flood or after the floods and about 10,000 people rendered homeless by floods that occurred between 1995 and 1997 in Accra and Tema (Gyau Boakye, 1997)

Also flood resulting from 148.40mm of rainfall during 4-hour duration in July 2002 displaced at least 2000 people in Accra and Tema communities 18, 19, and 20. (Hydrological services Department (HSD), 2002).

From the recent flood records there is the need for Ghana to make additional efforts to combat the adverse ~~impact of~~ floods damages. Plates1-1 and 1-2 below show the effects of floods in Accra during the floods of 3rd-4th July 1995 and 4th July 2007 respectively.

Plate 1-1: Flood damage in Accra
4th July 2007

Plate 1-2 Post and Telecom workers, Ghana rescued in 1995 floods

1.2 Problem Description

There is rapid hydrological response of urban catchments in Ghana due to the ongoing expansion of urbanized areas. The state of the art of urban drainage management in Ghana emphasises conveyance to the neglect of peak reduction methods, as all municipalities, regulatory authorities and communities practice channelization for urban flood management.

In the global trend of urban flood mitigation, many counties such as UK, USA, and China are moving away from the traditional method which seeks to protect against flooding by collecting and directly conveying floods to outfalls. Measures to reduce flood peaks and volume are usually incorporated into the total stormwater management practices, (Bedient and Huber, 1992)

Even though some level of storm water conveyance channels have been developed in the urban communities, severe flood damages continue to occur. The lack of effective flood mitigation methods, coupled with the ill effects of urbanization are the major causes of flooding in the urban communities. Some of the floods have been very devastating in terms of damage to property and loss of human lives depending on the magnitude and antecedent conditions of the soil.

1.2.1 Over reliance on direct conveyance of runoff

The current concept of flood mitigation in the country is channelization. Relying on this approach alone will require that most existing stormwater channels be expanded to accommodate the increasing flow. Achieving this objective will become extremely difficult as the areas for expansion are already built up. An integrated management approach will therefore be required for flood damage reduction. Policy makers have however been reluctant to embrace other methods such as stormwater storage and pumping etc. This study seeks to demonstrate the effectiveness of detention/ retention in flood management.

1.2.2 Ill effects of urbanization

The hydrological transformation of rainfall into runoff by a catchment is significantly modified by the land use changes such as urbanization; however hydrological consequences of land use policies are rarely considered at the level that they deserve in Ghana. There is the rush for land by estate developers resulting in encroachment on waterways and flood plains while the city authorities appear not to have noticed the importance of flood plains. Plates 1-3 and 1-4 below are typical examples of housing encroachment on waterways in the Mamahuma basin of Tema.

Plate 1-3: Structural encroachment on water ways. Plate 1-4: Structural encroachment on water ways

The increase in population of the urban areas in the country has led to the volume of waste generated. The Municipal Assemblies can however not manage the volume of waste generated and subsequently most of them end up in the water channels.

Furthermore there appears to be no coordination between the utility Agencies and the Departments responsible for the development of stormwater drainage system in the country. Water mains, electricity or telecommunication cable lines are constructed to cross inverts of storm drains. The effects of the utility service lines on solid waste transported in stormwater drains have led to the following:-

- Blockage of inlets of hydraulic structures such as culverts and bridges
- Reduction in the conveyance capacities of channels
- Reduction in the energy of flow in channels leading to siltation.
- Development of wide spread local floods

The above mentioned factors show the weakness of the conveyance approach alone to handle floods.

The figures 1-5 and 1-6 below illustrate the combined ill effects of waste transported and utility service lines on urban stormwater management in the country.

Plate 1-5: Debris at inlets of culverts

Plate 1-6: Effects of the activities of Utility Agencies

1.3 Justification of the study

The difficulty encountered in the current flood management approach which is centred on the direct conveyance of rainfall runoff by channelization is an indication that there is a weakness in the approach. In recent years several urban communities of the country and especially the residential estates in the Mamahuma basin of Tema have experienced serious floods resulting in the loss of property and lives. Rapid urbanization in the country has created the pressure on the use of perceived cheap lands on flood plains in urban areas and thus increasing the pressure on existing flood management structures. In Tema for example the expanding industries has required a large workforce which in turn has required housing. The areas best suited were the flat and easily developed flood plains along the Mamahuma River, areas which are prone to flooding. Another factor influencing the expansion of residential development to the Mamahuma basin is that people were getting fed up with the life in central business areas of the city and hence the ever increasing demand to the peri-urban areas of Mamahuma.

This mass movement to the uplands of the Mamahuma has created pressure on the existing drainage infrastructure downstream due to increase in stormwater runoff. To continue relying on channelization alone for flood management will require that most existing stormwater channels be expanded to accommodate the increasing flow. Achieving this objective will become extremely difficult as the areas for expansion are already built up. The problem has been created, hence the need to find alternative approach for delaying and attenuating the increased peak floods in the management of urban floods. This state of affairs is also developing in many urban areas in Ghana.

1.4 Objectives

The overall objective is to demonstrate the use of reservoir storage effects for delaying and reducing flood peaks to enhance management of urban floods in Ghana.

1.4.1 Specific objectives

To fulfill this overall objective the following specific objectives will be met:

- Explore the use of reservoir storage as a possible flood mitigation method
- Use an appropriate rainfall- runoff model to generate a hydrograph

- Route the hydrograph in a reservoir to predict its effect on the downstream channel.

CHAPTER TWO

2 Literature Review

This chapter explores the use of reservoirs in urban flood management and looks at other forms of urban flood management methods. It further reviews some of the available forms of rainfall runoff analysis methods. The chapter begins with the use of storage effects in flood management, and then some other important urban flood mitigation methods in the global trend. It is followed by review of rainfall runoff analysis methods and next by models that route the hydrograph through storage reservoir to predict its effects on channels and flood plains. It is finally followed by flood management approaches in Ghana.

2.1 Storage in flood management

Due to the rapid change in the land use of the urban area, urbanization has the tendency to increase both the volume and the velocity of rainfall-runoff. Various efforts have therefore been made to offset these effects in other to enhance the quality of life in urban areas by protecting human life and reducing flood hazard. Storage of stormwater in the basin provides one means of managing the increases in the storm volume by reducing flood peaks and redistributing the flow hydrograph.

Storage effect retarding the flood peak arrival time involves detaining or slowing runoff as means to reduce flood peaks and the storage in a pond or reservoir can be considered as a function of outflow discharge. In mitigation runoff peaks and volume in the global trend, storage in one form or the other is dominant. The most common storage systems used are

the Detention storage, Retention storage, Dams and even highways are sometimes designed with roadside swales for mitigation of runoff peaks and volume. In some states such as the USA and UK, regulations require localities to store runoff in developing urban areas (Bedient, Huber, 1992).

2.1.1 Detention storage

Detention storage is a form of storage of storm water runoff for a short period, usually less than a day, before it is released at a lower controlled rate. Detention storage may be provided at one or more locations both above ground and below. These locations may exist as impoundments, collection or conveyance facilities and on-site facilities, such as rooftops, parking for pavements, and basins, may be reservoir which is provided with a fixed or uncontrolled outlet (Interim Guidelines for Runoff Procedures, 1988). They are essentially useful for small streams and operate to reduce the flood peak by providing short-term storage of runoff and by restriction of the outflow rate. The discharge capacity of a detention basin with full reservoir is equal to the maximum flow which the channel downstream can convey without causing serious flood damage. The reservoir capacity is equal to the difference of flow volume of the design flood and volume of water released during the flood. As the flood occurs, the reservoir fills and the discharge increases till the flood has passed and the inflow becomes equal to the outflow. Release of stored water is continuous but at a lesser rate than inflow. Below is a sketch diagram of a detention reservoir.

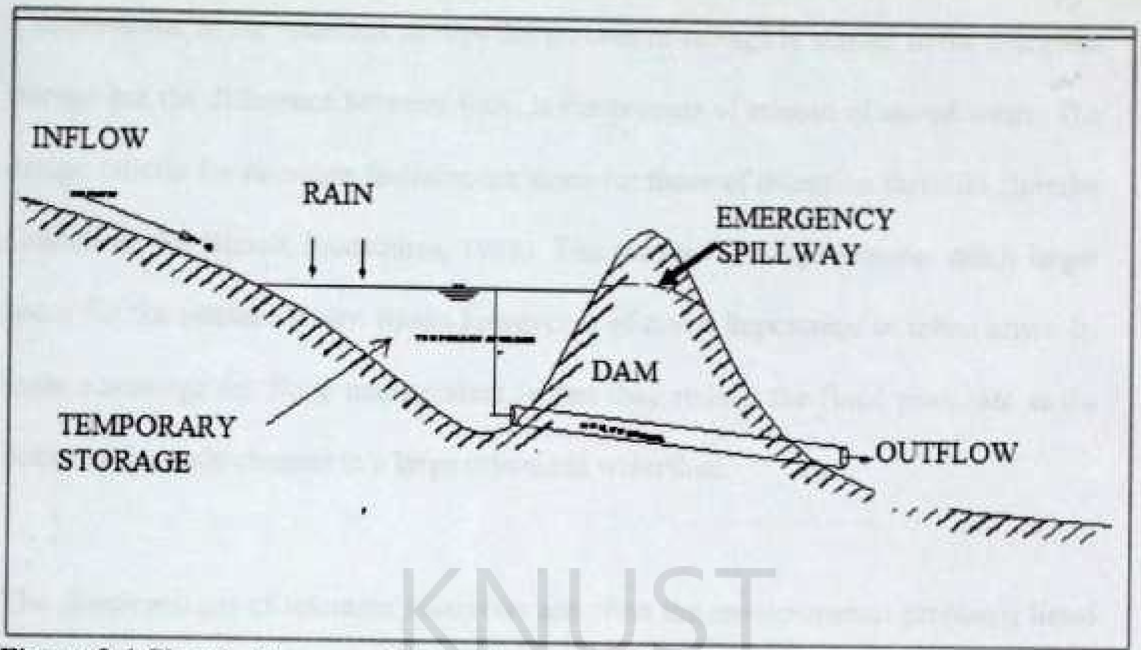


Figure 2-1 Sketch diagram of detention reservoir

In UK it is commonly used for minor flood alleviation and proponents of new developments are required to install detention basins to prevent exacerbating of downstream flood problems. (Wurbs and Wesley, 2002).

The advantages of Detention reservoirs are:-

- They reduce the flood peak rate at the outlet of the main channel in a large urbanized watershed
- They ensure draw down of the reservoir after a flood and prevent the use of the reservoir for conservation purposes at the expense of flood control.

2.1.2 Retention storage

Retention storage involves the storage of stormwater runoff for a period of several days or more and often is characterized by low release rates both during and after the rainfall. The storage is usually provided within a surface basin and controlled by a dam. The stored water may be removed by infiltration or by evaporation or slowly into a storm drain. In the retention storage the process of storage is similar to the detention storage but the difference between them is the process of release of stored water. The design criteria for retention facilities are same for those of detention facilities (Interim Guidelines for Runoff Procedures, 1988). The retention storage requires much larger space for the retained water. Space however is of much importance in urban areas. Its main advantage for flood management is that they reduce the flood peak rate at the outlet of the main channel in a large urbanized watershed.

The disadvantages of retention reservoirs are often the environmental problems listed below which can result if proper maintenance is not observed:-

- The failures of retention basins might result in loss of life and minor damage to homes, buildings and public utilities.
- Vegetation may grow in the presence of high nutrients stored water to becomes nuisance to the public
- Pollution of water quality due to discharge of raw sewage from septic tanks or open defecation.

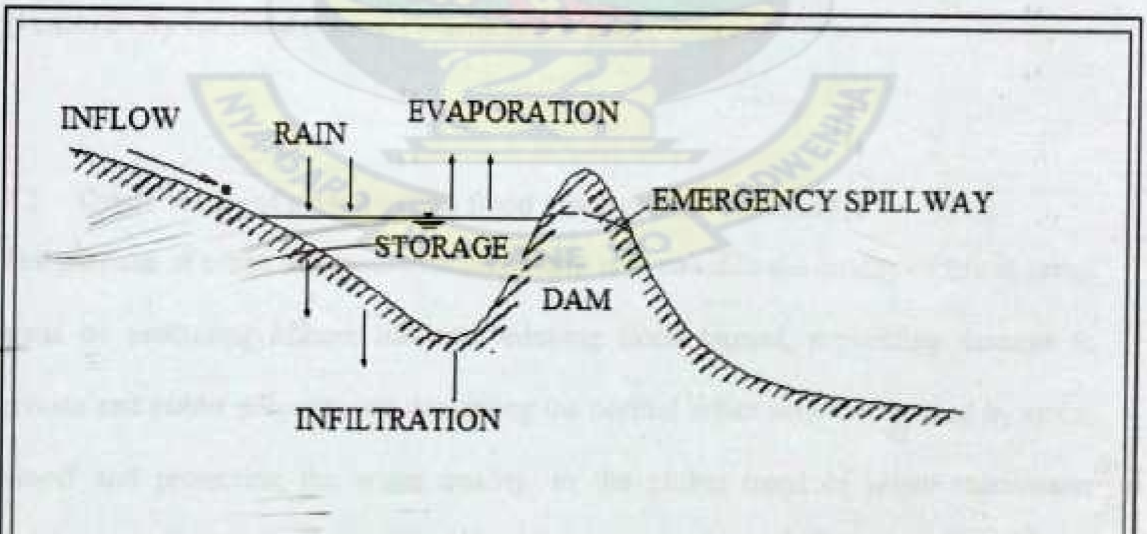


Figure 2-2 Sketch diagrame of retention reservoir.

2.1.3 Dams

Dams involve the storage of floodwater in reservoirs. The stored water is released in a controlled way over an extended time so that downstream channels do not get flooded. Dams offer one of the most reliable and effective methods of flood control. Many dams are multipurpose (e.g. water supply, hydropower, flood control and irrigation) and flood mitigation is usually a secondary objective. A major limitation of the dam is scarcity of suitable reservoir sites and high cost. The Clywedog dam of England; Hirakud and Damodar valley corporation reservoirs of India; Hoover, Grand Coulee and Folsom dams of the United States are some good examples of multipurpose reservoirs which are operated for flood control. (Wurbs and Wesley, 2002).

In the United States there are numerous dams and reservoir projects to regulate streams ranging from small creeks to major rivers with storage capacities of several hundred cubic metres. There are about 516 reservoirs in the United States which are owned and operated by the United States of America Civil Engineers (USACE) and have a total controlled storage capacity of 272,100 million m^3 out of which about 43% is exclusively for flood control (Wurbs and Wesley, 2002).

2.2 Other forms of global urban flood management methods

The purpose of urban stormwater management is to enhance the quality of life in urban areas by protecting human life and reducing flood hazard, preventing damage to private and public property and disrupting the normal urban activities caused by storm runoff and protecting the water quality. In the global trend of urban stormwater management, many developed countries are moving away from the traditional approach in urban drainage which is to protect against flooding by collecting and conveying stormwater runoff conveyance improvement to the management of stormwater runoff.

Apart from storage reservoirs some other urban flood mitigation measures are: - Levees or flood embankments, river diversions and channelization which are also known as structural methods. Flood warnings, development control, flood insurance, financial incentives, flood proofing, public information, emergency services and disaster aid are also some non structural methods.

2.2.1 Levees or Flood embankments

Levees or Flood embankments are earthen banks constructed parallel to the course of the river to confine it to a fixed course and limited cross sectional width. The heights of the levees will be higher than the design flood level with sufficient free board. The confinement of the river to a fixed path frees large tracts of land from inundation and consequent damage. This is most extensively used as a structural measure in the global world because it is relatively cheap and easy to construct. It has a very long relatively successful history. In the eyes of developers and planners the levees make scarce land flood free and available for unrestricted development (Wurbs and Wesley, 2002). While the protection offered by a levee against flood damage is obvious, what is often appreciated is the potential damage in the event of a levee failure. The levees being earth embankments require considerable care and maintenance. They also obstruct the drainage of low lying areas creating local ponding which needs special works underneath the levee for proper drainage of the low areas. In the event of being overtopped, they fail and the damage caused can be enormous and more than what it would have been if there were no levees (Wurbs and Wesley, 2002)

2.2.2 River diversions

Guided by the topography part of a flood may be diverted into a natural or man-made channel. Where ever they are feasible river diversion offers an economical alternative to other structural urban flood control measures. In Britain it is almost the norm for minor urban streams. There is a long history of major diversions in the East Anglican fens. In Andhra Pradesh in India a diverted channel has been constructed to transfer a part of flood water of the river Budamaru to river Krishna to prevent flood damage to the urban areas lying on the downstream reaches of the river Budamaru. (Subramanya, 1995)

2.2.3 Channelization

A marked reduction in the stage of a specific reach along a stream can be achieved by improving the hydraulic capacity of the channel. Channelization involves widening and deepening of the channel to increase the cross-sectional areas as well as its flood carriage capacity; reduction of the channel roughness, by cleaning of vegetation from the channel perimeter, or concrete lining of the channel bed and banks. Most urban centers use this method.

2.2.4 Flood Warnings

The function of the flood warning is the provision of an effective flood forecasting and warning services. Flood early warning systems are real-time event reporting system that is carried out by installing rain gauges and stream gauges at critical locations throughout water sheds that contribute flow to flooding in the urban areas. Data from the gauges are transmitted to a base station for analysis, display, and storage. The flood warning is disseminated among the concerned persons at the earliest possible time and at the fastest speed possible by the use of television, radio, and internet (American Society of Civil Engineers(ASCE) 1992).

In the lower Colorado River in Texas for example the data collection system used is called a Hydrometeorological Data Acquisition system (Hydromet). The system is designed to automatically acquire river level and meteorological data from each remote terminal unit (RTU); telemeter this data on request to the central station via the UHF/ microwave radio system; determine the flow rate at each site by using rating tables stored in the central system memory; format and output the data for each site; and maintain a historical file of the data for each site which may be accessed by the local operator, a computer, or remote dial up telephone (Subramanya ,1995)

2.2.5 Development control

Development control is achieved by restricting land use in flood prone areas (often designated as 10-year or 100 year flood plain) to compatible uses such as parks, agriculture etc. Although some damages may still occur during flooding, major damage to urban infrastructure is usually avoided (ASCE, 1992)

2.2.6 Flood Insurance

In the United States the National flood insurance programme was established in 1968 to restrict development in flood prone areas and provide federally- backed flood insurance. The programme is administed through the Federal Insurance Administration and Mitigation Directorate of the Federal Emergency Management Agency which is equivalent to the National Disaster Management Organization in Ghana (NADMO).

Over 19,000 communities representing most of the urban areas in the US participate in the programme (ASCE, 1992)

2.2.7 Flood proofing

Flood proofing involves raising the foundation of structures above a designated flood elevation using either imported earth fill material or pilings or stilts (Wurbs and James, 2002)

2.3 Review of some available rainfall - runoff analysis methods

Determining the hydrological response of urban catchments is of great interest for urban water management. The hydrological responses may be determined by means of either rainfall or runoff data. Most urban catchments in Ghana are ungauged, and their hydrological response can be deduced from available rainfall data which is the primary source of all waters.

When rain starts falling on a more or less pervious area, it is consumed but as rain continues (or snowmelt), the soil surface becomes covered with a thin film of water and runoff is generated and governed by the topography to find its way into the stream channels. The rainfall-runoff modeling of a river basin can be divided into two processes: the production function and the transfer function. The production function determines the proportion of gross rainfall actually involved in the runoff (i.e. the deduction of evapotranspiration, initial loss, infiltration and detention – storage). The transfer function spreads the net rainfall over time and space in the river basin.

Channel flows may be computed using either observed stream flows or rainfall techniques. Even though the Hydrological Services Department is responsible for collection of stream flow data throughout the country, there is a scanty or no stream flow data on the streams in the urban areas in Ghana.

There are various methods of modeling rainfall runoff and the most common ones being:-

- The Rational method
- Hydrographs
- Computer models

2.3.1 Rational Method

Rainfall runoff modeling in urban condition for the first time started around 1889 when Kuichling in Rochester, USA, obtained a certain rational solution of storm water drainage. Later in 1906 another similar methodology by Lloyd –Davis was proposed in Birmingham UK. Both approaches because of their similarity became known as the "Rational Method" (Chow et al., 1988).

The concept behind the method lies in the assumption that a steady, uniform rainfall rate will produce maximum runoff when all parts of a watershed are contributing to outflow, a condition that is met after the rainfall duration equals the time of concentration *tc*.

Even though it has frequently come under much criticism for its simplistic approach no other drainage design method has received such widespread use. The maximum drainage area for which the rational method is applicable is subject to judgment, but is found to be useful for small catchment areas from in the range of 0.04 km^2 to $12km^2$

(Wurbs and James, 2002). The rational method remains to be extensively used in Ghana.

2.3.2 Disadvantages of Rational Method

The magnitude of urban flood depends on the characteristics of the storm producing it and on the condition of the water shed at the time of the storm. However the Rational formula ignores important factors of antecedent conditions, variable loss coefficients and spatial and temporal rainfall variations. There is no consideration for the transfer function of runoff even though a more reliable prediction of the design hyetograph to obtain design hydrographs are necessary tools in obtaining more accurate rainfall-runoff results, especially when stormwater management systems are required. The rational method is a peak-runoff model that is not concerned with what happens to the other parts of the hydrograph after the peak has been reached for a particular design storm. It is therefore useful for sizing runoff conveyance structures such as drains, culverts, etc. in small catchments.

2.3.3 Modified Rational method

Some of the shortcomings of the rational methods were overcomed in the Transport and Road Research Laboratory (TRRL) hydrograph method introduced in 1963. The Wallingford Procedure (National Water Council, 1981) refined the hydrological principles resulting in a modified rational method. This method was developed so that the concepts of the rational method could be used to develop hydrographs for storage designs, rather than just flood peak discharges for storm sewer design. To reflect the spatial coverage of design storm over a basin an area reduction factor (ARF) is next applied to the point rainfall of the required return period and duration. The modified method can however be used for the preliminary designs of detention storage for watershed of up to 1.5km^2.

2.3.4 Hydrograph methods

The concept of the hydrograph is a tool for the transfer function of the net rainfall over time and space in the river basin in order to identify the general rainfall runoff characteristics and to provide the basis for the analysis of its component elements. A graphical representation of rainfall runoff data is processed in the form of hydrographs.

The graphical representation of the flow is a function of time rate (m^3/s) (Ghosh, 1986) therefore a continuous run-off model. Since urban areas respond quickly to rainfall transients in contrast to natural catchments which dampen out short –term fluctuations, to compute hydrograph special effort is required to obtain adequate data on rainfall events in the project area. The unit hydrograph is a model that can be used to derive the hydrograph by means of convoluting the unit hydrograph with excess rainfall (Nash, 1957 Sivapalan et al., 2002).

2.3.5 Unit Hydrograph (UH)

The unit hydrograph is defined as hydrograph which is produced by a rainfall of a given duration uniformly distributed over the catchment which produced a direct runoff volume of unit depth. The unit volume is equivalent to covering the watershed to a depth of 1 unit (cm or inch) (Reddy, 2007).

The unit hydrograph takes solely into account runoff transfer processes within the catchment, and does not include the transformation of rainfall into excess rainfall, i.e. runoff production. As such, it may be viewed as a hydrological fingerprint, or signature, representing the transfer function of the catchment (Beven, 2001). The unit hydrograph concept, initiated by Sherman (1932), applied only to that watershed and for the point on the watershed where stream flow data were measured by means of convoluting the unit hydrograph with excess rainfall (Nash, 1957; Sivapalan *et al.*, 2002). The process of combing the rainfall excess of each duration hour increment of rainfall with the duration hour unit hydrograph is called convolution (Ralph Wurbs and P. James, 2002). Hydrologists have developed various synthetic methods for identifying the unit hydrograph for other locations on the stream in the same watershed or for nearby watersheds of similar characters. These methods have been based on:

(i) Those relating hydrograph characteristics, peak flow rate and base time to watershed characteristics(Snyder,1938)

(ii) Those based on a dimensionless unit hydrograph (SCS,1972)

(iii) Those based on models of watershed storage(Clark,1943) and more recently,

(iv) Geomorphological features of the catchment.

The general frame work for the unit hydrograph approach is:

- For a particular watershed and rainfall duration the shape and time base for a runoff hydrograph are same regardless of the rainfall amount.
- The magnitudes of hydrograph vary between rainfall events in direct proportion to runoff volume.

- The magnitudes of runoff volume and flow rates are dependent on the antecedent moisture condition but the time distribution or shape of unit hydrograph does not depend on the antecedent moisture condition.

One very important assumption upon which the unit hydrograph is based is that the catchment has a linear time invariant response, however it is observed from a common fact that the ground condition at the time of the flood affects the speed with which the water runs over the ground or through the soil. In this vain hydrographs that will be derived from large floods will show shorter time element than those of smaller floods (Ghosh, 1986).

The unit hydrograph method has not received much application in Ghana probably due to difficulty of obtaining data on single interval isolated and uniform rainfall and the labour of the unit hydrograph computations. The procedure to derive the unit hydrograph involves collecting the rainfall data of the drainage basin and scanning through to find a storm of desired duration to give a firmly uniform distribution over the watershed and also within its duration and having a runoff volume of nearly 1 cm or greater. Many watersheds do not have reliable rainfall data in the country and this has deterred most engineers from using the approach. Today the trend has improved and there are number rainfall synoptic stations in the urban watersheds. However, since old habits die hard no serious attempts have been made to revisit the unit hydrograph methods but to stick to the easily applied Rational Method.

The unit hydrograph has a limitation on its application over basins larger than $10{,}000km^2$ because of the uniform rainfall distribution upon which the theory is based. In the real sense rainfall distribution may vary over a large basin.

2.3.5.1 Instantaneous Unit Hydrograph (IUH)

The hydrograph obtained when the effective rainfall is of unit amount and its duration is infinitesimally small which gives an impulse response function is called the Instantaneous Unit Hydrograph (IUH). It is a hypothetical response to a unit depth of effective rainfall deposited instantaneously on the watershed surfaces, as the rainfall is applied to the drainage area in zero time.

It is useful because the IUH characterizes the watershed response to rainfall without reference to the rainfall duration. This has therefore made the IUH to be related to its watershed geomorphology as can be seen in the mathematical modeling related to the watershed geomorphology known as Geomorphologic Instantaneous Unit Hydrograph (GIUH)

2.3.5.2 The Geomorphologic Instantaneous Unit Hydrograph (GIUH)

The transfer function of the net rainfall over time and space in the river basin can be modelled using the approach of the geomorphologic instantaneous unit hydrograph (GIUH). In recent years, the geomorphologic instantaneous unit hydrograph approach to evaluate the runoff from a watershed due to different storm patterns has gained wide acceptability for watersheds with scanty data. (Bhadra et al, 2008)

The GIUH relates the geomorphologic features of the catchment such as the catchment geometry, and topology, which are available by the use of geographic databases and geographical information system (GIS) processing (Rodriguez-Iturbe and Rinaldo, 1997).

The principle was developed on the basis of:-

- geomorphological state transition
- a numerical experimentation focusing the strahler's ordering scheme(Strahler, 1952).

By these two references the unit hydrograph is interpreted as the probability density function of the flow travel time to the catchment outlet.

In the study of urban catchments, consideration is given to the landuse, network as well as the various hydraulic conditions of the network, thus further development of the GIUH theory has incorporated the variations in:-

- hydraulic conditions
- dispersion in the drainage network (Snell and Sivapalan, 1994)
- rainfall field spatial variability (Rodriguez-Iturbe et al., 1982),

in addition to distinguishing the hill slope and network travel times among the whole travel time to the outlet (Gupta and Mesa, 1988; Rodriguez-Iturbe and Rinaldo, 1997).

The GIUH modeling approach is valuable in an urban environment featuring abundant geographic data. Rodriguez et al., 2004 in his study concludes that in an urban setting, characterized by its rapid evolution, the morphology-based methods could be especially useful in evaluating urban development impacts on hydrological disasters and risks.

The effectiveness of geomorphological model is actually revealed in rainfall-runoff modeling, where hydrologic data are desperately lacking, just as in ungauged basins (Fleurant et al, 2006).

The GIUH makes it possible to forecast the hydrograph shape and runoff variation versus time at the basin outlet, by analyzing the travel path of runoff particle. The path consists of the route through hill slope areas and channels leading to the outfall. The probability of this water particle follows a certain path among all possible paths from stream of lower order to those of higher order according to Strahler's order scheme (Horton, 1945, Schum m, 1964).

There is rapid hydrological response of urban catchments in Ghana due to the ongoing expansion of urbanized areas and this has placed increasing emphasis on related water management problems such as flooding and pollution control. Population densities and the size of these areas have led to considering the detailed behavior of water drainage systems at various scales. Changes in both landuse and water policy have made it necessary to take into account the rapidly-evolving morphology of urban catchments. Addressing urban flood management issues in an efficient manner thus requires special adaptation of hydrological modeling practices. Even though most of the urban watersheds in the country are ungauged, topographic maps and remote sensing images for these watersheds are now available in the country in different scales and for different times hence the choice of the Geomorphologic Instantaneous Unit Hydrograph (GIUH) to derive the hydrograph for the study.

2.3.6 Computer Models as tools for hydrograph analysis

Much as the computer has brought revolution in many fields so has it also in watershed hydrologic studies. Widespread access to computers and the investigation of sampling programs have led to the development of urban runoff models that have been calibrated and validated by comparisons with field data.

Many available computer models for simulating watershed hydrology are generalized for broad applicability with values for parameters being input by the model user to describe a particular watershed. The first uses of hydrologic models for urban flow simulation followed the development of the RRL Model (Road Research Laboratory) in the U.K., and the Chicago Model in the U.S. (Watkins, 1962, Kiefer, 1970).

Linsley, (1971) and Maclaren,(1975) made comparative studies of urban runoff models available in the early 1970's. During the last 25 years, there has been a proliferation of computer models that can be used for various aspects of the design of stormwater collection, storage and conveyance structures. Computer modelling became an integral part of hydrologic and hydraulic design and analysis in the early to mid 1970's when several federal agencies in the USA, began the development of software. Some of the most notable accomplishments of software development during that time were:

United States of American Civil Engineers (USACE) Hydrologic Engineering Centre

HEC-1 (Flood hydrograph package) (U.S.A.C.E., 1973)

HEC-2 (Water surface profiles) (U.S.A.C.E., 1976)

SWMM (Stormwater Management) Model (Metcalf and Eddy, 1971)

The Hydrologic Modeling System (HMS) developed by the Hydrologic Engineering Center is probably the most extensively applied of all the watershed computer models. In the HMS for example

- Rainfall depths for each time increment of a storm can be input in various formats
- Rainfall hydrograph for each sub basin can be developed with unit hydrograph.
- A unit hydrograph can be either input or synthesized by the model.

The model has been applied to watersheds ranging from small urban areas less than $1km^2$ to large river basins of several thousand km^2. Computer models are simplification of real world processes referenced to some observed watersheds and factors often used do not have direct bearing on other basins. Each individual model has its empirical basis, premises, assumptions and procedures for applying and is therefore applied with circumspection and sound engineering judgment.

The main differences between these computer models can be summarized in the below:-

Table 2-1 A summary of differences in some computer models

	Major system modeling	Minor system modeling	Hydrologic features	Hydraulic features	Ware quality features
Hydrologic software					
HEC-1	X		X		
Hydraulic software					
HEC-RAS	X			X	
EPA SWMM	X	X	X	X	X

2.4 Flood studies in Ghana

Flood studies in Ghana had mainly concentrated mainly on the five largest cities in the country with only a new interest cropping out in some other urban areas in the three Northern Regions due to the recent floods in those areas. So far drainage master plans have been prepared for five major urban areas in the country namely:-

- Accra drainage master plan
- Takoradi drainage master plan
- Tamale drainage master plan
- Kumasi drainage master plan
- Tema drainage master plan

The Tema drainage master plan was the first of its kind in Ghana and it was carried out by consulting firm Messrs Doxiadis during the development of the Tema Township (Doxiadis, 1952). This was followed by the Accra drainage master plan development (NEDECO, 1963) consulting company in 1963. The Tamale drainage master plan was prepared by Messrs Dar Al Handash in association with Twum Boafo & Partners (Dar al Handasah and Twum Boafo- 1995). The Kumasi and Takoradi drainage master plans were prepared concurrently by Messrs Watertech and Mott Macdonald (Watertech and Mott Mac Donalds, 2006) and Municipal Development collaborative Ltd., Ghana in association with Mott MacDonald,UK in 2006, respectively.

The terms of references for the preparation of these drainage master plans stipulated flood mitigation methods of conveyance approach only (Ministry of local Government, 2006)

All the drainage master plans prepared mainly focused on the central business areas of the various cities, hence with the fast expansion of the urban communities in the country these master plans needed to be updated to cover the entire watersheds of the communities. Typical cases were the updating the existing drainage master plans of Accra in 2004 and the Tema drainage master plans in 2006.

2.5 Flood management practices in Ghana

Flood management practices in Ghana involve the following:-

- Channelization
- Stream diversion
- Disaster management (by National Disaster Management Organization(NADMO)

The first two approaches are structural measures carried out in an effort to mitigate flooding while the last approach, a non structural measure, which is applied when flood disasters occur.

The Hydrological Services Department (HSD) of the Ministry of Water Resources, Works and Housing (MWRWH), is the agency charged with the responsibility of planning, development and maintenance of primary drains in the country, other Government departments such as Ministry of Local Government and Rural Development (MLGRD) and Department of Urban Roads (DUR) play part in urban flood management. Coordination among these institutions has not been very strong to ensure a common approach in flood management.

The HSD has outlined the following as some of the causes of floods in the urban communities in Ghana (HSD, 2008):

- Increase in runoff concentration as a result of rapid urbanization.
- Inadequate capacities of conveyance channels.
- Encroachment of building structures on waterways and flood plains.
- Inability to introduce peak discharge control measures during floods.

2.6 Funding for flood mitigation works

Flood control works in the country has been the sole responsibility of the government. Funding therefore has become very difficult especially where governments would like to utilize funds for projects that are visible like roads, water and housing, even though these facilities could be destroyed in a single extreme flood event. Some projects and the costs levels are shown in the table below:-

Table 2-1: Some cost of drainage projects

Project description	Locality	Year	Total channel length (km)	cost(million$)	source
Stormwater Drainage Project	Tamale	2002-2004	17	9.24	Addo AL-HANDASA – Consultant, 2008
Odaw Drainage Project	Accra	2003-2007	8.65	69.62	Howard, 2008 Ministry of Local Government

CHAPTER THREE

3 STUDY AREA

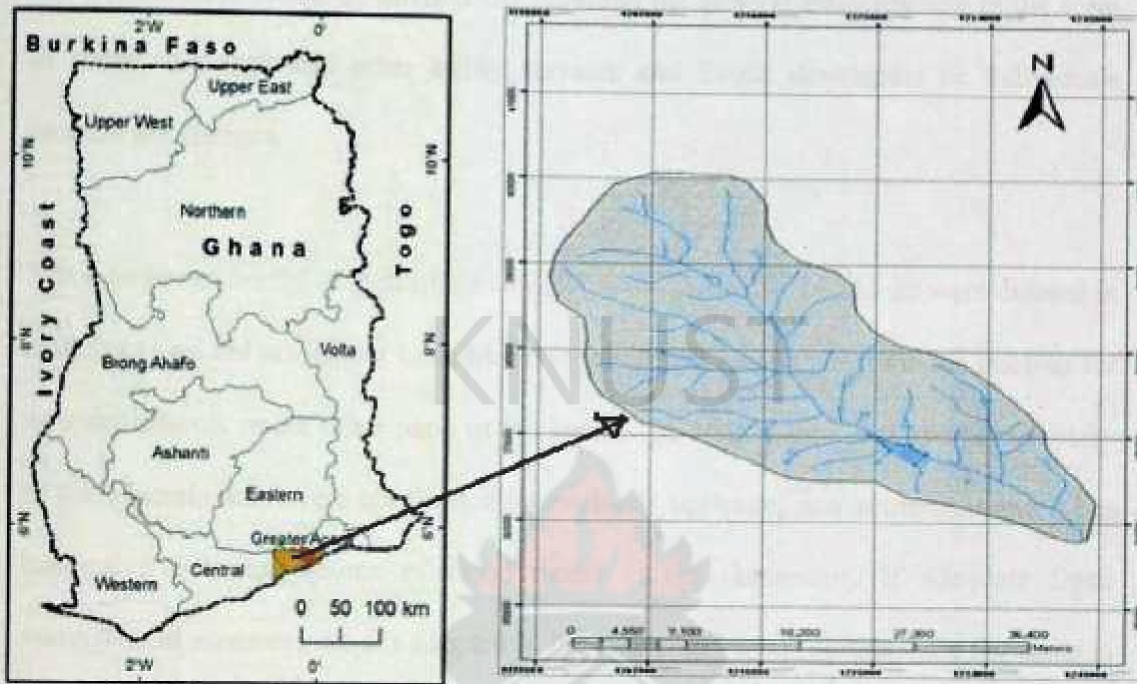


Figure 3-1: Map of study area

The study area falls within the Tema Municipality of Ghana. It is a sub basin of the Sakumo basin known as the Mamahuma basin. Tema is an industrial city of Ghana and has one of the busiest harbours in West Africa and also serves as an administrative capital of the Tema Municipal Assembly.

The 2000 Ghana Population Census and household survey puts the total population of the Tema Municipality at 511,459 made up of 252,109 males and 259,350 females. The municipality is also known to have a high population growth rate of 2.6%, which can be attributed to the rural to urban migration, thus making Tema to be overwhelmed with rapid urbanization, (Ghana Statistical Services, 2000).

Due to the pressure on the existing residential facilities in both Tema and Accra, a downstream section of the Mamahuma basin was acquired by the Tema Development Corporation (TDC) and developed as settlements. The settlements were developed on site and services bases, in which TDC provided the general infrastructure in the form of roads, drainage and other utility services and Estate developers or individuals provide the shelters.

Three large residential communities namely communities18, 19 and 20 were created in addition to an old settlement Lashibi. Soon this settlement has become the nucleus for new settlements in the other parts of the basin. The Mamahuma and another tributary of the Mamahuma have a confluence immediately upstream new settlement and it has become a potential source of flood threat to the community if adequate flood management measures are not adopted in the upstream basin. Furthermore the lands in the middle and upstream sections of the basin have been identified as the new direction for expansion of the Tema city (TDC, 2007).

The Mamahuma basin is bordered to east by the Dzorwulu basin at Ashaiman, to the south by Sakumo lagoon, to the west by the Onukpawahe basin. It can be identified by coordinates as 05^0 39' 34"N; 00^0 03' 51" W; and 05^0 46' 00"N; 00^0 10' 43" W. The Mamahuma stream takes its source from the Aburi hills near Oyarifa and runs through Frafraha village, East Adenta, West Ashaley Botwe, Nungua farms and finally crosses the Accra-Tema motorway to the confluence with the Onukpawahe stream. The stream has a number of retention reservoirs none of which is regulated for flood control. It has an area of $87km^2$, and with the current rate of urbanization it is anticipated would have highly been urbanized. This basin is still in its natural state with only about 2km of channelization on its tributary drains at Adenta. The Mamahuma stream has a confluence with another stream, the Onukpwahe, immediately upstream of newly developed settlements of communities 18, 19, and 20. Plate 3-1 shows the confluence of two streams. The combined flow of the streams then crosses the Ashaiman –Nungua Road to discharge into the Sakumo Lagoon.

Plate 3-1: Confluence of Mamahuma and Onukpawahe

3.1 Climates

The study area is characterized by two rainy seasons separated by a short dry spell occurring in mid July to August. The average annual rainfall in the study area is between 730mm to 800mm. More than 50% of the total precipitation falls in the period from April through July and from September to November as minor season. Average monthly temperature varies between 24^0 C and 30^0 C.

There is a short rainfall record for Tema; however Accra, Aburi, Akropong and Ada which surround it have longer rainfall records. These records have been analyzed and intensity- Duration frequency (IDF) curves have been developed for Accra and Ada which are synoptic stations (Dankwa 1974). Daily rainfall records for Accra dates back to 1935.

The IDF curve refers to a rainfall of frequency of occurrence of a given maximum intensity. The procedure to obtain the IDF relation involves analyzing every storm in a year to find the maximum intensities for the various durations. Each storm gives one value of maximum intensity for a given duration. The largest of all such values is taken to be the maximum intensity in that year for that duration. Similar frequency analysis is carried out for other durations. Then from the results of these analyses graphs of maximum rainfall intensity against the return periods for the various durations are developed as the IDF curves (Jaya, 2007). Engineers and other interested parties are interested in frequency of rainfall for the purposes of drainage, reservoir, flood mitigation and other water resources facilities and this is the information derived from the IDF curves.

In this study the magnitude of the return period of 3rd-4th July 1995 rainfall event, which has been used for the study, is determined to compare with IDF values established for Accra which is generally used to express extreme rainfall events. The study area shares common catchment boundaries with the Accra synoptic station.

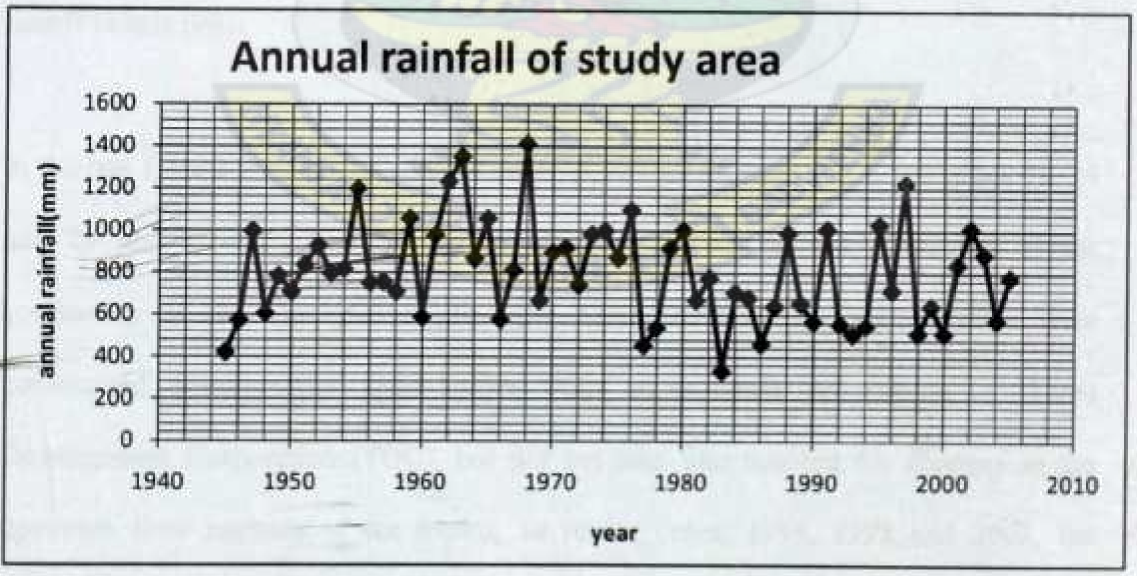


Figure 3-2: The Annual precipitation recorded for Accra from 1977- 2005 (Ref. Ghana Meteorological Agency)

Table 3-1: Floods in the study area

FLOODS IN THE STUDY AREA				
Date			Maxi. Rainfall Depth (mm)	Remarks
Day	Month	Year		
2^{nd}-3^{rd}	May	1988	158	Damage to property
3^{rd}-4^{th}	July	1995	249	Damage to property
13th	June	2002	148	Damage to property
4^{th}	July	2007	44	Damage to property

3.2 Recent Developments in the study area

The only human settlement in the study area before 1990 was at Ashaley Botwe. From mid 1990 settlements such as the Adenta Estate, eastern Ashaley Botwe, Frafraha and Lake Side Estates in the upstream of the Mamahuma began to spring up. As a consequence of the rapid urbanization of the upstream of both basins the rainfall runoff responses of the catchments have become radically altered due to the introduction of impervious surfaces in the form of concrete paving, tarmac, roofs etc, which inhibit infiltration and reduces surface retention. Thus the proportion of storm rainfall that goes to surface runoff is increased and the response is to produce more runoff in less time.

In the late 1990 high concentration of urban development such as Communities 18, 19 and 20 and Western Lashibi townships sprang up in the lower reaches of the confluence of the Onukpawahe and the Mamahuma. Storm water drains were constructed within these communities from an in-house programme by Tema Development Corporation (TDC), but did not take into account the changes in the upstream flow regimes of the basins. In recent years; 1995, 1998 and 2002, the settlements in the basin have experienced very high flood inundation resulting in the displacement of people as well as loss of properties. These frequent floods have raised the anxiety of the settlers in the communities and the authorities at large to find the causes of the flooding for the appropriate management solutions to be adopted.

3.3 Geology

The study area in general is underlain by the rocks of the Dahomeyan formation of Precambrian age, which form the basement complex of Ghana (Ghana, German Environmental and Engineering Geology Project-2004). They consist mainly of foliated biotite granite gneiss, granite augen gneiss-schist with occasional thin beds of essential stable minerals (quartz, garnet, hornblende, plagioclase feldspar), which weather into black clayey soils with low permeability and for the undeveloped portions of the area it can be supported only by grass- savanna and shrubs.

3.4 Land use and cover information in the study area

Limited resources and weakness of key state agencies over the years have made the task of preparing land use plans difficult. Various sub-urban areas both within the city and peripheral areas including the Mamahuma basins have developed without any realistic planning information. For the purpose of flood management analysis in the basin however five categories of land uses as shown in table 3-3 below were estimated.

Using the aerial photography map of 1999 of scale 1:25000, (Boamah, 2006) and conducting field surveys the estimates of the present day land use in the study area were estimated and compared with the 1999 estimates provided by the Geological Survey Department of Ghana as given in the Table 3.2 below:-

Table 3-2: Present land-use values in the Mamahuma basin

Land use	Area (m^2) as at 1974	% as at 1999	Area (m^2) as at 1999	% as at 2007
Total project area	120009989	-	120009989	-
Total built up area	4782273	3.99	35402947	29.50
Total Paved and road	3718820	3.1	28322357	23.60
Vegetative coverage	109617374	91.34	54964575	45.80
Total Water coverage	1891522	1.58	1324065	1.10

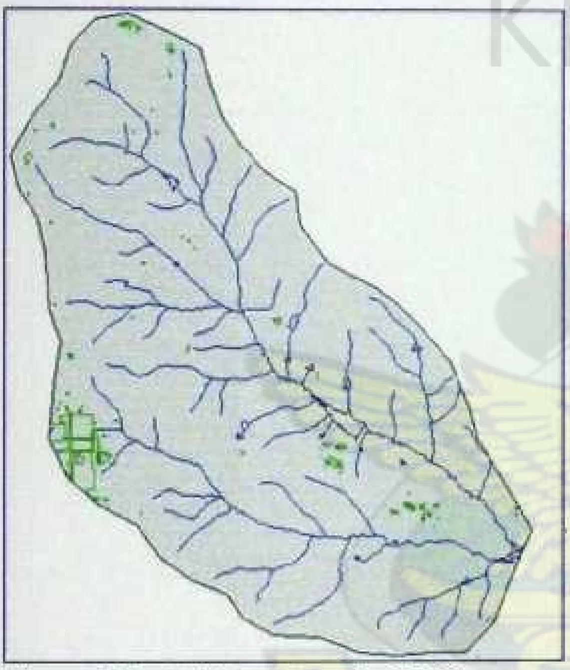


Figure 3-3: (a) Land use in 1989 **(b) Land use in 2007**

3.5 Soil characteristics

The soil data of the study area was obtained from the Soil Research Institute in addition to inspection at site. The soil type of the Onukpawahe/Mamahuma basins originates from the Dahomeyan system (metamorphic basement rocks of middle –late Precambrian age) and consists of quartz schist. Soil on top of elevations is observed to be lateritic, while clayey soil develops in flat and depressed areas. The clayey soil has very poor foundation and drainage properties because it expands and contracts according to moisture contents.

CHAPTER FOUR

4 METHODOLOGY OF RESEARCH

This chapter starts with a desk study which includes the Horton and strahler's stream ordering system from a GIS application as well as land use classifications and rainfall analysis procedures. A brief description of method used to generate hydrograph; carry out the field activities of Bathymetric surveys to establish the reservoir relationships; topographic surveys to determine the channel cross sections downstream of the reservoir. Finally, the use of the Hec-Ras model as a tool for hydraulic routing of inflow and outflow hydrographs through the channel downstream of the reservoir.

4.1 Desk study

Desk study involved collection of materials and information needed for the study in the project area and it includes the following:-

- Topographic map of study area,
- Information on rainfall,
- Soil characteristics,
- Landuse characteristics and
- Characteristics of drainage system – natural or man-made

The institutions liaised with for the needed information are: Survey Department, Meteorological Services Agency, Hydrological Services Department, Water Research Institute, Soil Research Institute, Geological Survey Department and Town and Country Planning Department in Accra and Tema.

4.2 Field topographic surveys of basin

Field surveys data include collection of land use and land cover survey as well as:-

- Topographic surveys to determine channel longitudinal and cross sectional profiles elevations
- Bathymetric surveys to establish the area-storage, elevation storage and elevation- outflow relationships of the reservoir.

The field surveys involved forming survey team for the field activities. Geographical Position System (GPS) equipment, total station survey equipment (Sokkia Set 3030R), an Echo Sounder (Humminbird Fishing System 727) and a digital camera (Sony cyber-shot 5.1mega pixels) were used to support this field work.

- The GPS Survey- the GPS equipment was used to obtain the co-ordinates of all established control points for the work.
- The Controls consist of concrete pillars fixed at vantage points along the channels. The obtained coordinates were used for the delineation of the channel configuration. Special controls were also established for the bathymetric surveys to enable the detail delineation of the Mamahuma reservoir.
- Total station surveys- This involved the picking of the ground elevations with respect to specific coordinates along the channel banks to determine channel longitudinal and cross sections configurations.
- Echo sounding- The Echo sounder equipment was used for bathymetric surveys to gather the elevations of the Mamahuma reservoir bed levels.

For the bathymetric survey two sets of survey teams were raised. One team was on ground carrying out the levels on the reservoir bank slopes while the other was on the reservoir to gather elevations on the reservoir bed using Echo sounding equipment. The following personnel and equipments were used on the Echo sounding team:-

- A boat measuring 3.0m long x 1.2m in middle section,
- 2 boatmen and 2 personnel on land to guide the direction of boat on reservoir
- An echo sounder and dry cell battery as a source of power supply to the Eco sounder.
- Sounder readings recorder, and a measuring tape

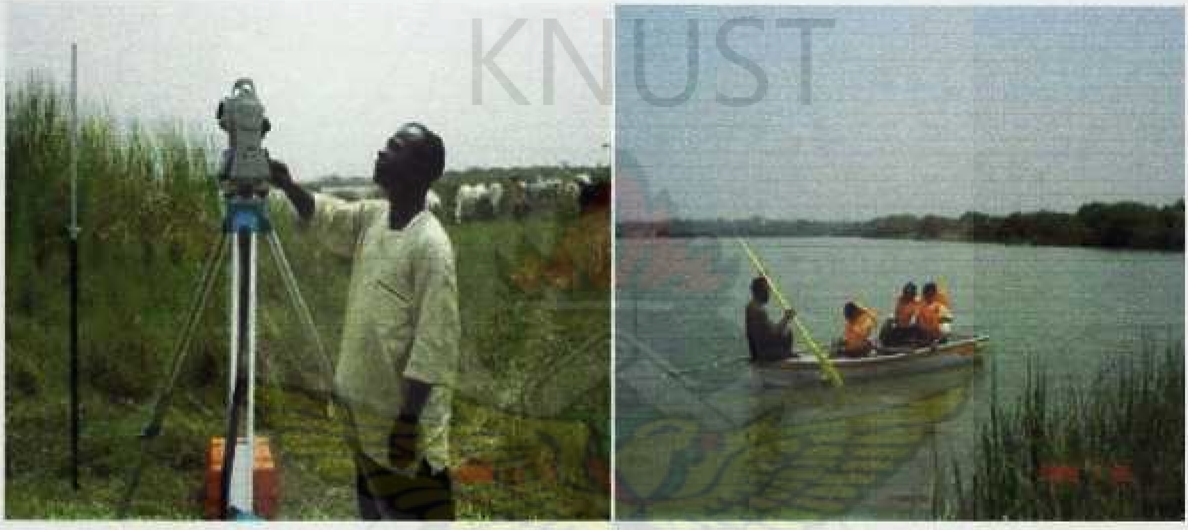


Plate 4-1: Topographic and bathymetric surveys

4.3 Rainfall data analysis

Rainfall data is collected and converted to effective rainfall, then used in the GUIH model through the process of convolution to derive the hydrograph. In the analysis of the rainfall data an important factor added is the antecedent conditions.

In the development of the Geomorphologic Instantaneous Unit Hydrograph (GUIH) the rainfall event of 3^{rd}-4^{th} July 1995 was used for the analysis. This event was selected based on extreme value analysis using the Gumbel's distribution function. It is recognized that other methods exist for maximum rainfall intensity analysis but Gumbel's method is preferred for its simplicity and rigid statistical method (Dankwa, 1974).

The figure below gives cumulative records of 3^{rd}-4^{th} July 1995 rainfall (Source: Water Resources Institute (WRI), 2000).

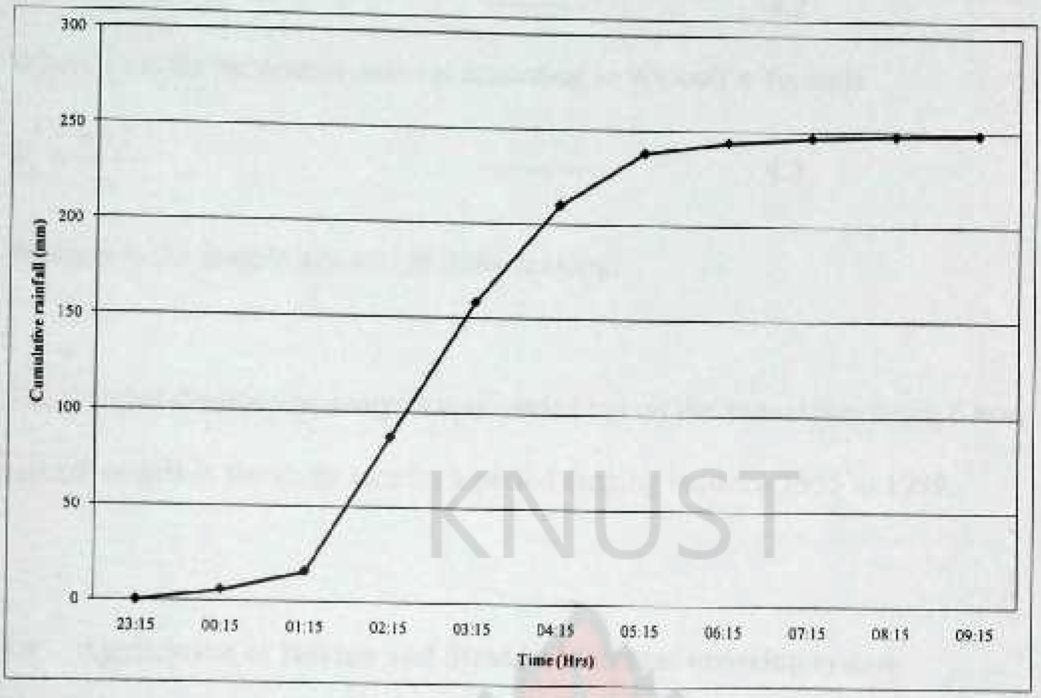


Figure 4-1 cumulative rainfall 3rd-4th 1995

4.3.1 The Gumbel distribution function

Gumbel distribution is perhaps the most widely used distribution for the estimation of floods of various recurrence intervals. According to Gumbel the probability of occurrence of an event X equal to or larger than a value of X_o is given by:

$$X_T = \bar{X} + K\sigma_x \qquad \text{---------} \qquad 4.1$$

X_T = extreme rainfall to be estimated for a return period

$\bar{X}$ = Mean of the sample

σ_X = standard deviation of the sample

K= Gumbel frequency factor read from tables.

$$y_T = -\left[\log_e \log_e\left(\frac{T_r}{T_r - 1}\right)\right] \qquad \text{----------} \qquad 4.2$$

Where Tr is the recurrence interval according to Weibull's formula

$$T_r = \frac{n+1}{m} \qquad \text{----------} \qquad 4.3$$

Where n is the sample size and m is the ranking.

The Gumbel distribution analysis was carried out on the annual maximum 6 hour rainfall events in the study area for a period ranging between 1955 to 1999.

4.4 Application of Horton and Strahler's stream ordering system

The first step is to digitize the catchment areas of the stream network from the topographic map of the basin. The next step is to digitize each sub catchment of channels in the basin. From the stream network the smallest recognizable channels, which normally flow only during the wet season are identified and designated as the stream order 1 (first order stream). Where two channels of the first order join a channel of the second order results downstream. Where two channels of order *i* join, a channel of order *i+1* result and the highest stream order in the basin is determined. Figure 4-1 below shows the Mamahuma basin with the Horton's ordering procedure.

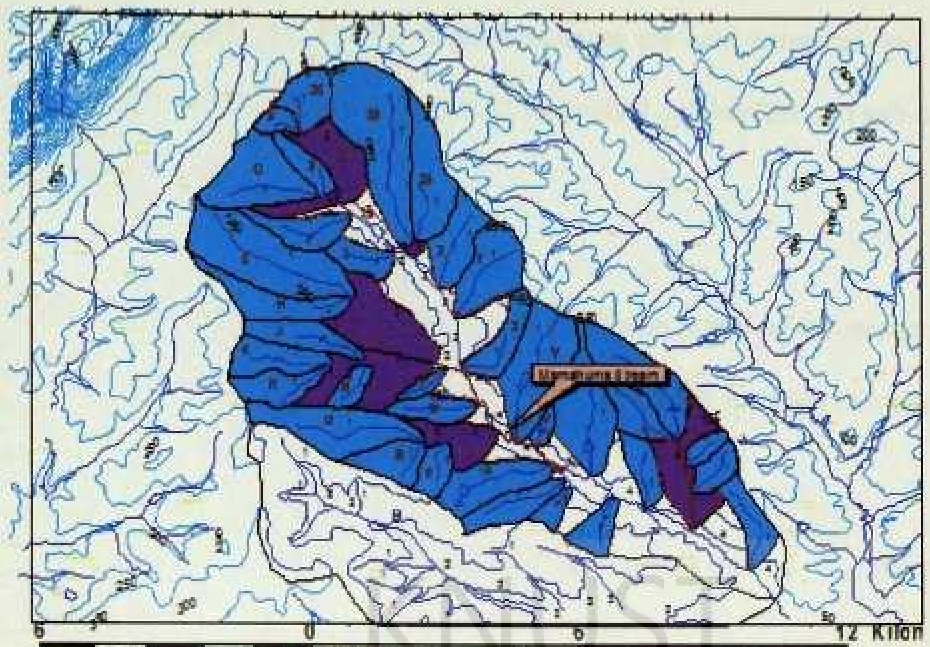


Figure 4-1: 1ST & 2ND order catchment-Mamahuma

4.5 GIUH development

The Fig.4.2 below gives the schematic representation of the GIUH process. The rainfall data and land use characteristics and geomorphology of the stream basin as input to derive the hydrograph (Q).

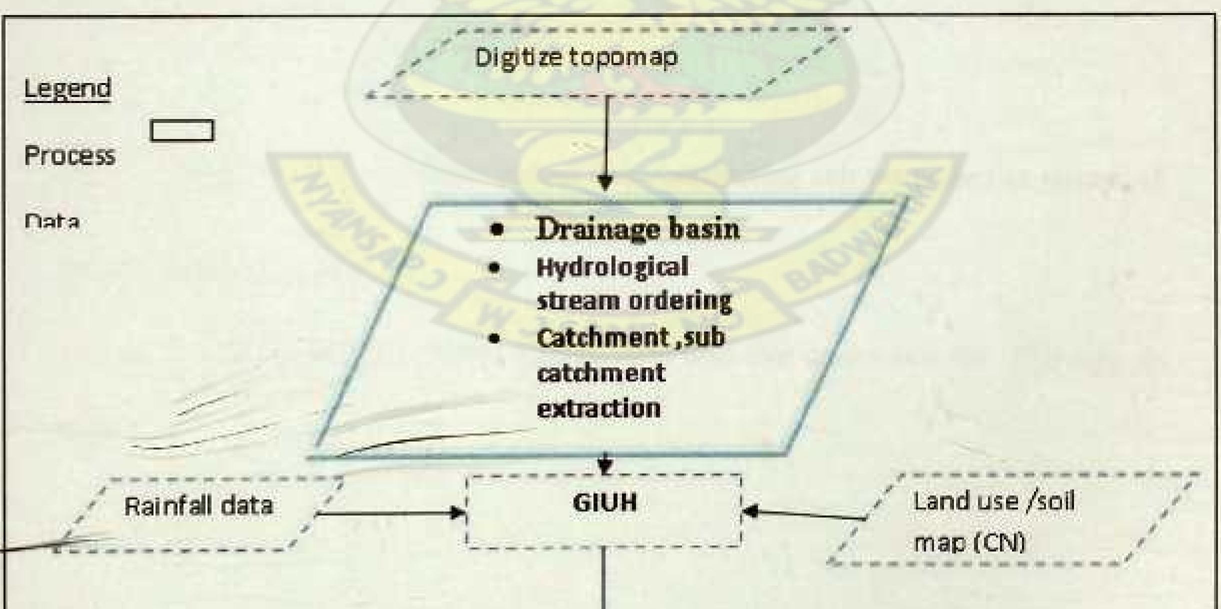


Figure 4-2: Schematic representation of GIUH process

The individual catchments and sub catchment areas are next extracted and computed for each order of stream showing layouts of individual catchments and sub catchments in the Mamahuma basin.

The next stage is the quantitative expressions of Horton's law of the stream numbers, stream areas and stream lengths, R_B, R_A and R_L respectively are then calculated. These are:-

$R_B = \frac{N_i}{N_{i+1}}$ which is described as bifurcation ratio, where N_i and N_{i+1} are number of streams in order i and $i+1$. Let Ω represent the highest order of stream in the catchment, i.e. $i = 1,2,-----\Omega$

$R_L = \frac{\bar{L}_{i+1}}{\bar{L}_i}$ where R_L is referred to as the length ratio and $\bar{L}_i$ is the average length of channel of order i

$\bar{L}_i = \frac{1}{N_i}\sum_{J=1}^{N_i} L_{ij}$ (Horton,1945) where L_{ij} represents the total length that drains into the j^{th} stream of order i.

$R_A = \frac{\bar{A}_{i+1}}{\bar{A}_i}$ where $\bar{A}_i$ is the mean area of the contributing sub watershed to stream of order i (Schumm-1956)

$\bar{A}_i = \frac{1}{N}\sum_{J=1}^{N_i} A_{ij}$ where A_{ij} represents the total area that drains into the j^{th} stream of order i

4.5.1 Derivation of the GIUH

The GIUH (Quan 2006; Rodriguez-Itube et al 1979) which is the impulse response function of the system is denoted by $U(t)$ and is determined as $U(t) = \frac{\partial}{\partial t} \Pr ob(T_B \angle t)$

or $U(t) = \frac{\partial}{\partial t}\left[\sum_{Si} \Pr ob.T_{Si} \angle(t) prob.(S_i)\right]$

Where $\Pr ob.()$: stands for the probability of the set given in the parenthesis

T_B : is the time of travel to the outlet

T_{Si} : is the travel time in a particular path

$\Pr ob.(S_i)$: is the probability of a drop which will travel all possible paths S_i to the outlet.

$\Pr ob.(T_{Si})$: is the probability density function of the total path travel time T_{Si}

Using the Strahler's ordering scheme (strahler, 1957) a pathway can be defined as a set of transitions between initial order of a droplet and the higher order stream until the outlet is eventually reached

For example a 3rd order stream, the paths traveled will be:-

Path $S_1 : a_1 \rightarrow r_1 \rightarrow r_2 \rightarrow r_3 \rightarrow outlet$

Path $S_1 : a_1 \rightarrow r_1 \rightarrow r_2 \rightarrow outlet$

Path $S_1 : a_1 \rightarrow r_3 \rightarrow outlet$

Path $S_2 : a_2 \rightarrow r_2 \rightarrow r_3 \rightarrow outlet$

Path $S_2 : a_2 \rightarrow r_3 \rightarrow outlet$

Path $S_3 : a_3 \rightarrow r_3 \rightarrow outlet$

Where a_i is denoted when the drop is in the hill slope state of order *i* and r_i is denoted when the drop is in channel state of order *i*

The probability of following any path to the outlet is given by:-

$$\Pr ob.(S_i) = \theta_i P_{ij} P_{jk} ------ P_{i\Omega}$$

Where θ_i is the Initial probability and is denoted by:

$$\theta_i = \frac{(\text{total area draining directly into stream order of i})}{(\text{total catchment area})}$$

This accounts for the drop of falling water on any hill slope area in the catchment either in the 1st 2nd, 3rd etc. stream of the catchment and is represented by θ_1 θ_2 θ_3 etc. It is assumed that Rainfall into the channel network is neglected.

The transition probabilities P_{ij} being transported from order *i* to *j* are obtained as

$$P_{ij} = \frac{(\text{number of stream of order i draining into stream of order j})}{(\text{total number of stream of order i})}$$

P_{ij} transition probability accounts for the changing stage of a drop from low order stream to the higher order. For example at the first order stream, the drop can either go to the 2nd order or the 3rd, etc. and this is represented by P_{12} and P_{13}.

The travel time T_S in a particular path must be equal to the sum of the elements of that path and is denoted as:-

$$T_{(si)} = T_{ai} + T_{ri} + T_{ri+1} ----- T_{r\Omega}$$

T_{aj} is the travel time on hill slope; T_{rj} is the travel time in each stream segment of *i* ($1 \leq i \leq \Omega 1$, where Ω is the highest order).

Assuming that these individual times are independent variables such that:-

$\int T_a$ is the probability density function (pdf) of T_{ai}

$\int T_{ri}$ is the probability density function (pdf) of T_{ri}

$\int T_{si}$ is the probability density function (pdf) of T_{si}

The probability of sum T_{si} is a multiple convolution integral of the following form:-

$$Prob(T_{si}) = \sum f_{Si}(t) = \sum fT_{ai}(t))*fT_{ri+1}(t)------fT_{i\Omega}(t)$$

$$fTai(t) = \alpha i exp(\alpha it)$$

is a gamma function corresponding to a travel time of a drop of water in given hill slope that obeys the exponential probability density function

$$fTri(t) = \beta i\, exp(-\beta i\, t)$$

is a gamma function corresponding to a travel time of a drop of water in given channel that obeys the exponential probability density function.

$$\beta i = V0/L,\ \alpha_i = \frac{v_0}{L_0}\ ,\ D = Ltot * RB^\wedge(\Omega - 1)$$

$$A(\Omega)*(R_L/R_B)^{(\Omega-1)}*(R_L/R_B-1)$$

The α, β are mean travel times for the hill slope and for stream channel flows respectively

Vo for hill slope velocity and Vs for stream velocity

Ω Where Ω is the highest order; where

D is the **drainage density** and it is the ratio of the total length of stream channel in the watershed to its area. The α_i is kept constant for any given hill slope i.e. $\alpha_1 = \alpha_2 = \alpha_3$ etc. while β_i is changing according to the average length of each given order stream.

The GIUH, $U(t)$ is computed as :-

$$U(t) = \sum fTai(t) * fTri(t) * fTri + 1(t) * --- * Prob(s)$$

This expression is solved by Excel spread sheet in time step shown in Appendix 3

4.6 Input of rainfall and land use data

Out of the various rainfall data collected, the rainfall event of 3rd-4th July 1995 (refer to Figure 4-1 above) was selected as a significant event in the basin and its abstracted runoff values used as input for the generation of the hydrograph from the GIUH model. Rainfall runoff process on a watershed is an open system, because not all the rainfall becomes runoff. There are some hydrological abstractions which include interceptions, surface storage, infiltration and evaporation hence the need to determine the effective rainfall which generates runoff. The effective runoff thus becomes Precipitation minus abstractions.

The effective (excess) rainfall is that rainfall which is neither retained on the land surface nor infiltrated into the soil(Chow et al, 1998) is computed according to National Conservation Services (NRCS) method which uses the Curve Number *(CN)* as its indicator. The Curve Number (CN) value depends on soil characteristics, land cover, antecedent moisture conditions and land use area. The *CN* values are estimated using information from tables developed by the National Resources Conservation Service (NRCS), (NRCS, 1985; Mc Cuen 1998).

The weighted curve number CN-II

CN-II = $\frac{\Sigma A_{LS} CN-II_{LS}}{A_{tot}}$ (NRCS, 1985; Mc Cuen 1998).

A_{LS} area of a particular land use and soil type CN-II is corresponding curve number and A_{tot} is the total area of basin.

CN-I = $\frac{CN-II}{2.334-0.01334CN-II}$

The direct runoff volume or the effective rainfall is calculated using the following empirical formulae

$$P_e = \frac{(P-0.2S)^2}{(P+0.8S)} \text{ for } \quad P \geq I_A = 0.2S$$

And P_e for $p \leq I_A = 0.2S$. (Chow et al, 1998)

P_e =Runoff volume (rainfall excess) to result from precipitation expressed as depth

P= total observed precipitation. (Cumulative rainfall)

S= the maximum potential abstraction after runoff begins

I_A = Initial abstraction before runoff begins

P_e,P and I_A have units in cm. or inches

$S = \frac{1,000}{CNI} - 10$ (in) and $S = \frac{2,540}{CNI} - 25.4$ (cm) (Chow et al, 1988).

4.7 Flood Routing Method

Flood routing is used in establishing the height of a flood peak at a downstream location in order to assess the protection that would result from the reservoir effect. Flood routing method may be divided into two namely (i) reservoir routing and (ii) the channel routing. The reservoir routing analyses the effect of reservoir storage on the flood hydrograph while the channel routing analyses the effect of storage of the channel reach on the flood hydrograph. In this study the reservoir routing method is adopted and the Hec-ras software is used as a tool for the channel routing.

The flood routing method is based on the Law of continuity- which states that the volume of water that is discharged from a reach during any time interval must equal the volume of inflow during the time interval plus or minus any increment in the stored water during the period. (Chaudhry, 1993). In equation form it becomes:-

$\overline{Q} = \overline{I} - \frac{\Delta S}{\Delta t}$ ----------- (1) (Chow et al, 1998)

Where $\bar{Q}$= Mean outflow during routing period $\Delta t\ (m^3/s)$

$\bar{I}$= Mean inflow during routing period $\Delta t\ (m^3/s)$

Δs = net change in storage during routing period $\Delta t\ (m^3)$

The equation can be modified as:-

$\Delta t \frac{(Q_1+Q_2)}{2}$=$\Delta t\ \frac{\bar{I}_{1+}\bar{I}_2}{2}$-$(S_2 - S_1)$ ----------------- (2)

Where Q, I, S and Δt are as above and the subscripts identify the beginning and the end of routing period Δt.

In the reservoir–storage method the equation (2) is rearranged to permit the rapid calculation of the outflow hydrograph as:-

$(\bar{I}_1 + \bar{I}_2)$+$\frac{2S_1}{\Delta t}$ - Q_1 = $\frac{2S_2}{\Delta t}$+Q_2-------------------------- (3)

It is assumed that the reservoir has a horizontal water level and the variation of inflow and outflow over the interval is also assumed approximately linear.

Let the time horizon be broken into intervals of duration Δt indexed by j that is

$t = 0, \Delta t, 2\Delta t,$---------------$j\Delta t, (j + 1)\Delta t$

Taking inflow values at the beginning and the end of the j^{th} time intervals as $\bar{I}_j$ and $\bar{I}_{j+1}$ respectively and the corresponding values of the outflows as Q_j and Q_{j+1} and the change in the storage over the interval S_{j+1} - S_j can be obtained by rewriting equation (3) as:-

S_{j+1} - S_j = $\frac{I_j+I_{j+1}}{2}\Delta t$ - $\frac{Q_j+Q_{j+1}}{2}\Delta t$ ----------------- (4)

In the equation (4), I_j and I_{j+1} are known from a given inflow hydrograph to be routed through the reservoir Q_j and S_j are the initial outflow from the reservoir and the initial storage in the reservoir. Q_{i+1} and S_{j+1} are the unknown quantities which must be determined.

The relation between the storage and the outflow provides the additional relationship required to obtain the Q_{i+1} and S_{j+1}. The storage discharge relationship of the reservoir is determined by surveying the area at the reservoir site(bathymetric) in detail and preparing a contour map of a reasonable interval(0.5-2.0m) depending on the size of the reservoir. From the map the areas enclosed by the various contours are planimetered and a curve of elevation versus area is prepared. With the known enclosed contours the incremental volumes of water stored between any two successive contours can then be determined by using $V=h/3(A_1+A_2)$ where A_1 and A_2 are the areas corresponding to two successive contours values and h is the difference between the contours. The information from the above relation is used to prepare the elevation –storage curve (Jaya, 2007).

The outflow is plotted against the elevation and the curve known as elevation – outflow curve as shown in figure 5.3. The elevation-storage curve and the elevation-outflow curves are utilized to prepare a curve of storage versus discharge as shown in Figure 5-4 in the results.

The reservoir for flood regulation is the Mamahuma reservoir built around 1963 for the storage of water for agricultural purposes. There were no records of the designed characteristics of the Mamahuma reservoir. It was therefore necessary to re-establish the reservoir's elevation, storage and outflow relationships hence the bathymetric data collection.

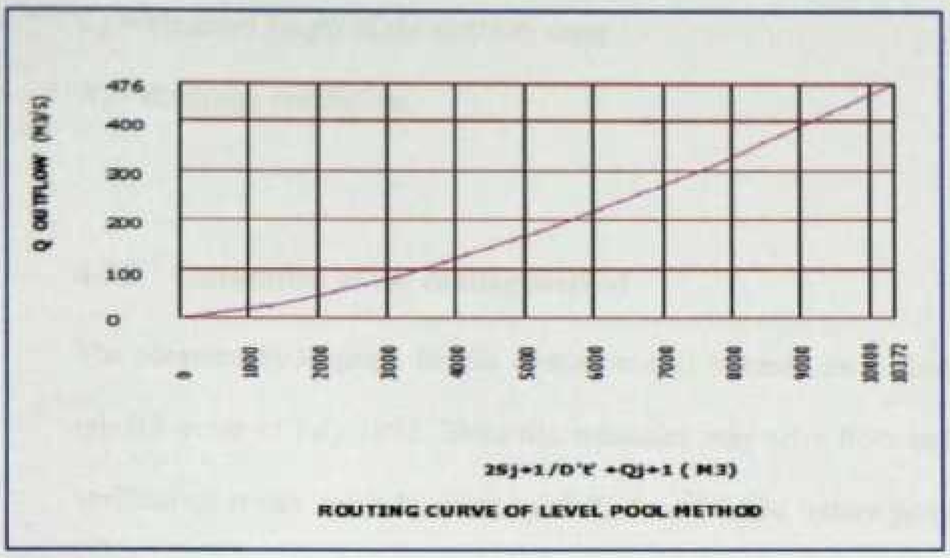


Figure 4-3 Discharge -storage curve

4.7.1 Weir discharge of the Mamahuma reservoir

The elevation–outflow hydrograph over the weir is governed by the hydraulics of the weir. The flow over the reservoir weir is uncontrolled i.e. it has no gates to regulate the flood spills. The outflow from the reservoir corresponding to any elevation can be determined by the discharge equation of the weir as shown below in the figure.

Figure 4-4: Existing uncontrolled weir on Mamahuma

The discharge equation of an uncontrolled sharp-crested weir is given by:

$Q = K_R\sqrt{2g}\, L_e\, Ho^{3/2}$ (Wurbs and James, 2002).

Where H_O= head over the spillway

L_e = effective length of the spillway crest

K_R= discharge coefficient.

4.7.2 Calibration of the routing method

The obtained hydrograph for the routing model is based on a historic single valued rainfall event of July 1995. Since discrepancies may arise from lack of quality data, verification is can made by using rainfall of established 'return periods' derived from the Intensity Duration Frequency curves (IDF) by Dankwa, 1974. The value with a return period is the amount that will be equalled or exceeded only once in the year on the average over a long period of time (Dankwa, 1974). The data from the IDF have been compiled using rainfall data observed from synoptic stations closed to the watershed with sufficiently long records extending for at least ten years' values. Even though the IDF values are themselves old and need updating its representation is broad base than a single valued rainfall event.

4.8 Hec-Ras for Hydraulic analysis

The predictions of hydraulic performance of channels with respect to derived flood hydrographs are necessary for urban storm water drainage planning. In this study the Hec-Ras Hydrologic model was used as a tool to observe the hydraulic behaviours and to predict the performance of the inflow and outflow hydrographs in the Mamahuma stream downstream the reservoir. The following description of the software and its applications are based on the literature materials obtained from Hec-Ras user Manual (Hec Ras May 2005).

The Hydrological Engineering Center, River Analysis System (Hec-Ras) is software that allows one to perform one-dimensional steady, unsteady flow river hydraulics and sediment transport calculations. Hec-ras is an integrated system of software, designed for interactive use in a multi-tasking environment. The system is comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. The Hec-Ras system contains three one-dimensional hydraulic analysis components for: Steady flow water surface profile computations; unsteady flow simulation; movable boundary sediment transport computations. A key element is that all three components use a common geographic data representation and common geometric and hydraulic computation routines. In addition to the three hydraulic analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed.

The main activities in HEC-RAS model includes:

- Entering Geometric data
- Entering flow data and boundary conditions
- Performing hydraulic calculations
- Viewing, exporting and/or printing results.

4.8.1 Entering Geometric data of stream channel.

The geometric data for this analysis consists of coordinates and elevations defining the Mamahuma channels longitudinal and cross sectional profiles. In all 20 cross sections were taken over to the reservoir and another 5 cross sections beyond the reservoir at average distance of 200m between each cross to the reservoir.

4.8.2 Channel layout

The x, y and z coordinates obtained from selected points along in a topographic surveys of the stream was imported into the Hec-Ras geometric data inter-face. After adjusting the schematic scale to accommodate all the coordinates and the resulting channel network was obtained (refer Appendix 2)

4.8.3 Cross Section Geometric Data

To enter the cross sectional data, the cross section icon on the left side of the Geometric Data Editor, second on that column, was selected. This activates the cross section Data Editor. Then, 'Add a new cross section' was selected under the Options menu to create each new cross section. For each cross section, the geometric data consists of the: Description, Lateral and elevation co-ordinates for each terrain point channel reach lengths, Manning's roughness coefficients 'n' values, main channel Bank, Left and Right bank stations, and contraction and expansion coefficients. A typical example of the input data for the channel and its output section is shown below:

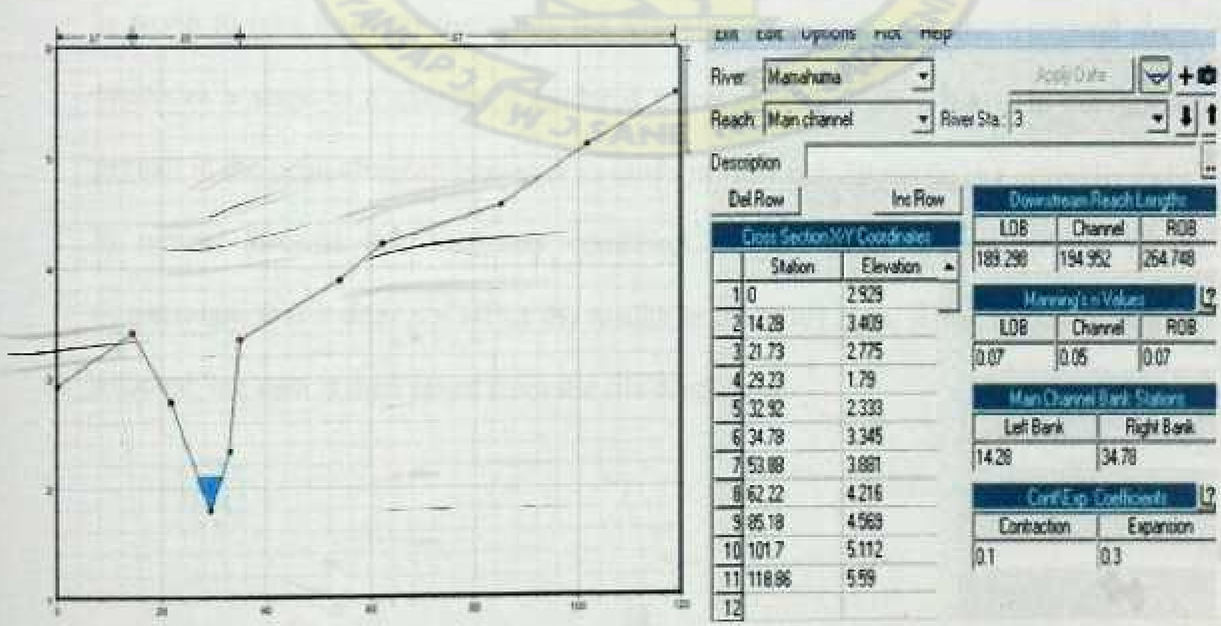


Figure 4-5: Typical cross section of Stream station 3 of the Mamahuma

4.8.4 Entering flow data

To enter the flow rate data, the unsteady flow data Editor was activated from the main program window by selecting Edit and then the unsteady flow Data. In this Editor, the Channel names and the reaches are picked up automatically according to the number of channels and reaches in the channel system schematic. The only data required here is the inflow hydrograph for each channel, after which the button okay is selected and then saved from the file drop down (Boamah, 2006).

4.8.5 Entering boundary conditions.

After the flow rates data were entered, the boundary conditions were entered by selecting the boundary conditions button at the top of the unsteady flow data Editor. This activates the Boundary condition Editor. There are four available external boundary condition types: Known water surface, Critical depth, Normal Depth, and Rating curve. Each channel needs two boundary conditions, one at the upstream end and the other at the downstream. In this study, the inflow hydrograph derived from the GIUH was used for the upstream boundary condition. For the downstream condition, the normal Depth from Manning's equation with channels frictional slope was used. It is worth to note however that using the Manning's equation with the frictional slope produces a stage in a channel considered to be normal depth if uniform conditions prevail in the open channel. However as uniform flow conditions do not normally exist in natural streams ~~this boundary~~ condition is used close to end of the reach downstream that it does not affect the results in the study area. After, the ok button is selected; the data is then saved from the file drop down.

4.8.6 Viewing, exporting and/or printing results.

The simulation is complete if there are no warnings displayed about any wrongful input data, corrections and results viewed and printed. (Hec- Ras User Manual version 3.1.3, May 2005).

4.8.7 Weir discharge of the Mamahuma reservoir

The elevation–outflow hydrograph over the weir is governed by the hydraulics of the weir. The flow over the reservoir weir is uncontrolled i.e. it has no gates to regulate the flood spills. The outflow from the reservoir corresponding to any elevation can be determined by the discharge equation of the weir as shown below in the figure.

CHAPTER FIVE

5 DATA ANALYSIS AND DISCUSSIONS

5.1 Gumbel's frequency analysis results

The Gumbel distribution frequency was applied to the annual maximum 6hr rainfall events. Figure 5-1 below presents the empirical probability plotted against the annual maximum 6hr rainfall events and the fitted Gumbel probability distribution. As shown in the plot, the annual maximum 6hr rainfall events give a uniform distribution of the around the fitted Gumbel distribution curve.

Having obtained a good fit with the Gumbel distribution function, various return periods were computed and presented in Table 5-1. From the table, it was observed that the rainfall event of the 3rd-4th July 1995 falls within a return period of 200 years.

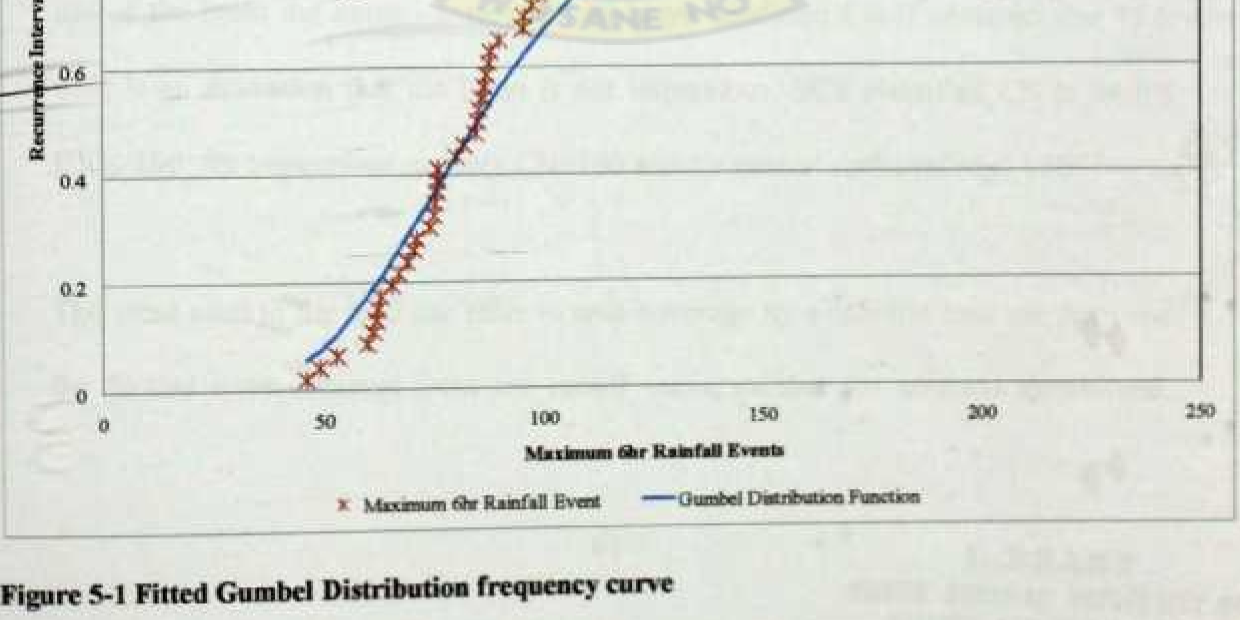


Figure 5-1 Fitted Gumbel Distribution frequency curve

Table 5-1 Computed return periods (Gumbel's distribution)

Return Period	y_t	K	X_T
2	0.367	-0.156	85.226
5	1.500	0.828	119.831
10	2.250	1.479	142.742
15	2.674	1.847	155.669
20	2.970	2.104	164.719
25	3.199	2.302	171.691
50	3.902	2.913	193.166
75	4.311	3.268	205.649
100	4.600	3.519	214.483
125	4.824	3.714	221.327
150	5.007	3.873	226.914
175	5.162	4.007	231.635
200	5.296	4.123	235.722
225	5.414	4.226	239.327
250	5.519	4.317	242.550

5.2 Effective rainfall analysed from the rainfall event of 3rd-4th July 1995

The obtained effective rainfall is with respect to the current land use in the basin. This may change depending on the degree of urbanization, land use and the peak runoff control measures adapted in the basin. Similarly the initial abstraction of 141.96 mm/min (0.0024m/hr) obtained may be changed for a lower or higher value.

In converting the effective rainfall an important factor considered was antecedent conditions based on the Soil Conservation Service (1972) SCS method. From the land use of the basin the normal antecedent moisture condition CN-II obtained was 45.5. This is an indication that the basin is not impervious. SCS classified CN to be $0 \leq CN \leq 100$ for impervious surfaces CN=100 and for natural surfaces $CN < 100$.

The areas used in the land use refer to area coverage by a specific land use item and the factors were obtained from the runoff curve number for selected agricultural, suburban and urban land uses. Tables 5.1 and 5.2 below show the computed weighted CN and the depths of the effective rainfall values.

Table 5-2: Weighted CN values

Land Use	Area	CN Value	
Grass/Herb with scattered trees	31.44	58.00	1823.52
Settlement	5.10	51.00	260.1
Shrub Thicket with trees	1.448	39.00	56.472
Mosaic of Thickets and Grass with scattered trees	24.02	45.00	1080.9
Moderately Dense Herb/Bush with scattered trees	24.15	30.00	724.5
Moderately Closed trees canopy with Herbs and Bush	1.19	25.00	29.75
	87.35		**3975.242**
Weighted CN values			**45.5104**

Table 5-3: Transformation of rainfall of 3rd-4th July 1995

Time	Time(15mins)	Rainfall depth(mm)	Rainfall(cum)	Effective rainfall(e_i) (mm)
23:15	0	0	0.00	0.00
0:15	1	5.7	5.70	0.00
1:15	2	9.9	15.60	0.00
2:15	3	71	86.60	0.00
3:15	4	71.8	158.40	0.37
4:15	5	52	210.40	6.02
5:15	6	28	238.40	11.54
6:15	7	6	244.40	12.92
7:15	8	2.9	247.30	13.61
8:15	9	1.2	248.50	13.91
9:15	10	0.8	249.30	14.10

5.3 Stream network parameters and probabilities

The Mamahuma stream network shows a stream of the fourth order and with a total catchment area of 87km^2 and total stream length of almost 88km.

The geomorphology of the basin is used in determining of the hydrological response of the basin through initial state probability, transition probability, and travel path probability obtained from the network analysis.

Table 5.3 shows that the streams in the first order occupy the about 68% of the stream basin. These streams receive flows from overland and most of them may disappear as the catchment develops.

The bifurcation ratios RB obtained for the 1st, 2nd and 3rd orders are 5, 3.5, and 2 respectively, fall within the range 3-5 proposed (Horton, 1945) showing that the basin is of relatively homogenous material.

Table 5-4: Analysis of the hydrological ordering system

Channel order	No. Streams	Total Area	Total Length	Mean Area	Mean Length
1	35	59.61	60.72	1.70	1.73
2	7	11.65	12.00	1.66	1.71
3	2	5.95	5.74	2.98	2.87
4	1	10.14	9.46	10.14	9.46
Total	45	87.35	87.92	6.34	6.32

Table 5-5: Stream ratios

Length Ratio	Area Ratio	Bifurcation Ratio
0.988	0.941	5
1.674	1.627	3.5
3.296	3.75	2

5.4 Derived flow hydrographs

From the geomorphologic instantaneous unit hydrograph analysis the hydrograph for basin based on the rainfall event of the3rd and 4th July 1995 rainfall event was derived.(refer Appendix 3) The hydrograph derived has a peak discharge of $45m^3/s$ and it represent the time taken by the total catchment to contribute discharge to the flow. The total time taken for the hydrograph to rise and fall is 6hrs. This translation time is needed in the reservoir routing to decide on the management of flood. The 1995 rainfall with a cumulative rain represent a rain fall of return period 75years.

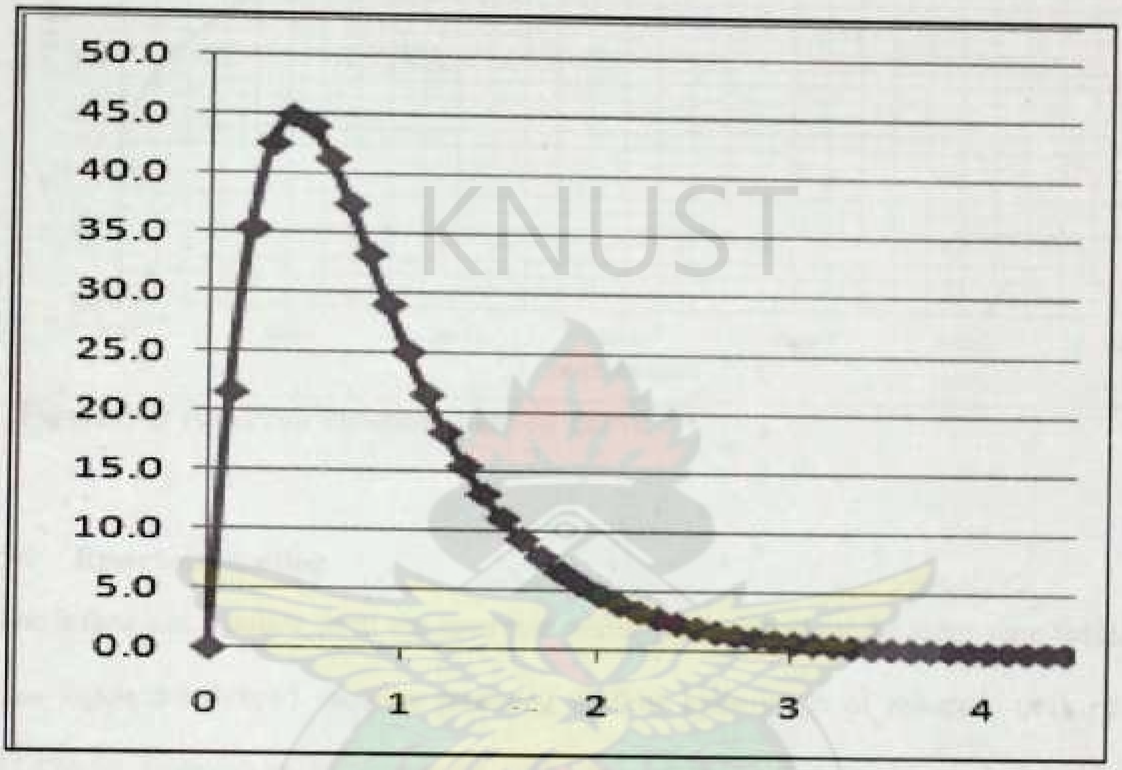


Figure 5-2: Computed inflow hydrograph from 1995 rainfall event

5.5 Reservoir Storage Relations

5.5.1 Elevation –outflow relation

The weir on the Mamahuma is a sharp crested weir. From the hydraulic equation of a sharp crested weir the outflow-elevation relations were obtained as shown below in the table. The weir crest elevation measured 19.504m. (Refer Appendices 4, 5 and 6 for other reservoir relations)

Figure 5-3: Reservoir elevation-outflow curve

5.6 Reservoir Routing

The inflow and outflow from the reservoir routing and plotted on the same time scale (see figure 5-4 below) shows a predicted outflow hydrograph of reduced peak of $11.67m^3/s$ hydrograph as compared with that of the inflow hydrograph $45m^3/s$. Similarly the time to peak, 11.67 hrs, in the outflow hydrograph is more than the time peak 0.4 hrs of the inflow hydrograph. These differences show the effects of the reservoir storage on the movement of flood wave through the reservoir. The attenuation which is the reduction in peak is $33m^3/s$ in the lag time, which is the difference in times to peak, is 1.1hrs. The peak outflow occurs at the intersection point of the inflow and the outflow hydrographs. This is an indication that between 0.4hrs-1.4hrs inflow was higher than outflow which is an indication of the accumulation of storage and this continues until the two curves intersect. After the intersection point the outflow curve is above the inflow curve which is also an indication of withdrawal from the storage. The storage therefore is maximum at the intersection point.

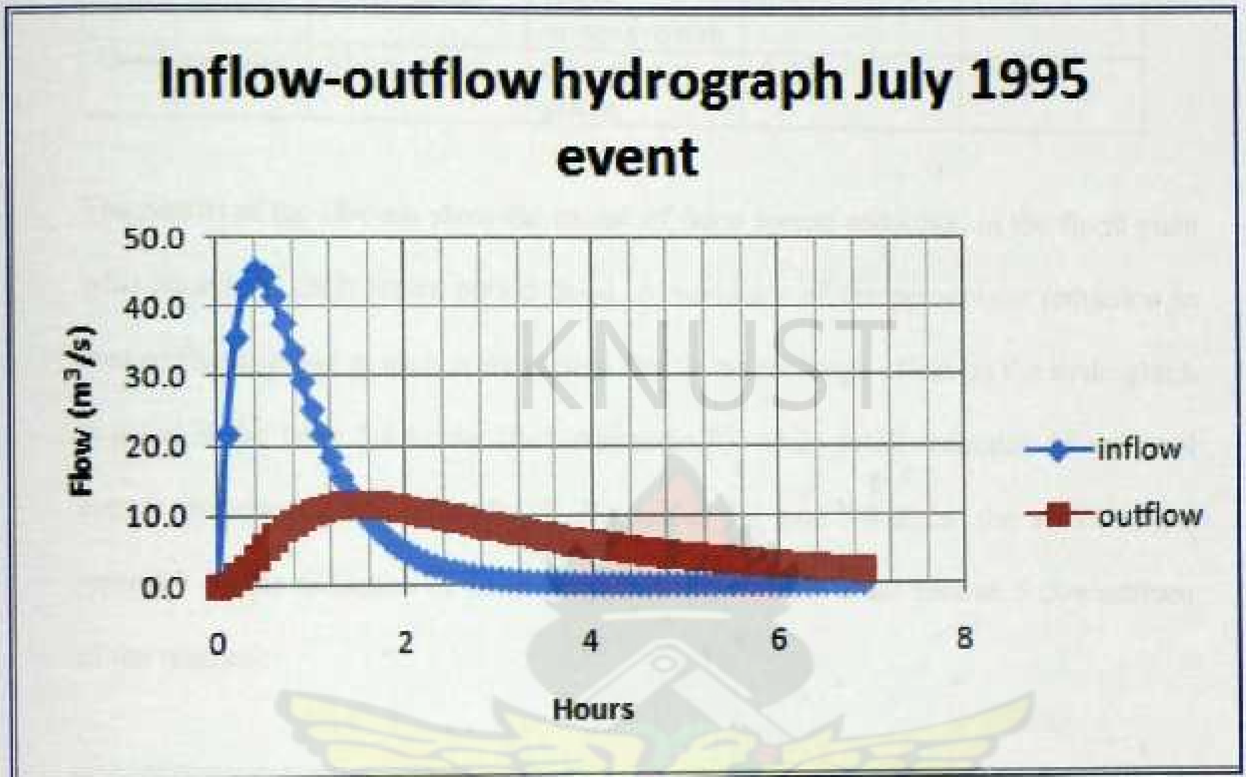


Figure 5-4: Reservoir routing curve

5.7 Hec –Ras flood routing effects

In the running of the Hec-Ras programme the surface roughness coefficient ('n') values of the Mamahuma channel was necessary as input. The values were obtained by visual inspection of channel noting the bed, bank materials, the presence of vegetation and shape to decide on the 'n' values with reference to standard values provided in literature, (refer Appendix 7) The figures below show general roughness characteristics of sections of the Mamahuma channel. And the table next shows the obtained values from the field inspection

Table 5-6: Channel Manning's roughness coefficient

channel	Surface characteristics		Roughness (n)	Coefficient
	Bed	Bank slope	Bed	Bank slope
Mamahuma	Short grass	Light undergrowth	0.15	0.30
Onukpawahe	Short grass	Clay/short grass	0.15	0.20

The results of the Hec-ras show the extent of flood spread reduction in the flood plain with regards to each return period flood. A summary of the percentage reduction in area of flood spread at each cross section due to the reservoir effect on the hydrograph is given in the table 5-6 below (Ref. Appendix 9), while detail reduction in sectional area is provided in the appendix 8. The tables 5-7 and 5-8 show the effect of the reservoir on the reduction of flood heights in the channel cross section 5 downstream of the reservoir.

Cross section	0	1	2	3	4	5	6	7	8	9	10
% Reduction in flood area	70	63	41	69	75	70	66	67	70	73	66
Cross section	11	12	13	14	15	16	17	18	19	20	**Ave.**
% Reduction in flood area	57	66	56	69	73	79	45	64	48	77	65%

Table 5-6 showing percentage flood spread reduction in channel downstream

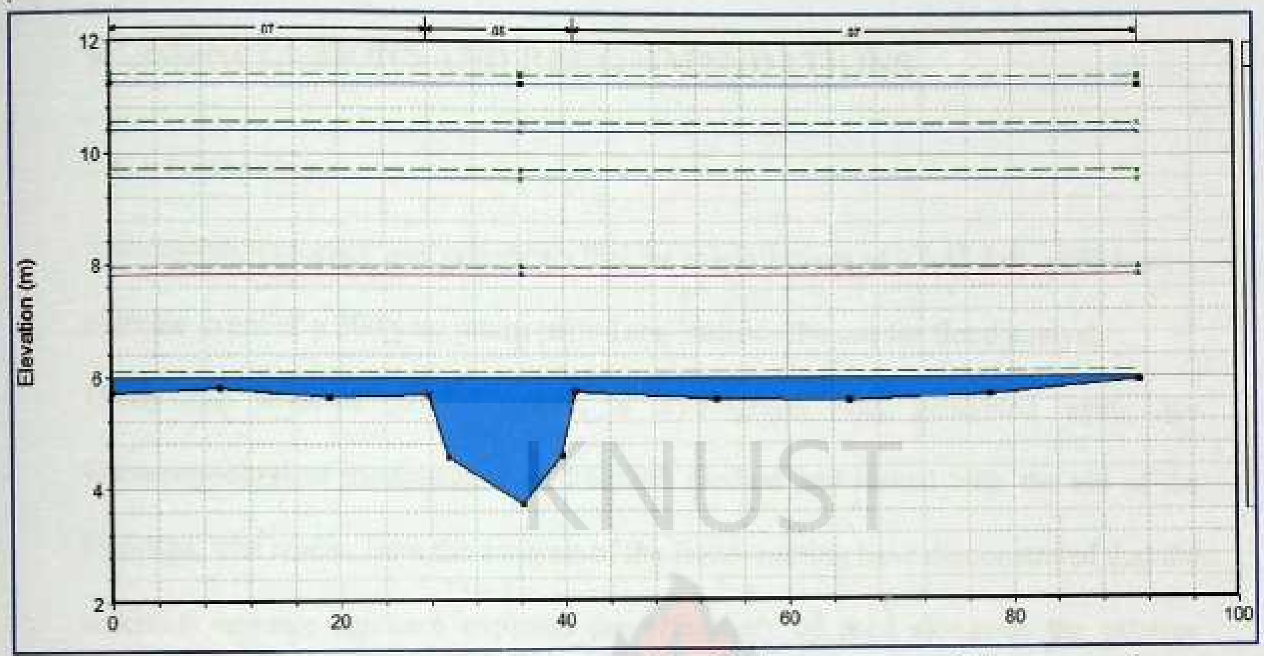


Figure 5-7: No reservoir effect, at cross section 5, downstream of the reservoir.

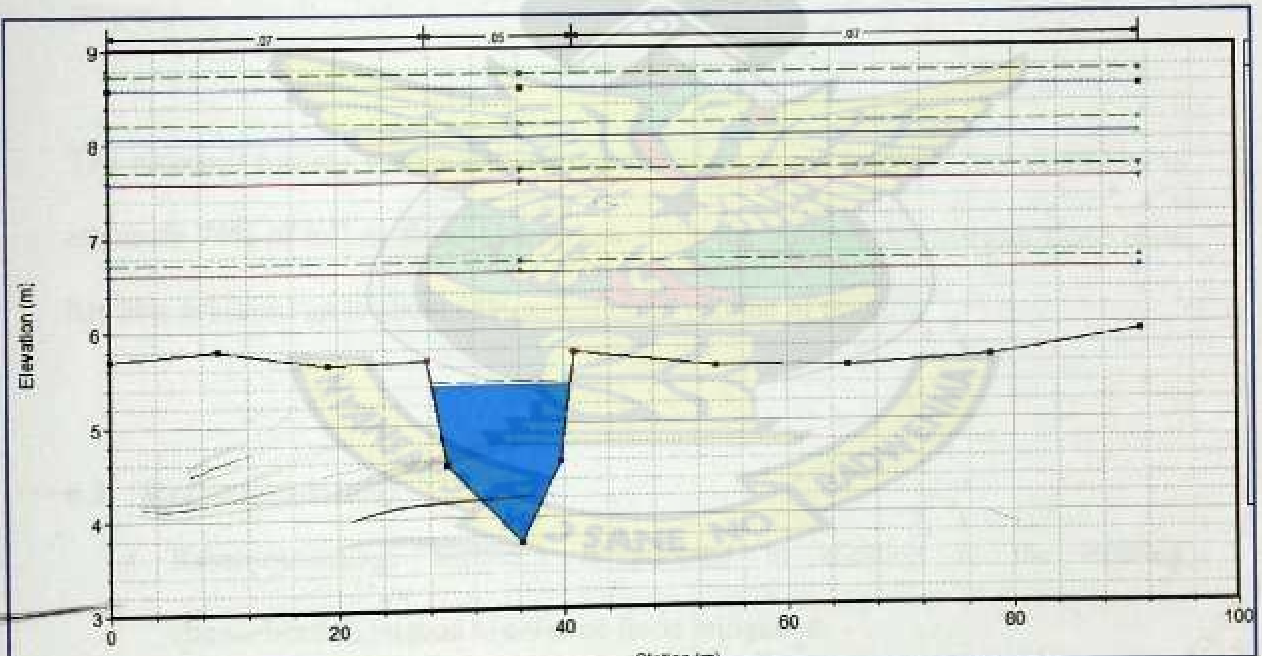


Figure 5-8 cross section 5 showing reservoir effect and other flood heights

CHAPTER SIX

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The rainfall frequency analysis shows that the rainfall event of 3rd-4th July 1995 is an extreme event of a 200 year return period and justifies the use for flood analysis.

Using the selected rainfall event, a hydrograph was generated using the Geomorphological instantaneous hydrograph method and routed with the aid of the Hec-Ras. The results from the analysis of the flood- routing have demonstrated that the reservoir -storage approach explored can effectively be used alongside the existing practiced method in reducing flood hazard, preventing damage to private and public property.

The reservoir routing further showed that the Mamahuma reservoir has a capacity to attenuate 74% of inflow flood hydrograph while the outflow downstream flood plain has also achieved up to about 65% reduction in volume of flood.

6.2 Recommendations

- Reservoir-storage approach is adopted in addition to the existing channelization method to enhance flood mitigation.
- Introduce decision makers such planners, engineers and politicians to the concept of reservoir storage in flood management planning and design.
- Existing irrigation reservoirs in the urban areas are protected from being reclaimed to be used as reservoir-storage for flood peak reduction.

- Conduct a comprehensive safety assessment on small reservoirs in urban communities to harness their potentials for urban flood management.
- Settlement planners in the country should be trained on the need to demarcate appropriate location in urban settlements for stormwater storage reservoirs.
- Policies should be evolved at national and district levels to approve the use of detention reservoirs as a flood control measure so that right- of way could be reserved for flood control and maintenance measures.
- The Hydrological Services Department which the agency charged with the responsibility of hydrological studies, planning and development all primary channels for flood mitigation champions the education drive for adapting the reservoir storage effect in urban flood management in the country.

REFERENCES

Bedient Philip B. and Wayne C. Huber Hydrology and Flood Plain Analysis, Addison-Wesley, 1992.

Chang-Xing J. A deterministic gamma-type geomorphologic instantanoeus unit hydrograph based on path type, Water Resources Research, 1992. - Vol. 28.

Chow Van Te, Maidment David R. and Mays Larry W. Applied Hydrology, Singapore : McGraw-Hill, 1998.

Clark C.O. Storage and unit hydrograph, American Society of Civil Engineers, 1945. - Vol. 110.

Dankwa and B. J. Maximum Rainfall Intensity-Duration Frequencies in Ghana,- Accra : Ghana Meteorological Services Department Note No. 23, 1974.

Department(H.S.D) Hydrological Services Causes of flooding in Accra, - Accra : H.S.D (unpublished), 2008.

Fleurant C., Kartiwa, B., Roland, B. Analytical model for a geomorphological instantaneous unit hydrograph, Wiley Interscience. - Hydrological Processes, 10 October 2002. - 6162. - 27 August 2008. - www.interscience.wiley.com.

Gaisic S.K. Population data nanalysis reports- Socio -Economic and Demographic Trends analysis, - Ghana : Ghana Statistical Services, 2005.

Global Water Partnership(GWP)/ West African Technical Advisory Committee (WATAC) Water for the 21st century Vision for West Africa, - The Hague : World Water Forum, 2000 (17-22 March).

Gupta B.L. and Gupta Amit Engineering Hydrology, - Delhi : N. C. Jain, 1999. - pp. 261-269.

GYAU Boakye Report on Accra floods, - Accra : Water Research Institute, 1997.

Horton R.E. Erosional development of streams, hydrophysical approach to quantitative morphology, Geological Science of American Bulletin, 1945. - Vol. 56.

Lopez J.J, Analysis of a unit hydrograph model based on watershed geomorpholoy represented as a cascade of reservoirs, Science direct. - Agricultural water management, march 2005. - 97. - 28 August 2008. - www.sciencedirect.com.

Muff R. and O. Boamah K. Environmental and Engineering Geology for Urban Planning in greater Accra Metropolitan Area // Maps of Accra. - Accra : Geological Survey Department, 2005.

Nash J.E. The form of the instantaneous unit hydrograph,: International Association Science Hydrology, 1992. - Vol. 45.

Nguyan Hong Quan Rainfall-Runoff modeling in the unguaged can le catchment Saigon River Basin, - 2006.

Rodriguez Fabrice, Andrieu Herve and Jean-Dominique Creutin Surface runoff in urban catchments: morphological identification of unit hydrograph from urban databanks, Science direct. - Journal of Hydrology, 2 July 2003. - 283. - 27 August 2008. - www.elsevier.

Rodriguez-Iturbe I. Valdes, J. The geomorphologic structure of hydrologic response, Water Resources Research, 1979. - 6 : Vol. 15.

Schumm S., Evolution of drainage system and slopes in bad lands at Pearth Amboy, Geological Society of America, 1956. - Vol. 87.

Shamseldin Araad Y. and Nash J.E. The Geomorphological Unit Hydrograph- a critical review, Hydrology and Earth System Sciences 2(1). - 1998. - pp. 1-8.

Sherman L.K Streamflow from rainfall by unit-graph, : Engineering News Record, 1932. - Vol. 108.

Strahler A.N. Quantitative geomorphology of drainage bains and channel network, Handbook of Applied Hydrology / book auth. chow Van Te. - New York : Macgraw-Hill, 1964.

Subramanya and K. Engineering Hydrology, - New York : Tata Mc Graw-Hill, 1995.

U.S Army Corps of Engineers Hec-Ras user mannual [User manual]. - Davis, Califonia : Hydrologic Engineering Centre, 2006. - Vol. 3.1.3.

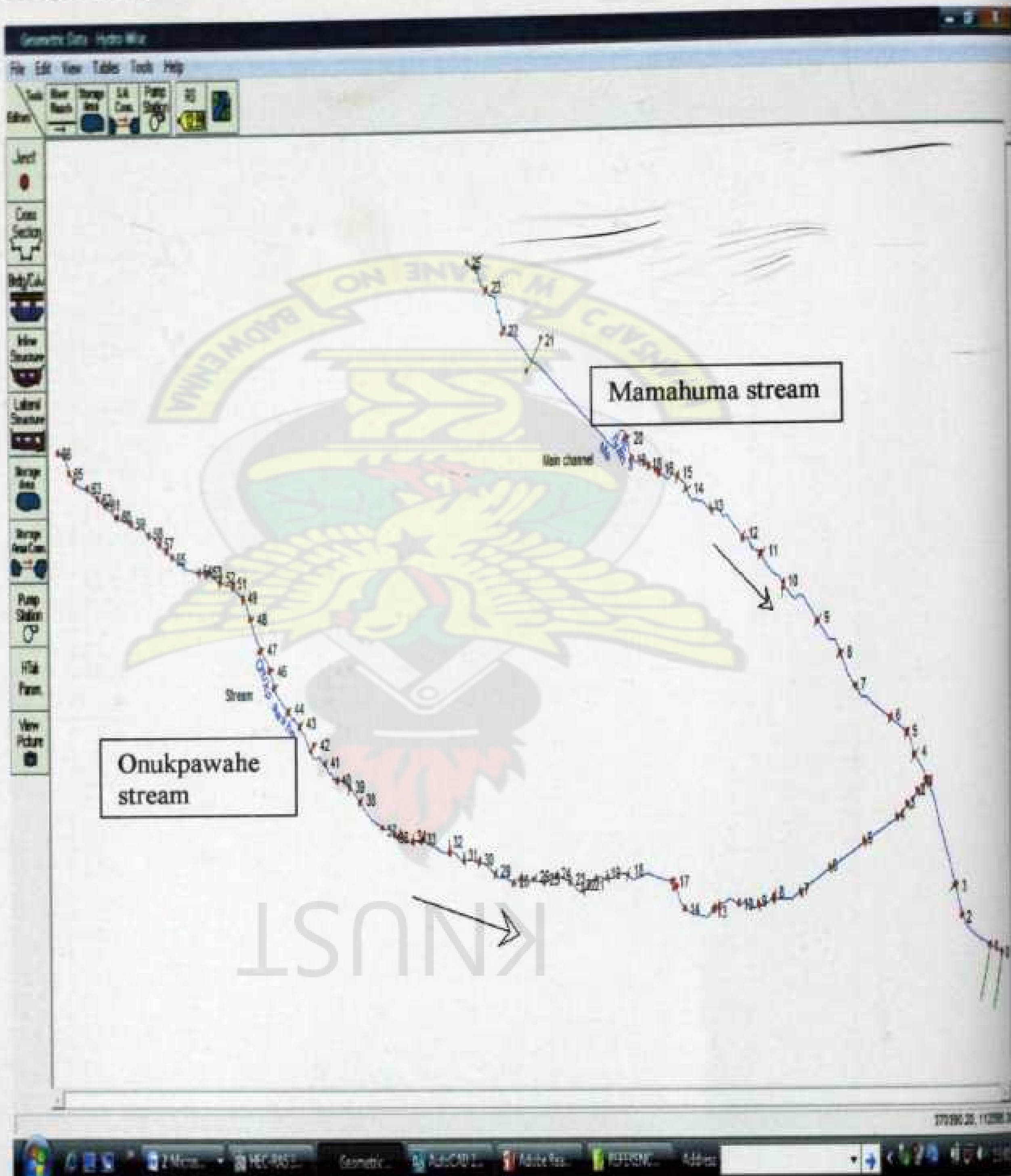
Valdes Juan B., Yolanda Fiallo and Rodriguez-Iturbe A Rainfall- Runoff Analysis on the Geomorphologica IUH, Water Resources Resaerch volume15 no 6. - 1979.

Wiley Interscience, www.interscience.wiley.com. - Hydrological Processes.

Wurbs [et al.] Water Resources Eengineering, Prentice Hall, 2002.

APPENDICES

APPENDIX 2: LAY OUT OF MAMAHUMA STREAM – OBTAINED FROM THE GEOMETRIC DATA INPUT IN THE HEC-RAS



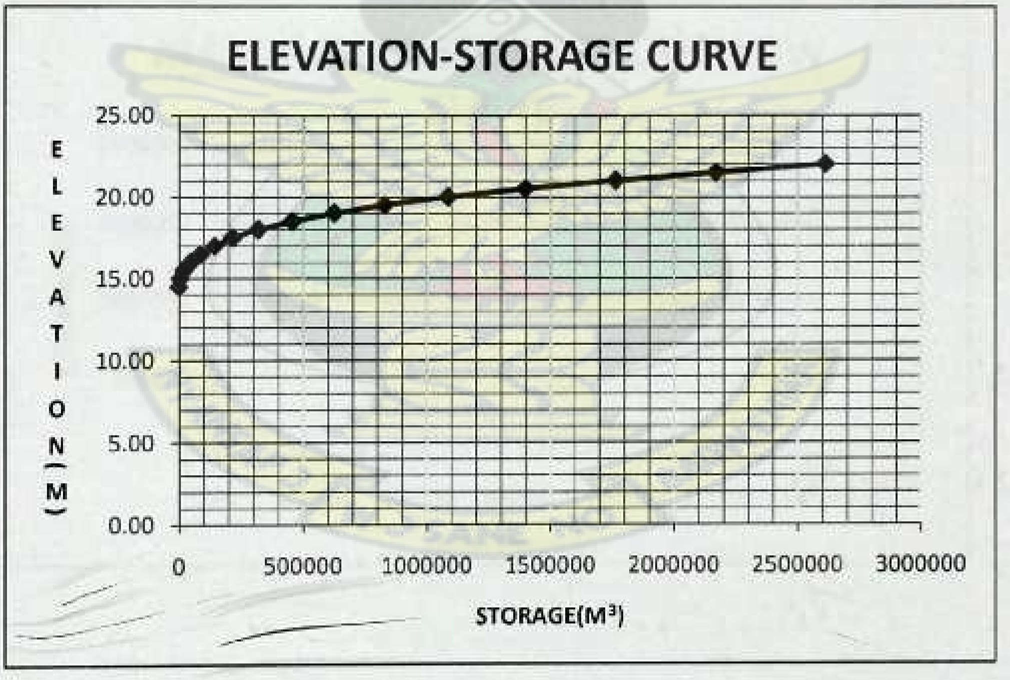
APPENDIX 3: GEOMOPHORLOGIC INSTANTANEOUS UNIT HYDROGRAPH (GUIH), CONVOLUTION RESULT

EFFECTIVE RAINFALL(mm)/GIUH(m3/sec)	0.00	0.00	0.00	0.00	0.37	6.02	11.54	12.92	13.61	13.91	14.10
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.55	0.00	0.00	0.00	0.00	0.58	9.35	17.92	20.07	21.15	21.60	21.91
2.54	0.00	0.00	0.00	0.00	0.95	15.32	29.36	32.88	34.64	35.38	35.88
3.06	0.00	0.00	0.00	0.00	1.14	18.44	35.34	39.58	41.70	42.59	43.19
3.23	0.00	0.00	0.00	0.00	1.20	19.46	37.31	41.78	44.02	44.97	45.60
3.17	0.00	0.00	0.00	0.00	1.18	19.09	36.60	40.99	43.19	44.11	44.73
2.97	0.00	0.00	0.00	0.00	1.11	17.88	34.27	38.38	40.44	41.31	41.89
2.70	0.00	0.00	0.00	0.00	1.00	16.23	31.10	34.84	36.70	37.49	38.02
2.39	0.00	0.00	0.00	0.00	0.89	14.41	27.61	30.92	32.58	33.28	33.75
2.09	0.00	0.00	0.00	0.00	0.78	12.59	24.13	27.02	28.47	29.08	29.49
1.81	0.00	0.00	0.00	0.00	0.67	10.88	20.85	23.35	24.60	25.13	25.48
1.55	0.00	0.00	0.00	0.00	0.58	9.32	17.86	20.00	21.08	21.53	21.83
1.32	0.00	0.00	0.00	0.00	0.49	7.94	15.21	17.04	17.95	18.34	18.60
1.12	0.00	0.00	0.00	0.00	0.42	6.73	12.91	14.45	15.23	15.56	15.77
0.95	0.00	0.00	0.00	0.00	0.35	5.69	10.91	12.22	12.88	13.16	13.34
0.80	0.00	0.00	0.00	0.00	0.30	4.81	9.21	10.32	10.87	11.11	11.26
0.67	0.00	0.00	0.00	0.00	0.25	4.05	7.77	8.70	9.17	9.36	9.49
0.57	0.00	0.00	0.00	0.00	0.21	3.41	6.54	7.33	7.72	7.89	8.00
0.48	0.00	0.00	0.00	0.00	0.18	2.87	5.51	6.17	6.50	6.64	6.73
0.40	0.00	0.00	0.00	0.00	0.15	2.42	4.64	5.19	5.47	5.59	5.67
0.34	0.00	0.00	0.00	0.00	0.13	2.04	3.90	4.37	4.60	4.70	4.77
0.28	0.00	0.00	0.00	0.00	0.11	1.71	3.28	3.68	3.87	3.96	4.01
0.24	0.00	0.00	0.00	0.00	0.09	1.44	2.76	3.09	3.26	3.33	3.38
0.20	0.00	0.00	0.00	0.00	0.07	1.21	2.32	2.60	2.74	2.80	2.84
0.17	0.00	0.00	0.00	0.00	0.06	1.02	1.95	2.19	2.31	2.35	2.39
0.14	0.00	0.00	0.00	0.00	0.05	0.86	1.64	1.84	1.94	1.98	2.01
0.12	0.00	0.00	0.00	0.00	0.04	0.72	1.38	1.55	1.63	1.66	1.69
0.10	0.00	0.00	0.00	0.00	0.04	0.61	1.16	1.30	1.37	1.40	1.42
0.08	0.00	0.00	0.00	0.00	0.03	0.51	0.97	1.09	1.15	1.17	1.19
0.07	0.00	0.00	0.00	0.00	0.03	0.43	0.82	0.92	0.97	0.99	1.00
0.06	0.00	0.00	0.00	0.00	0.02	0.36	0.69	0.77	0.81	0.83	0.84
0.05	0.00	0.00	0.00	0.00	0.02	0.30	0.58	0.64	0.68	0.69	0.70
0.04	0.00	0.00	0.00	0.00	0.02	0.25	0.48	0.54	0.57	0.58	0.59
0.04	0.00	0.00	0.00	0.00	0.01	0.21	0.40	0.45	0.48	0.49	0.49
0.03	0.00	0.00	0.00	0.00	0.01	0.18	0.34	0.38	0.40	0.41	0.41
0.02	0.00	0.00	0.00	0.00	0.01	0.15	0.28	0.32	0.33	0.34	0.35
0.02	0.00	0.00	0.00	0.00	0.01	0.12	0.24	0.27	0.28	0.29	0.29

0.02	0.00	0.00	0.00	0.00	0.01	0.10	0.20	0.22	0.23	0.24	0.24
0.01	0.00	0.00	0.00	0.00	0.01	0.09	0.17	0.19	0.20	0.20	0.20
0.01	0.00	0.00	0.00	0.00	0.00	0.07	0.14	0.15	0.16	0.17	0.17
0.01	0.00	0.00	0.00	0.00	0.00	0.06	0.12	0.13	0.14	0.14	0.14
0.01	0.00	0.00	0.00	0.00	0.00	0.05	0.10	0.11	0.11	0.12	0.12
0.01	0.00	0.00	0.00	0.00	0.00	0.04	0.08	0.09	0.09	0.10	0.10
0.01	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.07	0.08	0.08	0.08
0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.06	0.06	0.07	0.07	0.07
0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.05	0.05	0.06	0.06
0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.04	0.05	0.05	0.05
0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.03	0.04	0.04	0.04	0.04
0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.03	0.03	0.03	0.03
0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02	0.03	0.03	0.03
0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02	0.02	0.02	0.02
0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02	0.02	0.02	0.02
0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.02	0.02	0.02
0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

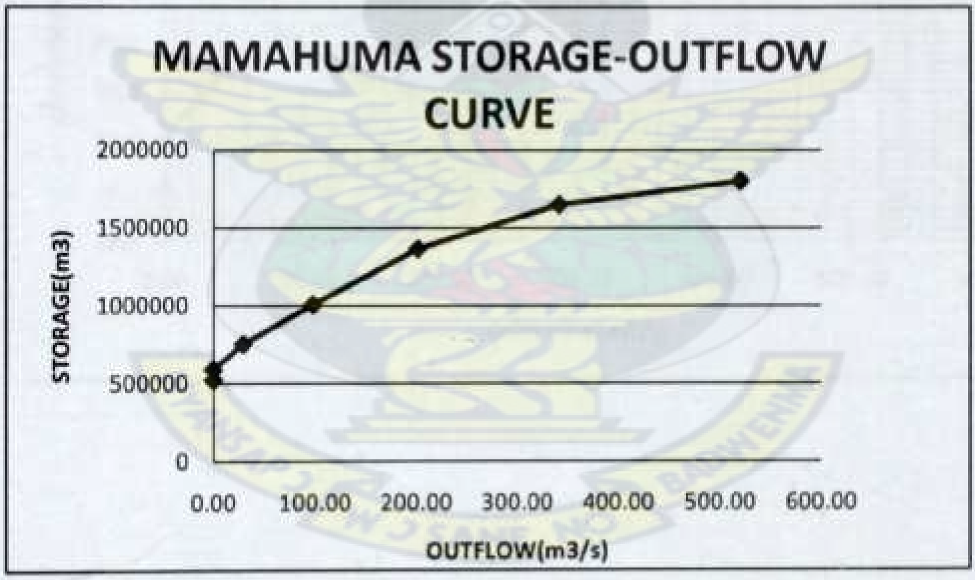
APPENDIX 4: RESERVOIR ELEVATION-STORAGE COMPUTATION

Contour Area(m2)	contour ht(m)	contour interval(m)	STORAGE(m3)
15100	14.50	0	0
48700	15.00	0.5	5533
116700	15.50	0.5	19880
199800	16.00	0.5	47270
276300	16.50	0.5	88455
364550	17.00	0.5	143995
480300	17.50	0.5	217118
706500	18.00	0.5	319583
883000	18.50	0.5	456862
1060850	19.00	0.5	624681
1319800	19.50	0.5	**829463**
1689200	20.00	0.5	1084788
2339000	20.50	0.5	1398621
3121200	21.00	0.5	1763097.25



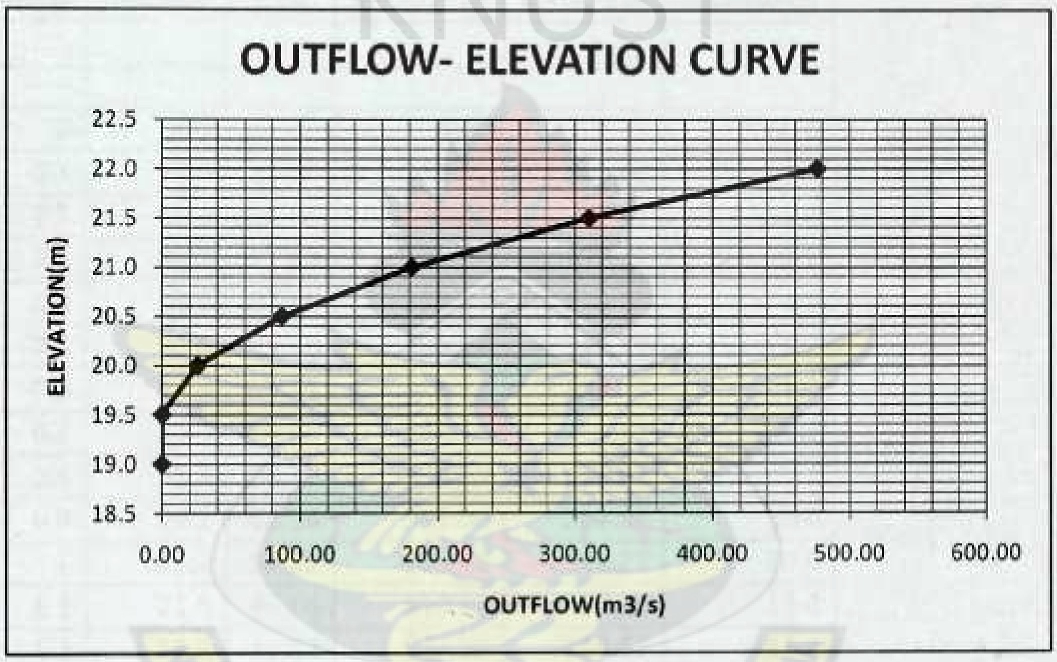
APPENDIX 5: RESERVOIR STORAGE-OUTFLOW RELATION

ELEVATION(m)	STORAGE(m^3)	OUTFLOW(m^3/s)	2S/DELTA 't' +O(m^3)	x	y
14.50	0	0	0.00	0.00	0
15.00	5533	0	30.74	0.00	15950
15.50	19880	0	110.44	0.00	41350
16.00	47270	0	262.61	0.00	79125
16.50	88455	0	491.42	0.00	119025
17.00	143995	0	799.97	0.00	160213
17.50	217118	0	1206.21	0.00	211213
18.00	319583	0	1775.46	0.00	296700
18.50	456862	0	2538.12	0.00	397375
19.00	624681	0	3470.45	0.00	522972
19.50	**829463**	**0.0**	**4608.13**	**0.00**	**589366**
20.00	1084788	29.4	6056.00	29.40	752250
20.50	1398621	97.6	7867.74	97.63	1007050
21.00	1763097.25	201.0	9996.01	201.02	1365050
21.50	2167682.75	341.0	12383.64	340.95	1650000
22.00	2610660	519.4	15023.06	519.40	1800000



APPENDIX 6: RESERVOIR ELEVATION –OUTFLOW RELATION

ELEVATION -OUTFLOW COMPUTATION				
ELEVATION	HEAD(H)	EFFECTIVE HEAD(Ho)	COEFFICIENT(KR)	OUT FLOW(Q)
19.0	0.0	0.00	0.403	0.00
19.5	0.0	0.00	0.403	0.00
20.0	0.5	0.47	0.459	25.90
20.5	1.0	0.97	0.518	87.08
21.0	1.5	1.47	0.576	181.26
21.5	2.0	1.97	0.635	310.24
22.0	2.5	2.47	0.694	476.23



APPENDIX 7: ROUTING THE HYDROGRAPH OF RAINFALL

EVENT OF JULY 1995

STORAGE(S)	Outflow(O)	Q/2*Dt	S+Q/2*Dt	S-Q/2*Dt
m^3	m^3/S			
1084788	25.9	4661.265	1089449	1080126
1398621	87.1	15673.84	1414295	1382947
1763097	181.3	32626.68	1795724	1730471
2167683	310.2	55842.49	2223525	2111840
2610660	476.2	85720.51	2696381	2524939

Time(t)	inflow(I)	(I+I)	S_1-Q_1/2*Dt	S_1+Q_2/2Dt	outflow(Q)
0	0.0		0.00		0.0
0.1	21.6	21.60	20.82	21.60	0.4
0.2	35.4	56.98	75.01	77.81	1.4
0.3	42.6	77.98	147.54	152.98	2.7
0.4	45.0	87.56	226.64	235.10	4.2
0.5	44.1	89.08	304.34	315.72	5.7
0.6	41.3	85.42	375.72	389.76	7.0
0.7	37.5	78.80	438.14	454.52	8.2
0.8	33.3	70.77	490.59	508.91	9.2
0.9	29.1	62.36	533.06	552.96	10.0
1	25.1	54.21	566.13	587.27	10.6
1.1	21.5	46.66	590.70	612.78	11.0
1.2	18.3	39.87	607.85	630.57	11.4
1.3	15.6	33.89	618.62	641.74	11.6
1.4	13.2	28.71	624.03	647.33	11.7
1.5	11.1	24.26	624.95	648.29	11.7
1.6	9.4	20.47	622.20	645.42	11.6
1.7	7.9	17.25	616.41	639.45	11.5
1.8	6.6	14.52	608.21	630.93	11.4
1.9	5.6	12.23	598.12	620.44	11.2
2	4.7	10.29	586.51	608.41	11.0
2.1	4.0	8.66	573.75	595.17	10.7
2.2	3.3	7.29	560.11	581.03	10.5
2.3	2.8	6.13	545.86	566.24	10.2
2.4	2.4	5.15	531.18	551.02	9.9

2.5	2.0	4.33	516.21	535.51	9.7
2.6	1.7	3.64	501.14	519.86	9.4
2.7	1.4	3.06	486.06	504.20	9.1
2.8	1.2	2.57	471.03	488.63	8.8
2.9	1.0	2.16	456.17	473.19	8.5
3	0.8	1.81	441.51	457.99	8.2
3.1	0.7	1.52	427.09	443.03	8.0
3.2	0.6	1.28	412.96	428.36	7.7
3.3	0.5	1.07	399.13	414.03	7.5
3.4	0.4	0.90	385.63	400.03	7.2
3.5	0.3	0.75	372.48	386.38	7.0
3.6	0.3	0.63	359.69	373.11	6.7
3.7	0.2	0.52	347.25	360.21	6.5
3.8	0.2	0.44	335.17	347.69	6.3
3.9	0.2	0.37	323.44	335.54	6.1
4	0.1	0.31	312.08	323.74	5.8
4.1	0.1	0.26	301.10	312.34	5.6
4.2	0.1	0.21	290.47	301.31	5.4
4.3	0.1	0.18	280.17	290.65	5.2
4.4	0.1	0.15	270.23	280.31	5.0
4.5	0.1	0.12	260.64	270.36	4.9
4.6	0.0	0.10	251.34	260.74	4.7
4.7	0.0	0.09	242.39	251.43	4.5
4.8	0.0	0.07	233.72	242.46	4.4
4.9	0.0	0.06	225.36	233.78	4.2
5	0.0	0.05	217.30	225.40	4.1
5.1	0.0	0.04	209.52	217.34	3.9
5.2	0.0	0.03	202.00	209.56	3.8
5.3	0.0	0.03	194.75	202.03	3.6
5.4	0.0	0.02	187.75	194.77	3.5
5.5	0.0	0.02	181.01	187.77	3.4
5.6	0.0	0.02	174.51	181.03	3.3
5.7	0.0	0.01	168.22	174.52	3.2
5.8	0.0	0.01	162.19	168.23	3.0
5.9	0.0	0.01	156.36	162.20	2.9
6	0.0	0.01	150.75	156.37	2.8
6.1	0.0	0.01	145.31	150.75	2.7
6.2	0.0	0.01	140.10	145.32	2.6
6.3	0.0	0.00	135.06	140.10	2.5
6.4	0.0	0.00	130.21	135.07	2.4
6.5	0.0	0.00	125.53	130.21	2.3

6.6	0.0	0.00	120.99	125.53	2.3
6.7	0.0	0.00	116.63	120.99	2.2
6.8	0.0	0.00	112.42	116.64	2.1
6.9	0.0	0.00	108.40	112.42	2.0

APPENDIX 8: ANALYSIS OF SIX HOUR MAXIMUM RAINFALL DATA - ACCRA

6 hour events	No	Y	m	T=(N+1)/m
105.5	1	238	1	46.00
157.2	2	165.6	2	23.00
80.2	3	163.3	3	15.33
122	4	157.2	4	11.50
48.9	5	124	5	9.20
71	6	122	6	7.67
46	7	118.1	7	6.57
81.8	8	106.9	8	5.75
62.8	9	105.5	9	5.11
59.8	10	102.2	10	4.60
87.9	11	100	11	4.18
75.8	12	99	12	3.83
70.4	13	97.6	13	3.54
86.6	14	95.7	14	3.29
100	15	95.5	15	3.07
61	16	90	16	2.88
61.7	17	87.9	17	2.71
165.6	18	87.6	18	2.56
74	19	87	19	2.42
86	20	86.6	20	2.30
85	21	86	21	2.19
87	22	85.3	22	2.09
65.3	23	85	23	2.00
95.7	24	84.6	24	1.92
52.8	25	81.8	25	1.84
62.2	26	80.2	26	1.77
95.5	27	76	27	1.70
163.3	28	75.8	28	1.64
87.6	29	75.7	29	1.59
75.7	30	75.3	30	1.53
75.3	31	74.9	31	1.48
90	32	74	32	1.44
74.9	33	71	33	1.39
106.9	34	70.4	34	1.35
85.3	35	69	35	1.31
84.6	36	67	36	1.28
124	37	65.3	37	1.24

69	38	62.8	38	1.21
67	39	62.2	39	1.18
99	40	61.7	40	1.15
238	41	61	41	1.12
76	42	59.8	42	1.10
102.2	43	52.8	43	1.07
118.1	44	48.9	44	1.05
97.6	45	46	45	1.02
No.of event	N=45			
	Mean	90.716		
	Std	35.169		
Return Period	T		Frequency factor	Estimated extreme rainfall
	2	0.367	-0.156	85.226
	5	1.500	0.828	119.831
	10	2.250	1.479	142.742
	15	2.674	1.847	155.669
	20	2.970	2.104	164.719
	25	3.199	2.302	171.691
	50	3.902	2.913	193.166
	75	4.311	3.268	205.649
	100	4.600	3.519	214.483
	125	4.824	3.714	221.327
	150	5.007	3.873	226.914
	175	5.162	4.007	231.635
	200	5.296	4.123	235.722
	225	5.414	4.226	239.327
	250	5.519	4.317	242.550

APPENDIX 9: HEC–RAS RESULT SHOWING EFFECT OF RAINFALL DIFFERENT RETURN PERIODS ON FLOOD ROUTING

HEC-RAS Plan: Plan 04 River: Mamahuma Reach: Main channel								
Reach	River Sta.	Profile	Q Total	Min Ch El	W.S. Elev	E.G. Elev	E.G. Slope	Flow Area
			(m3/s)	(m)	(m)	(m)	(m/m)	(m2)
Main channel	20		11.67	15.29	15.75	15.84	0.024941	11.53
Main channel	19		11.67	13.4	15.12	15.13	0.000392	30.47
Main channel	18		11.67	13.41	14.99	15	0.001021	25.57
Main channel	17		11.67	13.47	14.77	14.83	0.003764	12.24
Main channel	16		11.67	13.34	14.5	14.61	0.007665	10.51
Main channel	15		11.67	12.25	13.92	13.95	0.001473	16.64
Main channel	14		11.67	12.26	13.63	13.68	0.003523	19.66
Main channel	13		11.67	11.34	12.72	12.74	0.002193	26.32
Main channel	12		11.67	10.69	11.39	11.4	0.005165	21.97
Main channel	11		11.67	8.91	9.84	9.9	0.005893	10.92
Main channel	10		11.67	7.93	9.04	9.05	0.001403	21.9
Main channel	9		11.67	7	8.11	8.15	0.002787	13.33
Main channel	8		11.67	6.41	7.5	7.51	0.00132	27.58
Main channel	7		11.67	6.2	7.07	7.09	0.001973	21.75
Main channel	6		11.67	4.34	5.87	5.92	0.003476	11.51
Main channel	5		11.67	3.72	5.4	5.43	0.00173	13.64
Main channel	4		11.67	3.63	5.1	5.14	0.001922	20.66
Main channel	3		11.67	1.79	3.32	3.34	0.002022	17.19
Main channel	2		11.67	2.08	3.01	3.03	0.001311	18.5
Main channel	1		11.67	1.78	2.7	2.7	0.000602	57.91
Main channel	0		11.67	1.78	2.48	2.51	0.004002	24.7

APPENDIX 10: MAMAHUMA CHANNEL CROSS SECTIONAL DETAILS

Id	East	North	Ht	Code	Dist	Sta	Stat1
5	372856.9	113184.8	22.211	RT	0	0	48.16994982
6	372860.8	113192.4	22.075	RT	8.523002	8.523002	39.64694795
7	372864.1	113200.2	21.828	RT	8.539517	17.06252	31.10743057
8	372870.1	113212.6	21.365	RT	13.74399	30.80651	17.36344214
9	372870.5	113213.4	20.885	RI	0.894002	31.70051	16.4694399
10	372871.4	113215.2	20.634	CCL	1.974772	33.67528	14.49466776
11	372872.3	113217.7	20.868	LI	2.726142	36.40142	11.76852598
12	372872.7	113218.6	21.883	LT	0.994083	37.39551	10.77444299
13	372875	113222.9	21.886	LT	4.846063	42.24157	5.928380155
14	372877.9	113224.8	21.885	LT	3.505407	45.74698	2.422972761
15	372880	113226	22.188	LT	2.422973	48.16995	0
21	372951.8	113029.7	21.91	RT	0	0	87.45771878
22	372960	113037.7	21.533	RT	11.46483	11.46483	75.99288908
23	372969.9	113047	20.996	RT	13.61952	25.08435	62.3733645
24	372980.3	113055.9	20.507	RT	13.67714	38.76149	48.69622397
25	372981.1	113057.2	20.07	RI	1.519895	40.28139	47.17632858
26	372984.3	113060.5	19.529	CCL	4.61721	44.8986	42.55911892
27	372987	113064.2	20.166	LI	4.560418	49.45902	37.99870096
28	372988.7	113066.2	20.628	LT	2.625897	52.08491	35.37280435
29	372994.8	113073.2	20.713	LT	9.31145	61.39636	26.06135457
30	372998.2	113083.8	20.867	LT	11.13661	72.53297	14.92474539
31	373003	113098	21.561	LT	14.92475	87.45772	0
1	373222.5	112792.8	20.444	RT	0	0	70.85369319
2	373238.9	112810.6	20.119	RT	24.20926	24.20926	46.64443669
3	373250.1	112822.9	19.216	RI	16.6106	40.81985	30.03383947
4	373259.2	112834	18.762	CCL	14.39001	55.20987	15.64382703
5	373263.5	112839.7	19.038	LI	7.13164	62.34151	8.512187322
6	373269.7	112845.6	19.971	LT	8.512187	70.85369	0
1	373458.7	112492.7	21.082	RT	0	0	281.4722664
2	373473.4	112507.5	20.757	RT	20.89293	20.89293	260.5793407
3	373493.4	112526.8	20.362	RT	27.77594	48.66887	232.8033967
6	373533.3	112568.9	19.52	CCL	58.03038	106.6993	174.773016
13	373630.6	112673.7	20.264	LT	142.9855	249.6847	31.7875231
14	373651.9	112697.4	20.9	LT	31.78752	281.4723	0
37	374655.3	111923.7	16.361	RT	0	0	98.79473464
38	374660.8	111940.8	15.941	RT	17.88646	17.88646	80.90827258

39	374666	111955.8	15.756	RT	15.9674	33.85386	64.94087213
40	374671.2	111969.7	15.491	RT	14.79451	48.64837	50.14636667
41	374672.5	111971.7	15.407	RI	2.4074	51.05577	47.73896703
42	374674.7	111977	15.293	CCL	5.743429	56.7992	41.99553792
43	374676.9	111982	15.486	LI	5.408662	62.20786	36.58687588
44	374677.7	111984	15.677	LT	2.152025	64.35988	34.43485056
45	374684	111999.7	15.705	LT	16.94892	81.30881	17.48592714
46	374690.5	112015.9	15.403	LT	17.48593	98.79473	0
2	374816.9	111943.9	14.823	RT	0	0	32.3739085
3	374820.1	111953.4	14.11	RI	9.955806	9.955806	22.41810211
4	374821.6	111957.8	13.404	CCL	4.712782	14.66859	17.70532023
5	374822.4	111963.8	13.752	LI	5.984582	20.65317	11.72073796
6	374823.9	111967.5	14.258	LT	4.053544	24.70671	7.667194076
7	374826.9	111974.6	14.997	LT	7.667194	32.37391	0
56	375010.2	111850.7	14.767	RT	0	0	53.95010254
57	375011.8	111862.3	14.659	RT	11.68374	11.68374	42.26636478
58	375013.4	111872.5	14.799	RT	10.29629	21.98003	31.97007375
60	375014.8	111882.3	13.563	RI	9.933301	31.91333	22.03677314
61	375014.8	111883.1	13.414	CCL	0.837026	32.75036	21.19974686
62	375016.2	111886.9	13.726	LI	4.063839	36.81419	17.13590779
63	375016.4	111887.9	14.627	LT	0.918676	37.73287	16.21723211
65	375017.2	111896.3	14.771	LT	8.502706	46.23558	7.714526363
66	375018.6	111903.9	14.801	LT	7.714526	53.9501	0
47	375108.6	111851.5	14.371	RT	4.81651	19.59266	0
48	375106.5	111855.9	14.294	RT	2.944202	14.77615	4.816509628
49	375105.5	111858.6	13.653	RI	2.251562	11.83195	7.760711558
50	375104.7	111860.8	13.473	CCL	1.693655	9.580389	10.01227324
51	375104	111862.3	13.608	LI	1.426769	7.886734	11.70592787
52	375103.6	111863.6	14.106	LT	6.459965	6.459965	13.13269659
53	375101.1	111869.6	14.599	LT	0	0	19.592662
33	375124.9	111867.7	14.537	RT	0	0	72.55878771
34	375127.8	111880.3	14.396	RT	12.89864	12.89864	59.66015019
35	375129.3	111886.4	14.323	RT	6.327225	19.22586	53.33292488
36	375130.3	111890	14.067	RT	3.731734	22.9576	49.6011907
37	375130.5	111890.9	13.339	RI	0.897867	23.85546	48.70332379
38	375130.9	111892.3	13.407	CCL	1.511457	25.36692	47.1918672
39	375131.5	111895.1	13.568	LI	2.826281	28.1932	44.36558608
40	375132.1	111897.5	14.229	LT	2.470372	30.66357	41.89521404
41	375134.5	111907.8	14.515	LT	10.59701	41.26058	31.29820371
42	375138.1	111922.9	14.883	LT	15.46119	56.72178	15.83701045
43	375141.8	111938.3	15.373	LT	15.83701	72.55879	0
18	375282.1	111759.3	13.983	RT	0	0	101.480175
19	375297	111770.2	14.006	RT	18.43738	18.43738	83.04279723
20	375309.1	111779.5	14.087	RT	15.22968	33.66706	67.81311325

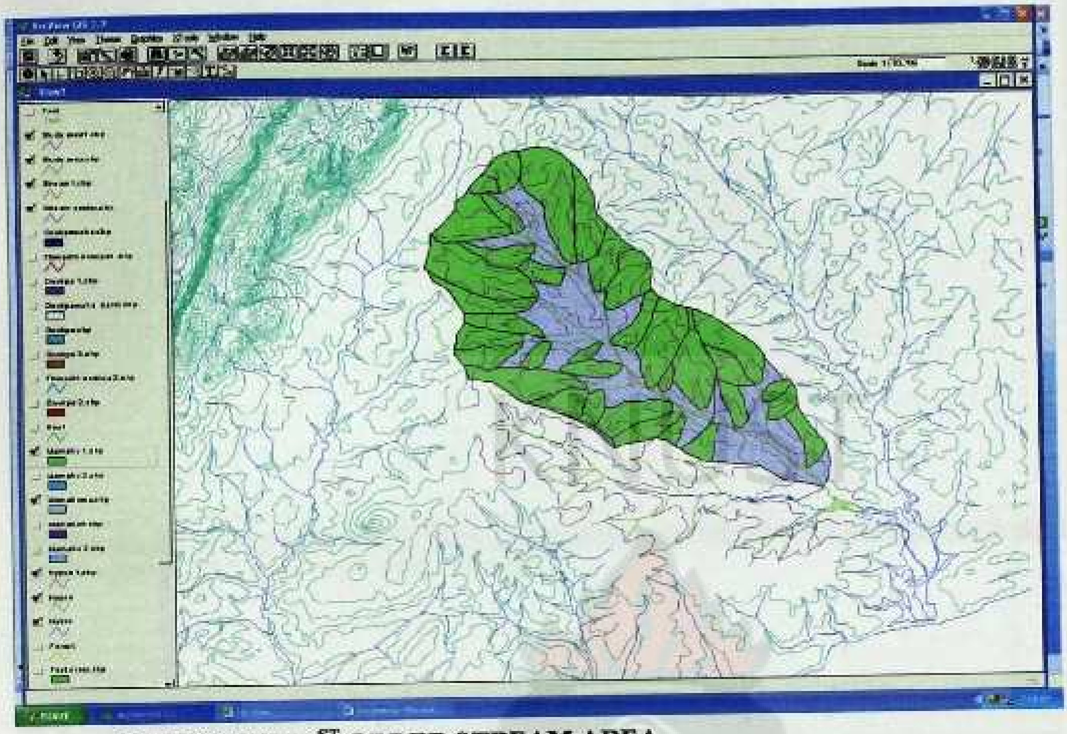
21	375320.1	111787.7	13.967	RT	13.77544	47.4425	54.0376742
22	375321.8	111789.1	13.127	RI	2.157624	49.60012	51.88005028
23	375326.2	111792.5	12.248	CCL	5.571207	55.17133	46.30884349
24	375328.2	111793.8	12.782	LI	2.428778	57.60011	43.88006582
25	375330.1	111795.2	13.09	LT	2.384703	59.98481	41.49536248
26	375338.4	111800.8	14.162	LT	9.991485	69.9763	31.50387711
27	375350.4	111810.7	14.102	LT	15.54228	85.51858	15.96159256
28	375363.4	111820	14.854	LT	15.96159	101.4802	0
2	375414.4	111674.2	14.132	RT	0	0	92.37844373
3	375419.2	111686.5	13.926	RT	13.14942	13.14942	79.22902146
4	375424.2	111699.3	13.741	RT	13.80278	26.9522	65.4262417
5	375429.8	111715.9	13.077	RT	17.51461	44.46681	47.91163329
6	375430.1	111717.4	12.396	RI	1.537874	46.00468	46.37375911
7	375430.6	111718.7	12.261	CCL	1.377276	47.38196	44.99648317
8	375431.3	111720.5	12.49	LI	1.968134	49.35009	43.02834877
9	375431.5	111721.3	13.403	LT	0.799321	50.14942	42.22902781
10	375433.6	111731.2	13.279	LT	10.12851	60.27792	32.10052185
11	375440.9	111745	13.409	LT	15.58285	75.86077	16.51766996
12	375448.1	111759.9	13.596	LT	16.51767	92.37844	0
71	375734.6	111586.5	12.595	RT	0	0	88.52131729
72	375741.8	111599.4	12.419	RT	14.77621	14.77621	73.74510546
73	375748.9	111612.4	12.258	RT	14.77095	29.54716	58.9741539
74	375753.5	111620.6	12.213	RT	9.413815	38.96098	49.56033916
75	375754.9	111622.8	12	RI	2.648339	41.60932	46.91200006
76	375755.6	111624.6	11.674	RI	1.888956	43.49827	45.02304426
77	375756	111625.3	11.343	CCL	0.828592	44.32686	44.19445268
78	375756.8	111626.4	11.579	LI	1.354638	45.6815	42.83981473
79	375757.3	111627.3	12.506	LT	1.011863	46.69337	41.82795209
80	375759.1	111630.8	12.558	LT	3.907338	50.6007	37.92061373
81	375763.3	111638.7	12.187	LT	8.945081	59.54578	28.97553307
82	375769.6	111649.9	12.957	LT	12.8649	72.41068	16.11063599
83	375777.7	111663.8	13.572	LT	16.11064	88.52132	0
107	376089	111361.3	11.804	RT	0	0	91.38146212
108	376095.4	111375.1	11.52	RT	15.25835	15.25835	76.12310821
109	376099.1	111384.2	11.217	RT	9.80659	25.06494	66.31651838
110	376102.7	111394.4	10.99	RT	10.75121	35.81616	55.56530543
111	376103.4	111396.2	10.691	RI	1.972509	37.78867	53.59279637
112	376103.8	111397.2	10.696	CCL	1.089543	38.87821	52.50325288
113	376104.3	111398.2	10.776	LI	1.075737	39.95395	51.42751593
116	376107.6	111404.6	11.304	LT	7.201347	47.15529	44.22616848
117	376115	111421.5	11.166	LT	18.48539	65.64069	25.74077425
118	376121.5	111436.2	10.874	LT	16.0974	81.73809	9.643372906
119	376125.7	111444.9	10.905	LT	9.643373	91.38146	0
124	376286.8	111193.1	11.764	RT	0	0	118.6243672
125	376295.8	111203.6	11.384	RT	13.79768	13.79768	104.8266829
126	376307.5	111219.3	11.183	RT	19.6332	33.43089	85.19347889
127	376317.1	111233.5	10.671	RT	17.08918	50.52007	68.10429699
128	376318.6	111236.1	9.295	RI	3.065188	53.58526	65.03910891

129	376324.7	111242	8.906	CCL	8.416142	62.0014	56.62296668
130	376329.5	111249.4	9.793	LI	8.868548	70.86995	47.75441889
131	376330.4	111250.6	10.353	LT	1.463702	72.33365	46.29071639
132	376339.5	111262.4	10.942	LT	14.923	87.25665	31.3677191
133	376348.7	111272.5	10.816	LT	13.70298	100.9596	17.66473778
134	376360.3	111285.9	10.911	LT	17.66474	118.6244	0
140	376581.4	111057.1	9.88	RT	0	0	128.2521465
141	376593.1	111072.2	9.71	RT	19.0822	19.0822	109.169945
142	376607	111089.5	9.591	RT	22.17413	41.25633	86.99581395
143	376618.2	111102.2	9.068	RT	16.96042	58.21675	70.03539163
144	376621.5	111106.1	8.755	RI	5.091507	63.30826	64.94388449
145	376626.8	111110.9	8.238	CCL	7.140507	70.44877	57.80337744
146	376632.9	111118.2	7.934	LI	9.502576	79.95134	48.30080152
147	376643.7	111130.1	9.101	LT	16.07926	96.0306	32.22154171
148	376653.4	111141.3	9.873	LT	14.80639	110.837	17.41515593
149	376665	111154.3	10.236	LT	17.41516	128.2521	0
49	377004.3	110773.7	8.224	RT	0	59.28529	115.1897153
48	377003.6	110789.5	8.111	RT	15.81054	59.28529	99.37917563
47	377002.8	110808	8.312	RT	18.49922	43.47475	80.87995597
46	377002.2	110822.7	8.37	RT	14.65033	24.97553	66.22962843
45	377002.2	110826.4	7.391	RI	3.740348	10.32521	62.48928072
44	377001.7	110833	6.998	CCL	6.584858	6.584858	55.90442233
43	377001.1	110839.6	7.475	LI	6.616981	39.27459	49.28744137
42	377001.1	110841.2	8.203	LT	1.637313	32.65761	47.65012863
41	377000.7	110856.6	8.775	LT	15.45997	31.0203	32.19015809
40	377000.2	110872.2	8.962	LT	15.56033	15.56033	16.62983082
39	376999.5	110888.8	9.151	LT	16.62983	0	0
36	377164.9	110518.4	7.72	RT	0	0	96.96531575
35	377177.1	110531.3	7.276	RT	17.80048	17.80048	79.16483682
34	377189.2	110543.8	7.203	RT	17.31648	35.11696	61.84835606
33	377198.3	110554.1	7.346	RT	13.82401	48.94097	48.02434535
32	377200.1	110554.8	6.494	RI	1.910346	50.85132	46.11399957
31	377202.4	110557.7	6.41	CCL	3.707002	54.55832	42.40699742
30	377204.3	110560.4	6.572	LI	3.313537	57.87186	39.09346033
29	377211.2	110567.9	7.076	LT	10.21034	68.08219	28.88312395
28	377219.9	110577.5	7.371	LT	12.88197	80.96416	16.00115477
27	377231.1	110588.9	7.727	LT	16.00115	96.96532	0
1	377421	110422.4	6.63	RT	0	0	58.23630582
2	377429.4	110429.4	6.535	RT	10.94446	10.94446	47.29184119
3	377434.9	110433.7	6.474	RT	7.03346	17.97792	40.25838159
4	377437.1	110435.7	6.195	RI	2.968893	20.94682	37.2894887
5	377438.9	110437.6	6.197	CCL	2.634765	23.58158	34.65472343
6	377441.4	110439.8	6.296	LI	3.293667	26.87525	31.36105648
7	377444.2	110442.3	6.493	LT	3.728331	30.60358	27.63272575
8	377451.7	110452.3	7.223	LT	12.5888	43.19238	15.04392402
9	377460.4	110464.6	7.125	LT	15.04392	58.23631	0

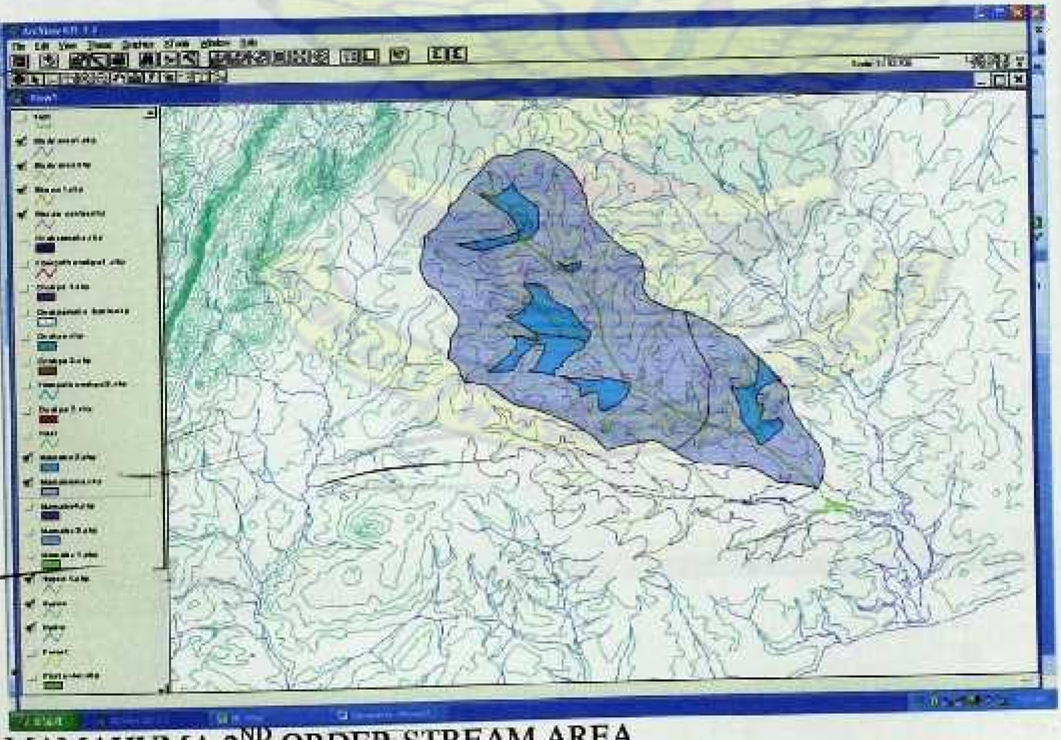
38	377812.4	110191.5	7.512	RT	0	0	84.62576195
39	377818.9	110198.6	7.416	RT	9.608125	9.608125	75.01763653
40	377826.5	110208.9	7.015	RT	12.77452	22.38265	62.24311466
41	377830.7	110215.7	6.579	RT	7.967073	30.34972	54.27604142
42	377836.2	110221.8	5.671	RT	8.314279	38.664	45.9617628
44	377837.8	110223.3	4.84	RI	2.196728	40.86073	43.76503501
43	377837.9	110223.4	4.85	RI	0.036056	40.89678	43.72897949
45	377839.5	110225.5	4.341	CCL	2.726607	43.62339	41.00237245
47	377842.2	110229.7	5.246	LI	4.979057	48.60245	36.02331511
48	377844.3	110233.8	6.021	LT	4.574106	53.17655	31.44920885
49	377847.5	110238.9	6.528	LT	6.022247	59.1988	25.42696168
50	377850.8	110242	6.637	LT	4.54636	63.74516	20.88060138
51	377856.7	110248.4	6.614	LT	8.681096	72.42626	12.19950577
52	377864	110258.2	6.793	LT	12.19951	84.62576	0
94	377869.8	110004.6	5.93	RT	0	0	91.29266677
95	377881.2	110011.5	5.662	RT	13.36342	13.36342	77.92924977
96	377891.3	110019.1	5.567	RT	12.5698	25.93322	65.35944814
97	377900.7	110026.2	5.584	RT	11.80542	37.73864	53.55402612
98	377910.2	110034.6	5.741	RT	12.65074	50.38938	40.9032876
99	377911.3	110034.8	4.587	RI	1.16897	51.55835	39.73431797
101	377914.1	110037	3.723	CCL	3.506487	55.06484	36.22783113
102	377918.4	110042	4.553	LI	6.666942	61.73178	29.56088932
104	377919.8	110043.2	5.664	LT	1.79304	63.52482	27.76784889
105	377927	110048.1	5.627	LT	8.750595	72.27541	19.01725388
106	377935.6	110052.4	5.796	LT	9.612428	81.88784	9.404825623
107	377943.7	110057.3	5.704	LT	9.404826	91.29267	0
78	377985.5	109894.8	5.25	RT	0	0	124.931762
79	377996.1	109902.5	4.977	RT	13.05882	13.05882	111.8729465
80	378006.5	109909.3	4.955	RT	12.44248	25.5013	99.43046183
81	378016.5	109917.2	4.965	RT	12.75634	38.25764	86.67412426
82	378027.5	109925.3	4.966	RT	13.65309	51.91073	73.02103026
83	378039	109934.2	5.104	RT	14.52147	66.4322	58.49956115
84	378040.8	109935.2	3.799	RI	2.10246	68.53466	56.39710069
85	378044.8	109938.1	3.63	CCL	4.873642	73.4083	51.52345882
86	378046.3	109940.1	4.164	LI	2.548556	75.95686	48.97490276
88	378047.7	109941.5	5.905	LT	1.932251	77.88911	47.04265174
89	378059	109950.2	5.236	LT	14.30226	92.19137	32.74039108
90	378065.5	109954.9	4.973	LT	7.960692	100.1521	24.77969859
91	378077.2	109963.4	5.054	LT	14.49108	114.6431	10.28861633
92	378085.3	109969.7	4.96	LT	10.28862	124.9318	0
72	378344.2	109093.8	5.59	RT	0	0	118.8604742
73	378360.9	109097.6	5.112	RT	17.16287	17.16287	101.697604
74	378377	109101.4	4.569	RT	16.51504	33.67791	85.1825642
75	378393.4	109105.7	4.216	RT	16.96617	50.64408	68.21639162
76	378407.4	109109	3.881	RT	14.33588	64.97996	53.88051128
77	378425.9	109113.7	3.345	RT	19.09964	84.0796	34.78087377
78	378427.7	109114.1	2.333	RI	1.865653	85.94525	32.91522107
79	378431.3	109114.8	1.79	CCL	3.682984	89.62824	29.23223682
80	378437.5	109119.1	2.775	LI	7.507077	97.13531	21.72515983

81	378444.1	109122.4	3.409	LT	7.44524	104.5806	14.27991947
82	378454.5	109132.2	2.929	LT	14.27992	118.8605	0
95	378572	108983.8	2.952	LT	0	0	
96	378572	108982.5	2.266	LI	1.226863	1.226863	
97	378571.8	108979.6	2.083	CCL	2.928198	4.155061	
98	378573.3	108962.5	2.158	RI	17.21981	21.37487	
99	378573.4	108961.3	3.495	RT	1.163912	22.53878	
100	378574.4	108951.9	3.259	RT	0	0	378.1039455
112	378919.6	108854.8	2.187	RT	358.5614	358.5614	19.54252125
113	378921.7	108857.3	2.144	RI	3.253929	361.8154	16.28859178
114	378926.9	108863.2	1.783	CCL	7.869137	369.6845	8.41945479
115	378930.4	108867.1	1.923	LI	5.274495	374.959	3.144959459
116	378932.5	108869.4	2.087	LT	3.144959	378.1039	0

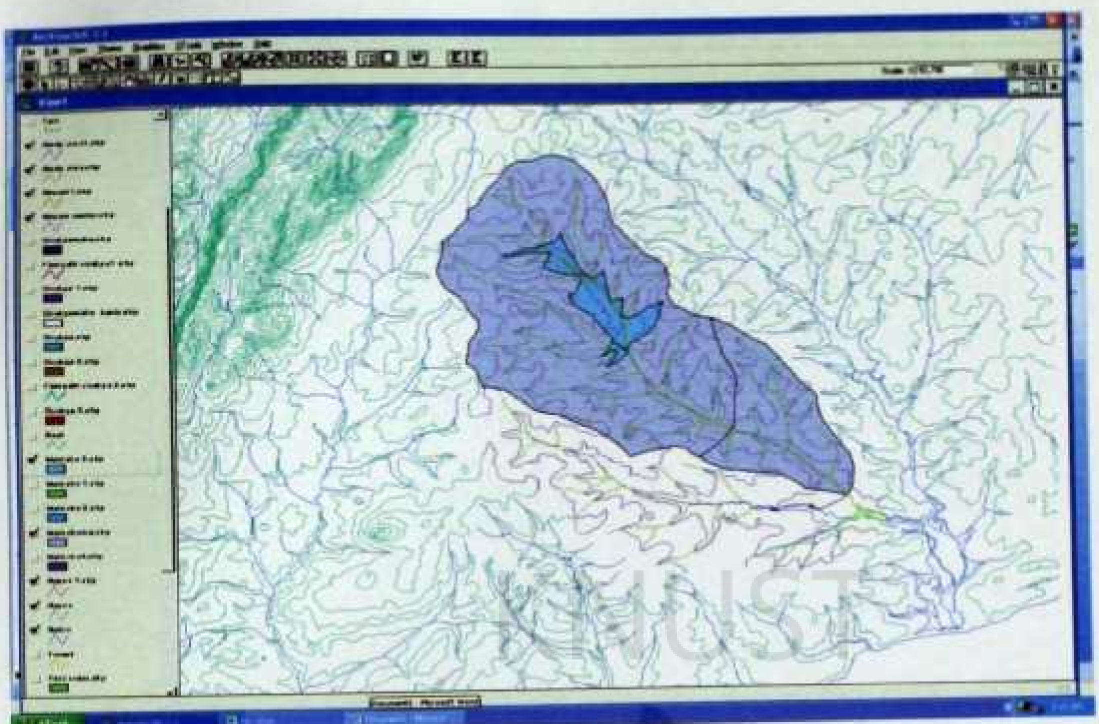
APPENDIX 11: MAMAHUMA STREAM ORDERING BY HORTON'S SYSTEM



MAMAHUMA 1ST ORDER STREAM AREA



MAMAHUMA 2ND ORDER STREAM AREA



MAMAHUMA 3RD ORDER STREAM AREA

MAMAHUMA 4TH ORDER STREAM AREA